



Bridge Standards and Procedures Manual

VOLUME 1

Supplement to CHBDC S6:19



Ministry of
Transportation
and Transit

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Preface

The BC Ministry of Transportation and Transit (BC MoTT) Supplement to CSA S6:19 is to be read and utilized in conjunction with the CSA S6:19 Canadian Highway Bridge Design Code. Included in this supplemental document are referenced bridge design code clauses where; additional text is provided that supplements the design clause, changes are noted that either delete or modify text, or additional commentary is provided for the reference of the designer. All Commentary within this document is denoted by *italicized text*. The text under each specific clause is considered additional and supplemental to the information provided in the CSA S6:19 Canadian Highway Bridge Design Code.

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1.1 Scope

1.1.1 Scope of Code

Add the following paragraphs:

The Canadian Highway Bridge Design Code, CSA S6:19 (CHBDC) applies subject to each of the CHBDC sections specified herein by section number and title, being amended, substituted or modified, as the case may be, in accordance with the amendments, substitutions and modifications described herein as corresponding to each such CHBDC section.

The Canadian Highway Bridge Design Code, CSA S6:19 (CHBDC) shall apply for the design, evaluation, and rehabilitation design of Ministry bridges and other Ministry structure types that are referenced in the scope of CHBDC.

The “BC Ministry of Transportation and Infrastructure Supplement to the Canadian Highway Bridge Design Code, CSA S6:19” (Supplement to CHBDC S6:19) shall also apply for the design, evaluation, rehabilitation design and construction of Ministry bridges and other Ministry structures types that are referenced within the scope of CHBDC.

In the event of inconsistency between the Supplement to CHBDC S6:19 and the CHBDC, the Supplement to CHBDC S6:19 shall take precedence over the CHBDC.

In the event of inconsistency, between Project specific Contracts and Terms of Reference prepared by or on behalf of the Ministry, and the Supplement to CHBDC S6:19 or the CHBDC, the Project specific Contracts and Terms of Reference shall take precedence over the Supplement to CHBDC S6:19 or the CHBDC. In cases where precedence may be unclear, the Ministry Chief Bridge Engineer will provide direction regarding the design requirements.

1.3 Definitions

1.3.2 General administrative definitions

Add the following administrative definitions:

Approval/Approved/approval by the owner/approved by the owner/approval of the owner – means approved in writing by the Regulatory Authority.

Consent/Consented to – means consented to by the Ministry engineer who has the authority, responsibility and technical expertise to provide consent as allowed herein and documented.

Design Exception – means a deviation from Ministry design standards. Design Exceptions require Approval.

Engineering Association – means the Association of Professional Engineers and Geoscientists of the Province of B.C., currently doing business as Engineers and Geoscientists BC.

Ministry – means the BC Ministry of Transportation and Transit (BC MoTT).

Regulatory Authority – means the persons who may from time-to-time hold, or be acting in the position of, the Office of Chief Engineer of the BC Ministry of Transportation and Transit.

1.3.3 General technical definitions

Add the following technical definitions:

Animal Overpass - means a structure carrying animals or wildlife over a road, highway or other facility

Animal Underpass – means a structure carrying a highway, road or other facility over an animal or wildlife path.

BCL – means British Columbia Loading

BC Supplement to TAC Geometric Design Guide - means the compilation of Ministry recommended design practices and instructions comprising supplemental design guidelines which are published by the Ministry and which are to be used concurrently with the Transportation Association of Canada's (TAC) Geometric Design Guide for Canadian Roads.

CHBDC – means the Canadian Highway Bridge Design Code CSA S6:19.

Design-Build Standard Specifications (DBSS) – means the BC Ministry of Transportation and Infrastructure Design-Build Standard Specifications for Highway Construction relating to material specification, construction methodology, quality testing requirements and payment which are published by the Ministry and which are applicable to Ministry Design-Build bridge and highway construction projects unless otherwise specified. (Note – Where this Supplement to CHBDC S6:19 uses the term SS, then the corresponding DBSS section shall apply to Design-Build projects.)

Embankment – earth or rock slope, with or without a foundation unit, that has been altered by cuts or fills, structurally stabilized, subject to ground improvement, or drainage modification and is adjacent to, or supporting, a highway or bridge.

Flyover – means a structure carrying one-way traffic over a highway from one highway to another highway.

Footbridge – means a structure providing access to pedestrians over water and land but not over a road.

Highway – has the same definition as given in S6:19 and includes a Provincial public undertaking, within the meaning of the Transportation Act, S.B.C. 2004, c. 44.

Low Volume Road (LVR) Structure – means a bridge or structure, as designated by the Ministry, on a side road with an average daily traffic ADT (for a period of high use) total in both directions, not exceeding 400. Numbered Routes are not considered as a Low Volume Road unless otherwise Approved.

Numbered Route – means a highway, within the meaning of the Transportation Act, S.B.C. 2004, c. 44, **designated by number by the Ministry.**

Overhead – means a structure carrying a highway over a railway or a railway and other facility.

Overpass – means a structure carrying a highway over a road or lesser highway.

Pedestrian Overpass – means a structure carrying pedestrians over a road, highway or other facility.

Pedestrian Underpass – means a structure carrying pedestrians under a road, highway or other facility.

Railway Underpass – means a structure carrying a railway or a railway and other facility over a highway or roadway.

Recognized Products List – means a data base of products which is to be used as a guide by the Engineer and Constructor to identify products which may be acceptable to the Ministry. The address is as follows:

http://www2.gov.bc.ca/assets/gov/driving-and-transportation/transportation-infrastructure/engineering-standards-and-guidelines/recognized-products-list/recognized_products_list.pdf

Scuppers -means horizontal openings through a curb or barrier for roadway drainage.

Special Provisions (SP) – means the project specific construction specifications relating to material specification, construction methodology, quality testing requirements and payment which are prepared by or on behalf of the Ministry and are applicable to Ministry construction projects.

Standard Specifications (SS) – means the BC Ministry of Transportation and Infrastructure Standard Specifications for Highway Construction relating to material specification, construction methodology, quality testing requirements and payment which are published by the Ministry and which are applicable to Ministry Design-Bid-Build bridge and highway construction projects unless otherwise specified. (Note – Where this Supplement to CHBDC S6:19 uses the term SS, then the corresponding DBSS section shall apply to Design-Build projects.)

S6-06 – means the Canadian Highway Bridge Design Code CAN/CSA-S6-06

S6-14 – means the Canadian Highway Bridge Design Code CSA-S6-14

S6-19 – means the Canadian Highway Bridge Design Code CSA S6:19

TAC Geometric Design Guide for Canadian Roads – means the roadway design guidelines published by the Transportation Association of Canada which is to be used concurrently with the BC Supplement to TAC Geometric Design Guide.

Tunnel – means a covered roadway or pathway through or under an obstruction such as a highway fill, a mountain or a river etc.

Underpass – means a structure carrying a road or lesser highway over a highway.

1.4 General requirements

1.4.1 Approval

Add the following paragraphs:

Exceptions from the Supplement to CHBDC S6:19, including for the purpose of application of codes other than S6:19, may be obtained with prior written Approval.

The following products, materials or systems shall not be incorporated into Ministry bridge projects unless specifically Consented to by the Ministry:

- a) Steel grid decking;
- b) Impressed current cathodic protection system;

- c) Modular deck joints;
- d) Bridge deck heating systems;
- e) Proprietary composite steel/concrete girders;
- f) Full depth precast deck panels;
- g) Timber components;
- h) Mechanically Stabilized Earth (MSE) walls with dry cast concrete block facings;
- i) Walls with wire facings;
- j) Mechanically Stabilized Earth (MSE) walls with polymeric reinforcement used as abutment walls or wing walls;
- k) Fibre-reinforced polymer (FRP) structural products;
- l) Polymer composite based structural products;
- m) Welded shear keys for precast concrete beams and slabs;
- n) Discontinuous spans between substructure elements; and
- o) Sheet piles carrying vertical loads.

Commentary: *Ministry Consent is required for the use of above listed products, materials, or systems to ensure that there is a thorough technical review undertaken on a project-by-project basis to assess the risks and to enable informed decisions regarding whether their use is considered acceptable.*

1.4.2 Design

1.4.2.3 Design life

Add the following paragraph:

For any calculations which are time dependent including but not limited to fatigue, corrosion, creep and shrinkage, the length of time shall be specified as 100 years.

1.4.2.6 Economics

Delete the first sentence and replace with the following:

After safety, total life cycle costs shall be a key consideration in selecting the type of structure but may not be the determining consideration on all projects.

1.4.2.7 Environment

Delete the last paragraph and replace with:

Particular attention shall be paid to the preservation of fish, wildlife, native vegetation and associated habitat. Structures on fish-bearing streams shall be designed to pass fish in accordance with Approved guides, standards, methods and criteria.

1.4.2.8 Aesthetics

Commentary: *General guidelines for bridge aesthetics are set out in the Ministry's Manual of Aesthetic Design Practice.*

1.4.4 Construction

1.4.4.3 Construction methods

Commentary: Reference the BC Ministry of Transportation and Infrastructure Commercial Vehicle Safety and Enforcement (CVSE) programs and the Ministry Bridge Standards and Procedures Manual – Volume 2 Procedures and Directions, for guidelines associated with transportation of bridge girders in BC.

1.4.4.5 Plans

The following provisions shall be added to the end of the fourth paragraph:

Approved specifications for construction and rehabilitation shall include the Ministry's SS, DBSS and SP for bridge construction. In the event of any inconsistency or conflict between these Ministry construction specifications and the CHBDC S6:19, the Ministry construction specifications shall take precedence and will govern.

1.5 Geometry

1.5.2 Structure geometry

1.5.2.1 General

Delete the first paragraph and replace with:

Roadway and sidewalk widths, curb widths and heights, together with other geometrical requirements not specified in S6:19 or this Supplement, shall comply with the BC Supplement to TAC Geometric Design Guide, or in their absence, with the TAC Geometric Design Guide for Canadian Roads.

Change the first sentence of the second paragraph to read:

Protected sidewalks, bicycle and multi-use pathways shall be separated from traffic by a barrier or guide rail. For design speeds ≤ 60 km/h, a raised curb may be used with the curb having a face height of 200 mm and a face slope steeper than one horizontal to three vertical.

Add the following paragraphs and Table 1.5.2.1 – Sidewalk Widths

Accommodation of cyclists shall be in accordance with the Ministry Cycling Policy.

Commentary: The Ministry's Cycling Policy can be found at the following link:

<http://www2.gov.bc.ca/gov/content/transportation/driving-and-cycling/cycling/cycling-regulations-restrictions-rules/cycling-policy>

Guidance for Active Transportation can be found in the "British Columbia Active Transportation Design Guide (2019 Edition)" at the following link:

<https://www2.gov.bc.ca/gov/content/transportation/transportation-infrastructure/engineering-standards-guidelines/traffic-engineering-safety/active-transportation-design-guide>

Design widths for shoulder bikeways shall be in accordance with the BC Supplement to TAC Geometric Design Guide.

The following table of sidewalk widths shall be used to determine the sidewalk width for various site conditions. The widths specified shall be the clear distance from the back of parapet or face of curb to the railing. Sidewalks are to be located on the side of the highway which is predominantly used by either pedestrians or cyclists. In dense urban areas, consideration shall be given to providing a sidewalk on both sides of the bridge. Where shoulder widths are provided that are 2.0 m or greater, consideration shall be given to accommodating cyclists on the roadway.

Table 1.5.2.1

Sidewalk widths

Type of Traffic	Direction	Minimum Width (metres)
Pedestrian Only	Bi-directional	1.5 ¹
Pedestrian Only	Bi-directional	1.8 ²
Pedestrian and Cycle	Uni-directional	2.5 ³
Pedestrian and Cycle	Bi-directional	3.5 ³

Notes:

1. Sidewalk width applies where the approach roadways has no sidewalk
2. Minimum sidewalk width or match sidewalk width approaching structure
3. These widths are intended for high volume urban areas. Reductions will be considered on a project specific basis as Consented to by the Ministry.

Commentary: *In most cases, the bridge deck width will incorporate the lane and shoulder width dictated for the highway. Generally, this information shall be provided by the Ministry's Highway Designer or designate. In the case of long bridge structures, consideration may be given to reducing the stipulated shoulder width on the structure. The BC Supplement to TAC Geometric Design Guide and the TAC Geometric Design Guide for Canadian Roads may be used for guidance.*

1.5.2.2 Clearances

1.5.2.2.1 Roadways and sidewalks

Delete and replace with the following:

Minimum vertical clearance to bridge structures shall be 5.0 m over all paved highway surfaces, including any on- or off-ramp(s) that pass underneath. The minimum vertical clearance to pedestrian structures, sign bridges, and other lightweight structures spanning the highway shall be 5.5 m.

Minimum vertical clearances for pedestrian/cycle tunnel structures shall be 2.5 metres. The minimum vertical clearance for pathways under structures shall be 2.5 meters. If the pathway is designated for shared equestrian use, the clearance shall be increased to 3.5 metres.

Long-term settlement of supports, superstructure deflection and future pavement overlay shall be accounted for in the vertical clearances.

Consideration shall be given to providing horizontal separation between adjacent structures for maintenance access.

The proposed horizontal separation shall be determined with consideration of pounding effects during seismic events.

For gaps greater than 0.3 m and up to 3 m between adjacent structures, fall arrest provisions shall be provided to prevent people from errantly falling through the gap.

Commentary: *For adjacent wide structures, consider the clearances required for the operation of an under bridge inspection unit when setting the gap spacing.*

1.5.2.2.2 Railways

Commentary: *The designer shall reference the Ministry's Bridge Standard and Procedures Manual, Volume 2 – Procedures and Directions, Section 5.0 Regulatory Submission Requirements with regards to procedures and drawing requirements for regulatory submissions.*

All regulatory submissions required for grade separated rail crossings to Transport Canada and the railway companies will be made by the Ministry's Rail/Navigable Waters Specialist (part of the Ministry Transportation Systems and Road Safety Engineering department), unless otherwise Consented to by the Ministry.

1.5.2.2.3 Waterways

Commentary: *The designer shall reference the Ministry's Bridge Standard and Procedures Manual, Volume 2 – Procedures and Directions, Section 5.0 Regulatory Submission Requirements with regards to procedures and drawing requirements for regulatory submissions.*

All regulatory submissions to Transport Canada required for water crossings will be made by the Ministry's Rail/Navigable Waters Specialist (part of the Ministry Transportation Systems and Road Safety Engineering department), unless otherwise Consented to by the Ministry.

Add the following clause:

1.5.2.3 Pedestrian/cycle bridges

A maximum gradient of 1:12 shall be used for wheelchair traffic on ramps. The clear distance between the railings shall comply with Clause 1.5.2.1 but shall not be less than 2.0 m.

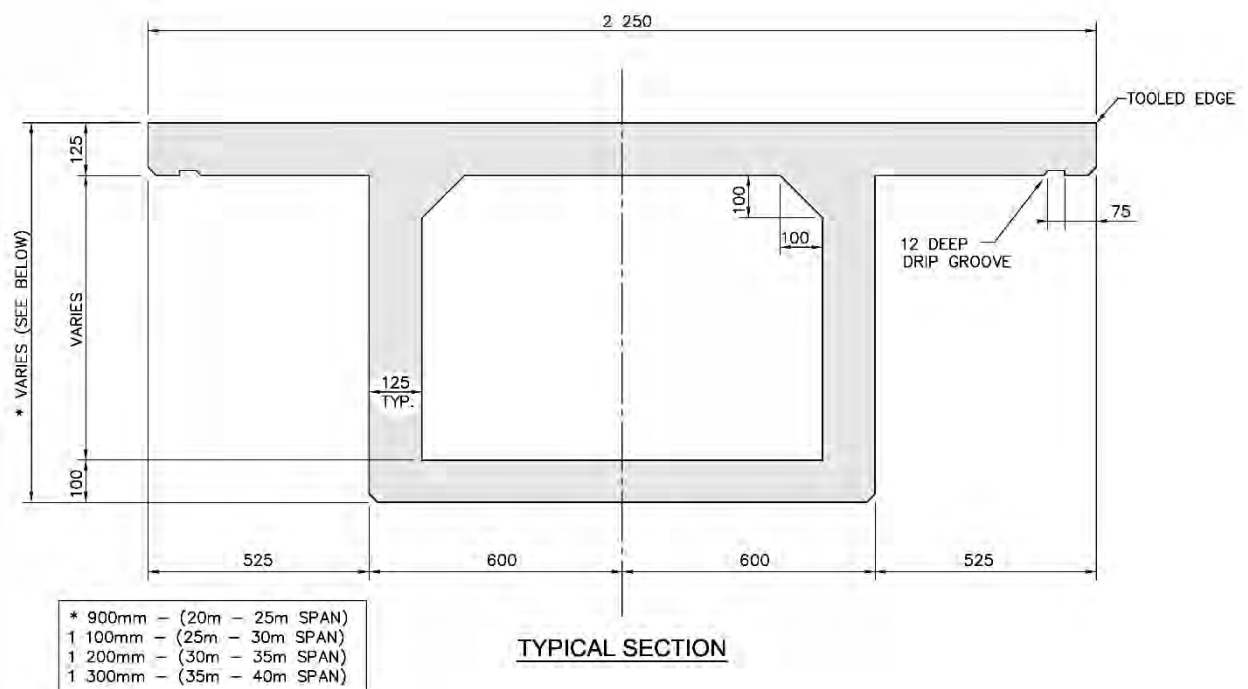
At locations where there is a change in gradient at the piers, the provision of a smooth curve over the piers shall be considered for improving aesthetics.

A crossfall shall be provided on the deck surface of pedestrian/cycle bridges to ensure adequate drainage.

Commentary: Figure 1.5.2.3 details a modified concrete single cell box beam that has been utilized throughout BC as a pedestrian bridge structure.

The Designer should consult with the Ministry for the use of S7 Pedestrian, cycling, and multiuse bridge design guideline.

Figure 1.5.2.3



1.6 Barriers

1.6.1 Superstructure barriers

Add the following:

Unless otherwise Approved, all roadside edge barriers on bridges shall be permanent and shall be anchored.

The standard sidewalk railing, when incorporated on a structure, shall extend a minimum of 3 m beyond the bridge abutments.

1.7 Auxiliary components

1.7.2 Approach slabs

Delete clause and replace with the following:

Approach slabs shall be provided as follows:

at all Lifeline bridges,

at all Major Route and Other bridges in Seismic Performance Categories 2 and 3,

unless otherwise Consented to by the Ministry, at all Numbered Route bridges where a total settlement greater than 50 mm is anticipated between the abutment and the roadway fill, and,

as Consented to by the Ministry based on project specific conditions.

Approach slabs are not required for low-volume road structures on gravel surfaced roads.

Approach slabs shall be 6 m in length (as measured perpendicular to the abutment), located at least 100 mm below finished grade, anchored to the abutments. and shall be designed to match the full width of the bridge deck. Cover and reinforcing type shall be as per the requirements for deck slabs.

Approach slabs shall be designed as a one-way slab in the longitudinal direction to support CL-800 loading or Special Truck and Special lane loading if applicable, whichever produces the maximum effect. The slab shall be assumed to span 75% of its length (4.5 m). Approach slabs shall be fully supported by fill and shall extend transversely to the limits of the roadway. The joints around the approach slab shall be sealed.

Approach slabs shall have a 100 mm minimum asphalt overlay but do not require a waterproofing membrane unless specified otherwise by the Ministry.

Commentary: *all semi-integral and integral abutment bridges should have approach slabs, though they may not be applicable for short simple span standard prestressed box girder bridges or where push-through deck systems are utilized.*

1.7.3 Utilities on bridges

1.7.3.1 General

Add the following:

The Ministry "Utility Policy Manual" shall apply regarding installation of utilities on or near bridges.

Commentary: *The Ministry's Utility Policy Manual can be found at the following link:*

https://www2.gov.bc.ca/assets/gov/driving-and-transportation/funding-engagement-permits/highway-permits/utility_policy_manual.pdf

1.7.3.2 Location and attachment

Add the following paragraphs:

Conduits for utilities shall not be placed in deck slabs less than 250 mm thick.

No more than two utility conduits shall be located within a concrete barrier and the nominal inside diameter of any such conduit shall not be more than 50 mm. Conduits shall be located vertically above one another with a minimum of 50 mm clearance between adjacent conduits. The bottom conduit shall be located so that there will be at least 50 mm of clearance for fresh concrete to flow under the conduit when the concrete barrier is cast. Conduits should be located towards the center of the barriers to maximize clearance to barrier reinforcing.

***Commentary:** Concrete bridge and combination barriers can serve as a convenient location for running electrical conduit over the bridge length. The size and number of conduits should be limited such that their presence does not have an adverse effect on the crash performance of the barrier or the ability to place and consolidate concrete. The conduit(s) should be located at the base of the barrier, within the rebar cage. The junction boxes to service the conduit should be located in the rear (non-impact) face of the barrier. If additional conduits are required consideration should be given to separate embedded conduits located in thickened deck sections or exposed conduits suspended under the bridge.*

1.8 Durability and maintenance

1.8.2 Bridge deck drainage

1.8.2.1 General

Add the following:

Runoff water from the surface of bridges and/or approach roads shall be conveyed to discharge at locations that are acceptable to environmental agencies and the Ministry.

When deck inlets are required, they shall use air drop discharge unless otherwise directed by environmental agencies. Water shall not be discharged onto railway property, pavements, sidewalks or unprotected slopes. Discharge into rivers and creeks require approval by the appropriate environmental regulatory agency.

1.8.2.2 Deck surface

1.8.2.2.1 Crossfall and grades

Delete the first paragraph and replace with the following:

Bridge deck drainage of the roadway shall be achieved by providing a minimum 2% transverse crossfall and by providing a minimum longitudinal grade of 1%, except where, for limited lengths, vertical curves or superelevation transitions preclude this. In cases where there is extreme topographical hardship, the absolute minimum longitudinal grade may be reduced to 0.5% with the Consent of the Ministry.

All sidewalks, safety curbs, tops of barriers, raised medians, or other deck surfaces that are raised above the roadway, and are wider than 300 mm, shall have a minimum transverse crossfall of 2% to direct surface runoff away from median longitudinal expansion joints. Deck runoff from sidewalks can be directed to the outside of the bridge, subject to approvals from the regulatory environmental agencies.

Commentary: *For long term durability, it is preferable to control all drainage and direct it to deck drains. Directing drainage over the fascia can lead to freeze-thaw durability problems in colder climates.*

1.8.2.2.2 Deck finish

Add the following:

Concrete bridge decks shall be textured by tining in accordance with SS 413.31.02.05. Concrete bridge decks which are to receive a waterproof membrane and asphalt topping shall be given a smooth float finish. Sidewalks shall receive a transverse broom finish.

1.8.2.3 Drainage systems

1.8.2.3.1 General

Delete the first sentence of the first paragraph and replace with the following:

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 will not encroach more than 1.20 m onto the traffic lane.

1.8.2.3.2 Deck drain inlets

Add the following:

Future settlement shall be considered when locating deck drain inlets.

1.8.2.3.3 Downspouts and downpipes

Add the following paragraph:

Scuppers shall not be used unless Consented to by the Ministry.

Commentary: *Improper detailing of scuppers leads to extensive maintenance problems. Use of metal inserts has given rise to corrosion and delamination of the concrete curbing. Large openings can present a hazard due to snagging of a vehicle's wheel during impact. Note that some fabricators use the term scupper for catch basin type systems similar to those in SS582 which do not require Consent.*

Add the following to the first sentence in the second paragraph:

The portion of the drainage system extending through the deck and the downspout shall be hot dip galvanized steel pipe.

Delete the last sentence in the fourth paragraph and replace with the following:

Downspouts shall project a minimum of 500 mm below any adjacent component, except where prohibited by minimum vertical clearances.

Support brackets shall be considered for deep girders and steel trusses.

Add the following:

Erosion protection shall be provided at discharge areas from downpipes and downspouts similar to the splash pad detail shown on SP504-03, constructed of 10 kg class riprap.

Commentary: Energy dissipators may be required to mitigate erosion due to high velocity discharges.

Typical drain inlet and downspout details are shown in Figures 1.8.2.3.3a and 1.8.2.3.3b.

Figure 1.8.2.3.3a
Deck drain setting detail

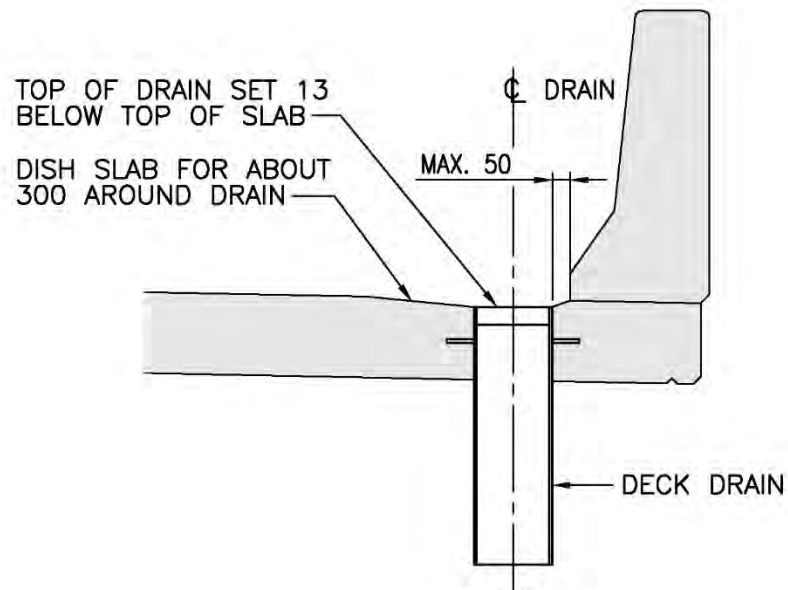
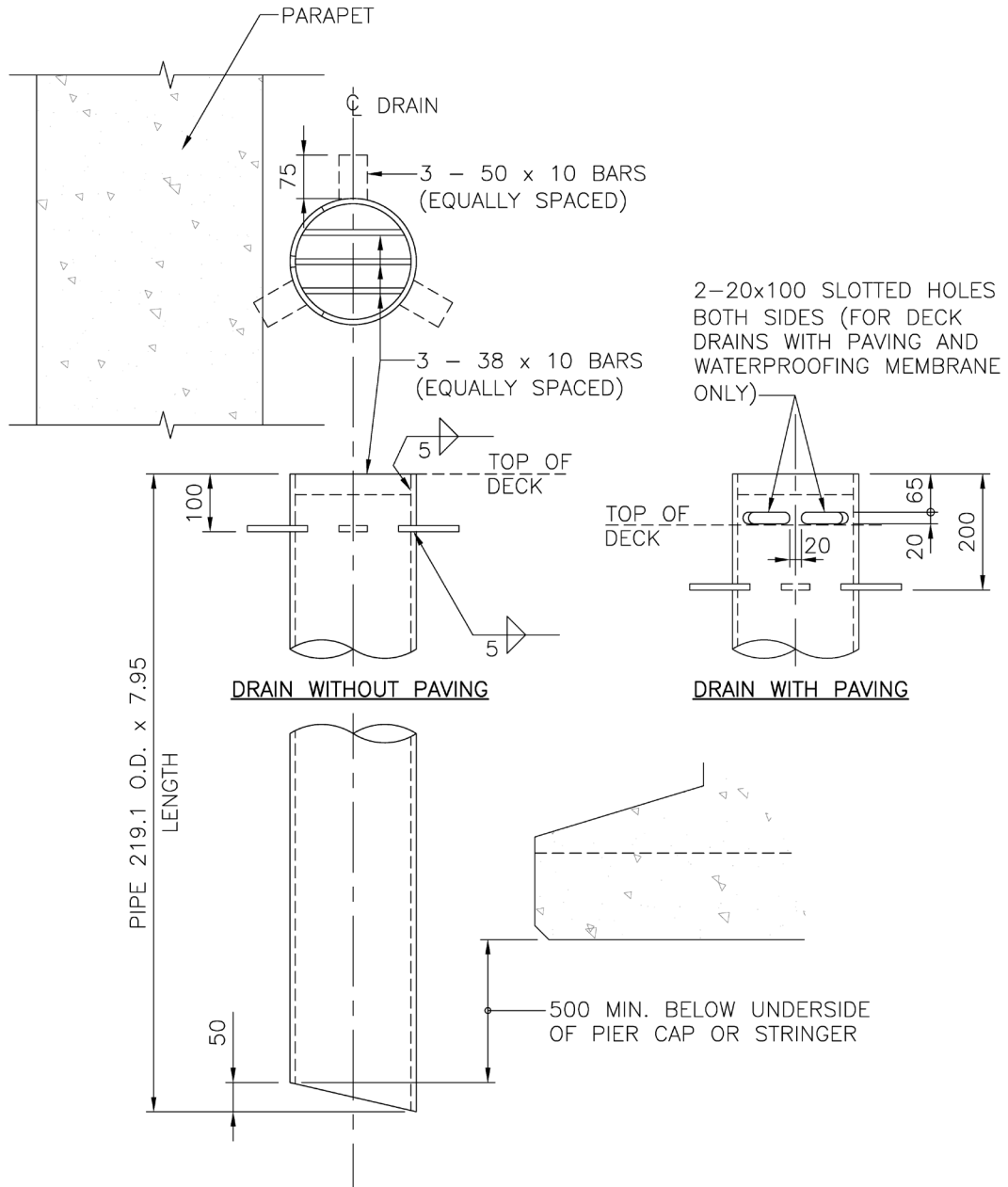


Figure 1.8.2.3.3b
Deck drain fabrication detail



1.8.2.5 Runoff and discharge from deck

Add the following paragraph:

If catch basins are required just beyond the limits of the structure, a continuous length of barrier or curb and gutter shall be provided to connect the bridge curb or barrier to the catch basin to prevent washout of the fill at the ends of the wingwalls.

1.8.3 Maintenance

1.8.3.1 Inspection and maintenance access

1.8.3.1.1 General

Add the following paragraphs:

Permanent equipment access to the stream bed level shall be provided in the design to enable future removal of debris build up at the inlet of buried structures and culverts at locations where the height from the roadway surface to the stream bed level exceeds 5 metres.

The following minimum clearances shall be maintained between the top of the finished fill in front of the abutment and the underside of the superstructure to facilitate inspection:

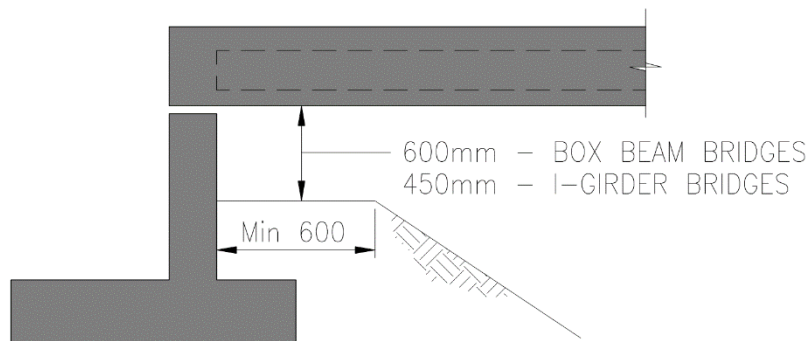
I-Girder Bridges (Steel or Prestressed Concrete) 450 mm

Box Beam Bridges 600 mm

These clearances are applicable for isolated zones of the structure. Where extensive areas of the bridge have clearance restrictions the minimum clearance shall be as Consented to by the Ministry under a site specific access plan.

A minimum 600 mm wide horizontal bench shall be provided as shown in Figure 1.8.3.1 to facilitate inspection and maintenance access unless otherwise Consented to by the Ministry.

**Figure 1.8.3.1
Abutment berm detail**



1.8.3.1.2 Removal of formwork

Add the following sentence to the end of the first paragraph:

All other formwork shall be removed.

Add the following paragraph:

Partial depth precast panels acting compositely with the concrete deck shall not be considered as formwork. Foam supports for the partial depth precast panels may be left only if the panels are not prestressed.

Commentary: *AASHTO Bridge Design Specification Clause C9.7.4.3.2 and C9.7.3.4 address the issues of the reflective cracking in decks caused by prestressing in partially prestressed concrete panels due to concrete creep and shrinkage.*

1.8.3.1.3 Superstructure accessibility

Commentary: *Access to steel girders for inspection purposes shall be considered in the design in high traffic volume areas in consultation with the Ministry. Designs for inspection access shall be in accordance with Work Safe BC Occupational Health and Safety Regulations (OHS).*

1.8.3.1.5 Access to primary component voids

Add the following to the end of the second paragraph:

Drains into voids in bridge girders shall be screened so that the larger mesh opening dimension does not exceed 15 mm.

1.8.3.3 Bearing maintenance and jacking

Delete and replace the third paragraph with the following:

In the design of jack-bearing locations, the assumed factored jacking force shall be the greater of twice the unfactored dead load or the sum of the factored dead load and full factored live load.

Sufficient vertical and horizontal space shall be provided between the superstructure and the substructure to accommodate the jacks required for bearing replacement. For steel girders the web stiffeners of the end diaphragm must be located accordingly.

Commentary: *A minimum vertical clearance of 150 mm is suggested for typical short span bridges (<20 m), though may be omitted for standard prestressed concrete box girders. Larger spans should be assessed for the size of jack required.*

1.9 Hydraulic design

1.9.1 Design criteria

1.9.1.1 General

Delete and replace the first paragraph with the following:

The hydraulic design of bridges, buried structures, culverts and associated works shall comply with the requirements of the TAC Guide to Bridge Hydraulics, (latest edition) and Part 3 of the Water Sustainability Act (WSA) Regulation.

At a minimum, for buried structures and culverts:

- headwater depth to diameter ratio (HW/D) shall not exceed 1.0 at the design flow.
- outlet control headloss shall be less than 0.3 m under design (flow and tailwater) conditions.

Add the following:

Wave effects for structures in the proximity of lakes and coastal bodies of water may require special consideration.

1.9.1.2 Normal design flood

Delete the first paragraph and replace with the following:

The design flood shall be the peak instantaneous flow including incorporation of climate change resilience in design, with the return periods as follows:

Bridges, retaining walls and river training & channel control works	200-year
Buried structures and culverts \geq 3 m span	200-year
Low-Volume Road – bridges, buried structures, culverts, retaining walls and river training & channel control works	100-year
Temporary Structures (2 years or less in duration)	10-year*

*Calculation of an alternative return period based on an equivalent risk for the duration of detour may be accepted when Consented to by the Ministry for durations greater than 2 years. Climate change does not need to be addressed, but shall be documented on the Design Criteria Sheet for Climate Adaptation.

In accordance with the WSA Regulation the 1 in 200 year maximum daily flow or the hydraulic capacity of the stream channel shall be shown on the Plans as required for environmental approvals.

The design shall also meet the requirements of the regulatory agencies.

Commentary: Floodplain maps are available for a number of locations throughout the Province and show the areas affected by the 200-year flood. The maps are generally drawn to a scale of 1:5,000 with 1 metre contour intervals and show the natural and man-made features of the area. In some cases these maps may be outdated and therefore only suitable for conceptual or early stages of design. The design flood level shall be re-evaluated for the site, accounting for present-day conditions, during detailed design.

For information on maps refer to:

<https://www2.gov.bc.ca/gov/content/environment/air-land-water/water/drought-flooding-dikes-dams/integrated-flood-hazard-management/flood-hazard-land-use-management/floodplain-mapping>

Where fish and fish habitat are involved, additional measures may be necessary to meet the requirements of the regulatory agencies.

For buried structures and culverts, consideration should be given to increasing the size and durability of the structure and/or providing additional measures (e.g. bypass culverts) to ensure maintainability (as per Clause 1.8.3.2) given the high cost of replacement, maintenance and renewal. Consideration should include such items as:

- Traffic volumes,
- Depth of cover
- Detour and alternate route availability
- Required maintenance frequency and maintenance accessibility
- Hydrotechnical issues

1.9.1.3 Check flood

Delete and replace with:

Consideration of a check flood is not required for Ministry structures.

1.9.1.5 Design flood discharge

Delete and replace the paragraph with the following:

The design floods shall be estimated by the following methods, unless otherwise Consented to by the Ministry.

- a) For drainage areas greater than 10 km², the recommended design flow calculation methods are:
 - Station Frequency Analysis
 - Regional Frequency Analysis

Commentary:

Annual maximum daily and peak instantaneous flows are available from Water Survey of Canada (WSC) gauging stations.

For information on Frequency Analysis, refer to: TAC Guide to Bridge Hydraulics, Section 3.2 (June 2001)

- b) For urban and small drainage areas less than 10 km², the recommended design flow calculation is the Rational Method.

If the drainage area is close to the upper limit, the designer shall check the results using other methods (e.g. measured flow data, Soil Conservation Service (SCS) Unit Hydrograph Method, regional frequency analysis, etc.) and confirm with an assessment of channel forming flow and channel flow capacity (by site observation, regime analysis etc.).

Commentary: For information on the Rational Formula Method, refer to the TAC Guide to Bridge Hydraulics, Section 3.4.1 (June 2001) and the BC Ministry of Transportation, Supplement to TAC Geometric Design Guide, (April 2019).

1.9.4.1 Scour calculations

Add the following:

The scour shall be calculated using methods as described in the TAC Guide to Bridge Hydraulics or another method Consented to by the Ministry.

1.9.5 Protection against scour

1.9.5.1 General

Delete and replace with the following:

Scour protection requirements for structure foundations shall be such that structural failure will not occur as a result of the design flood.

1.9.5.2 Spread footings

Add the following:

Abutments and piers subject to potential scour on Low Volume Road Bridges shall have piled foundations or be adequately protected from scour in accordance with Clauses 1.9.5.2.1 and 1.9.5.2.2 unless otherwise Approved.

Abutments and piers subject to potential scour on non-Low Volume Road Bridges, shall have piled foundations unless otherwise Approved.

Spread footings used for abutments and piers subject to potential scour shall have protective aprons.

Commentary: Use of spread footings for abutments and piers where Clauses 1.9.5.2.1 and 1.9.5.2.2 are not met, may be considered acceptable on low-volume roads or in other special circumstances provided a Design Exception, is approved.

The Design Exception in this situation entails a documented design rationale by the Coordinating Professional Engineer with input from the hydrotechnical engineer or geoscientist, structural engineer and geotechnical engineer to determine the hydrotechnical risks the spread footing is operating under and how those risks are mitigated. This assessment shall address site specific features including but not limited to abutment location, geotechnical conditions, stream morphology, natural channel

characteristics, sediment and bank material, debris risk, use and function of the road, alternate routes and any other applicable factors.

1.9.5.2.2 Protection of spread footings

Add the following paragraph:

Riprap and/or MSE walls shall not be used as means to justify spread footings founded at a depth less than specified in Clause 1.9.5.2.1 unless Approved.

Commentary: *The use of riprap may be considered for protecting spread footings located adjacent to the stream on low-volume road bridges.*

Riprap installations are not equivalent to piling, sheet piling, concrete or steel inverts, or concrete revetment. Protection of the spread footings must remain effective for the design life of the structure and provide stability to the structure foundation with the streambed at its ultimate elevation. Riprap is inherently prone to damage during floods. Performance can also be significantly affected by issues such as rock quality, weathering, installation details and maintenance. The Design Exception described in BC Supplement commentary 1.9.5.2 shall address the effective protection of spread footings against scour.

Spread footings some distance from the channel and founded on erodible material at an elevation higher than the streambed are vulnerable to failure from scour (see S6:19 commentary). Placing spread footings at streambed level or lower makes them less vulnerable.

1.9.5.3.2 Penetration and strength

Commentary: *Consideration of estimated scour depth and determination of the final bed elevation at the location of the piles should be assessed on a site specific basis as a collaboration between hydrotechnical, geotechnical and structural professionals. Ultimate bed elevation will differ across the channel, horizontal projection of the lowest ultimate bed elevation to all foundation elements may lead to excessively conservative pile designs.*

1.9.5.3.3 Abutments supported on piles

Replace the first sentence with:

The bottom elevation of an abutment designed to be directly exposed to flowing water at the design flood shall be set at least 1.0 m below the ultimate stream bed elevation.

Commentary: *For abutments not directly exposed to water, it is not necessary that the abutment fills be prevented from washing out.*

1.9.5.5 Protective aprons

Replace the second paragraph with the following:

Riprap stone sizes for pier aprons shall be determined using a velocity at least 1.5 times the estimated pier approach velocity at the design flow. The thickness of riprap aprons shall be not less than 1.5 times the median size of the stone

Add the following paragraph:

Riprap should conform to SS 205, unless otherwise Consented to by the Ministry.

1.9.6 Backwater

1.9.6.1 General

Delete the first sentence and replace with:

Backwater shall be calculated using methods Consented to by the Ministry. HEC-RAS numerical analysis may be used for determining the backwater profile.

1.9.7 Soffit elevation

1.9.7.1 Clearance

Delete and replace the first paragraph with the following:

Unless otherwise Approved, the clearance between the soffit and the Q200 design flood elevation shall not be less than 1.5 m for bridges; and not less than 0.5 m on low-volume road bridges for the Q100 flood elevation. For buried structures and culverts greater than or equal to 3 m span, the clearances shall be adequate to pass the anticipated ice flows and floating debris as well as accommodating sediment bed load at the site for the Q200 design flood or for the Q100 design flood on low-volume roads. For temporary structures the clearance shall be as indicated previously except using the Q10 (or alternative return period) flood elevation defined.

Both vertical and horizontal clearances shall be addressed. Increased clearance should be considered for crossings subject to ice flows, debris, debris flows and debris torrents.

Commentary: *Specific risk assessments may be required for debris torrents/flows if the required clearances become excessively large. For navigable waters, the Canadian Navigable Waters Act requires a vertical clearance that allows passage of the largest air draft vessel at the 100-year flood level or the HHWLT (Higher High Water, Large Tide). This allowance also includes a calculation of maximum wave height. For small watercourses capable of carrying only canoes, kayaks and other small craft a clearance of 1.7 m above the 100-year flood level is usually considered to be adequate. For small watercourses less clearance may be considered if cost and road design factors are affected significantly. Transport Canada, having authority of works over or in Navigable Waters, can require other clearance requirements. For minor waterways, the Ministry is to carry out a navigational assessment and determine the requirements for design and navigation. Vessel Surveys and studies may also be required to determine clearance requirements and navigable areas and channel(s) within the waterway. Applications and communications with the Transport Canada and Port Authorities shall be coordinated by the Ministry's Rail, Navigable Waters Specialist.*

For additional information, refer to Ministry's Bridge Standard and Procedures Manual, Volume 2 – Procedures and Directions, Section 5.0 Regulatory Submission Requirements.

1.9.9 Channel erosion control

1.9.9.3 Slope revetments

Add the following paragraphs:

Riprap revetments shall be used for protecting the bank slopes and bridge end fills of abutments, in conformance with SS 205 unless otherwise Consented to by the Ministry. The riprap toe shall be designed to prevent undermining of revetments in accordance with the TAC Guide to Bridge Hydraulics. At minimum, the revetment shall protect the bridge end fills, and the upstream and downstream ends shall be keyed into the bank slopes.

Commentary: *Top of slope revetment should be placed a minimum of 0.3 m above the design high-water level, and preferably 0.6 m. The dimension 0.3 m is consistent with the guidelines set by BC FLNRO for the riprap design and construction for dikes - Riprap Design and Construction Guide. For LVR structures this may be revised when Consented to by the Ministry.*

1.9.11 Culverts

1.9.11.2 Culvert end treatment

Add the following:

Cut-off walls shall be used at both ends of the culvert where there is a possibility of uplift, piping or undermining, unless otherwise Consented to by the Ministry. Cut-off walls shall extend 1 m below the culvert invert or to the estimated depth of scour, whichever is greatest, or as otherwise Consented to by the Ministry.

Commentary: *This will alleviate failure of culverts from uplift and piping during extreme flood events which has occurred at some Ministry sites.*

1.9.11.6 Closed-invert culverts

1.9.11.6.6 Soil-steel structures

Add the following:

Cut-off walls shall be used at both ends for closed-bottom type soil-metal structures where there is a possibility of uplift, piping or undermining. Collars or headwalls are required where there is a possibility of deformation due to debris or ice impact, or hydrostatic forces.

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2.3 Design for durability

2.3.5 Materials

2.3.5.5 Backfill material

Add the following paragraphs and Table:

Backfill for structures shall be Bridge End Fill (BEF) meeting the material, placement and compaction requirements of SS 201.40. In addition to SS 202.04.02, where Bridge End Fill is used for MSE Wall structural fill, primary quality testing shall also include all additional testing as required to confirm that the material meets the electrochemical criteria for the wall system. Bridge End Fill shall extend a minimum of 8 m (perpendicular) from the abutment for the full width of the roadway for bridges, as recommended by the designers for conventional retaining walls and for the full reinforced soil area of MSE walls unless otherwise Consented to by the Ministry.

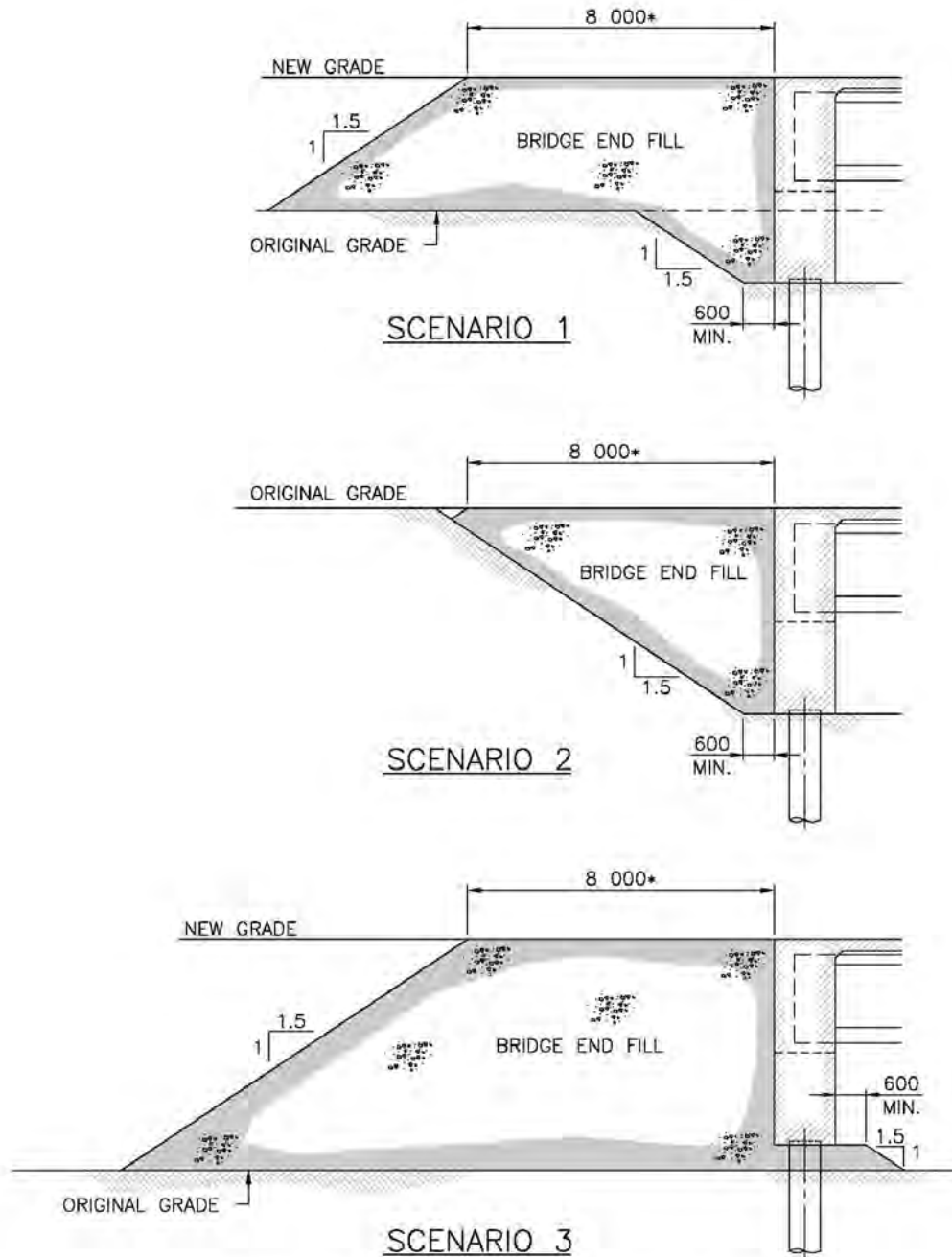
An aggregate drainage course shall be provided along the backside of all foundation and retaining walls located in cut areas with positive drainage.

The gradation of drainage course aggregate shall be as follows:

Sieve Size (mm)	Passing Per Nominal Maximum Size
40	100
20	0 - 100
10	0

Commentary: Typical BEF envelopes are shown in the sketches below in Figure 2.3.5.5. An envelope shorter than 8 m is generally acceptable on LVR structures and for shorter simple span structures. In some situations, the use of supplementary dimpled drainage board may be warranted to facilitate rapid drainage to the drain course or to protect the membrane between superstructure and substructure. When considered adjustments to the BEF envelope, the designer should consider: the length and details of an approach slab (if used), construction methodology, deck finishing requirements and equipment set up, approach fill settlement, drainage etc.

Figure 2.3.5.5
Bridge End Fill Envelopes



* DISTANCE MAY BE REDUCED FOR LOW VOLUME ROAD BRIDGES OR FOR SHORT SPAN BRIDGES.

BRIDGE END FILL EXAMPLES

2.3.5.6 Other materials

Add the following paragraph:

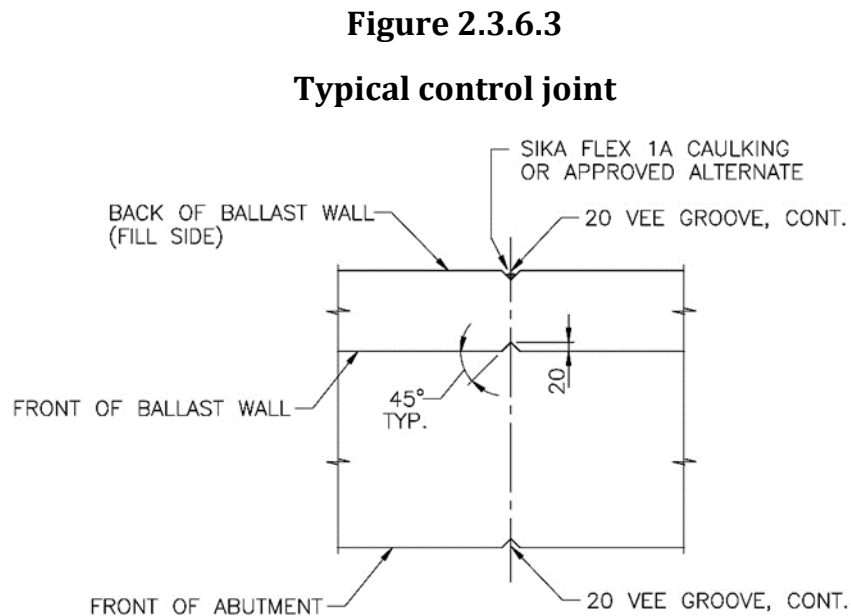
An acceptable pre-molded joint filler for structures consists of a minimum 25 thick Evazote 50 or alternate as Consented to by the Ministry. Application shall be in accordance with the manufacturer's instructions.

2.3.6 Structural details

2.3.6.3 Joints in abutments, retaining walls, and buried structures

Add the following and Figure 2.3.6.3 after the first paragraph:

Typical details for concrete control joints are shown in Figure 2.3.6.3.



NOTES:

1. ABUTMENT/BALLAST WALL SHOWN OTHER WALLS SIMILAR.
2. MAXIMUM SPACING OF CONTROL JOINTS = 3.0m
3. JOINTS TO BE LOCATED AT HORIZONTAL DRAINS THRU WALL AND AT ABRUPT ABUTMENT OR WALL SECTION CHANGES. INTERMEDIATE JOINTS TO BE LOCATED TO MEET MAX. SPACING OF 3.0m
4. CONTROL JOINTS (AND HORIZONTAL DRAINS) ARE TO BE LOCATED TO AVOID BEARING SEATS.
5. LOCATIONS OF CONTROL JOINTS ARE TO BE SHOWN ON ABUTMENT OR WALL ELEVATION.

2.3.6.4 Expansion and/or fixed joints in decks

Delete the first sentence and replace with:

Wherever practical, expansion and/or fixed joints in decks shall be avoided or placed in the approach pavements. Where expansion joints cannot be avoided, and are Consented to by the Ministry, they shall be detailed to prevent damage to components of the structure from water, de-icing salts, chemicals, and roadway debris. Joints shall be designed such that they can be easily accessed for flushing, maintenance, inspection, seal replacement and repair.

Commentary: *Joint seals shall be assessed for serviceability throughout the full temperature range at the site.*

2.3.6.5 Drainage

Amend the second sentence in the second paragraph as follows:

Downspouts shall extend a minimum of 500 mm below adjacent members, except where prohibited by vertical clearance requirements.

2.3.6.9 Waterproofing membranes

Add the following paragraphs after the first paragraph:

Requirements for membranes are identified in Section 8.

The Ministry's Recognized Products List shall be used as a reference to identify potential products for bridge deck waterproofing systems which are accepted by the Ministry. The link is as follows:

<https://www2.gov.bc.ca/gov/content/transportation/transportation-infrastructure/engineering-standards-guidelines/recognized-products-list>

2.3.6.10 Soil and rock anchors

Replace the second sentence with :

Unless otherwise Consented to by the Ministry, soil and rock anchors shall be a PTI - Class 1, Double Corrosion Protection (DCP) system.

Commentary: *Rock bolts other than DCP may be used for the purposes of rock slope design per SS and DBSS 206 and T- 04/17 Geotechnical Design Criteria when not integral to the stability of a structure:*

2.3.6.13 Utilities

Add the following paragraph:

The Ministry's "Utility Policy Manual" shall be followed for procedures and guidelines regarding the installation of utilities on or near bridges.

Commentary: The Ministry's Utility Policy Manual can be found at the following link:

https://www2.gov.bc.ca/assets/gov/driving-and-transportation/funding-engagement-permits/highway-permits/utility_policy_manual.pdf

2.8 Climate and exposure considerations

Add the following paragraph:

The designer shall follow the requirements of Technical Circular T-04/19 - Resilient Infrastructure Engineering Design - Adaptation to the Impacts of Climate Change and Weather Extremes.

Commentary: Guidance can also be found in the Engineers and Geoscientists British Columbia Professional Practice Guidelines, Civil and Transportation Infrastructure - Developing Climate Change-Resilient Designs for Highway Infrastructure in British Columbia, at the following link:

<https://www.egbc.ca/getmedia/b60921fc-a820-41be-868f-02f0d3d92892/EGBC-BCMOTI-Climate-Resilient-Design-Highway-V2-0.pdf.aspx>

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3.2 Definitions

Add the following definition:

Supervision – monitoring of the passage of a permit vehicle by a BC registered professional engineer familiar with bridge design or their designate to ensure bridge crossing restrictions of a permit are followed including weighing of a permit vehicle if required by the overload permit. The engineer shall have the authority to stop further movement of the permit vehicle if in non-compliance the bridge crossing requirements of a permit. The engineer shall submit a record detailing their observations to the Ministry upon completion of the move, including records of vehicle and axle weight, dimension measurements and any other requirements specified in the bridge crossing conditions of the permit.

3.5 Load factors and load combinations

3.5.1 General

When special load vehicle lanes are mixed with normal traffic loaded lanes, each lane will be assigned its corresponding different live load factor based on the traffic in the lane. For example, a special load vehicle lane will get a special load live load factor and the other lanes will get normal traffic live load factors.

Add the following to Table 3.1 Load factors and load combinations:

	Permanent Loads			Transitory Loads				Exceptional Loads				
Loads	<i>D</i>	<i>E</i>	<i>P</i>	<i>L*</i>	<i>K</i>	<i>W</i>	<i>V</i>	<i>S</i>	<i>EQ</i>	<i>F</i>	<i>A</i>	<i>H</i>
Ultimate Limit States‡												
ULS Combination 5A	α_D	α_E	α_P	λ	0	0	0	0	1.00	0	0	0

λ shall be equal to 0.30 unless otherwise Consented to by the Ministry.

ULS Combinations 5 and 5A shall only be used for force-based design (FBD) when FBD is permitted in accordance with Table 4.11.

Effects of live load on bridge inertia mass shall not be included in dynamic analysis.

Commentary: *If a vertical design spectrum is considered explicitly in a site-specific study, the load factor for dead load, α_D , shall be taken as 1.0 in ULS Combination 5 and 5A.*

Add the following two columns to: Table 3.2 Live load factors ultimate limit states:

Live load factor		
Special loads travelling on bridges without Supervision meeting crossing restrictions in Clause 3.8.3.2.3		
Load	Short spans	Other Spans
ULS Combination 1	1.70	1.50
ULS Combination 2	1.60	1.40
ULS Combination 3	1.40	1.25

Commentary: These load factors are consistent with the PS load factor approach in Section 14.

Calibration of load factors and resistance factors in Table 3.2 of S6:19 and the Ministry supplement to CHBDC are based on a minimum annual reliability index of 3.75 for traffic loading, including special load vehicles with no travel restriction or supervision, and 3.50 special load vehicles travelling alone on a bridge under supervision in accordance with Clause 3.8.3.

3.6 Dead loads

Add the following paragraphs:

Dead loads shall include an allowance for an additional 50 mm concrete overlay over the full area of the bridge deck to account for future deck rehabilitation and also to partially account for any unanticipated dead loads that may be added to the structure following construction.

For bridges with waterproof membrane and asphalt overlay on a concrete deck, the dead load for design shall include the design asphalt thickness of 100 mm of asphalt (see Section 2.7), and no allowance for future additional overlay thickness is required.

Add the following to **Table 3.4 Unit material weights**:

Material	Unit Weight, kN/m³
Wood	
Untreated Douglas Fir	5.4
Creosote treated sawn timber and glulam, >114 mm	6.6
Creosote treated truss chords, < 114 mm	7.0

3.8 Live loads

3.8.3 Traffic Loads

3.8.3.1 Normal traffic

3.8.3.1.1 CL-W loading

Delete the third paragraph and replace with the following:

A loading of CL-800 shall be used for the design of bridges under the authority of the Ministry.

3.8.3.1.2 CL-W Truck

Commentary: The Ministry has increased the weight of the design vehicle from BCL-625 to CL-800 as there has been a trend towards heavier term permit overload vehicles approaching 80T GVWs, which must travel with no bridge crossing restrictions or conditions. Single trip overload weights have also been increasing so this will help to have fewer bridge crossing restrictions/conditions and permit denials.

3.8.3.2 Special loads

Add the following clauses:

3.8.3.2.3 Geographically Specific Special Loads

In addition to CL-800 loading, structures located in the specific geographic regions indicated below shall also be designed for the indicated special loads. A refined method of analysis shall be used to distribute live loads. Analysis and dynamic load allowance shall be based on the crossing restrictions indicated. Axle spacings and weights for Special Trucks EPLL1 and EPLL2 are shown in Figures 3.8.3.2.3 i, ii and iii. Special Lane load shall be considered for EPLL1 loading only.

The Plans shall show the design vehicle diagrams, design crossing restrictions, and the ULS live load factors used for the Special Loads.

3.8.3.2.3.1 Special Load EPLL1

EPLL1 shall have the following crossing restrictions:

EPLL1 loading shall be placed in one lane and allowed to travel mixed with normal traffic. Both truck and lane loading shall be considered.

EPLL1 shall apply in the following specific geographic regions:

Sparwood Area

Hwy 3 between the BC/AB border and the south entrance to Douglas Fir Road in Sparwood, Highway 43, Corbin Road and Fording River Road.

Peace District

- H97 from Prince George to Hasler
- H29N from Chetwynd to Hudsons Hope
- Chowadee Rd #187U
- Cypress Cr Rd #187
- Graham R Rd #123
- Upper Halfway Rd #117
- Fort Nelson Airport Connector
- Fort Nelson Airport Drive
- Rolla Rd #3 south from Rd #222
- Peace River Sweetwater Rd#6 from Rolla Road Rd#3 to Highway H97
- Braden Rd #22
- Jackfish Lake Rd #12
- Rd #137
- Rd #101
- Rd #146
- Rd #146 east
- Beaton Montney #271
- Montney Hwy #114
- Becker #285W
- Prespatou Rd #193
- Buick Cr Rd #154
- Mile 30 Rd #169
- Triad Rd # 169A
- Rosefield Rd #142
- Doig Rd #188
- Siphon Cr Rd #184

3.8.3.2.3.2 Special Load EPLL2

EPLL2 shall have the following crossing restrictions:

- Centerline of the Special Load to remain within 600 mm of the centerline of the available bridge roadway between barriers in the direction of travel of the EPLL2 vehicle.
- For undivided bridge roadways - No other vehicles on the bridge while the Special Load crosses
- For divided bridge roadways - No other vehicles on the bridge travelling in the same direction of the EPLL2 vehicle and with normal traffic allowed on the other side of the barrier(s),
- Crossing speed to be less than 10 km/h
- Travelling on bridge without supervision

EPLL2 shall apply in the following specific geographic regions:

Peace District

- Highway 2 from the BC/Alberta border to the junction with Dangerous Goods Route
- Highway 52

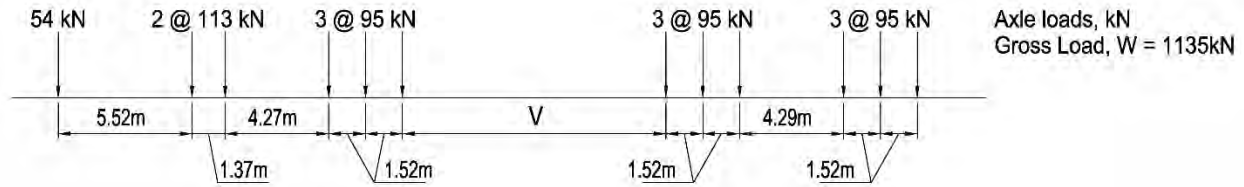
- Highway 29S from Chetwynd to Highway 52
- Highway 97 from Hasler north to Mile 83.5 on the Alaska Highway/Highway 97
- Highway 49
- Highway 29N from Charlie Lake to Canyon Dr #520R
- Highway 77
- Dangerous Goods Route
- Rd #259 (Fort St John Underpass Bypass)
- Rd 22 / Braden Rd
- Rolla Rd # 3 between Highway 2 and Rd #222
- Rd #148
- Rd#269
- Cecil Lake Rd #103
- Beatton River Airport Rd #151
- Beryl Prairie Rd #118
- Beryl Prairie Arterial Rd #715R
- Darrel Cr Rd #115
- Canyon Dr #520 from Highway 29 to Rd 715R

Other Districts

- Highway 23 between Shelter Bay and the Mica Dam
- Highway 1 between the north and south sections of Highway 23.
- Highway 22 between the BC/US border at Paterson and Highway 3B near Rossland
- Highway 3B between Highway 3 near Nancy Greene Provincial Park and Highway 22A at Waneta Junction
- Highway 3 between Highway 3B near Nancy Greene Provincial Park and the Ootischenia Interchange
- Highway 22 between Castlegar and Trail
- Highway 22A between Highway 3B at Waneta Junction and the BC/US border
- Highway 3A between the Ootischenia Interchange and Blewett Road
- Broadwater Road in Castlegar between the Keenleyside Dam and Highway 3A
- Highway 97 between Highway 39 (near the Parsnip River Bridge No. 1185) and the Old Caribou Highway (south of Prince George).

Figure 3.8.3.2.3 i

EPLL1

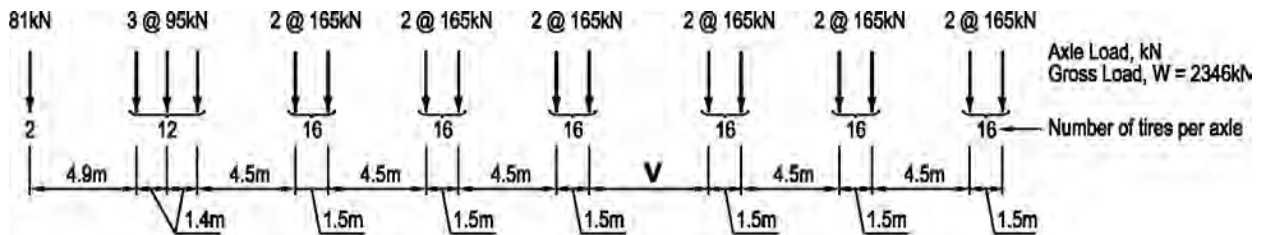


V = Variable Spacing = 10m to 16m. Spacing to be used is that which produces the maximum load effect

Transverse wheel spacings and the clearance envelope for EPLL1 truck load shall be similar to those indicated for the CL-W truck in Figure 3.2 of CHBDC.

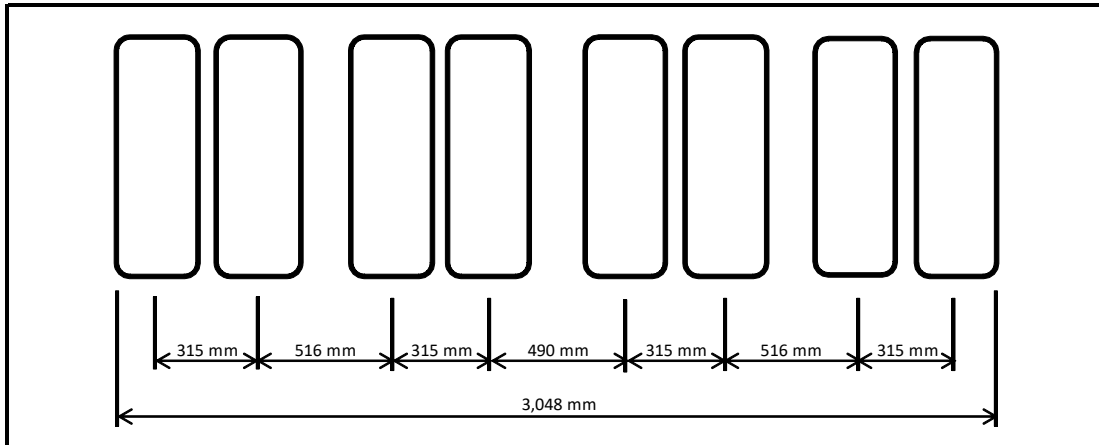
Figure 3.8.3.2.3 ii

EPLL2



V = Variable Spacing - 4.5m to 18m inclusive. Spacing to be used is that which produces the maximum load effect.

For the EPLL2 truck, transverse wheel spacings for 16 tire groups (8 tire axles) shall be as indicated in Figure 3.8.3.2.3 iii. Transverse wheel spacings for 2 and 12 tire axles shall be similar to those indicated for the CL-W truck in Figure 3.2 of CHBDC. The clearance envelope for the EPLL2 truck shall be assumed to extend 0.3 m on each side beyond the out-to-out width of tires shown in Figure 3.8.3.2.3 iii.

Figure 3.8.3.2.3 iii**Transverse Wheel Spacings for 16 Wheel Tandem Group of EPLL2**

Commentary: The extraordinary vehicle configurations described in this section are based on overload evaluation requests in different geographic regions and anticipated future demands. The oil and gas industry is prevalent throughout the Peace District. Compressors, pipe rack modules and drilling equipment frequently need to be hauled in and out of remote locations within the District to and from Alberta. Future supply and servicing of this industry from Prince George is contemplated and therefore full length of the John Hart Highway is included in this geographic region. Maintenance and upgrading of existing, and construction of new hydro power facilities on the Peace, Columbia and Kootenay Rivers requires the transport of turbine runners and transformers. Several coal mines are found in the area around Sparwood. Bridges in this area have been designed or load rated for EPLL1 loading to allow for the transport of mining equipment between different mining operations.

3.8.4 Application

3.8.4.1 General

Revise (c) to the following:

For the FLS, the traffic load shall be one CL-800 Truck that causes maximum effects only, increased by the dynamic load allowance and placed at the centre of one travelled lane. The Lane Load shall not be considered.

For the SLS Combination 2, the traffic load shall be one CL-800 Truck or the Special Truck that causes maximum effects only, increased by the dynamic load allowance and placed at the centre of one travelled lane. The Lane Load shall not be considered.

Commentary: Special load vehicles are rare compared to other live loads and therefore fatigue design for special load vehicles is not required.

Add the following at the end of this clause:

(c) Design shall address both the Special Truck and Special Lane loading for special load EPLL1. Design for the EPLL2 special load need only address the Special Truck loading since there is no Special Lane loading for EPLL2. The design lane(s) that the EPLL1 and EPLL2 special load occupies and other lanes that are loaded shall be selected to maximize the load effect. The normal traffic in other loaded lanes shall address both truck and lane loading.

3.8.4.3 Local components

Add the following:

For local components, the axle loads of a CL-625 truck shall be used.

Commentary: The design vehicle has been increased to CL-800 to capture the global demands of permit trucks; however no increases are anticipated for allowable axle loads. Increasing the design loading for local components would lead to unnecessary conservatism.

3.8.4.4 Wheels on the sidewalk

Add the following:

For wheels on the sidewalk, a CL-625 truck shall be used.

3.8.4.5 Dynamic load allowance

3.8.4.5.3 Components other than buried structures

Delete the last paragraph and replace with the following:

The dynamic load allowance given in Items (a) to (d) may be reduced by applying the modification factors from Clause 14.9.3 for a Special Truck travelling at reduced speed.

3.16 Construction load and loads on temporary structures

3.16.1 General

Insert the following paragraphs:

It shall be the responsibility of the Contractor to ensure that loads developed as a result of the construction methods can be properly carried unless a specific construction methodology is required by the designer. Assumed construction staging and loads shall be indicated on the Plans by the designer if a specific methodology is required.

All temporary support during construction shall be removed unless otherwise Consented to by the Ministry.

A3.2 Wind loads on highway accessory supports and slender structural elements

A.3.2.1 General

Delete the first paragraph and replace with the following:

Highway accessory supports include overhead and cantilever sign structures, overhead and cantilever traffic signal structures and luminaire structures. Highway accessory supports shall be designed for horizontal drag loads due to natural wind buffeting at the serviceability and ultimate limit states. The fatigue limit state designs required by Clause 12.5.5.4.1, shall use the design loadings in accordance with AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals 11.7 Fatigue Design Loads. Each of the AASHTO fatigue loading types shall be applied separately.

Galloping load from the AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals 11.7 Fatigue Design Loads does not apply for cantilever traffic signal support structures with approved vibration mitigation devices, and trussed or double mast-arm sign support structures.

Fatigue Importance Factor I_F shall be in accordance with AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals 11.6 Fatigue Importance Factors.

Fatigue Limit State Pressure Range for luminaire structures with heights of 16 m or greater design, P_{FLS} shall be in 310 Pa.

Slender structural elements, other than highway accessory supports, shall be designed for horizontal drag loads due to natural wind buffeting at the SLS and ULS and for horizontal drag loads due to natural wind buffeting, vertical loads due to truck induced wind buffeting and across-wind loads due to vortex shedding excitation at the FLS.

Delete the ULS Combination A2 from the Table A3.2.1.

Commentary: *The Ministry adopted AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals for fatigue design loading approach for highway accessory structures. Vortex shedding effect calculation in accordance with the S6:19 is not practical for the design of this type of structures and can be replaced with the equivalent static equations presented in the AASHTO LRFD Specification. All units shall be converted to Metric units.*

A3.3 Vessel collision

A3.3.5.3 Probability of aberrancy

A3.3.5.3.1 General

Replace the first sentence with the following:

Unless Consented to by the Ministry, the probability of vessel aberrancy, PA (the probability that a vessel will stray off course and threaten a bridge) shall be determined by the following approximate method:

Replace the definition of BR with the following:

BR = aberrancy base rate (0.6×10^{-4} for ships and 1.2×10^{-4} for barges)

Commentary: *The Ministry does not keep a data base of vessel collision with its structures. The values for BR are taken from AASHTO LRFD 2014 and are based on analysis of historical data for high use waterways. If site specific historical data is available, a statistical analysis may be used.*

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4.2 Definitions

Add the following:

Commentary:

Capacity design is a seismic design method in which the Designer selects, designs and details a primary lateral load resisting system to behave in a ductile and predictable manner while supporting specified gravity loads at deformations well beyond the elastic limits of the lateral load resisting system.

Capacity design involves an explicit selection of a plastic mechanism as the lateral earthquake resisting system. This system includes pre-selected plastic hinges (structural fuses) to allow the designer to control and limit forces in the non-yielding regions or components of the ductile substructure. Controlling the capacities of structural fuses allows the design forces on both the fuses and on adjacent structural components to be controlled. Detailing and proportioning the fuses and the adjacent components delays brittle failure modes until large post-elastic deformations occur, providing a significant degree of structural integrity and resilience to the bridge system for seismic loads. The method may also be applied to base-isolated bridges (where isolation bearings become the structural fuses) or to other energy-dissipating lateral load resisting systems.

Probable resistance: The probable resistance is larger than the expected nominal resistance and represents the increased resistance that ductile substructure element (plastic hinges) can develop accounting for effects such as rebar strain hardening, concrete confinement, expected material properties, etc. Adjacent capacity protected members and/or undesirable failure modes are designed for the force effect resulting from the plastic hinges (structural fuses) attaining their probable resistance.

Static Pushover analysis - Pushover analyses are used to determine capacity design demands and to assess structural behaviour and damage at each stage of inelastic deformation of the lateral load resisting system. Section capacities account for degradation with increasing ductility demands. Local deformations and strains from the analysis allow for damage and performance assessments at all specified earthquake levels, demonstrating whether the design performance criteria have been met.

Add the following definitions:

Analytical Plastic Hinge Length: the calculated equivalent length of a plastic hinge region used for analytical purposes, and over which the plastic curvature is assumed constant for estimating plastic rotation and plastic curvature.

Earthquake-Resisting System (ERS): a system that provides sufficient strength, ductility or energy dissipation for the bridge, ensures a load path for gravity loads, and controls seismic performance of the bridge.

Earthquake-Resisting Element (ERE): bridge elements within the ERS that transfers lateral loads, undergoes inelastic deformation, dissipates energy, or increases bridge damping.

Extended pile bent: Replace the definition with the following: gravity and lateral load-resisting substructure comprising piles that extend above grade without an at-grade pile cap, connecting directly to the pier cap beam supporting the bridge superstructure. Also includes Type I and Type II shafts. (Note: Where “pile bent” is used in this Section, it may be interpreted as an extended pile bent.)

Expected nominal resistance: the resistance based on expected material properties, calculated with the material resistance factors taken as 1.0.

Impedance: the product of V_s and mass density.

Impedance Ratio: the impedance of the material above the impedance contrast divided by the impedance of the material below the impedance contrast.

Plastic Hinge: a region of a structural member that undergoes flexural yielding and plastic rotation while retaining flexural strength.

Plastic Hinge Region/Zone: a region/zone of a structural member expected to, or with the potential to, form a plastic hinge and that therefore requires special seismic detailing to provide ductility.

Type I Pile Shaft: a drilled or driven shaft foundation having the same confined core diameter as that of the supported column but may have the same or different concrete cover and area of transverse and longitudinal reinforcement as the supported column. As defined by Caltrans Seismic Design Criteria, Version 2.0, April 2019.

Type II Pile Shaft: a drilled or driven shaft foundation that is larger in diameter than the supported column and has a reinforcing cage larger than and independent of columns. As defined by Caltrans Seismic Design Criteria, Version 2.0, April 2019.

Commentary:

To ensure ductile behaviour, S6-19 and the Supplement to S6-19 require enhanced seismic detailing for plastic hinge regions/zones and do not permit lap splices in these regions/zones.

The analytical plastic hinge length is used for calculating the plastic rotation and plastic curvature. The analytical plastic hinge length is different than plastic hinge region/zone. The AASHTO Guide Specification for LRFD Seismic Bridge Design (2nd edition) or the Caltrans Seismic Design Criteria Ver 2 may be used for calculating the analytical plastic hinge length.

4.3 Abbreviation and symbols

4.3.2 Symbols

Add the following:

$PGA(X)$ = peak ground acceleration, expressed as a ratio to gravitational acceleration, for seismic site designation X .

$PGV(X)$ = peak ground velocity, in m/s, for seismic site designation X .

$S_a(T, X)$ = 5%-damped spectral acceleration, expressed as a ratio to gravitational acceleration, at period T for seismic site designation X .

X = seismic site designation, either X_v or X_s .

X_s = seismic site designation in terms of Site Class, where S is the Site Class determined in accordance with Clause 4.4.3.2.(2).

X_V = seismic site designation in terms of \bar{V}_{S30} , where V is the \bar{V}_{S30} value calculated from in situ measurements of shear wave velocity.

ϵ_{ye} = strain corresponding to the expected yield strength of structural steel.

ϵ_{sh} = strain at the onset of strain hardening of structural steel.

ϵ_{ue} = strain corresponding to the expected tensile strength of structural steel.

Commentary:

Factored loads (eg. P_f , V_f) are replaced by the design forces in accordance with code provisions when using capacity design approach.

The expected tensile strength of structural steel, ϵ_{sh} and ϵ_{ue} should be based on mean values of tested material properties.

4.4 Earthquake effects

4.4.2 Importance categories

Replace the first sentence with:

The Ministry will designate bridges into one of the following three importance categories:

Commentary:

Low Volume Road (LVR) bridges are typically designated as "Other" bridges unless otherwise designated by the Ministry.

Seismic design should be considered on a case-by-case basis for temporary bridges planned to be in place for more than two years.

For temporary Lifeline and Major-Route bridges, a return period of at least 100 years should be considered.

Seismic design should be considered for all partially constructed bridges that carry or cross over public vehicular traffic, rail lines or navigable waters when the duration of construction is expected to exceed two years. For Lifeline and Major-Route bridges, a return period of at least 100 years should be considered.

4.4.3 Seismic hazard

4.4.3.1 General

Replace the first paragraph with the following:

The 5% damped horizontal spectral response acceleration values, $S_a(T, X)$, in units of g and where T is in seconds, for seismic site designation X , are established for the National Building Code of Canada (NBCC). These values shall be determined from NRCan 2020 National Building Code of Canada seismic hazard

tool (<https://www.seismescanada.rncan.gc.ca/hazard-alea/interpolat/nbc2020-cnb2020-en.php>) for the required probability of exceedance in 50 years.

Replace the second paragraph with the following:

The peak ground acceleration, $PGA(X)$, peak ground velocity, $PGV(X)$, and the 5% damped spectral response acceleration values, $S_a(T, X)$ for seismic site designation X for periods T of 0.2 s, 0.5 s, 1.0 s, 2.0 s, 5.0 s, and 10.0 s shall be determined for the hazard levels specified in Clause 4.4.6.2 for performance-based design, and for 2% in 50 years (2475-year return period) hazard level for force-based design.

Replace the fourth paragraph with the following:

For structures on all seismic site designations other than X_F , site-specific response analysis may also be conducted to obtain design spectral values when Consented to by the Ministry.

For all Lifeline bridges and Major-Route bridges in Seismic Performance Category 3, where the period of vibration of the elastic soil columns at foundation locations is within 0.5 to 2 times the period of vibration of the structure, the design response spectrum may be established using 1-D or 2-D site-specific response analysis if any of the following conditions are met, as agreed or directed by the Ministry:

- Non-gradational sites where shear wave velocity does not increase gradually with depth to a depth of at least 30 m, or
- High impedance contrast sites where there is an abrupt change of impedance over a few meters, with an impedance ratio greater than 4 or less than 0.25.

For structures on seismic site designation X_F with liquefiable soils:

- The requirements of Clause 6.14.8 shall be met.
- For structures with a fundamental period of vibration equal to or less than 0.5 s that are built on liquefiable soils, the seismic site designation, X and the corresponding values of $S_a(T, X)$ and $PGA(X)$ may be determined by assuming that the soils are not liquefiable.
- For Major-Route bridges in Seismic Performance Category 1 and 2, and for all Other bridges, the design response spectrum shall be the code-based design response spectrum using non-liquefied soil properties.
- For all Lifeline bridges and for Major-Route bridges in Seismic Performance Category 3, the design response spectrum shall be established using 1-D or 2-D site-specific response analysis.

For structures on seismic site designation X_F without liquefiable soils:

- For Major-Route and Other bridges in Seismic Performance Category 2 and Other bridges in Seismic Performance Category 3, the design response spectrum is the envelope of the code-based design response spectra for seismic site designation X_E for $T \leq 0.8$ sec and seismic site designation X_V or X_E for $T \geq 0.8$ sec.
- For all Lifeline bridges and Major-Route bridges in Seismic Performance Category 3, the design response spectrum shall be established using 1-D or 2-D site-specific response analysis.

When the design response spectrum is established using 1-D or 2-D site-specific response analysis, a high degree of site understanding as defined in Clause 6.5.3 is required.

For all seismic site designations, the design response spectra from site-specific response analysis shall not be less than 80% of the code-based spectra for the applicable seismic site designation. For liquefiable soil, non-liquefied soil properties shall be used when establishing the code-based design spectra.

Commentary:

In National Building Code of Canada 2020, the 6th generation seismic hazard model is adopted along with new seismic site designation definition. All references to Ste Class are now changed to seismic site designation X_V or X_S .

For projects in the regions where hydraulic fracturing (fracking) or deep wastewater disposal are planned or have been carried out, site-specific hazard analysis should be conducted to include injection-induced earthquakes as part of the seismic design.

For site-specific response analysis, the seismic site designation X_F is categorized into two groups:

- 1. Liquefiable soil that is horizontally continuous across the entire footprint of an abutment, pier, or geotechnical system, and*
- 2. Other seismic site designation X_F soils exhibiting ground profile characteristics of Site Class F as outlined in Table 4.1-B, except for the final criterion that pertains to liquefiable soils.*

The analysis requirements for seismic site designation X_F soils are addressed in clause 6.14.

Design response spectra from site-specific response analysis should have a horizontal plateau for short period ranges. The design response spectrum should not increase with an increase in period.

For sites where the soil deposition is not horizontal, 2-D site-specific response analysis is more appropriate than 1-D site-specific response analysis.

Spectral response acceleration values from the NBCC 2020 hazard tool are developed for gradational sites.

4.4.3.2 Site properties

Replace this clause with the following:

- 1) Except as provided in Sentence 2), the seismic site designation shall be X_V determined using the time-weighted average shear wave velocity in top 30 m, \bar{V}_{s30} , calculated from in-situ measurements of shear wave velocity, as follows:
 - a) The site-specific shear wave velocity profile of the ground shall be directly measured to at least 30 m below ground surface, or to at least 5 m depth below a sound rock layer.
 - b) The time-weighted average shear wave velocity in top 30 m, \bar{V}_{s30} , shall be calculated as

$$\bar{V}_{s30} = \frac{30}{\sum [h_i/V_{s,i}]}$$

where $V_{s,i}$ is in units of m/s, h_i in units of meters, and $\sum h_i = 30$ m.

- c) For the ground profiles described in Table 4.1-A, the site designation shall be determined in accordance with the Table, and
 - d) For all other ground profiles, the seismic site designation shall be X_V , where V is the value of \bar{V}_{s30} as defined in this clause.
- 2) Where \bar{V}_{s30} calculated from direct in-situ measurements is not available, the seismic site designation shall be X_S , where S is the Site Class determined in accordance with Table 4.1-B.

Table 4.1-A
Exceptions for Site Designation Using \bar{V}_{s30} Calculated from In-Situ Measurements
 (See Clause 4.4.3.2.(1))

Grond Profile Characteristics		Site Designation
Average Shear Wave Velocity in Top 30 m, \bar{V}_{s30} , Calculated from In-Situ Measurements, in m/s	Additional Characteristics	
$\bar{V}_{s30} > 760$	Ground profile contains more than 3 m of softer materials between rock and the underside of footing or mat foundations	X_{760}
$\bar{V}_{s30} > 140$	Ground profile contains a cumulative thickness more than 3 m of soil with all the following characteristics: <ul style="list-style-type: none"> • Plasticity Index, $PI > 20$, • Moisture content, $w \geq 40\%$, and • Undrained shear strength, $\bar{s}_u < 25$ kPa 	X_E
$\bar{V}_{s30} > 140$	Ground profile with any of the following characteristics: <ul style="list-style-type: none"> • Quick and highly sensitive silts and clay ($S_r > 8$), • Peat, organic silts and/or organic clays with a cumulative thickness greater than 3 m, • High-plastic clays ($PI > 75$) more than 8 m thick, • Soft to firm ($\bar{s}_u < 50$ kPa) silts and/or clays with a cumulative thickness more than 30 m, • Municipal solid waste and/or other landfill materials, or • A horizontally continuous liquefiable layer across the entire footprint of an abutment, pier or geotechnical system. 	X_F
$\bar{V}_{s30} \leq 140$	N/A	X_F

Table 4.1-B
Site Classes, S , for Seismic Site Designation X_S
 (See Clause 4.4.3.2.(2))

Site Class, S	Ground Profile	Ground Profile Characteristics		
		Average Shear Wave Velocity in Top 30 m, \bar{V}_{s30} , in m/s	Average Standard Penetration Resistance in Top 30 m, \bar{N}_{60} , in Blows per 0.3 m	Average Undrained Shear Strength in Top 30 m, \bar{s}_u , in kPa
A	Hard rock	$\bar{V}_{s30} > 1500$	Not applicable	Not applicable
B	Rock	$760 < \bar{V}_{s30} \leq 1500$	Not applicable	Not applicable
C	Very dense soil and soft rock	$360 < \bar{V}_{s30} \leq 760$	$\bar{N}_{60} > 50$	$\bar{s}_u > 100$
D	Stiff soil	$180 < \bar{V}_{s30} \leq 360$	$15 < \bar{N}_{60} \leq 50$	$50 < \bar{s}_u \leq 100$
E	Soft soil	$140 < \bar{V}_{s30} \leq 180$	$10 < \bar{N}_{60} \leq 15$	$40 < \bar{s}_u \leq 50$
		Any ground profile other than Site Class F that contains a cumulative thickness more than 3 m of soil with all the following characteristics: <ul style="list-style-type: none"> • Plasticity Index, $PI > 20$, • Moisture content, $w \geq 40\%$, and • Undrained shear strength, $\bar{s}_u < 25$ kPa 		
F	Other soils	$\bar{V}_{s30} \leq 140$	$\bar{N}_{60} \leq 10$	$\bar{s}_u \leq 40$
		Ground profile with any of the following characteristics: <ul style="list-style-type: none"> • Quick and highly sensitive silts and clay ($S_t > 8$), • Peat, organic silts and/or organic clays with a cumulative thickness greater than 3 m, • High-plastic clays ($PI > 75$) more than 8 m thick, • Soft to firm ($\bar{s}_u < 50$ kPa) silts and/or clays with a cumulative thickness more than 30 m, • Municipal solid waste and/or other landfill materials, or • A horizontally continuous liquefiable layer across the entire footprint of an abutment, pier or geotechnical system. 		

Commentary: It is preferable to determine the seismic site designation as X_v . This typically results in a lower design response spectrum than a seismic site designation X_S based on Site Class.

4.4.3.3 Site coefficients

Delete this clause.

Commentary: Site coefficients are no longer used as the spectral response accelerations from the 6th generation seismic hazard tool (NBCC 2020) already account for site properties using site designations.

4.4.3.4 Design response spectral (acceleration and displacement)

Replace this clause with the following:

The design spectral acceleration, $S(T)$ shall be determined as follows, using log-log interpolation for intermediate values of T :

$$\begin{aligned}
 S(T) &= S_a(0.2, X) \text{ or } S_a(0.5, X), \text{ whichever is greater for } T \leq 0.2 \text{ s} \\
 &= S_a(0.5, X) \text{ for } T = 0.5 \text{ s} \\
 &= S_a(1.0, X) \text{ for } T = 1.0 \text{ s} \\
 &= S_a(2.0, X) \text{ for } T = 2.0 \text{ s} \\
 &= S_a(5.0, X) \text{ for } T = 5.0 \text{ s} \\
 &= S_a(10.0, X) \text{ for } T = 10.0 \text{ s}
 \end{aligned}$$

The design spectral displacement values $S_d(T)$ at periods $T = 0.0, 0.2, 0.5, 1.0, 2.0, 5.0,$ and 10.0 s shall be determined using $S_d(T) = 250 S(T) T^2$ (in millimeters). Values for intermediate values of T shall be determined using log-log interpolation.

Commentary:

The value of $S(T)$ for $T_i < T < T_j$ can be determined using log-log interpolation as follows:

$$S(T) = 10^{\log(S(T_i)) + \frac{\log(T) - \log(T_i)}{\log(T_j) - \log(T_i)} [\log(S(T_j)) - \log(S(T_i))]}$$

When interpolating for the intermediate values of design spectral accelerations and displacements, log-log interpolation significantly improves the accuracy of seismic demands compared to linear interpolation.

4.4.3.6 Input ground motion records for time history analyses

Add the following to the first paragraph:

If Consented to by the Ministry, two or more site-specific target response spectra may be allowed. Input ground records shall have similar spectral shape to the target response spectrum.

Delete the second paragraph and replace with:

Eleven or more ground motion records shall be used in design for each target spectra. When only one suite of ground motion records is used in the analysis, the mean response quantity shall be used in design. When two or more suites of 11 or more ground motion records are used, the design seismic demand shall be taken as the largest of the mean values of each suite.

Add the following to the end of the third paragraph:

Vertical ground motions may be determined by carrying out an independent hazard calculation using vertical ground motion prediction equations to develop a vertical response spectrum. The methodology and ground motions shall be Consented to by the Ministry.

Add the following paragraph:

Deviations to the selection and use of ground motions shall be Consented to by the Ministry.

Commentary: *The Commentary on CSA S6:19, Canadian Highway Bridge Design Code should be used for guidance and additional references for the selection and use of ground motions.*

4.4.5.2 Single-span bridges

4.4.5.2.1 Analysis requirements

Add the following:

Single-span bridges with seismic site designation X_F shall be designed using PBD as specified for multi-span bridges in Clause 4.4.5.3.

In the restrained directions, the substructure of all single-span bridges shall be designed for the connection force effect from Clause 4.4.10.1, or for the seismic design forces obtained from EDA or ESA.

Commentary: *Continuous and reliable load path to transfer all seismic inertial loads from point of application to surrounding soils is essential for earthquake resistance systems. The substructure design of single-span bridges is only needed in the restrained directions between the superstructure and the substructure.*

4.4.5.3 Multi-span bridges

4.4.5.3.1 Analysis requirements and design approach

Add the following note to Table 4.11:

Bridges which may be subjected to the effects of liquefaction shall be designed using performance-based design.

Add the following note to Table 4.12:

EDA and ISPA are required for bridges which are subjected to the effects of liquefaction.

Add the following note to Table 4.13:

For PBD, ISPA is required unless the structure behaves in an essentially elastic manner.

Add the following:

Free-field response spectrum for ESA, EDA, and ISPA or free-field time history ground motions for NTHA shall be established using either code specified values or site response analysis. Free-field response

spectrum or free-field time history ground motions may be computed at an elevation determined jointly by the structural and geotechnical engineer and Consented to by the Ministry.

Commentary: *The depth-to-fixity may be used for determining the foundation input motion. The depth-to-fixity may be derived by equating the lateral stiffness of the cantilever to that of the elastic soil-pile system (Chai 2002). The depth-to-maximum-moment is different than the depth-to-fixity.*

Add the following:

As a minimum, the following geotechnical engineering input shall be incorporated in the structural analysis methods described in Table 4.12 and Table 4.13:

- Elastic Dynamic Analysis (EDA): These analyses shall be carried out on structural model(s) with an appropriate treatment of soil-structure interaction that capture, as a minimum, the effects of foundation flexibility important to global structural response.
- Inelastic Static Pushover Analysis (ISPA): These analyses shall be carried out on a full or partial model of the bridge system incorporating the effects of foundation flexibility using methods outlined in Clause 6.14.5 for shallow foundations, Clause 6.14.6 for deep foundations, and Clause 6.14.7 for abutment and approach fill interaction.
- Where applicable (e.g., liquefaction-induced lateral spreading or settlements), the effects of kinematic loading from inelastic ground deformations on the displacement and other effects of inertial loading shall be evaluated and combined using the combinations described below:
 - 100% inertial demands obtained from Clause 6.14.8.2.2, and
 - 100% kinematic demands $\pm 50\%$ inertial demands, combined in accordance with Clause 6.14.8.3 and this Supplement.
- Nonlinear time history analysis (NTHA): These analyses shall be carried out on a full or partial model of the bridge system incorporating the nonlinear behaviour of foundation soils and foundation elements. For NTHA, explicit pile foundation modelling is required. Computer software used for this purpose shall have the capability to incorporate nonlinear soil effects, pre-and post-earthquake stress-strain-strength characteristics of soils, and nonlinear structure effects. These analyses shall be either 2-D or 3-D. Unless otherwise specified by the Ministry, analyses shall be carried out for all input ground motions defined in Clause 4.4.3.6.
- A range of possible soil stiffness shall be evaluated for ESA, EDA, ISPA, and NTHA analysis based on accepted geotechnical methods using soil parameters based on field and laboratory testing. A study shall be made on the sensitivity of bridge seismic response to the variation in the soil stiffness.

Commentary:

Tables 4.12 and 4.13 apply to structural analyses including appropriate modelling for important soil-structure interaction effects in all analysis types.

Foundation flexibility is important in the seismic design, whether for stand-alone piers or for piers within bridge systems, as it can affect the location and progression of plastic hinging, the local ductility demands at hinges, and the demand calculations for capacity protected elements.

Project specific seismic specifications will be provided by the Ministry for Lifeline, Major-Route bridges, or other specific projects when needed.

Add the following clause:

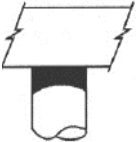
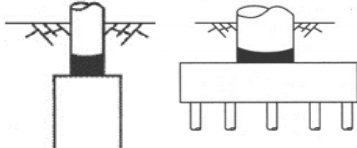
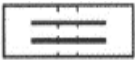
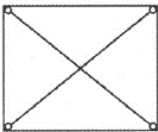
4.4.5.4 Earthquake Resisting System (ERS) and Earthquake Resisting Element (ERE)

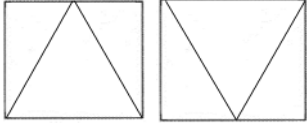
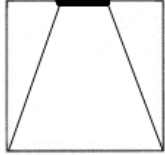
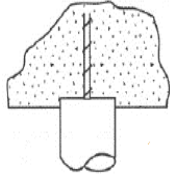
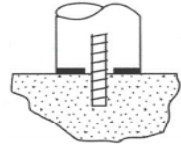
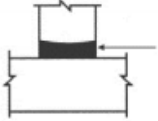
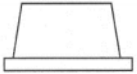


For Seismic Performance Category 2 and 3, all bridges and their foundations shall have a clearly identifiable ERS and ERE to achieve the seismic design requirements. The ERS shall be designed to ensure a continuous and reliable load path to transfer all seismic inertial loads from point of application into the surrounding soil.

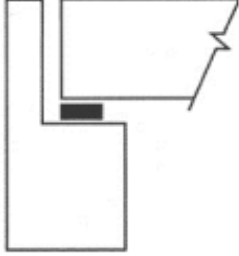
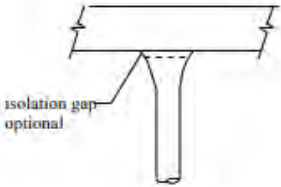
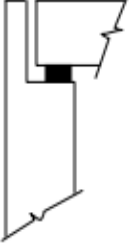


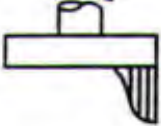
Limitations on the use of EREs are provided in Table 4.4.5.4-1. EREs are categorized into Permissible, Not Permissible, and Permissible when Consented to by the Ministry.

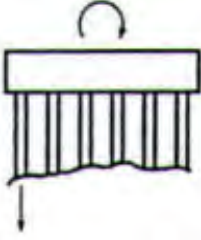

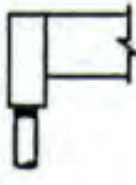

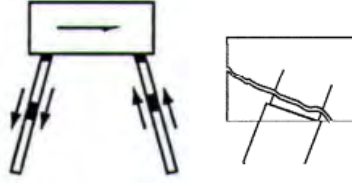
Use of EREs not identified in Table 4.4.5.4-1, require Consent by the Ministry. Where practical, preference shall be given to ERSs and EREs that promote low-damage and high-resilience performance.

Table 4.4.5.4-1: Earthquake-Resisting Elements (EREs)

Item	Illustration	Description	Conditions of Use
1		Column plastic hinges below cap beams, including extended pile bents.	Permissible
2		Near surface and at surface column plastic hinges above capacity-protected foundations, including spread footings and pile caps.	Permissible
3		Seismic isolation bearings or bearing designs to accommodate total seismic displacement.	Permissible
4		Ductile concentrically braced frames.	Permissible in the substructure.

5		Chevron-braced and V-braced systems	Permissible in the substructure.
6		Ductile eccentrically braced frame.	Permissible in the substructure when Consented to by the Ministry and only when there are no girders supported on the cap beam that has the shear link
7		Piles with 'pinned-head' conditions	Permissible when Consented to by the Ministry
8		Columns with moment reducing or pinned hinge details	Permissible in substructure when Consented to by the Ministry
9		Plastic hinges at base of wall piers and rectangular columns in weak direction	Permissible
10		Pier walls with or without piles	Permissible
11		Spread footings meeting the overturning criteria of Clause 6.14	Permissible
12		<p>Passive abutment resistance required as part of ERS.</p> <p>Passive abutment resistance shall be based on 70% of ultimate capacity as determined in accordance with Clause 6.14.7.1.</p>	Permissible

13		Seat abutments whose backwall is designed to fuse.	Permissible
14		Columns with architectural flares – with or without an isolation gap	Permissible when Consented to by the Ministry.
	<p>As per the requirements of CALTRANS SEISMIC DESIGN CRITERIA VERSION 2.0 APRIL 2019 Clause 7.6.3 and the AASHTO Guide Specification for LRFD Seismic Design, 2011 Section 8.14 for isolation gaps.</p>		
15		Seat abutments whose backwall is designed to resist the expected impact force in an essentially elastic manner	Permissible
16		Sliding of spread footing abutment allowed to limit force transfer.	Permissible when Consented to by the Ministry
17		Ductile End-diaphragms in superstructure.	Permissible when Consented to by the Ministry
18		<p>Foundations permitted to rock</p> <p>Use rocking criteria according to AASHTO Guide Specification for LRFD Seismic Design, 2011 Appendix A.</p>	Permissible when Consented to by the Ministry.

		<p>NTHA with comprehensive sensitivity analysis of foundation, structure, and ground motion assumptions required.</p>	
19		<p>More than the outer line of piles in group systems allowed to plunge or uplift under seismic loadings.</p>	<p>Permissible when Consented to by the Ministry.</p>
20		<p>Wall piers on pile foundations that are not strong enough to force plastic hinging into the wall, and are not designed for the seismic elastic forces</p>	<p>Permissible when Consented to by the Ministry.</p>
21		<p>Plumb piles that are not capacity-protected for integral abutment piles, semi-integral abutment piles or pile-supported seat abutments that are not fused transversely</p>	<p>Permissible when Consented to by the Ministry.</p>
22		<p>In-ground hinging in Type I or II pile shafts.</p>	<p>Permissible when Consented to by the Ministry.</p>
23		<p>Batter pile systems in which the geotechnical capacities and/or in-ground hinging define the plastic mechanisms.</p>	<p>Not permissible.</p>

24		Plastic hinges in the superstructure	Not permissible.
25		Frame with cap beam plastic hinging	Not permissible.
26		Bearing systems that do not provide for the expected displacements and/or forces (e.g., rocker bearings)	Not permissible.

Item 2: For Type II pile shafts, the shaft shall be designed as a capacity-protected element and column longitudinal reinforcement shall be embedded into oversized shafts per Caltrans Seismic Design Criteria Version 2.0, April 2019, clause 8.3.2. Sensitivity analysis, using appropriate bounds on the soil-pile interaction model, shall be performed on shear and moment demands.

Items 2, 20, 21 and 22:

In-ground plastic hinges are only permissible if reasonably accessible for inspection and repair. In-ground plastic hinge locations shall be Consented to by the Ministry.

Deep in-ground plastic hinge locations are only permissible for liquefaction load cases and only when Consented to by the Ministry. Post-earthquake serviceability, repair, and return to service criteria of Table 4.16 shall be demonstrated to the satisfaction of the Ministry. Installation of devices that can assist in the post-earthquake assessment of piles and in-ground plastic hinges are required.

Steel pipes subjected to in-ground hinging shall be made of steels satisfying the requirement of API Specification 5L, 46th Edition. Additional requirements for fabrication welds and splices are required for ASTM A252 material per the Special Provisions template.

Commentary:

Table 4.4.5.4-1 is based on the AASHTO Guide Specifications for Seismic Bridge Design, 2011, by the American Association of State Highway and Transportation Officials, Washington, D.C., U.S.A. and this has been used with their permission.

For ERS and ERE requiring Consent of the Ministry, the designer should prepare a project specific seismic design criteria memo for the Ministry's review. The memo should cover items such as the rationale for selected ERS and ERE, energy dissipation mechanism, post-earthquake seismic behaviour (inspection, reparability, return to service), life cycle cost, constructability, durability, reliability, risks, maintenance requirements, any peer-reviewed experimental and/or analytical publications, relevant past projects, proposed seismic performance criteria, etc. The memo should be submitted as early as possible, e.g.: during concept development.

"Reasonably accessible" locations for in-ground hinges vary and should be considered on a case-by-case basis. For general guidance in typical situations, the Ministry considers "reasonably accessible" to mean less than 8 meters below the ground, mean water or tide level for steel piles and less than 4 meters for concrete piles below the ground, mean water or tide level.

If deep in-ground plastic hinges are expected, installation of devices such as inclinometer tubes in shafts can assist in the post earthquake assessment of the performance of the piles and pile hinges.

4.4.6 Performance-based design

4.4.6.1 General

Replace third paragraph with:

Lifeline bridges in Seismic Performance Category 2 and 3 require independent peer review unless otherwise specified by the Ministry.

The independent peer review shall be done by a firm other than the firm employing the Engineer of Record.

The independent peer review shall be done by recognised subject-matter experts acceptable to the Ministry in relevant fields, including but not limited to:

- a) Performance-based structural seismic design of bridges, including foundations and supporting soils.
- b) Performance-based geotechnical seismic design of bridges, including foundations and supporting soils.
- c) Earthquake hazard definition and selection and modification of ground motions for use in nonlinear time history response analysis, including effects of soil-structure interaction.
- d) Application of structural and geotechnical analysis software for use in nonlinear time history response analysis and interpretation of analysis results.

Commentary: *The requirement for independent peer review is in addition to Engineers and Geoscientists BC requirements for independent reviews, such as independent reviews of structural designs and independent reviews of high-risk professional activities or work. The scope of work for independent peer review is described in the Engineers and Geoscientists BC document entitled Performance-Based Seismic Design of Bridges in BC, Section 2.4 and in the Commentary to S6:19, Clause C4.4.6.1. Recognition for subject matter experts may be evidenced by one or more of the following accomplishments in the field of bridge earthquake engineering and seismic design: industry achievements substantially above those ordinarily achieved by practicing engineers; industry awards and/or acknowledgements pertaining to*

seismic engineering of bridges; active technical member invited to collaborate with a standards organisation or code committee pertaining to seismic engineering of bridges; leadership role in a centre of expertise dealing with earthquake-resistant design; peer reviewed publications addressing seismic engineering of bridges; and/or international solicitation as consultancy expert in the seismic engineering of bridges.

4.4.6.3 Performance criteria

Replace the clause with:

The performance criteria for different performance levels are given in Table 4.16. The assessment of damage performance levels specified in Table 4.16 shall be carried out using nonlinear time history method or by using inelastic static pushover analysis up to the design displacement (see Clause 4.5.3.5). When assessing performance in performance-based design, the behaviour of the structural components may be determined using the expected material properties as defined in Clauses 4.7.2 and 4.8.2.2.

Table 4.16:

Add the following for “Immediate”, “Limited”, and “Service Disruption” Service:

Repair work shall restore the structure to meet the original design loading requirements.

Commentary: *The intent is the damage shall be repairable such that the repairs restore the structural capacity for its original design loads, both vertical and lateral loads.*

Effects of live load on bridge inertia mass shall not be included in the dynamic analysis.

Replace description for “Minimal Damage” to:

Minimal Damage

- Live Load: 50% of unfactored multi-lane normal traffic (without Dynamic Load Allowance) shall be applied concurrent with seismic demands.
- General: Bridge shall remain essentially elastic with minor damage that does not affect the service performance level of the structure.
- Concrete Structures: Concrete compressive strains shall not exceed 0.006 and flexural reinforcing steel strains shall not exceed 0.010.
- Steel Structures: Steel strains shall not exceed the lesser of 0.003 and $1.5\varepsilon_{ye}$. Local or global buckling shall not occur.
- Connections: The service performance level of connections shall not be compromised.
- Displacements: Residual displacement, settlement, translation or rotation, of the structure or foundations, including retaining and wing walls, shall not compromise the service performance level.
- Bearings and Joints: Shall not require replacement except for possible damage to joint seal.
- Restrainers: Negligible damage and no loss of displacement capacity to restraining systems or connected elements shall occur.

- Foundations: Foundation movements shall be limited to only slight misalignment of the spans or settlement of some piers or approaches that does not interfere with normal traffic, provided that no repairs are required.
- Steel Piles: For steel piles and the casing of concrete infilled steel pipes, steel strains shall not exceed the ε_{ye} . For concrete infilled steel pipes, concrete strains shall not exceed 0.004.
- Concrete Piles: Piles shall remain essentially elastic and concrete strains in concrete piles shall not exceed 0.004.

Replace description for “Repairable Damage” to:

Repairable Damage

- Live Load: 100% of unfactored multi-lane normal traffic (without DLA) in a minimum of 50% of the lanes, but not less than one lane in each direction, shall be applied concurrent with seismic demands. The operational traffic lanes shall be shown on the plans.
- General: The bridge may experience inelastic behaviour, but primary members shall be repairable in place.
- Concrete Structures: reinforcing steel tensile strains shall not exceed 0.025.
- Steel Structures: Steel strains shall not exceed the larger of 0.008 and $2/3\varepsilon_{sh}$. Buckling of primary members shall not occur. Secondary members may buckle provided that stability is maintained. Net area rupture of primary members at connections shall not occur.
- Connections: Primary connections shall not be compromised.
- Displacements: Residual displacement, settlement, translation or rotation, of the structure or foundations, including retaining and wing walls, shall not compromise the service performance level.
- Bearings and Joints: Replacement of elastomeric bearings is permitted provided that service requirements are not compromised. Damage to other structural bearings shall not compromise the integrity of the structure nor compromise the service requirements. Replacement of joints is permitted.
- Restrainers: Restrainers shall not rupture and shall retain their ability to mitigate span loss in aftershocks. Damage to restrainer supporting elements such as end diaphragms or substructure shall not require bridge closure to repair.
- Steel Piles: for steel piles and the casing of concrete infilled steel pipes, steel strains shall not exceed the lesser of 0.003 and $1.5\varepsilon_{ye}$. For concrete infilled steel pipes, concrete strains shall not exceed 40% of its ultimate confined strain limits.
- Concrete Piles: Reinforcement strains shall not exceed 0.01 and concrete strains in concrete piles shall not exceed 0.006.
- Ground deformations shall be mitigated such that permanent foundation offsets are small and repair objectives can be met. Foundation offsets shall be limited such that repairs can bring the structure back to the original operational capacity.

Replace description for “Extensive Damage” to:

Extensive Damage

- Live Load: a minimum of one lane of unfactored normal traffic (without DLA) shall be applied concurrent with seismic demands for one- and two-lane bridges. One lane of unfactored normal traffic (without DLA) in each direction, but not less than 30% of lanes, shall be applied concurrent with seismic demands for bridges with more than two lanes. The emergency traffic lanes shall be shown on the plans.
- General: Inelastic behaviour is expected. Members may have extensive visible damage, such as spalling of concrete and buckling of braces, but significant strength degradation is not permitted. Members shall be capable of supporting loads, including P-delta effects, without collapse.
- Concrete Structures: Extensive concrete spalling is permitted but the confined core concrete shall not exceed 80% of its ultimate confined strain limit. Reinforcing steel tensile strains shall not exceed 0.05.
- Steel Structures: Steel strains shall not exceed the lesser of 0.06 and $0.5 \epsilon_{ue}$. Global buckling of gravity load supporting elements shall not occur.
- Connections: There may be significant joint distortions, but damaged connections must maintain structural integrity under gravity loads.
- Structural displacements: There may be permanent structural offsets as long as they do not prevent use by restricted emergency traffic after inspection or the bridge, nor preclude return of full service to the bridge after major repairs.
- Bearings and Joints: The bearings may be damaged, or girders may become unseated from bearings, but girders shall have adequate remaining seat length and connectivity to carry restricted emergency traffic. Bearings and joints may require replacement.
- Restrainers: Restraining systems may be damaged but shall not fail.
- Foundations: Foundation lateral and vertical movements must be limited such that the bridge can be used by restricted emergency traffic. Foundation offsets shall be limited such that repairs can bring the structure back to the original operational capacity.
- Steel Piles: For steel piles and the casing of concrete infilled steel pipes, steel strains shall not exceed the larger of 0.008 and $2/3 \epsilon_{sh}$. For concrete infilled steel pipes, concrete strains shall not exceed 80% of its ultimate confined strain limits.
- Concrete Piles: Reinforcement strains shall not exceed 0.025. Concrete spalling is permitted.

Replace description for “Probable Replacement” to:

Probable Replacement

- Live Load: 30% of unfactored multi-lane normal traffic (without DLA), shall be applied concurrent with seismic demands.
- General: Bridge spans shall remain in place, but the bridge may be unusable and may have to be extensively repaired or replaced.

- Concrete Structures: Damage does not cause crushing of the confined concrete core. Reinforcing steel tensile strains shall not exceed 0.075, except that for steel reinforcing of 35M or larger the strains shall not exceed 0.060.
- Steel structures: Steel strains shall not exceed ϵ_{ue} .
- Extensive distortion of beams and column panels may occur.
- Members shall be capable of supporting loads, including P-delta effects, without collapse
- Fractures at some moment connections may occur that don't significantly increase the risk of collapse. Shear connections shall remain intact.
- Displacements: Permanent offsets shall be limited such that the bridge can be evacuated safely.
- Foundations: Foundation movements shall not lead to collapse of the bridge superstructure, collapse of approach walls within the bridge approach embankment interface zone, nor prevent safe evacuation of the bridge.
- Steel Piles: For steel piles and the casing of concrete infilled steel pipes, steel strains shall not exceed the lesser of 0.06 and $1/2 \epsilon_{ue}$. For concrete infilled steel pipes, concrete strains shall not exceed its ultimate confined strain limits.
- Concrete Piles: Reinforcement strains shall not exceed 0.05. Concrete spalling is permitted but the confined core concrete shall not exceed 80% of its ultimate confined strain limit.

4.4.6.4 Consideration of aftershock effects

Delete the clause.

Commentary:

In general, superstructures, ductile substructures, restrainers and foundations designed to S6:19 PBD methods are considered to have inherently met expectations for aftershocks without additional assessment. This is because the design methods and detailing result in a robust structure and is capable of sustaining multiple additional cycles of seismic loading.

Although there are no standardized methodologies for the aftershock assessment, the potential effects of aftershocks on the performance levels of Lifeline and Major-Route bridges in SPC 3 may be needed if required by the Ministry.

4.4.7 Force-based design

4.4.7.1 General

Add the following:

For regular bridges of slab, beam-girder, or box girder construction, with a structurally continuous reinforced concrete deck designed as a horizontal diaphragm between substructure elements, and where the superstructure is not integral with the substructure, a detailed analysis of earthquake effects on superstructure components shall not be required. Cross-frames and diaphragms, bearings, bracing connections and connections between the girders at the abutments and piers shall be analysed.

4.4.7.2 Response modification factor

Delete the last paragraph.

4.4.9 Load factors and load combinations

4.4.9.2 Earthquake load cases

Delete the last paragraph and add the following:

The effects of vertical ground motion for FBD and PBD shall be accounted for by using the load factors on dead load specified in Table 3.3 or by using a dead load factor of 1.0 and vertical effects arising from a dynamic analysis including vertical accelerations explicitly, in combination with the orthogonal effects described above.

Commentary:

Orthogonal load combinations in this section were developed primarily for force-based design approaches on piers but should also be used to make allowances for coupling of displacement demands and response in orthogonal directions. The bi-direction combination applies for verifying the performance criteria of Clause 4.4.6.3.

These directional combinations were not calibrated for abutment or retaining wall design. Abutments and walls are normally designed using earthquake loads in each direction separately. For skewed abutments it is common to check abutment stability using pressures perpendicular to the ballast wall. This approach is acceptable, including for integral abutment bridges, for skew angles 20 degrees or less. For higher skew angles, concurrent directional combinations in orthogonal directions should be investigated more explicitly. Structurally, the effects of displacements normal to the abutment should be considered in detailing for seat lengths and global structural response.

4.4.10 Design forces and support lengths

4.4.10.1 General

4.4.10.2 Seismic Performance Category 1

Add the following:

The substructure shall be designed for the forces determined in accordance with this clause, or for the seismic design forces obtained from EDA or ESA.

Commentary: *A continuous and reliable load path to transfer all seismic inertial loads from point of application to surrounding soils is essential for earthquake resistance systems.*

4.4.10.4 Seismic Performance Category 3

4.4.10.4.2.2 Seismic design forces for capacity-protected elements for force-based and performance-based design

Delete the last paragraph and replace it with:

Where a seismic lateral load-resisting system relies on elastic forces rather than on capacity design principles to control demands, brittle failure modes in lateral-load resisting elements shall use design forces of 1.25, 1.35, and 1.5 times the elastic forces for Other, Major-Route, and Lifeline importance category respectively unless required otherwise by project-specific criteria. The elastic forces need not be greater than forces obtained from capacity design principles using probable resistance. Connectors shall be designed to transmit, in their restrained directions, the maximum force effects determined from the elastic seismic forces using the above increase, but these forces need not exceed the force that can be developed by the ductile substructure element attaining 1.25 times its probable resistance.

Commentary: The design of columns or pier walls which are part of the energy dissipating system are sometimes governed by other load cases rather than seismic load cases. As such, the probable moment resistance of these components may be greater than the elastic seismic demands. Consequently, capacity design based on the forces obtained from probable resistance tends to be overly conservative. To maintain enough margin of the safety between the brittle failure modes and ductile behaviour modes, the elastic seismic forces need to be increased. The margin of safety is provided as a function of the importance designation of the bridge.

4.4.10.4.3 Yielding mechanisms and design forces in ductile substructures

Add the following to the fourth paragraph:

Design forces for ductile substructure elements for Major-Route and Lifeline importance categories shall be taken as the unreduced elastic design forces increased by 1.35 and 1.5 for respectively, unless required otherwise by project-specific criteria.

Commentary: Note that “reasonably accessible” for inelastic hinge locations is described in Clause 4.4.5.4.

4.4.10.7 Hold-down devices

Commentary: Uplift restraint is regarded as a beneficial feature in bridges in zones of high seismic hazard. Alternative hold-down details are subject to Ministry consent.

4.5 Analysis

4.5.3 Multi-span bridges

4.5.3.5 Static pushover analysis

Add the following:

The static pushover analysis shall be taken to the expected displacement demands considering the bi-directional effects of seismic loading. Displacement demands shall capture global bridge response considering the behaviour of the individual pier or support within the global model unless the designer demonstrates that relevant information can be obtained with a local model.

Commentary:

Static pushover analyses are used to define the sequence of inelastic action in ductile structures, to develop member design forces for ‘capacity protection’ in ductile substructures, and to assist in defining deformation capacity. They may also be used to assist in defining stiffness and hysteretic properties for use in inelastic dynamic analyses.

The pushover analysis should be used to identify the expected ultimate failure mode and displacement to identify the margins of reserve and resiliency inherent in the design, and to assist the Ministry in evaluating the design. Local pier models are often adequate for ISPA, but global response effects (e.g. torsion in plan from variations in pier stiffnesses) should also be considered. In some cases, for example integral superstructure-to-pier connections, a pushover model must consider the restraint imposed by the bridge on the local pier response. The model used should be appropriate to capture the important aspects of seismic behaviour.

4.7 Concrete structures

4.7.4 Seismic Performance Category 2

Replace the second paragraph with the following:

The transverse reinforcement at the top and bottom of a column and in potential plastic hinge zones of beams, columns, shafts or piles shall be as specified in Clauses 4.7.5.2.5 and 4.7.5.2.6.

4.7.5 Seismic Performance Category 3

4.7.5.2 Column requirements

4.7.5.2.4 Column shear and transverse reinforcement

Commentary:

The amount of transverse reinforcing steel required within plastic hinge regions need not be carried through the remaining length of the columns.

Acceptable refined seismic shear design methodologies for plastic hinge regions of columns, which takes into account typical bridge column proportions, reinforcing quantities, details and degradation of concrete shear strength is presented in “Displacement-based Seismic Design of Structures, Priestley, Calvi and Kowalsky, IUSS Press, 2007”.

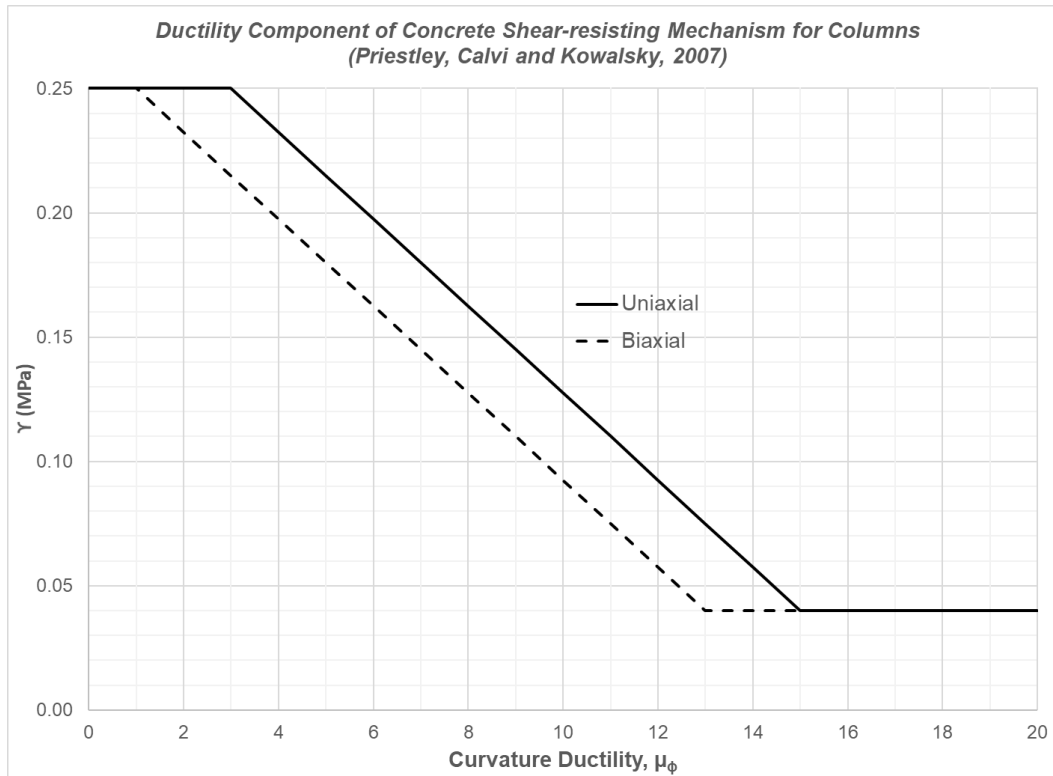
$$V_n = \phi_C V_C + \phi_S V_S + V_P$$

V_C = Concrete Shear-Resisting Mechanism

$$V_C = \alpha\beta\gamma\sqrt{f'_{ce}} 0.8A_g$$

$$\alpha = 1.0$$

$$\beta = 0.5 + 20\rho_l \leq 1.0$$



V_s = Steel Shear-Resisting Mechanism (based on spirals or ties crossed by a crack with $\cot \theta$ measured from vertical, using $\theta = 35^\circ$ for design)

For Rectangular Columns:

$$V_s = \frac{A_v f_y (D - c - c_o) \cot \theta}{s}$$

For Circular Columns:

$$V_s = \frac{\pi A_v f_y (D - c - c_o) \cot \theta}{2s}$$

V_p = Axial Load Component

$$V_p = 0.85P \tan \zeta$$

Definitions:

γ = factor for degradation in V_c with increasing curvature ductility, MPa

s = spiral spacing

A_v = Area of reinforcing bar used for spirals (for rectangular columns use total area of all shear bars at the section)

f_y = hoop steel nominal yield stress

D = Column Depth / diameter (out to out)

c = depth from extreme compression fibre to neutral axis under the loading considered

c_o = cover to centre of the peripheral spiral cage

$\theta = 35^\circ$ for design

P = axial load from bridge weight plus plastic mechanism effects

ζ = angle of inclination of a compression strut through the column, measured from the member's longitudinal axis

Plastic Hinge Region in Tall Columns or those having high axial loads:

“Tall” columns consider those with clear height to column diameter (H/d), or to least rectangular dimension, greater than 15.

“High axial loads” considers those with greater than 30% of the crush load ($f'_c A_g$) of the reinforced concrete section, including axial loads from bridge self weight, any specified live loads to be combined with seismic demands, and from seismic demands.

4.7.5.2.5 Transverse reinforcement for confinement at plastic hinge regions

4.7.5.2.7 Splices

Replace the first sentence of the second paragraph with:

Lap splices in longitudinal reinforcement and lap splices in spiral reinforcement shall not be located in plastic hinge regions. “No-Splice Zones” shall be clearly identified on the plans.

Add the following at the end of the third paragraph:

Welded splices and mechanical splices shall only be used when Consented to by the Ministry.

Commentary:

For long plastic hinge regions, the Ministry may accept mechanical splices in longitudinal reinforcements in plastic hinge regions based on the following:

- If the longest length of commercially available reinforcement steel (not less than 18 m) is used to ensure that splicing is avoided where possible and minimized elsewhere, and
- In addition to the requirements of Section 8.4.4.4, the mechanical couplers shall be capable of developing 125% of the maximum tensile strength of the spliced bars.

4.7.5.4 Column connections

Add the following after the first paragraphs:

For Lifeline and Major-Route bridges in Seismic Performance Category 3, the design of column connections, including member proportions, details, and reinforcement, shall be designed as capacity-

protected elements based on beam-column joint design methodologies as described in Caltrans Seismic Design Criteria Version 2.0. Headed bars may only be used when Consented to by the Ministry.

Commentary: *Rational design of beam-column joints is required for important bridges in high seismic zones. In the absence of an explicit design, "Other bridges" are to have beam-column joints reinforcing extend the full depth of the joint. Beam column joints in bridges of SPC 1 should be designed for force transfer as described in Section 8 of S6:19.*

Caltrans provides additional guidance for the use of headed bars in Memo to Designers 20-21 "Seismic Requirements for Headed Bar Reinforcement", 2016.

4.8 Steel structures

4.8.3 Sway stability effects

Commentary: *Guidance on incorporating P-Delta effects can be found in ATC-32 Clause 3.21.15.*

4.8.4 Steel substructures

4.8.4.4 Seismic Performance Category 3

4.8.4.4.5 Buckling restrained braced frames

Commentary: *It is preferable to use analyses that emphasize the deformation demands within the brace when used in bridge applications.*

4.10 Seismic base isolation and supplemental damping

4.10.4 Performance based design

4.10.4.3 Performance criteria

Delete Table 4.19 and replace with:

Unless Consented to by the Ministry, the displacement capacity of isolator and damping units shall not be less than $1.5d$ plus offset displacement and shall be determined from prototype testing.

Displacement capacity is defined as the displacement that can be achieved without failure. Failure includes but is not limited to the following:

- Component shear failure,
- Component bond failure,
- Surface cracks on elastomers wider or deeper than two-thirds of the cover thickness,
- Material peeling,
- Scoring of stainless steel plate,
- Permanent deformation, or
- Leakage.

Unless Consented to by the Ministry, no strain hardening shall occur prior to $1.25d$ plus offset displacement.

4.10.5 Analysis procedures

4.10.5.3 Uniform-load and single-mode spectral analysis

Replace item d) in first paragraph with:

d) the bridge is not located in a seismic site designation X_F .

4.10.6 Design displacements for seismic and other effects

Delete the second paragraph and replace with the following:

The offset displacement is the resultant of the displacements in each of the two orthogonal directions due to 50% of the deformations due to temperature changes and 100% of the deformations induced by concrete shrinkage and creep.

Commentary: *Deformations from creep and shrinkage may be determined considering the timing of bearing placement or re-setting to mitigate such effects if shown on the plans. The value of 100% for thermal demands in S6:19 in combination with seismic demands for 5% or 10% in 50 years is believed conservative and uncalibrated.*

4.11 Seismic evaluation of existing bridges

Commentary: *The Ministry has established a seismic risk reduction policy for its highway bridges. This policy includes the following initiatives:*

- *Stringent earthquake design standards for planned new bridges.*
- *A program of “seismic retrofitting” to improve the earthquake resistance of existing structures.*

The Ministry has designed bridges to meet modern, evolving earthquake design standards since 1983. These newer bridges may sustain damage but are not expected to collapse in the design earthquake. Structures designed or built prior to 1983, or those having poor seismic detailing or arrangements, are considered potentially vulnerable to collapse or major damage from earthquakes.

In 1989, the Ministry initiated a program of seismic retrofitting to improve the earthquake resistance of existing bridges constructed prior to 1983. The main objectives of the program are as follows:

- *Minimizing the risks of bridge collapse;*
- *Preserving important highway routes for disaster response and economic recovery after earthquakes;*
- *Reducing damage and minimizing loss of life and injury during and after earthquakes.*

A detailed description of the seismic retrofitting program is provided in the report “Bridge Seismic Retrofit Program”, BC Ministry of Transportation & Highways, Engineering Branch, February 2000.

Details are contained in the Ministry document, “Seismic Retrofit Design Criteria”, June 30, 2005.

Seismic Retrofit Criteria going forward will be based on S6-19 as modified in this Supplement.

Since S6-14, S6 has made a major shift in the seismic analysis and design of bridges compared to previous codes. It has moved from the use of a force-based design approach with a single level (475-year design event) to a philosophy of performance-based design using multiple earthquake design levels (475, 975, and 2475-year return period events). The Ministry's seismic retrofit criteria, and project-specific seismic criteria adopted beginning also circa 2005, included performance-based and displacement-based requirements and methods.

The Ministry will use the S6:19 performance-based analysis and design approach for evaluation and retrofit of its bridges, as modified within this Supplement. The basic strategy and philosophy behind the Ministry's seismic retrofit program will remain unchanged.

Sections 4.11 and 4.12 in this Supplement, which build on provisions elsewhere in Chapter 4 and this Supplement, provide the Ministry's general requirements for analysis and design of seismic retrofits that will be used going forward.

4.11.1 General

Add the following paragraph:

Existing bridges shall be evaluated based on performance-based principles using seismic performance levels and hazard levels designated by the Ministry. Seismic evaluations shall assess the expected performance of the bridge at the required hazard levels.

4.11.3 Seismic hazard and evaluation

Add the following paragraph:

Unless otherwise specified by the Ministry, the hazard having a 2% in 50-year probability of exceedance shall be used for seismic evaluation.

Commentary: *The baseline hazard for seismic evaluation of existing bridges prior to S6-14 code was a 10% in 50-year probability of exceedance. The Ministry's objective is to assess and retrofit those bridges in its Seismic Retrofit program that are expected to have remaining economic lives in excess of 20 years following renewal or retrofit, to at least a collapse prevention state for a hazard having a 2% in 50-year probability of exceedance. For bridges expected to have shorter functional lives, but which are targeted for seismic retrofit, then a hazard not lower than 10% in 50-years shall be specified.*

4.11.4 Performance criteria for performance-based design approach

Commentary: *The previous performance criteria for bridge seismic retrofit prior to S6-14 was determined using a staged approach. This staged approach will be used going forward as well. In the current stage, the objective will be to continue to reduce the risk of bridge collapse. The ultimate objective is to work towards achieving performance criteria equivalent to new bridges using a staged approach. There may be aspects of existing bridges that preclude economical achievement of the ultimate objective.*

4.11.5 Performance criteria for force-based design approach

4.11.5.1 General

Delete the second paragraph.

4.11.5.2 Limited evaluation

Delete clause.

Commentary: *The prescribed limited evaluation shall not be used for the seismic assessment or retrofit of Ministry bridges. This does not preclude the reliance on elastic component strengths having adequate reserve margin as a lateral load resisting mechanism in existing bridges. Evaluations shall use displacement-based method using ISPA or NTHA wherever practicable. The latter may be applicable to base isolation or added damping strategies. For screening-level evaluation of bridges as part of seismic retrofit planning and prioritizing, elastic methods may be appropriate. Elastic methods shall not be sufficient analyses for decisions related to renewal / retrofit versus replacement, or as meeting the requirements for seismic assessment to this Supplement.*

4.11.6 Load factors and load combinations for seismic evaluation

Add the following after the second paragraph:

The assessment of biaxial effects on failure modes shall be addressed explicitly in the evaluation of existing bridges.

Commentary: *Biaxial bending in poorly detailed, brittle components may lead to spalling, loss of structural integrity of the core of the member and potential collapse. Evaluation for the potential for these failure modes is therefore essential in existing bridges, and retrofit measures considered must also address this potential.*

4.11.9 Required response modification factor for force-based design approach

Delete clause.

4.11.10 Response modification factor for existing substructure elements

Delete clause.

4.11.12 Bridge access

Modify sentence by deleting “for bridges located in Seismic Performance Category 3”.

Add the following paragraph:

Damage to embankments and abutments shall be evaluated.

4.11.13 Liquefaction of foundation soils

Delete first paragraph, including sub-clauses (a) and (b), and replace with the following:

The potential for liquefaction of the foundation soils shall be evaluated as required to determine performance.

Add the following clauses:

4.11.15 Seismic evaluation report

A Structure Seismic Evaluation Report shall be prepared for Ministry review and acceptance. The report will incorporate findings from a Detailed Condition Assessment Report and a Structure Evaluation Report, when provided or created prior to the creation of the Structure Seismic Evaluation Report. The Structure Seismic Evaluation Report is intended to define all the vulnerabilities for the existing structure and to provide recommendations and cost estimates for seismic retrofit actions to achieve the performance objectives for the site and classification and shall contain the following as a minimum:

- The specified performance objectives.
- A summary of design response spectra and, where applicable, ground motion time histories.
- Desktop assessment of liquefaction at the site for the hazards specified by the Ministry.
- Description of the methodology and parameters for structural and geotechnical assessment.
- Procedures for establishing material properties and design/constructed details, and the methodology used for determining ductility demands and capacities of existing structural components/connections.
- A description of the reference materials used, and all assumptions made as part of this work. Provide recommendations for any additional field and/or desktop work to verify or alter them.
- Identification and prioritization, based on expected performance, of seismically deficient areas of the structure and foundations.
- Description of the current seismic load paths through the structure [typically an elastic Load Path Capacity Assessment], key components, their criticality, behaviour, reliability and their assessed seismic performance.
- Summary of the displacement demands and capacities from the analysis of the current structure.
- Discussion of vulnerable components for the current structure, that could affect use, expected damage, the nature of the associated short-term actions and time to restore service, the type of restored service [emergency vehicle access lane in each direction only, full access with load limits, full access] and the stabilization work and/or full repair work, if applicable, to restore the structure to its pre-event service level.
- Description of recommended conceptual retrofit measures, their capacity improvement ratio, including schematic sketches, quantities, cost estimates, and appropriate back-up data to achieve performance measures.
- Discussion of high demand vulnerable components, for the retrofitted structure, that could affect use, expected damage, the nature of the associated short-term actions and time to restore service, the type of restored service [emergency vehicle access lane in each direction only, full access with load limits, full access] and the stabilization work and/or full repair work, if applicable, to restore the structure to its pre-event service level.

4.12 Seismic rehabilitation

4.12.1 Performance criteria

Add the following paragraphs at the beginning of the clause:

Performance-based design shall be required for all seismic rehabilitation (retrofit) of bridges of all importance classifications and performance categories. The Ministry will designate the importance classification.

Unless otherwise specified by the Ministry, the minimum performance levels to be used for seismic rehabilitation shall be in accordance with Section 4.11.4.

Commentary: *The goal of Clause 4.12 is to identify and implement a cost-effective seismic upgrading strategy that meets the prescribed performance requirements, and which can be integrated into other renewal works planned for each bridge. The Ministry will specify objectives, requirements and implementation staging in project-specific Seismic Criteria. Principles to guide the seismic upgrading strategy include:*

- *The assessment of seismic vulnerabilities and design of upgrading works shall use displacement-based methods wherever applicable. Elastic demands and designs may be unavoidable for some existing bridges, but where used shall provide the performance requirements with an appropriate margin of reserve strength. Force reductions based on ductility factors as in a force-based design approach shall not be used.*
- *Given the limited economic lives of some existing bridges in the retrofit program, seismic upgrades to be implemented are likely to be the best or sole opportunity to upgrade these bridges. The retrofit level to be implemented should in general therefore be implemented as a single stage retrofit, although more than one contract package may be adopted.*
- *For bridges to be renewed to extend their economical lives potentially beyond approximately 20 years, seismic assessments shall be performed, and vulnerabilities shall be identified through analysis and assessment for a 2% in 50-year hazard. Sufficient information including analysis, assessment and retrofit strategy should be completed, based on analyses and methods outlined in this Supplement, such that an informed decision can be made regarding renewal or replacement of the bridge. For any bridge for which a seismic retrofit is contemplated, other than for an initial screening of an inventory of bridges, a displacement-based performance assessment using static pushover models shall be used. Where substructures are found to remain essentially elastic, and whose capacities would not be exceeded, a push-over assessment becomes moot.*

4.12.2 Response modification factor for force-based design approach

Delete the clause.

Commentary: *The Ministry does not use force-based design for seismic rehabilitation.*

Add the following clause:

4.12.5 Seismic retrofit strategy report

A Seismic Retrofit Strategy Report shall be prepared for Ministry review and acceptance. The Seismic Retrofit Strategy Report shall contain the following as a minimum:

- Project-specific seismic retrofit design criteria.
- A summary of design response spectra and, where applicable, ground motion time histories.
- Description of methodology and parameters for structural and geotechnical modelling, analysis, and design.
- Procedures for establishing properties of existing materials and the methodology used for determining capacities of existing structural components.
- Description of the seismic load path through the structure, key components, their importance and behaviour and their assessed seismic performance.
- Summary of the results and demands from the analysis.
- Identification and prioritization of seismically deficient areas of the structure, including geotechnical deficiencies.
- Description of conceptual retrofit measures and their design philosophies including preliminary drawings, estimated costs, appropriate back-up data, and aesthetic considerations.
- Discussion of expected damage and the nature of the repairs anticipated, if applicable, to restore the structure, under traffic as required, to the specified service level.
- Summary of the recommended retrofit scheme to proceed with in the detailed design phase.
- Discussion of the long-term reliability and required maintenance of the proposed retrofit measures.
- All summary test hole/test pit logs.

The report shall be submitted for Ministry review prior to undertaking the detailed design. It shall be updated to include any modifications made as a result of the Ministry review. A final version of the report shall also be provided after construction to include any modifications resulting from the construction work.

5.5 Requirements for specific bridge types 2
5.5.7 Rigid frame and integral abutment types 2

5.5 Requirements for specific bridge types

5.5.7 Rigid frame and integral abutment types

Add the following:

Design and analysis shall follow published design criteria from a recognized source applicable to the type of jointless bridge under consideration.

The designer shall provide details regarding construction constraints, sequencing of work etc. on the Plans. The details for the interface between approach pavement and approach slabs shall be compatible with anticipated movements.

Commentary: *Several examples exist of Ministry bridges with inadequate consideration of the expected thermal movements at the approach pavement and approach slab interface which have led to premature deterioration of the pavement and loss of roadway fill under the approach slab irrespective of whether the approach slab is buried. Consideration of the use of additional control joints and sleeper slab type systems may be warranted for longer integral abutment type bridges.*

Some recognized design guides are:

- BA 42/96 including Amendment No. 1 dated May 2003, Design Manual for Roads and Bridges, ISBN 115524606 [www.tso.co.uk].
- *Integral Bridges: A Fundamental Approach to the Time-Temperature Loading Problem*, George England, David Bush & Neil Tsang, ISBN 0-7277-2845-8.
- *England, G.L., Tsang, N.C.M., Towards the Design of Soil Loading for Integral Bridges- Experimental Solution*, Imperial College London, 2001
- *NJDOT Design Manual for Bridges and Structures, Section 15 – Integral Abutment Bridges.*
- *Ontario Ministry of Transportation, Structural Office Report #SO-96-01, Integral Abutment Bridges*
- *Ontario Ministry of Transportation, Bridge Office Report #BO-99-03, Semi-Integral Abutment Bridges*
- *Ontario Ministry of Transportation, Structural Office Report #SO-99-04, Performance of Integral Abutment Bridges*
- *The 2005 – FHWA Conference: Integral Abutment and Jointless Bridges (IAJB 2005). 2005. U.S. Department of Transportation, Federal Highway Administration, Washington, DC.*

Experience in North America with jointless superstructures of limited backwall height using integral pile-supported end-diaphragms, or semi-integral abutment designs has demonstrated that superstructures of this type may be designed longer than the 60 m limit in BA 42/96, provided that the effects described therein are properly accounted for.

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6.1 Scope

Add the following:

Section 6 - Foundations and geotechnical systems, addresses some construction practice and requirements. The designer shall consider all design-related construction requirements and provide the necessary design information and recommendations in the geotechnical reports and the Plans.

6.2 Definitions

Replace the following:

Double corrosion protection – a proven system of double covering of the tendon to protect against corrosion consisting of encapsulation of the tendon inside a plastic sheath pre-filled under factory conditions with grout or corrosion protection compound (grease or wax) designed to minimize crack width in the pre-grouted assembly. The whole assembly is then grouted into the anchor hole.

Add the following:

Embankment – earth or rock slope, with or without a foundation unit, that has been altered by cuts or fills, structurally stabilized, subject to ground improvement, or drainage modification and is adjacent to, or supporting, a highway or bridge.

Two-Stage MSE Wall – MSE walls where in the first stage, the retention structure is built with a flexible facing, allowing foundation settlement to occur, and the second stage adds a cast-in-place or precast wall facing.

Geosynthetic reinforced soil (GRS) – Geosynthetic reinforced soil (GRS) consists of alternating layers of compacted granular fill and closely spaced (≤ 300 mm) geosynthetic reinforcement.

Note: Geosynthetic reinforced soil (GRS) is internally supported and has distinct behavior from Mechanically stabilized earth (MSE). Reinforcement spacing is the key differentiation between GRS and MSE.

Replace the following:

Geotechnical System – a group of interrelated elements designed to transmit loads to the ground or to retain the ground.

Note: examples of such systems include deep foundations, shallow foundations, retaining structures, ground anchors, embankments, and their components.

Add the following:

Natural slope – the unaltered earth or rock slope adjacent to a highway or structure, where the highway performance depends on the natural slope's performance.

6.4 Design Requirements

6.4.1 Limit states

6.4.1.1 General

Add the following:

Serviceability Limit State (SLS) Combination 1, given in Table 3.1, shall be used for global (overall) stability of embankments, geotechnical systems, and natural slopes which affect the performance of the highway or structure.

6.7 Geotechnical report

6.7.3 Design information

Delete the last sentence and replace it with the following:

Signing and sealing of the Geotechnical report shall be in accordance with the Association of Professional Engineers and Geoscientists of the Province of British Columbia requirements.

Commentary: Requirements for signing and sealing are governed by the Association of Professional Engineers and Geoscientists of the Province of British Columbia.

6.9 Geotechnical resistance

6.9.1 General

Add the following:

The following benchmarks in Table 6.2a provide guidance for determining the Degree of Understanding for use of Table 6.2 for deep foundations:

Table 6.2a
Benchmarks for Degree of Understanding for Deep Foundations

Test Method/ Model	Degree of Understanding		
	Low	Typical	High
Compression			
Static Analysis	<ul style="list-style-type: none"> Design based on SPT blow counts and soil sample descriptions from boreholes representative of conditions at project site. 	<ul style="list-style-type: none"> Design based on SPT blow counts and soil sample descriptions from boreholes representative of conditions at each bridge pier and abutment. 	<ul style="list-style-type: none"> Design based on CPT or BPT data representative of conditions at each bridge pier and abutment. <p>OR</p> <ul style="list-style-type: none"> Design based on BPT data representative of conditions at each bridge pier and abutment, and Measure bounce chamber pressure and consider BPT friction.
Static Test	<ul style="list-style-type: none"> Design based on a single test pile for bridge pier as per ASTM D1143, and Results extrapolated to other bridge piers by consideration of borehole, CPT or BPT data, and Test pile size and length shall be similar to the production piles. 	<ul style="list-style-type: none"> Design based on a single test pile for bridge pier as per ASTM D1143, and Test pile instrumented with at least a tell-tale. Force applied at pile head above ground, and Test pile size and length shall be similar to the production pile, but toe condition and installation method shall be the same as production piles, and Results extrapolated to other bridge piers by consideration of borehole, CPT or BPT data. <p>OR</p> <ul style="list-style-type: none"> Design based on a single pile test with single level high capacity, sacrificial loading unit embedded in the foundation unit instrumented with force measurement, and Test pile size and length shall be similar to the production pile, but installation method shall be the same as production piles, and Results extrapolated to other bridge piers by consideration of borehole, CPT or BPT data. 	<ul style="list-style-type: none"> Design based on a single test pile for bridge pier as per ASTM D1143, if bridge piers are separated less than 500 m, and Design based on two test piles for bridge pier as per ASTM D1143, if bridge piers are separated more than 500 m, and Test pile instrumented with at least toe tell-tale and strain gauges attached to pile at appropriate elevations. Force applied at pile head above ground, and Test pile length shall be similar to the production pile, but size, installation method, and toe condition shall be the same as production piles, and Results extrapolated to other bridge piers by consideration of borehole, CPT or BPT data. <p>OR</p> <ul style="list-style-type: none"> Design based on one test pile with two levels of high-capacity, sacrificial loading units embedded in the foundation unit if bridge piers are separated less than 500 m, and Design based on two test piles with two levels of high-capacity, sacrificial loading units embedded in the foundation unit if bridge piers are separated more than 500 m, and Test pile size and length shall be similar to the production pile,

			<p>but installation method shall be the same, and</p> <ul style="list-style-type: none"> • Results extrapolated to other bridge piers by consideration of borehole, CPT or BPT data.
Dynamic Analysis	<ul style="list-style-type: none"> • Wave equation analysis (WEAP unless otherwise Consented to by the Ministry) performed before construction for multiple driving systems <p>OR</p> <ul style="list-style-type: none"> • Wave equation analysis (WEAP unless otherwise Consented to by the Ministry) performed using pile driving blow count data from previous installations at the site. 	<ul style="list-style-type: none"> • Wave equation analysis (WEAP unless Consented to by the Ministry) performed with pile driving blow count data on production piles for the full depth and known driving system. 	<ul style="list-style-type: none"> • Wave equation analysis (WEAP unless Consented to by the Ministry) performed using pile driving blow count data on production piles for full depth, damage observations and measured blow rate data for diesel hammer or using known efficiency for a hydraulic hammer.
Dynamic Test	<ul style="list-style-type: none"> • Pile dynamic testing (PDA unless otherwise Consented to by the Ministry) and dynamic analysis (CAPWAP unless Consented to by the Ministry) conducted on an adjacent bridge pier or abutment used with pile driving blow count data obtained for the pile. <p>OR</p> <ul style="list-style-type: none"> • Design based on a single rapid load test on a pile for bridge pier or abutment as per ASTM D7383, and • Results extrapolated to other bridge piers and abutments by consideration of borehole, CPT or BPT data, and • Test pile size and toe condition may not be the same as production piles. 	<ul style="list-style-type: none"> • Pile dynamic testing (PDA unless otherwise Consented to by the Ministry) and dynamic analysis (CAPWAP unless otherwise Consented to by the Ministry) conducted at each bridge pier and each abutment, and • blow count data for other piles at the same piers or abutments collected with a hammer having consistent driving energy. 	<ul style="list-style-type: none"> • Pile dynamic testing (PDA unless otherwise Consented to by the Ministry) and dynamic analysis (CAPWAP unless otherwise Consented to by the Ministry) conducted at each bridge pier and each abutment, and • Have borehole, CPT or BPT data to define the ground conditions, and • Have consistent driving energy delivered from the driving system with measured blow rate data for diesel hammers or known efficiency for a hydraulic hammer.

Table 6.2a (continued)
Benchmarks for Degree of Understanding for Deep Foundations

Test Method/ Model	Degree of Understanding		
	Low	Typical	High
Tension			
Static Analysis	<ul style="list-style-type: none"> Design based on SPT blow counts and soil sample descriptions from boreholes representative of conditions at project site. 	<ul style="list-style-type: none"> Design based on SPT blow counts and soil sample descriptions from boreholes representative of conditions at each bridge pier and abutment. 	<ul style="list-style-type: none"> Design based on CPT or iBPT data representative of conditions at each bridge pier and abutment. <p>OR</p> <ul style="list-style-type: none"> Design based on BPT data representative of conditions at each bridge pier and abutment, and Measure bounce chamber pressure and consider BPT friction.
Static Testing	<ul style="list-style-type: none"> Design based on a single test pile for bridge pier as per ASTM D3689, and Results extrapolated to other bridge piers and abutments by consideration of borehole or CPT data, and Test pile size and length shall be similar to the production piles. 	<ul style="list-style-type: none"> Design based on a single test pile for a bridge pier as per ASTM D3689, and Test pile size and length shall be similar to the production piles, but the installation method shall be the same as production piles, and Results extrapolated to other bridge piers by consideration of borehole, CPT or BPT data. <p>OR</p> <ul style="list-style-type: none"> Design based on a single pile test with single level high-capacity, sacrificial loading unit embedded in the foundation unit instrumented with force measurement, and Test pile size and length shall be similar, but installation method shall be the same as production pile, and Results extrapolated to other bridge piers by consideration of borehole, CPT or BPT data. 	<ul style="list-style-type: none"> Design based on a single test pile for bridge pier as per ASTM D3689, if bridge piers are separated less than 500 m, and Design based on two test piles for bridge pier as per ASTM D3689, if bridge piers are separated more than 500 m, and Test pile size and installation method shall be the same as production piles, and Test pile length shall be similar to production piles, and Results extrapolated to other bridge piers by consideration of borehole, CPT or BPT data. <p>OR</p> <ul style="list-style-type: none"> Design based on one test pile with two levels of high capacity, sacrificial loading units embedded in the foundation unit instrumented with force measurements, if bridge piers are separated less than 500 m, and Design based on two test piles with two levels of high capacity, sacrificial loading units embedded in the foundation unit instrumented with force measurements, if bridge piers are separated more than 500 m, and Test pile size and length shall be similar to production piles, but installation method shall be the same, and Results extrapolated to other bridge piers by consideration of borehole, CPT or BPT data.

Note: Pile relaxation must be considered when using pile driving blow count or PDA data, for example, in some very stiff soils or some weak rocks. Restrike data should be used if these conditions may be present.

Designs shall be based on information available at the time of design; and higher resistance factors shall not be used based on the intent to do load testing or dynamic monitoring during construction. Higher resistance factors may be used based on data from load testing or dynamic monitoring that has been done to confirm resistance during construction.

Design and acceptance procedures for deep foundations shall be based upon the use of blow counts established from dynamic analysis or dynamic testing unless otherwise Consented to by the Ministry.

In Table 6.2 under the column entitled “Application”, replace “Embankments (fill)” with “Embankments”

The geotechnical resistance factors given in Table 6.2 for Global Stability of Embankments shall be used for geotechnical systems, and for natural slopes that affect the performance of the highway or bridge. The resistance factors given in Table 6.2 have been developed with the intent of achieving the following Factors of Safety (FOS) against global failure:

Table 6.2b
Resistance Factors, Consequence Factors and Factors of Safety for Global Stability of Embankments, Geotechnical systems and Natural slopes that affect the performance of the highway or bridge
 (to be used in conjunction with Table 6.2)

Degree of Understanding	Low			Typical			High		
Resistance Factors for Global Stability – Permanent from S6-19	0.60			0.65			0.70		
Resistance Factors for Global Stability – Temporary from S6-19	0.70			0.75			0.80		
Consequence Factor from S6-19	High	Typical	Low	High	Typical	Low	High	Typical	Low
	0.90	1.00	1.15	0.90	1.00	1.15	0.90	1.00	1.15
FOS for Global Stability – Permanent	1.85	1.67	1.45	1.71	1.54	1.34	1.59	1.43	1.24
FOS for Global Stability – Temporary	1.59	1.43	1.24	1.48	1.33	1.16	1.39	1.25	1.09

The resistance and consequence factors (and the corresponding FOS values) in Table 6.2b shall be used with the load factors specified for the SLS Combination 1 in Table 3.1 of Chapter 3. This use is consistent with the methodology followed when computing the FOS values on global stability of embankments using the currently available computer software.

The FOS values for the Temporary condition apply to short term loading conditions lasting up to two years. Temporary condition does not apply to seismic or other transient loading conditions.

The following benchmarks in Table 6.2c provide guidance for determining the Degree of Understanding for use of Table 6.2 for global stability of embankments:

Commentary: *Assessment of existing embankments and/or natural slopes not altered by the project may be required on a case-by-case basis as requested by the Ministry.*

**Table 6.2c
Benchmarks for Degree of Understanding for Embankments**

Degree of Understanding	Low Understanding	Typical Understanding	High Understanding
Global Stability	<ul style="list-style-type: none"> • Shear strength parameters established based on subsurface data from nearby sites and published correlations with the consistency/density of site soils supplemented with geological evidence, and • Stability of embankment evaluated using accepted computer software that incorporates the method of slices and limit equilibrium method of analysis, and • Embankment fill density and strength based on Ministry standard specifications and published parameters. 	<ul style="list-style-type: none"> • Shear strength parameters established based on a minimum of one borehole and published correlations with the consistency/density of site soils supplemented with geological evidence, and • Stability of embankment evaluated using accepted computer software that incorporates the method of slices and limit equilibrium method of analysis, and • FOS computed for an inferred groundwater profile, and • Embankment fill density and strength based on Ministry standard specifications and published parameters. 	<ul style="list-style-type: none"> • Site-specific soil stratigraphy and consistency/density of soils established based on a minimum of two boreholes or 2 CPTs along the slope profile with laboratory testing to determine shear strength parameters, and • Groundwater profile established based on in-situ measurements, and • Low spatial variability of the subsurface soil conditions, and • Stability of embankment evaluated using accepted computer software that incorporates the method of slices and limit equilibrium method of analysis. Both force and moment equilibrium of slices shall be satisfied, and • Sensitivity of the computed FOS evaluated for differing groundwater profiles and anticipated variations in shear strength parameters, and • Embankment fill density and strength based on Ministry standard specifications and laboratory or in-situ testing. Fills placed with engineering supervision.

Note: For low-volume road bridges, modifications to the resistance factors may be considered when Consented to by the Ministry.

6.13 Integral and semi-integral abutments

6.13.3 Design requirements

6.13.3.1 General

Delete the third paragraph and replace with:

Sufficient lateral pile restraint shall be provided for integral abutments. Integral abutments shall not be used where the soil is susceptible to liquefaction, slope instability, sloughing, or boiling unless Consented to by the Ministry.

6.14 Seismic design

6.14.2 Seismic design and performance requirements

6.14.2.1 Performance requirements for foundations and geotechnical systems

Delete the third paragraph and replace with:

Unless specified otherwise by the Ministry, the following seismic performance criteria shall be met for geotechnical systems within the bridge approach embankment interface zone:

- a) Lifeline geotechnical systems shall have
 - i. 100% of the travelled lanes available for use following ground motions with a return period of at least 975 years. Any repair work shall not cause service disruption; and
 - ii. 50% of the travelled lanes available for use following ground motions with a return period of at least 2475 years. If damaged, normal service shall be restorable within one month.
- b) Major-route geotechnical systems shall have 100% of the travelled lanes available for use following ground motions with a return period of at least 475 years. Any repair work shall not cause service disruption.
- c) Other geotechnical systems shall have 50% of the travelled lanes, but not less than one, available for use following ground motions with a return period of at least 475 years. If damaged, normal service shall be restorable within one month.
- d) Lifeline, major route, and other geotechnical systems shall meet the life safety requirement of no collapse following ground motions with a return period of at least 2475 years and it shall be possible to evacuate the bridge safely.

Commentary: *The potential impact of adjacent structures or geotechnical systems to the embankment bridge interface zone on the seismic performance of the embankment bridge interface zone should be considered.*

6.14.2.3 Seismic performance criteria

Delete the clause and replace with:

Unless otherwise specified by the Ministry, the following seismic performance criteria shall be met for geotechnical systems outside the bridge approach embankment interface zone if the geotechnical system falls within Seismic Performance Category 2 or 3, as defined in Table 4.10:

- a) Lifeline geotechnical systems shall have at least 50% of the travelled lanes, but not less than one, available for use following ground motions with a return period of at least 975 years. If damaged, normal service shall be restorable within one month.
- b) Major-route geotechnical systems shall have at least 50% of the travelled lanes, but not less than one, available for use following ground motions with a return period of at least 475 years. If damaged, normal service shall be restorable within one month.

- c) Other geotechnical systems shall have at least 50% of the travelled lanes, but not less than one, restorable for use within one month following ground motions with a return period of at least 475 years.
- d) Large permanent foundation deformations of geotechnical systems may be acceptable provided the specified post-seismic travelled lane functionality can be achieved.
- e) Lifeline, major route, and other retaining structures taller than 6 m and in Seismic Performance Category 3 shall meet the life safety requirement of no collapse following ground motions with a return period of 2475 years.

Commentary: Retaining structures in item (e) includes, but is not limited to, such types as retaining walls, light weight fill, MSE systems, reinforced soil systems, and soil-nail systems.

The Ministry may approve alternate requirements for the seismic design of geotechnical systems on a case-by-case basis by using a risk-based approach that considers the economic and societal consequences of the performance and the seismic hazard levels. For instance, the Ministry may consider alternative requirements for seismic performance of a retaining wall that is dependent on the performance of an existing large slope where the wall has little destabilizing effect on the slope and where stabilization of the slope for seismic loading is impractical.

Seismic design should be considered on a case-by-case basis for temporary geotechnical systems exceeding two years in service. For temporary lifeline and major route geotechnical systems, a return period of at least 100 years should be considered.

Seismic design should be considered for all partially constructed geotechnical systems with construction exceeding two years. For lifeline and major route geotechnical systems, a return period of at least 100 years should be considered.

6.14.3 Geotechnical investigation

Replace item a) with the following:

- a) site designation in accordance with Clause 4.4.3.2;

6.14.4 Geotechnical resistance factors and analysis

6.14.4.1 Geotechnical resistance factors

Replace the contents of Table 6.3 with:

Design scenario	Seismic resistance factor*
Capacity-protected elements	ϕ_{gu} static values + 0.2
Forced-based design	ϕ_{gu} static values + 0.2
Performance-based design	ϕ_{gu} static values + 0.2

* ϕ_{gu} static values from Table 6.2; in no case shall the seismic resistance factor be greater than 1.0.

Commentary: For analysis purposes, a nominal resistance factor of 1.0 may be used if a sensitivity analysis using appropriate bounds on soil parameters is completed.

6.14.4.2 Analysis methods

Replace the second paragraph with:

The geotechnical analysis and design methods shall be subjected to independent peer review when specified herein or as directed by the Ministry.

Delete the third and fourth paragraph and replace with:

Where the potential for liquefaction is present, the requirements of Clause 6.14.8 for liquefaction evaluation and effects shall be met.

For retaining structures, the minimum analysis requirement shall be as specified in Table 6.14.4.2-1. For embankments, the minimum analysis requirement shall be as specified in Table 6.14.4.2-2.

**Table 6.14.4.2-1
Minimum analysis requirement of retaining structures (all zones)**

SPC	Lifeline		Major-route		Other	
	$H_s \leq 6m$	$H_s > 6m$	$H_s \leq 6m$	$H_s > 6m$	$H_s \leq 6m$	$H_s > 6m$
1	No seismic analysis is required.					
2	SDBM	RDAM	SDBM/FBM*		FBM	
3	SDBM	RDAM	SDBM	RDAM	SDBM	RDAM

*SDBM shall be used within the bridge interface zone where performance-based design is used for a bridge.

**Table 6.14.4.2-2
Minimum analysis requirement for embankments**

SPC	Lifeline			Major-route			Other		
	Within IZ		Outside IZ	Within IZ		Outside IZ	Within IZ		Outside IZ
	$H_s \leq 18m$	$H_s > 18m$		$H_s \leq 18m$	$H_s > 18m$		$H_s \leq 18m$	$H_s > 18m$	
1	No seismic analysis is required.								
2	SDBM	RDAM	SDBM	SDBM/PSA*		PSA	PSA		
3	SDBM	RDAM	SDBM	SDBM	RDAM	SDBM	SDBM	RDAM	PSA

*SDBM shall be used within the bridge interface zone where performance-based design is used for a bridge.

Legend:

Outside IZ = Outside the embankment bridge interface zone

Within IZ = Within the embankment bridge interface zone

SDBM = Simplified displacement-based method including Newmark-based type methods.

FBM = Force-based method including Mononobe-Okabe or Generalized Limit Equilibrium (GLE). Global stability shall be assessed by PSA.

RDAM = Rigorous dynamic analysis method including complex 2D or 3D finite element or finite difference dynamic analysis.

PSA = Pseudo static analysis. In this type of analysis, the seismic loading is modeled as a statically applied inertial force, the magnitude of which is a product of a seismic coefficient and the weight of the potential sliding mass.

H_s = Exposed height of retaining walls, top of embankment to bottom of embankment, or in the case of a geotechnical system, height of retaining wall plus embankment height

When FBM is used, and a retaining structure is able to move horizontally a minimum of 50 mm, without compromising the performance of the retaining structure or adjacent structures, the horizontal and vertical seismic coefficients shall not be less than one-half of the corresponding peak ground accelerations at the ground surface as estimated using Clause 4.4.3.3.

When PSA is used for stability analysis of embankments or retaining structures, the horizontal and vertical seismic coefficients shall not be less than one-half of the corresponding peak ground accelerations at ground surface as estimated using Clause 4.4.3.3. SDBM or RDA shall be used when the pseudo-static limit equilibrium analysis indicates a factor of safety less than 1.3.

When RDAM is used, the following requirement shall be met:

1. SDBM shall also be conducted for comparison.
2. The ground motions shall be established according to criteria 4.4.3.6.
3. A high degree of site understanding as defined in Clause 6.5.3 is required.

Commentary: *NCHRP Report 611 – Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes and Embankments provides methods for the seismic analysis and design of embankments and slopes using the displacement-based approach.*

Geotechnical systems such as abutments, retaining walls, and embankments are often designed using an equivalent seismic coefficient generally varying between 0.5 and 1.0 of the peak ground accelerations and assuming rigid behavior of the soil mass. Permanent deformations may be ignored when the pseudo static equilibrium analysis indicates a factor of safety greater than 1.3.

6.14.6 Deep foundations

6.14.6.1 Analysis

Delete the third paragraph and replace with:

Piles shall be explicitly incorporated into the structural model when piles are subjected to liquefaction induced kinematic effects.

Add the following:

Sensitivity studies using appropriate bounds on strength and stiffness used in soil-pile interaction modelling shall be carried out during design to exclude unacceptable failure modes, or unacceptable performance and/or unrealistic performance of the foundations or structure.

Reduction in lateral soil resistance developed at the soil-pile interface due to soil liquefaction shall be incorporated in the design. The effect of liquefaction on both the shape and the magnitude of the p-y curves shall be considered.

Commentary: Consider the soil layers with excess porewater pressure ratio (r_u) of 0.7 or higher as liquefied. The p-y curves for liquefiable soil layers may be developed using soft-clay p-y models with the residual strength of the liquefied soil. For soil layers with r_u less than 0.7, a reduction factor of $(1 - r_u)$ should be used on the static (pre-liquefaction) p-y curves to model the softened conditions.

An alternative method with SPT N data may be considered as follows:

When an excess pore pressure ratio (r_u) close to 100% is predicted in a given foundation soil layer, the soil reaction computed from the non-liquefied “p-y” curve should be reduced by multiplying by the p-multiplier (m_p) shown below:

$$m_p = 0.0031N + 0.00034N^2$$

where, N = clean sand equivalent corrected blow count $SPT(N_1)_{60cs}$

Where the predicted excess pore pressure ratio in a given soil layer is less than 100%, the p-multiplier should be proportionally scaled by the ratio of $100/r_u$ for that layer.

Modification to the p-y curves to account for the weakening effect the liquefied soil has on overlying and underlying non-liquefied strata should be considered [Ref. CALTRANS Memo to Designers 20-15 Attachment 1, dated May 2017].

The sensitivity of the strength and stiffness of the p-y curves on the predicted performance of piled foundations should be assessed during design by factoring the values by $\frac{1}{2}$ to 2 as recommended by “Recommended Design Practice for Pile Foundations in Laterally Spreading Ground” Ashford (2011).

6.14.6.3 Axial resistance

Delete and replace with:

The axial resistance of deep foundations in liquefiable soil shall be evaluated for the following conditions:

1. The factored axial resistance in the liquefied and non-liquefied conditions shall be greater than the combination of dead load, seismic demand and the accompanying live load specified in Table 4.16.
2. The factored axial resistance in the post-liquefaction condition shall be greater than the combination of down drag, unfactored dead load and the accompanying live load specified in Table 4.16.

Commentary: The Ministry uses performance-based design for deep foundations in liquefiable soil.

Geotechnical resistance factors are identified in Table 6.3 of the BC Supplement to CHBDC S6:19.

The dead load used for the liquefied and non-liquefied conditions includes vertical acceleration which may be accounted for by a factor or be explicitly modelled in the analysis.

6.14.7 Abutments and retaining walls

6.14.7.1 Abutment and approach fill interaction

Delete the second paragraph and replace with:

The seismic design of abutments shall consider:

- a) forces arising from seismically induced lateral earth pressures in accordance with Clause 6.14.7.2;
- b) additional forces arising from wall inertia effects, including the weight of soil that is immediately above the heel of the wall; and
- c) the transfer of seismic forces from the superstructure through bearing supports.

Add the following:

Unless Consented to by the Ministry, design of abutments shall include the following two cases:

1. Combine 100 percent of forces obtained from (a) with 50 percent of forces obtained from (b) and (c)
2. Combine 50 percent of forces obtained from (a), but not less than the static active pressure, with 100 percent of forces obtained from (b) and (c)

If the inertial load of the bridge pushes the abutment into the backfill, the abutment shall be designed for the forces arising from the passive pressure condition and forces obtained from (b) and (c).

Delete the first sentence of the last paragraph and replace with:

Approach slabs providing a structural transition from approach fills to abutments shall be in accordance with Clause 1.7.2 of the BC Supplement to CHBDC S6:19.

Commentary: *Research carried out using centrifuge tests of reduced scale walls by Atik & Sitar (2010) has indicated that the wall inertial forces and lateral earth pressures can be out of phase. The load combinations above reflect an approximation to capture this behaviour and are taken from AASHTO LRFD Bridge Design Specification, 9th Edition.*

The effective abutment stiffness and ultimate passive resistance can be determined based on CALTRANS SDC Ver.2 Clause 6.3.1.2. The backfill passive pressure force varies nonlinearly with the abutment displacement. The bilinear model in CALTRANS SDC Ver. 2 Clause 6.3.1.2 is based on experimental studies using engineered structural backfill to a relative compaction of at least 95%.

6.14.7.2 Seismic forces on retaining walls

Add the following:

The point of application of the dynamic portion of the earth pressure is 0.6 H above the base of the wall.

6.14.8 Liquefaction

6.14.8.1 Evaluation of liquefaction potential

6.14.8.1.2 Liquefaction assessment

Delete and replace with:

A liquefaction assessment shall be conducted for all foundation soils and adjacent soils that impact the behaviour of structures, bridges and geotechnical systems.

Future ground water levels due to climate change effects over the full life of the structure shall be considered in the liquefaction assessment.

Commentary: *Liquefaction should be considered for all structures where failure could have safety implications for highways users. This would include sign bridges, large cantilever sign structures and high-mast lighting but would not typically include standard signal and luminaire poles.*

6.14.8.1.3 Liquefaction potential of foundation soils

Delete the last two paragraphs and add the following:

When evaluating liquefaction potential with various methods, each method shall be followed independently.

The evaluation shall incorporate variability of the in-situ penetration resistance and earthquake ground motions.

Commentary: *Saturated low-plastic silts exhibiting sand-like behaviour, sands, sand-silt mixtures, gravels confined by low permeability soil layers, and gravel-sand mixtures, all have a high potential for liquefaction. Silts and sand-silt mixtures with $PI < 7$ are classified as soils exhibiting sand-like behaviour.*

There are two commonly accepted methods of evaluation for liquefaction potential referred to as CPT-based and SPT-based methods. Other evaluation methods, including Vs-based liquefaction evaluation, may only be used for high level initial screening.

The CPT-based method of assessment for liquefaction potential is preferred because of the repeatability of the test, production of near continuous penetration resistance and pore pressure profiles, accurate identification of soil stratigraphy, and the availability of a data base correlating CPT resistance to liquefaction triggering from past earthquakes. The CPTs should be paired with boreholes to correlate/confirm soil types and fines content.

Use the SPT-based method of assessment for liquefaction potential, when SPT blow counts or equivalent SPT blow counts are available from BPTs, when CPT data is not available or CPTs are not feasible in the soils investigated. Energy measurements are required when SPT data is used for assessment for liquefaction potential. Site-specific correlations of energy corrected BPT blow counts with equivalent SPT blow counts are required when using BPT data in assessment for liquefaction potential.

When both SPT and CPT results are available for a given site, CPT data may be correlated to SPT data or vice versa and used along with the applicable method of assessment for liquefaction potential.

Alternatively, the assessment for liquefaction potential results may be weighted using engineering judgement to account for epistemic uncertainty associated with calculating the representative cyclic resistance ratio profile. CPT data should not be converted to equivalent SPT values for later use with the SPT-based liquefaction triggering method.

The assessment for liquefaction potential using the simplified stress-based method should be carried out in a manner consistent with how the method was developed. Using techniques and adjustment factors from one variant of a method with other variants is not appropriate and shall not be performed.

The evaluation for liquefaction potential shall be conducted in accordance with the following table:

Table: 6.14.8.1.3
Minimum methods for evaluation of liquefaction potential *

SPC	Lifeline	Major route	Other
1	Routine	Simplified	Simplified
2	Routine	Simplified	Simplified
3	Rigorous	Routine	Simplified

* More complex evaluation methods may be used when agreed to or directed by the Ministry.

Simplified: Simplified analysis shall be based on “Method 1: Simplified stress-based method of analysis” as per the CHBDC commentary C.6.14.8.1 using following criteria:

1. Peak ground acceleration shall be adjusted for the site using non-liquefied soil properties. In SPC=1, PGA corresponds to the ground motions with a return period of 2475 years.
2. Magnitude shall be the mean earthquake magnitude obtained from the de-aggregation of PGA.
3. At least a typical degree of site understanding as defined in Clause 6.5.3 is required.

Routine: Routine analysis shall be based on the 1D dynamic site response analysis with equivalent linear models using non-liquefied soil parameters satisfying the following criteria:

1. Simplified method shall be conducted as well for comparison.
2. The ground motions for liquefaction analysis shall be established according to the criteria in Clause 4.4.3.6.
3. A typical degree of site understanding as defined in Clause 6.5.3 is required and downhole shear wave velocity measurement shall be collected.

Rigorous: Rigorous analysis shall be based on 2D or 3D non-linear effective stress analysis using the following criteria:

1. Address pre-triggering, triggering, and post-triggering aspects of liquefaction.
2. Routine 1D analysis shall also be conducted for comparison.
3. The ground motions shall be established according to the criteria in Clause 4.4.3.6.
4. A high degree of site understanding as defined in Clause 6.5.3 is required and downhole shear wave velocity measurement shall be collected.
5. Requires an independent peer review as per Clause 4.4.6.1.

For Routine and Rigorous analysis, sensitivity studies shall be completed to mitigate the potential for unacceptable failure modes and unrealistic performance of the foundations, structures, and

geotechnical systems. The sensitivity studies shall consider variations of the soil, foundation, and structure stiffness and strength parameters including the assumed depth to firm ground.

Commentary: *For Routine analysis, the equivalent-linear model provides reasonable results for strains less than about 1%. Equivalent-linear analysis should be used with caution where large strains are likely to occur.*

When Routine or Rigorous analysis is required as per Table 6.14.8.1.3, such analysis can be omitted if a liquefaction potential assessment, conducted using Simplified analysis methods, indicates no liquefaction potential with a high factor of safety.

Computer programs with non-linear effective-stress models are available to assess liquefaction triggering and the consequences of liquefaction. Computer programs capable of modelling pre-triggering, triggering, and post-triggering aspects of soil liquefaction responses are considered suitable for Rigorous analyses. Also, the effects of soil-structural interaction and ground improvement can be included.

Detailed geological and geotechnical site characterization, selection and calibration of constitutive models used in analysis, determination of material parameters and their spatial variability, general limitations of numerical modeling, development of input ground motions that appropriately reflect the seismic hazard of the bridge site, and detailed documentation of methodology, assumptions and findings are important factors affecting the quality of the analysis results. Practitioners should refer to Boulanger and Beaty (2017) for more details on checks and balances required when conducting Rigorous analysis.

For lifeline structures and structures in Seismic Performance Category 3, when uncertainty exists with regards to cyclic resistance of fine-grained soils, the liquefaction susceptibility should be evaluated using laboratory cyclic shear testing of representative undisturbed soil samples.

6.14.8.2 Effects on bridge foundations, culverts, and geotechnical systems

Revise this clause as follows:

Change “culverts” to “culverts and buried structures” at all occurrences.

6.14.8.2.1 General

Add to the existing clause:

If liquefiable soils are identified, then the methods of analysis for estimating liquefaction-induced ground movements shall be based on Table 6.14.8.2.1.

Table 6.14.8.2.1

Minimum methods of estimating liquefaction-induced ground movements*

SPC	Lifeline	Major route	Other
1	Simplified	Simplified	Simplified
2	Simplified	Simplified	Simplified
3	Rigorous	Simplified	Simplified

* More complex evaluation methods may be used when agreed to or directed by the Ministry.

Simplified: Simplified method includes empirical-based approaches, semi-empirical approaches, and Newmark-based analysis using the following criteria:

- Peak ground acceleration shall be based on non-liquefied soil properties. In SPC=1, PGA corresponds to the ground motions with a return period of 2475 years.
- Ground acceleration shall be based on the method used in Clause 6.14.8.1.3.
- Magnitude shall be the same as used in Clause 6.14.8.1.3.
- A typical degree of site understanding or better as defined in Clause 6.5.3 is required.

Commentary: *Excessive load or displacement demands caused by lateral spreading and settlements are commonly mitigated using ground improvement techniques or structural enhancement. Both options should be considered to develop the most appropriate solution. CALTRANS- “Memo To Designer 20-15, Lateral Spreading Analysis for New and Existing Bridges” provides guidance on how to calculate the foundation restraining action.*

In addition to Youd et al. (2002) and Newmark-based analysis using residual strength in the liquefied condition, semi-empirical approaches, such as Faris et al. 2006, Zhang et al. 2004, Idriss and Boulanger 2008, can be considered to estimate liquefaction induced lateral movement and settlements.

For estimates of liquefaction induced lateral displacement using the simplified method, at least two approaches from the above methods should be selected to evaluate a likely range of potential lateral displacements. Engineering judgement should be used to determine lateral displacement values to be used in the assessment of the structure or geotechnical system performance. The assumptions, limitations, and applicability of the chosen methodologies should be assessed.

Rigorous: Rigorous analysis shall be based on 2D or 3D non-linear effective stress analysis or non-linear total-stress analysis using following criteria:

1. The type of analysis shall be Consented to by the Ministry.
2. An independent peer review shall be carried out.
3. The analysis shall address pre-triggering, triggering, and post-triggering aspects of liquefaction.
4. The Simplified method of analysis shall be conducted for comparison.
5. The ground motions shall be the same as those used for the rigorous analysis for liquefaction potential assessment from Clause 6.14.8.1.3.
6. A high degree of site understanding as defined in Clause 6.5.3 is required.
7. Sensitivity analysis shall be undertaken of the effects of liquefaction using a deformation range from one-half to double the deformation amount obtained from the rigorous analysis.

Commentary: *Simplified and Rigorous analysis include the evaluation for liquefaction induced flow failure using post-liquefaction soil parameters. The minimum factor of safety against flow slide failure should be 1.0.*

The state-of-practice constitutive models used in evaluating the consequences of soil liquefaction such as lateral displacements predict different results due to epistemic uncertainty. To address the epistemic uncertainty, rigorous analysis may be carried out using more than one state-of-practice constitutive model and the results may be weighted using engineering judgement when estimating the consequences such as displacements.

6.14.8.2.2 Liquefaction around bridge foundations

Add the following to subclause (b):

Effects of increased spectral accelerations at periods longer than 1 second due to soil liquefaction shall be evaluated by the designer using 2D or 3D rigorous ground response analysis.

6.14.8.2.3 Mitigation measures

Add the following:

Where soil improvement is achieved using vertical reinforcement, as referred to in Clause 6.14.8.2.3 (b), by installing rigid inclusions with a load transfer platform constructed between the underside of the foundation and the top of rigid inclusions, the rigid inclusions shall be designed to carry the vertical and lateral gravity and seismic loads.

The loads transferred from the bridge foundation onto the rigid inclusions shall be assessed based on Rigorous 2D or 3D methods as per Clause 6.14.8.1.3.

Commentary: *There is a recent trend of using rigid inclusions to vertically reinforce foundation soils to enhance both vertical and lateral stiffness of foundation soils. These designs utilize shallow foundations that are not structurally connected to the rigid inclusions. The foundation system involves the construction of a load transfer platform, often in the form of a compacted granular layer, between the underside of the foundation and the rigid inclusions. The composite foundation system eliminates the need to design for large bending moments and tensile forces transferred at the pile cap-deep foundation interface when using conventional pile foundations. Designers should carry checks to confirm that the rigid inclusions have capacity to support the vertical and lateral loads and displacements imposed on them. The unreinforced rigid inclusions are susceptible to brittle failure and therefore the tension and shear stresses in rigid inclusions for both static and seismic loads transferred from the geotechnical/foundation system are required to be evaluated during the design.*

6.14.8.3 Combined kinematics and inertial loads

Delete the second and third paragraphs and replace with:

All bridges in Seismic Performance Category 2 and 3 shall consider the potential simultaneous occurrence of inertial loads from the structure and kinematic loads on foundations, considering the phasing and locations of these loads on foundation elements. The foundations shall be designed such that the structural performance is acceptable when subjected to combined kinematic and inertial loads.

In the absence of a Rigorous soil-structure interaction analysis of the soil-foundation system, the effects of kinematic loading shall be evaluated and combined with inertial loading as follows:

1. 100% kinematic demand \pm 50% inertial demand

Inertial demands shall be computed from the requirement in Clause 6.14.8.2.2 (b).

In cases where NTHA is used, kinematic effects may be incorporated by applying displacement time-histories obtained from Rigorous analysis to the base of the non-linear p-y springs (ref. Clause 6.14.8.2.1) distributed along the piles of the structural model.

The percent contribution of inertial loads identified above may be reduced when supported by Rigorous soil-structure interaction analysis of the soil-foundation system. Such analyses shall be subjected to an independent peer review and Consented to by the Ministry. The inertial contribution shall not be less than 25%.

In cases where soil softening does not reduce the inertial effect, then a special assessment shall be undertaken to develop an appropriate combination of inertial plus the applicable kinematic effects.

Commentary: *It is important to recognize that the combination of kinematic loads with inertial loads in a pseudo-static analysis will be subject to considerable uncertainties. Emergence of well-established criteria for the combination of these loads is highly unlikely.*

The contribution of inertial loads to be combined with the kinematic loads has been shown to be dependent on a number of complex factors including, but not limited to, the following (ref. Koshravifar & Nasr, 2021 submitted for publication):

- a) *Differences in the location of maximum pile bending moments due to kinematic and inertial load: the depth at which the maximum bending moment occurs due to kinematic loads that are distributed along the embedded length of piles is different from the depth at which the maximum bending moment occurs due to inertial loads applied at the top of the piles (ASCE 64-14).*
- b) *Depth of soil liquefaction: the inertial loads on foundations installed in sites with shallow soil liquefaction are anticipated to be larger than for sites with deep soil liquefaction.*
- c) *Duration of ground shaking: the inertial loads from long-duration ground motions are likely to be larger than from short-duration ground motions, since soil liquefaction may be triggered earlier on during strong shaking in sites subjected to long-duration ground motions. On average, the increase is reported to be about 15% (ref. Koshravifar & Nasr, 2021).*
- d) *Pile groups versus individual piles: the inertial loads on piles within pile groups are likely to be smaller than for individual piles due to sheltering effects.*
- e) *Pile stiffness relative to surrounding soil: the inertial loads from foundations supported on large diameter and stiff piles are likely to be larger than on slender small diameter piles.*
- f) *Phase of ground motions: the inertial loads should be added to the kinematic loads unless it can be demonstrated that the ground motions are out-of-phase with the ground displacements.*

6.14.9 Associated seismic hazards

6.14.9.1 Stability and deformation of slopes

Delete the first paragraph and replace with:

Embankments comprising soils that are not susceptible to liquefaction or cyclic mobility shall be analyzed using requirements of Clause 6.14.4.2.

Delete the first sentence of the third paragraph.

Delete the fourth paragraph and replace with:

Embankments comprising soils that are susceptible to liquefaction, or cyclic mobility shall be analyzed using requirements of Clause 6.14.8.

Commentary: The factor of safety threshold has been included in the Supplement Cl. 6.14.4.2.

6.19 Mechanically stabilized earth (MSE) structures

6.19.2 Design

Add the following clause:

6.19.2.1 General

The maximum height for MSE walls using extensible soil reinforcing shall be 9 m. The maximum height of MSE walls using inextensible soil reinforcing shall be 12 m.

Inextensible soil reinforcement shall be steel. Extensible reinforcement shall be geogrid.

Only MSE Wall systems listed in the Ministry Recognized Products List may be used. MSE Walls shall meet all requirements given in the Recognized Products List.

Wire used in wire facing or soil reinforcing components of all MSE walls shall be galvanized and shall have a minimum thickness determined based on a 100-year design life and corrosion-resistance durability requirements.

MSE walls in Seismic Performance Category 2 and 3 must have anchored connections of the facing to the soil reinforcing that do not rely on friction. MSE walls in SPC 2, using facing blocks conforming to SS942, may use friction for their connections when Consented to by the Ministry.

Two-stage MSE walls shall only be used where Approved by the Ministry.

Commentary: Two-stage MSE walls have had significant performance issues including failure. If a two-stage wall is approved, the designer shall liaise with MSE wall supplier(s) to confirm wall system details prior to tendering. Only wall systems that meet the project-specific criteria shall be shown on the Plans. Two-stage MSE walls shall be constructed so that there is no void space between the initial stage 1 wall and the final stage 2 facing after construction. The connections used to connect the second stage fascia panels to the main gravity wall structure shall be designed to minimize movement between panels during shaking during seismic load cases.

a) Mechanically Stabilized Earth (MSE) Walls at Bridge Abutments and the embankment bridge interface zone

Inextensible soil reinforcing shall be used. Geogrid extensible soil reinforcing shall only be used with Consent of the Ministry based on a project-specific evaluation.

The walls shall have precast reinforced concrete facing panels. Alternative facings may be acceptable for LVR structures when Consented to by the Ministry.

A reinforced concrete coping shall be used along the top of the walls.

Any portion of an MSE wall within the embankment bridge interface zone (6.14.2.2) shall also be considered as an abutment wall.

b) Other Mechanically Stabilized Earth (MSE) Walls

Inextensible or geogrid extensible soil reinforcing may be used.

Non-geogrid extensible soil reinforcing may only be used with the Consent of the Ministry based on a project specific evaluation.

MSE walls with wire mesh facing, dry cast concrete block facing, or rock stack facing shall only be used when Consented to by the Ministry.

Wire mesh facing shall only be used in Ministry Service Areas 1, 2, 3, 4, 6 and 27 unless otherwise Approved. The design shall include provisions to ensure long-term durability for the wire facing when exposed to spray or surface runoff containing de-icing chemicals.

Commentary for MSE walls: Corrosion of wire-faced MSE walls has occurred prematurely on Ministry walls. Wire-faced walls need to be carefully designed for site-specific environment and exposure conditions. Exposure to drainage, runoff and spray containing de-icing salts requires a corrosion evaluation during the design phase. The Service Areas listed above where wire-faced walls may be considered have been chosen since they are areas where these facings have not been reported to have premature corrosion in service and where the walls are subject to rain that can help remove de-icing chemicals from the facing. Even in these listed Service Areas, careful consideration of the site-specific corrosion conditions is needed to verify the appropriateness of the use of wire-faced walls.

The designer needs to consider the extent of quality control and quality assurance testing for the soil reinforcement for the specified wall systems and add these requirements to the Plans.

6.19.9.5 Global and Compound Stability

Add the following:

Global stability is the responsibility of the project geotechnical Engineer of Record. The geotechnical engineer of record shall use Table 6.2b herein for resistance factors and factors of safety for global stability.

Compound stability responsibility can reside with the geotechnical Engineer of Record and/or the MSE wall supplier. If responsibility is to reside with the MSE wall supplier, soil parameters for retained soil required for limit equilibrium stability analysis shall be provided to the supplier and the supplier shall be informed of this responsibility. Factors of safety for compound stability shall not be less than those of Table 6.2b for global stability of geotechnical systems.

Delete the last paragraph and replace with:

For compound stability, the restraining force of each soil reinforcement layer intersected by the failure surface shall be the lesser of either the long-term strength of each extensible reinforcement layer, the 100-year corroded strength of each inextensible reinforcement layer or the strength mobilized by the restrained length.

6.19.14.3 False abutments

Add the following:

False abutments shall consider soil-structure interaction unless the piles are isolated using casing.

Commentary: *Utah Department of Transportation Document UT-13.04, Lateral Resistance of Piles Near Vertical MSE Abutment Walls, presents comprehensive research and analysis related to the complex soil-structure interaction behavior.*

When designing a false abutment MSE wall with an embedded pile without isolation, an integrated design team including the bridge engineer, geotechnical engineer and potential MSE wall suppliers should be involved. Several iterations of the design may be expected for this type of wall configuration.

The lateral pile resistance for loading towards the MSE wall face should be reduced to account for the presence of the wall. Figure 6.14 in UT-13.04 presents p -multipliers to account for this reduction as a function of offset from wall face and MSE strap length. These p -multipliers should be used in conjunction with p -multipliers for group effects.

Lateral loading on the pile will induce additional load in the MSE reinforcement which must be resisted with additional soil reinforcement tied to the pile or pile cap. Appropriate soil-structure interaction analysis that incorporate non-linear soil springs (p - y curves or similar) should be completed to estimate this load. The estimated additional reinforcement load from the soil-structure interaction analysis should be compared with empirical envelopes presented in UT-13.04 Figures 6.15 and 6.16 and the larger of the two values used for design.

After estimating the additional soil reinforcement loads, the MSE wall supplier should be consulted to confirm the estimated additional loading can be reasonably resisted using readily available MSE strap types.

Add the following clause:

6.20 Lightweight fills

All lightweight fills shall be adequately protected against wheel loads, ground water, road salts, weather, fire, flotation under flood conditions and fuel spills.

Where walls are used to contain flammable lightweight fills, the walls shall provide a 2-hour fire rating.

Any geotechnical system or landscaping above the lightweight fills shall be designed such that the protective membrane covers for the lightweight fill shall not be compromised.

Flotation forces corresponding to inundation of the lightweight fill under the 200-year flood level shall be addressed in the design, regardless of any flood protection provided for the area in which the lightweight fill is to be constructed.

Expanded Polystyrene (EPS) lightweight fills shall meet the following requirements:

- EPS shall be supplied in the form of blocks. It shall be classified as to surface burning characteristics in accordance with CAN/ULC-S102.2-03-EN, having a flame spread rating not greater than 500.

- The minimum compressive strength, measured in accordance with ASTM D1621, shall be 125 kPa at a strain of not more than 5%.
- The density of EPS shall not be less than 22 kg/m³.
- EPS blocks shall be fully wrapped with minimum 0.254 mm (10-mil) thick, black polyethylene sheeting.
- Polyethylene sheeting joints shall be overlapped by a minimum of 0.5 m.
- EPS blocks shall have a minimum 1.2 m granular cover vertically and horizontally.

Shredded rubber tires or hog fuel (wood waste) shall not be used as fill.

Add the following clause:

6.21 Retaining Walls

Retaining wall types shall meet the durability and aesthetic requirements specified for the project and shall be subject to the Consent of the Ministry.

Design issues not addressed by S6:19 or herein shall meet the requirements of the latest edition of AASHTO LRFD Bridge Design Specifications.

Surface drainage and drainage of the backfill material shall be addressed in the design of the walls and details shall be shown on the Plans.

Additional full-length anchors, tie-backs, soil reinforcing and/or soil nails shall be installed to allow for future extraction for long-term inspection and testing. The minimum number of additional elements provided for each wall shall be the greater of 2% of the number of elements required by design, or 2. Additional full-length double corrosion protection anchors for long-term inspection and testing may be omitted when Consented to by the Ministry.

Add the following clause:

6.22 Geosynthetic reinforced soil (GRS)

Geosynthetic reinforced soil (GRS) methods may be used when agreed to or directed by the Ministry. GRS design shall follow S6:19, with the exception of methods for internal stability analysis, which shall be in accordance with FHWA-HRT-017-080.

Commentary: *At this time, use of GRS is not considered appropriate on Major or Lifeline routes as the Ministry evaluates this design method. This is to be reevaluated under future revisions of the Code and Supplement.*

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7.1 Scope

Add the following:

Section 7 applies only to buried structures with a span greater than or equal to 3 m.

Commentary: *The CHBDC Commentary (C7.1 Scope) indicates that Section 7 applies only to buried structures with spans greater than 3 m.*

For buried structures with spans less than 3 m, see Section 1000 of the Ministry Supplement to TAC Geometric Design Guide.

Specifications for materials, fabrication and construction of buried structures should be in accordance with SS 303 Culverts and SS 320 Corrugated Steel Pipe, where applicable.

Some types of buried structures are procured using a proprietary design process where the supplier provides the engineering for the structure as part of the tendered contract. Simple structures up to 6 m in span should be designed and included in the tender documents rather than utilizing a proprietary design process.

7.5 Design

Add the following:

When live load effects are governed by the single axle no.4, the CL-625 truck shall be used.

Commentary: *The axle no.4 CL-800 truck loading is considered to be overly onerous compared to actual single axle loads.*

7.5.1 Sustainability and durability

7.5.1.2 Durability

7.5.1.2.1 General

Commentary: *Refer to Cl. 1.9.1.2 for additional commentary.*

7.5.1.2.2 Steel structures

Add the following:

The design life for Soil-Metal Structures, for the purpose of durability and corrosion allowance calculations, shall be 100 years.

Design shall be in accordance with the following Corrugated Steel Pipe Institute (CSPI) Technical Bulletins unless otherwise Consented to by the Ministry:

Technical Bulletin 1 – Performance Guideline for Corrugated Steel Pipe Culverts (300mm to 3,600mm Diameter) – November 2020

Technical Bulletin 13 – Performance Guideline for Buried Steel Structures – February 2012 using the AASHTO corrosion loss model.

Inverts shall be assessed for deterioration for erosion and bedload abrasion and additional protection shall be provided when necessary. Abrasive stream flows should be avoided wherever possible by appropriate hydraulic measures.

Commentary: *The Ministry has noted the premature failure of buried structures due to corrosion and/or abrasion because of inadequate consideration of site conditions. Options to mitigate these effects include armour plates, energy dissipaters concrete liners, thicker galvanic coating and polymer coatings.*

Note that CSA G401 coating thickness is total both sides.

7.5.1.2.5 Design detail

7.5.1.2.5.2 Membranes

Add the following:

The top slab of concrete structures within 600 mm of the finished roadway surface shall be treated with a waterproofing membrane system. The membrane may be omitted on LVR structures when Consented to by the Ministry.

7.5.3 Load factors

Add the following:

When checking buried structures for buoyancy (refer also to Clause 3.11.3), the designer shall consider the potential effects of soil-structure interaction and soil particle behaviour.

Commentary: *Section 7 refers generally to Section 3, Clause 3.5.1, for load factors but design of buried structures against buoyancy effects is not addressed. For buried structures, wall friction is usually dependent on actual soil-structure interface properties achieved during construction, and thereafter, so a conservative minimum value is appropriate for the buoyancy check. Also, a conservative assumption of actual soil state (minimum active or minimum at-rest) is appropriate to assure safety against buoyancy.*

7.5.7 Geotechnical considerations

7.5.7.4 Material for engineered fill

Add the following:

Materials for Zones 1 and 2 shall meet the requirements for Bridge End Fill in accordance with SS 201.40. Zone 3 material may be Bridge End Fill or a Group III material deemed suitable by the designer.

7.5.8 Seismic requirements

7.5.8.1 General

Replace a) in second paragraph with:

a) locations where the design spectral response acceleration for a period of 0.2 s, $S_a(0.2, X)$ at a probability of 2%/50 years, is greater than 0.7 g; or

7.5.14 Site supervision and construction control

Add the following:

For all types of buried structures, the Plans shall specify the following design information:

- Type of Buried Structure;
- Design Life
- Highway Design Loading;
- Unit Weight of Backfill;
- Depth of Cover, H ;
- Depth of Cover, H_c , at intermediate stages of construction;
- Construction Live Loading assumed in the design (corresponding to H_c);
- Geometric Layout and Key Dimensions;
- Foundation and Bed Treatment;
- Foundation Allowable Bearing Capacity;
- Extent of Structural Backfill;
- Conduit End Treatment;
- Hydraulic Engineering Requirements, as appropriate;
- Roadway Clearance Envelope, as appropriate; and,
- Concrete Strength, as appropriate.
- Backfill and drainage details including material properties, placement and compaction

For Soil-Metal Structures and Metal Box Structures, the Plans shall also specify the following design information:

- Design life based on corrosion allowance calculations;
- Minimum plate thickness and coating system;
- Corrosion Loss Rates (for substrate metal and for coating system);
- Abrasion Level and abrasion treatment;
- Electrochemical Properties of Soil Materials and Water in contact with the structure;
- Seam Strength at Critical Locations;
- Conduit Geometry including: Rise, D_v , Span, D_h , Radius at Crown, R_c , Radius at Spring-line, R_s , and Radius at Base, R_b . etc.

7.6 Soil-metal structures

7.6.3 Design criteria

7.6.3.1 Thrust

7.6.3.1.3 Live loads

Replace item (b)(i) with the following:

(b)(i) within the span length, position as many axles of the CL-800 Truck or Trucks (and/or Special Truck if specified) at the road surface above the buried structure as would give the maximum total load;

7.6.3.4 Design criteria for longitudinal connections

7.6.3.4.1 General

Add the following:

The values of unfactored seam strength for bolted steel plates, S_s , shown in Commentary Figure C7.4 may be used for standard corrugation profile with bolted connections.

7.6.6 Special features

Add the following:

Special features shall be used only when Consented to by the Ministry. All structures having special features shall use refined analysis unless otherwise Consented to by the Ministry.

Commentary: Cl. 7.6.6. of S6.1:19 (Commentary on S6:19, Canadian Highway Bridge Design Code) provides examples of some of the special features that can be used to improve structural performance and provides guidance for transverse stiffeners and GRS structures.

7.7 Metal box structures

7.7.1 General

Add the following:

The geometric limitations provided in AASHTO LRFD Bridge Design Specifications (2020) Table 12.9.4.1-1 shall be met (e.g., maximum radius at crown and minimum radius at haunch), unless refined methods of analysis are utilized.

7.7.3 Design criteria

7.7.3.1 Design criteria for crown and haunches

7.7.3.1.3 Live loads

Replace the definition of A_L at the end of this clause with the following:

where A_L is the weight of a single axle of the CL-800 Truck (or Special Truck if specified) for $D_h < 3.6$ m, or the combined weight of the two closely spaced axles of the CL-800 Truck (or Special Truck if specified) for $D_h \geq 3.6$ m, and k_4 is a factor for calculating the line load, as specified in Table 7.9

7.9 Reinforced concrete buried arches

7.9.4 Construction

7.9.4.4 Waterproofing

Delete and replace with the following:

On paved highways, the top slab of concrete structures within 600 mm of the finished roadway surface shall be treated with a waterproofing membrane system. The membrane may be omitted on structures on gravel surfaced highways. For arches over highways or railways the use of membranes between adjacent segments shall be required unless otherwise Consented to by the Ministry.

When a membrane is required, the arch designer shall consider hydrostatic pressure, if applicable, and the appropriate interface friction in the analysis.

Commentary: *Seepage and the formation of icicles has been observed on buried arch structures at the joints between adjacent segments.*

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8.4 Materials

8.4.1 Concrete

8.4.1.1 Compliance with CSA A23.1/23.2 and CSA A23.4

Delete and replace with the following:

Materials, methods of material testing, and construction practices shall comply with the following, unless otherwise Consented to by the Ministry, based on the order of precedence as listed below:

- Ministry Supplement to CHBDC S6-19
- Ministry Standard Specifications for Highway Construction (SS and DBSS)
- CSA A23.1/A23.2 and CSA A23.4

8.4.1.2 Compressive strength

8.4.1.2.1 Specified compressive strength

Commentary: The specified concrete strength for prestressed I girders and box girders should not exceed 55 MPa at 28 days or 37.5 MPa at release. These strength limits enable the prestressed girders to be manufactured with efficient turnaround times for release and handling. Modifications to these strength limits should be discussed with the Ministry and the precast girder manufacturers to enable efficient design and manufacture.

8.4.1.5 Modulus of elasticity

8.4.1.5.1 General

Add the following:

The design values of modulus of elasticity at transfer and 28 days shall be shown on the plans for all prestressed girders.

Commentary: With use of new materials in concrete mix designs for prestressed girders, such as use of GUL cement, the Ministry will be gathering data for the modulus of elasticity from prestressed concrete girder suppliers.

Designers should check that their designs accommodate the values of modulus of elasticity of concrete available from prestressed concrete girder suppliers.

8.4.2 Reinforcing bars and deformed wire

8.4.2.1 Reinforcing bars

Add the following:

Carbon steel reinforcing bar layouts shall be based on standard reinforcing bar lengths of 12 m for 10M bars and 18 m for 15M bars and greater. Stainless steel reinforcing bar layouts shall be based on standard reinforcing bar lengths of 11.89 m.

Commentary: *Standard reinforcing bar lengths are based on typical bar lengths which are currently available from reinforcing steel suppliers.*

8.4.2.1.1 Specification

Add the following:

Reinforcing bars shall be in accordance with SS412 and DBSS 412.

Other reinforcing bar types are permitted for use when Consented to by the Ministry.

The designer shall consider and address the difference between metric and imperial bar sizes when specifying the use of solid stainless reinforcing bars or low carbon/chromium reinforcing steel. Design for stainless steel reinforcing shall be based on Canadian metric bars and a conversion table for allowable substitutions with imperial or non-Canadian metric bars shall be provided in the Plans. Design for low carbon/chromium steel reinforcing shall be based on imperial bars and a conversion table for allowable substitutions with metric bars shall be provided in the Plans.

Commentary: *Solid stainless reinforcing bars are available in both European metric and imperial sizes, while low carbon/chromium reinforcing steel meeting the requirements of ASTM A1035 is currently only available in imperial sizes.*

8.4.2.1.2 Welding

Replace the second paragraph with the following:

Welded splices and tack welding of reinforcing shall only be allowed when Consented to by the Ministry.

8.4.2.1.3 Yield strength

Add the following:

Grade 400W reinforcing bars shall be specified for flexural reinforcement in plastic hinge regions, unless otherwise Consented to by the Ministry.

For bridge decks, the design yield strength shall be 420 MPa for stainless steel and low carbon/chromium reinforcing steel. The Plans shall note that details such as lap lengths shall be adjusted by the contractor at their cost to the satisfaction of the engineer if higher yield strength material is proposed by the contractor during construction.

Commentary: *Use of Grade 400W bars is intended to ensure plastic hinge regions possess expected ductility characteristics.*

For Grade 400W reinforcing bars, an upper limit for yield strength of 525 MPa is a requirement of CSA-G30.18.

There are corrosion resistant and stainless steel reinforcing grades with yield strength greater than 420 MPa and a stress-strain curve differing from Grade 400 reinforcing steel. These differences shall be considered in the design, in particular with respect to assumption for moment redistribution or seismic design considerations.

8.6 Design considerations

8.6.1 General

Add the following:

Connection details for precast concrete components shall be designed to address the potential for cracking in the concrete at the connections. The designer shall pay special attention to the size and type of welds for embedded connections to mitigate the potential for damage to the concrete from the heat of welding.

Commentary: The Ministry has observed concrete cracking near welded connections at precast concrete components. Details should be carefully designed to address this issue.

8.7 Prestressing requirements

8.7.4 Loss of prestress

8.7.4.1 General

Commentary: The losses tabulated in Table C8.2 may be unconservative for prestressed girders where the span to depth ratio is high.

8.8 Flexure and axial loads

8.8.4 Flexural components

8.8.4.5 Maximum reinforcement

Replace the second sentence with the following:

This requirement may be waived if Consented to by the Ministry if it is demonstrated that the consequences of reinforcement not yielding are acceptable.

8.9 Shear and torsion

8.9.3 Sectional design model

8.9.3.8 Determination of ϵ_x

Commentary: For the design and evaluation of prestressed girders, the capacity-enhancing effect of negative strains (compressive) near supports may be taken into account using generally accepted approaches.

8.11 Durability

8.11.2 Protective measures

8.11.2.2 Concrete quality

8.11.2.2.1 General

Delete the entire clause and replace with the following:

Concrete mix design parameters shall be determined in consultation with the Ministry and shall comply with the requirements given in Table 8.4 unless otherwise Consented to by the Ministry. The information, for each relevant classification of concrete, shall be included in the Special Provisions for the Project.

For structural concrete not covered by Table 8.4, the maximum water to cementitious materials ratio shall be 0.45 unless otherwise Consented to by the Ministry.

Entrained air content for all concrete mixes shall be in accordance with CSA A23.1, Table 4, Air content category 1, except that the air content shall be measured at the point of placement.

Mix design parameters for concrete subject to Very Severe degree of exposure to sulfate attack (CSA A23.1 Class of Exposure S-1) shall be determined in consultation with the Ministry and shall be Consented to by the Ministry.

Cement type shall be GU or GUL unless otherwise Consented to by the Ministry.

Low-density and Semi-low-density concrete shall not be used in any bridge component.

Commentary: A23.1-24, Clause 8.2 Mass Concrete considers mass concrete may include elements having a minimum dimension equal to or greater than 1 metre. The requirements of Clause 8.2 shall be considered when developing mix design requirements and Special Provisions, including but not limited to:

- *Temperature requirements*
- *Cementitious material contents*
- *Temperature monitoring*
- *Thermal control plans.*

Table 8.4
Cast-in-Place Concrete mix design parameters

Exposure⁽¹⁾⁽²⁾	Minimum Compressive Strength at 28 days (MPa)	Nominal Maximum Size of Coarse Aggregate (mm)	Slump⁽³⁾ (mm)	Maximum w/cm Ratio by Mass
Deck Concrete⁽⁴⁾⁽⁷⁾: Deck Slab⁽⁶⁾, Approach Slab⁽⁶⁾, Parapet and Median Barrier, Bridge sidewalk, Diaphragm				
Late chloride exposure ⁽¹³⁾	35	20 ⁽⁸⁾⁽¹⁰⁾	80 ± 20	0.38
Early chloride exposure ⁽¹⁴⁾	35	20 ⁽⁸⁾⁽¹⁰⁾	80 ± 20	0.38
Substructure Concrete: Piers⁽⁵⁾, Abutments⁽⁵⁾, Retaining Walls⁽⁵⁾, Footings⁽⁵⁾, Pipe Pile In-fills⁽⁹⁾, Working Floors				
All chloride and sulphate exposures except Class S-1 ⁽¹¹⁾⁽¹⁵⁾	30	20 ⁽¹⁰⁾	120 ± 30	0.40
Deck Overlay Concrete⁽⁴⁾⁽⁶⁾⁽⁷⁾:				
Late and Early chloride exposure ⁽¹⁴⁾	45	14	80 ± 20	0.38
Keyways between Box Stringers⁽⁴⁾⁽¹²⁾:				
Late chloride exposure ⁽¹³⁾	35	14	80 ± 20	0.38
Early chloride exposure ⁽¹⁴⁾	35	14	80 ± 20	0.38
Concrete Slope Pavement⁽⁵⁾:				
All chloride and sulphate exposures except Class S-1 ⁽¹¹⁾⁽¹⁵⁾	30	20 ⁽¹⁰⁾	30 ± 20	0.40

Notes:

- 1) Early chloride exposure applies to concrete that is not allowed to air-dry for a period of at least one month after the end of the curing period, before exposure to de-icing chemicals. Late chloride exposure applies to concrete that is allowed to air-dry for a period of at least one month after the end of the curing period, before exposure to de-icing chemicals.
- 2) Sulphate exposure classes as per CSA A23.1, Table 3.
- 3) Slump as measured at the final discharge end of the placement system, after addition of admixtures.
- 4) Resistance to chloride ion penetration test results shall be less than 1,000 coulombs at 91 days for late chloride exposure and shall be less than 1,500 at 28 days for early chloride exposure, using test method A23.2-23C or ASTM C1202.
- 5) Resistance to chloride ion penetration test results shall be less than 1,500 coulombs at 91 days using test method A23.2-23C or ASTM C1202.
- 6) For exposed concrete decks that have no added wearing surfaces, the maximum proportion of aggregate passing the 5 mm sieve shall be less than 38% of the total mass of the aggregate. For deck overlay concrete with 14 mm aggregate, the maximum proportion of aggregate passing the 5 mm sieve shall be less than 42% of the total mass of the aggregate.
- 7) Shrinkage limit of the concrete shall be in accordance with CSA A23.1 Clause 8.8 Low-shrinkage concrete and shrinkage test shall be in accordance with CSA A23.2-21C.
- 8) The requirements of Note 7) shall be waived if the total cementitious materials content in the concrete mixture is equal to, or less than 400 kg/m³ with the use of a Nominal Maximum Size of Coarse Aggregate of 28 mm.
- 9) For deep piles and drilled shafts, the use of a tremie concrete mix that incorporates 14 mm Nominal Maximum Size of Coarse Aggregate and plasticised slump of up to 190 mm ± 40 mm with a Visual Stability Index (VSI) of 0 or 1 in accordance with CSA A23.2-19C are permissible.
- 10) 28 mm Nominal Maximum Size of Coarse Aggregate may also be used.
- 11) The concrete mix shall satisfy the additional requirements regarding sulphate attack in accordance with CSA A23.1-24 Table 3. Cementitious materials testing is required per CSA A3004-C8; maximum expansion at 6 months shall be less than 0.05% or less than 0.10% at 1 year for S-2 exposure, and at 6 months shall be less than 0.10% for S-3 exposure. Requirements for S-1 exposure shall be determined in consultation with the Ministry. No additional requirements regarding sulphate attack are required for lower sulphate concentrations than those for S-3 class of exposure according to CSA A23.1 Table 3.
- 12) Pre-blended bagged concrete mixes shall only be used when Consented to by the Ministry. Pre-blended bagged concrete mixes may be considered where limited ready mix concrete supply is available.
- 13) The total cementitious content shall not exceed 400 kg/m³. The use of supplementary cementitious materials shall be in accordance with one of the following:
 - a) Maximum 20% Type F fly ash by mass of cementitious materials, or
 - b) Maximum 35% slag by mass of cementitious materials, or
 - c) a combination of Type F fly ash and slag, with one of these components meeting its

minimum percentage from a) or b) above, and with a combined maximum of 35% by mass of cementitious materials.

- 14) The use of 5-8% silica fume is required. In addition to the silica fume, following shall also apply:
- a) 10-15% Type F fly ash by mass of cementitious materials, or
 - b) 15-25% Slag by mass of cementitious materials, or
 - c) a combination of Type F fly ash and slag, a combination of Type F fly ash and slag, with one of these components meeting its minimum percentage from a) or b) above, and with a combined maximum of 25% by mass of cementitious materials.
- 15) The use of supplementary cementitious materials shall be in accordance with one of the following:
- a) 15-35% Type F fly ash by mass of cementitious materials, or
 - b) 25-40% Slag by mass of cementitious materials, or
 - c) a combination of Type F fly ash and slag, with one of these components meeting its minimum percentage from a) or b) above, and with a combined maximum of 40% by mass of cementitious materials.

Commentary: *Changes to the target slump may be Consented to by the Ministry for substructure concrete in special circumstances such as in heavily congested elements or elements with other constraints that make it difficult for concrete placement and consolidation.*

Concrete mix with silica fume as the sole SCM has potential of deck cracking. Addition of the fly ash or slag can mitigate this issue. Concrete mix with silica fume should be avoided for late chloride exposure.

8.11.2.3.2 Concrete placement

Add the following:

The deck casting sequence and the detail for construction joints shall be shown on the Plans. Typically, deck slabs shall be cast in the direction of increasing grade (uphill).

For simply supported span structures, each span shall be cast in one continuous operation unless otherwise Consented to by the Ministry.

For continuous structures, concrete shall be cast full width in stages to limit any post-construction cracking in the deck concrete to less than 0.20 mm at the surface of the structural deck. In specifying the deck pour sequence, the designer shall pay particular attention to the adverse effects of stress reversal within freshly cast concrete deck slabs.

Commentary: *Concrete placement temperatures are identified in the SS and DBSS and should not be specified separately by the designer without the Consent of the Ministry.*

A deck casting sequence is required in order to minimize the potential for deck cracking due to improper concrete placement sequencing.

Several factors limit the quantity of concrete which can be placed in one continuous operation. Special consideration shall be given if the continuous placement exceeds a volume of 200 cubic metres or if the bridge deck exceeds four lanes in width.

Structures are to be cast full width to uniformly load the superstructure and to avoid differential deflection between stringers. The positive moment regions are generally cast first followed by the negative moment areas.

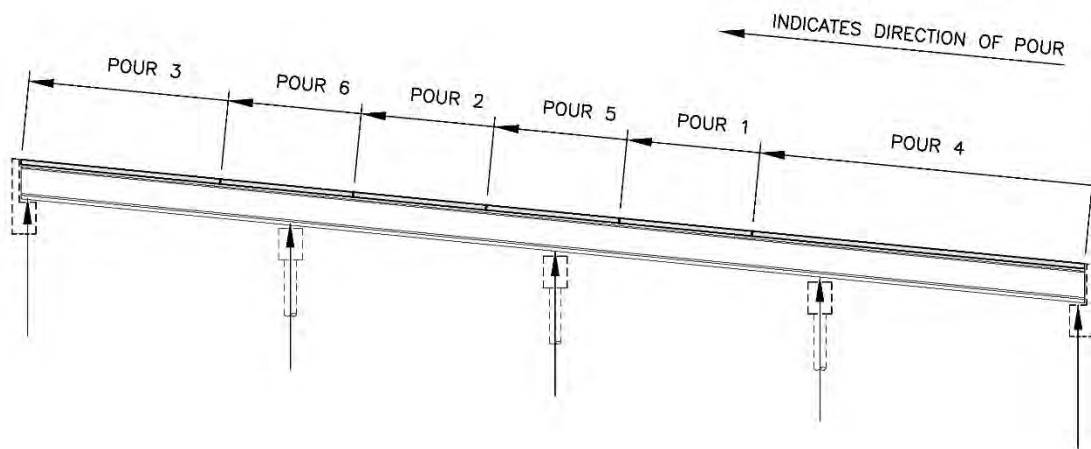
The following is generally the Ministry's deck casting procedure:

Concrete in positive-moment zones: All concrete in these zones to be cast prior to concrete in negative-moment zones.

Concrete in negative-moment zones: Concrete in these zones are typically not be cast until adjacent concrete in positive- moment zones have been cast, unless cast monolithically with the positive-moment concrete as shown below in pour sequence 4.

Concrete deck above concrete diaphragm without wearing surface: Concrete deck above diaphragms shall be cast after diaphragm concrete has set to avoid subsidence effect.

Figure C8.11.2.3.2
Sample schematic of deck pour sequence



Unless higher strengths are required by the designer, deck concrete shall attain a strength of 15 MPa before parapets are placed and 25 MPa before heavy loads, such as concrete trucks, are allowed on the bridge.

Concrete placement sequence for integral abutments shall be given special consideration to reduce stresses induced by deflection of the girders. Unless otherwise Consented to by the Ministry, the full width and length of deck shall be cast prior to the end diaphragms being cast integral with the abutment.

Commentary: For integral abutments, techniques for reducing stresses induced by deflection of the girders may include delaying the casting of the abutments and/or the deck in the abutment area until after all other deck concrete has been cast.

8.11.2.3.5 Slip-form construction

Add the following:

Extruded concrete barriers shall not be used.

8.11.2.3.6 Finishing

Delete the entire clause and replace with the following:

The methods to be used for finishing surfaces of concrete to ensure a durable surface shall comply with the relevant SS and DBSS clauses, unless otherwise Consented to by the Ministry.

Surface finishes shall be in accordance with Table 8.11.2.3.6, unless otherwise Consented to by the Ministry, and shall be specified in the Special Provisions.

Table 8.11.2.3.6
Surface finishing requirements

Surface	Finish	Relevant SS or DBSS clause
Surfaces submerged or buried	Class 1	211.17
Top and inside (exposed) face of parapets, curbs	Class 3	211.17
Outer face of parapets, curbs; outer edges of deck	Class 2	211.17
Abutments and retaining walls	Class 2	211.17
Piers	Class 2	211.17
Bearing seats	Steel Trowel	211.14
Top of deck	Tined ⁽¹⁾	413.02.02 (e)
Approach slabs	Float Finish	211.14
Sidewalks	Transverse Coarse Broom	211.14

Underside of Deck	Class 1 (or better)	211.17
Slope Pavement	Transverse Coarse Broom ⁽²⁾	211.14

Notes:

(1) Decks to receive waterproofing membranes shall be finished in accordance with SS 419.33 or DBSS 419.33 as applicable.

(2) Exposed Aggregate finishes may be considered.

Consideration shall be given to surfaces exposed to close public view such as piers and abutments on underpasses where a Class 3 finish may be considered and underside of decks where a Class 2 finish may be preferred.

Exposed concrete surfaces of large abutments or retaining walls that are clearly visible to the public may require a special architectural finish. The selection of surface finishes shall also consider future removal of graffiti, including the application of anti-graffiti coatings.

8.11.2.4 Concrete cover

Add the following:

All references to “minimum cover” shall be considered as the minimum specified cover. The tolerances specified in SS 412 and DBSS 412 shall apply to the minimum specified cover.

The soffits of deck slabs cantilevered from the exterior girder shall be considered as exposed to harsh conditions and shall have additional cover as noted in S6:19 under Table 8.5.

The cover and tolerances given in Table 8.5a below shall govern over Table 8.5 from S6:19.

Add Table 8.5a as follows:

Table 8.5a
Minimum specified concrete covers and tolerances
(See Clause 8.11.2.4)

Component	Reinforcement/ steel ducts	Cast-in-place concrete (mm)	Precast concrete (mm)
Top Surfaces of Bridge Decks and Approach Slabs	Reinforcing Steel	⁽¹⁾ +6 -0	⁽¹⁾ +6 -0
	Pretensioning strands	–	100 ±5
Ministry Standard Precast Box Girders	Reinforcing steel		
	- Top surfaces	–	⁽¹⁾ +10 -5
	- Vertical surfaces	–	40 +10 -5
	- Soffits	–	30 +10 -5
	- Inside surfaces	–	30 +10 -5
	Pretensioning strands		
	- Top surfaces	–	200 ±5
	- Vertical surfaces	–	50 ±5
	- Soffits	–	40 ±5
	- Inside surfaces	–	35 ±5
Ministry Standard Precast I-Beams	Reinforcing steel		
	- Top surfaces	–	30 +10 -5
	- Vertical surfaces	–	30 +10 -5
	- Soffits	–	30 +10 -5
	Pretensioning strands		
	- Top surfaces	–	100 ±5
	- Vertical surfaces	–	40 ±5
	- Soffits	–	40 ±5

Notes:

(1) For the minimum specified cover and reinforcing steel type, see Table 8.11.2.5.2 in Clause 8.11.2.5.2

Commentary: Additional concrete cover as specified in the notes below Table 8.5 in S6-19 are not required for the top surface of concrete decks.

The term “minimum cover” should be avoided on the Plans as it creates confusion for installers. The term “specified cover” is the preferred term and the appropriate placing tolerances would apply. For example, for the vertical reinforcing in the Ministry Standard Precast Box Girders, a “specified cover” of 40 mm with placing tolerances of +10 mm and -5 mm will provide the correct installation.

Designers must be aware of, and account for, placing tolerances and specified cover requirements. Consideration shall be given to the cover requirements on mechanical splices.

8.11.2.5 Corrosion protection for reinforcement, ducts and metallic components

Add the following clauses:

8.11.2.5.1 General

Ends of prestressing strands shall be painted with a Ministry accepted organic zinc rich paint where the ends of stringers are incorporated into concrete diaphragms or are otherwise embedded in concrete.

Ends of prestressing strands shall be given a minimum 3 mm thick coat of thixotropic epoxy in 100 mm wide strips applied in accordance with the manufacturer's requirements where ends of stringers are not embedded in concrete. For prestressed box girders, the entire ends of the girder shall be coated.

If galvanized reinforcing steel is used, all reinforcing steel in the component shall be galvanized. Galvanized bars and uncoated bars shall not be permitted to be in contact with each other as specified in SS 412.11.03 and DBSS 412.11.03.

Precautions that are to be taken for cold-bent reinforcing steel that is to be galvanized shall include:

- increasing the minimum bend diameter to meet the requirements for epoxy coated steel as provided in SS Table 412-C
- ensuring Grade W (weldable) reinforcing is used in accordance with SS 412.11.03 and DBSS 412.11.03.

and

- stress relieving the reinforcing steel after bending and prior to galvanizing in accordance with SS 412.11.03 and DBSS 412.11.03. (Stress relieving procedures vary with the thickness of the material).
- Galvanized reinforcing bars are not to be bent after galvanizing.

Commentary: *Galvanized reinforcing steel and uncoated steel should not be used in combination due to the possibility of establishing a bimetallic couple between zinc and bare steel (i.e., at a break in the zinc coating or direct contact between galvanized steel and black steel bars or other dissimilar metals.*

The designer is cautioned regarding the potential for embrittlement of reinforcing steel which is cold-bent and then galvanized. (Straight reinforcing bars are not prone to embrittlement).

8.11.2.5.2 Corrosion protection for bridge decks, parapets, curbs and approach slabs

As a minimum, all reinforcing steel within the upper 50% of bridge decks and approach slabs including the top mat of deck reinforcing steel and any steel projecting into this zone and all reinforcing steel in cast-in-place parapets shall be protected against corrosion.

Corrosion protection for reinforcing steel shall be achieved by using corrosion resistant reinforcing and/or waterproofing membranes in accordance with the Table 8.11.2.5.2 below, unless otherwise Consented to by the Ministry.

Table 8.11.2.5.2
Corrosion protection options for top mat reinforcing steel for bridge decks, parapets, curbs and approach slabs

Rebar type ⁽¹⁾ for top mat of deck and approach slabs, and for the parapets and curbs	Minimum deck thickness (mm)	Specified Top Cover ⁽¹⁾ (mm)	Where used ⁽²⁾
Stainless steel	225	60 ⁽³⁾	Main roads where there is no waterproofing membrane
Stainless steel	225	50	Main roads where there is also a waterproofing membrane
Low carbon/chromium to ASTM A1035 Type CS	225	70	Other roads where there is no waterproofing membrane
Low carbon/chromium to ASTM A1035 Type CS	225	50	Other roads where there is also a waterproofing membrane
Black, epoxy coated, galvanized, ASTM A1035 Type CM	225	70	Gravel roads where there is no waterproofing membrane

Notes:

(1) Rebar type in accordance with Clause 8.4.2.1.1.

(2) Main roads = includes all structures on all primary highways and other highways with a current AADT of 2000 or greater.

Other roads = includes all other structures.

Gravel roads = gravel roads and roads with an ADT of less than 400 vehicles.

(3) Cover may be reduced to 50 mm for MoTT Standard Bridge Parapet Barriers (2784-7)

For cable supported structures, stainless steel reinforcing shall be used in both the top and bottom mats of the deck.

For pedestrian bridges with a clear walkway width of less than 3 m, black steel reinforcing bars may be used. For pedestrian bridges of 3 m and wider, corrosion protection of deck reinforcing steel shall be in accordance with the requirements for "Other roads" in Table 8.11.2.5.2.

Other corrosion protection for reinforcing steel, including stainless-steel clad reinforcing and composite reinforcing steel (GFRP, CFRP etc.) may only be used with Consent of the Ministry.

Commentary: The BC numbered highway functional classification can be found at: https://www2.gov.bc.ca/assets/gov/driving-and-transportation/transportation-infrastructure/planning/inventories/bc_numbered_hwy_functional_classes_map.pdf.

Black steel is generally used for bottom mat reinforcing.

8.11.2.5.3 Corrosion protection for components subject to spray or surface runoff containing de-icing chemicals

Except in Ministry Service Areas 1, 2, 3, 4, 6 and 27, steel reinforcement, anchorages, and mechanical connections specified for use within 75 mm of a surface exposed to moisture containing de-icing chemicals shall use corrosion resistant material in accordance with Table 8.11.2.5.3, unless otherwise Consented to by the Ministry. This shall include the following components:

- components and surfaces under expansion joints, such as bearings and girders, ballast walls, end diaphragms, bearing seats, etc. for a horizontal distance from the joint of 1.5 x the superstructure depth.
- exposed surfaces of piers, abutments, retaining walls where buildup of snow containing de-icing chemicals in contact with the component will occur
- components on main roads adjacent to or up to 3.0 m above the pavement surface subject to spray containing de-icing chemicals.

Commentary: Corrosion resistant material should also be considered as follows for concrete decks with or without curbs but with open railings:

- For the underside of the deck, past the drip groove for a minimum distance of 1.0 m.
- For soffits that are level or slope inward, the portion from the exterior edge to the full soffit width.
- For girders, the exterior surface and soffit of the girder.

Table 8.11.2.5.3
Corrosion protection for components subject to de-icing chemicals

Corrosion resistant material type ⁽¹⁾	Where used ⁽²⁾
Stainless steel	Main roads
Low carbon/chromium ASTM A1035 Type CS	Other roads
Black, epoxy coated, galvanized, ASTM A1035 Type CM	Gravel roads

Notes:

(1) Rebar type in accordance with Clause 8.4.2.1.1.

(2) Main roads = includes all structures on all primary highways and also on other highways with a current AADT of 2000 or greater.

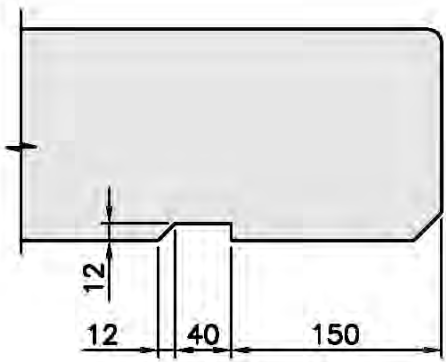
Other roads = includes all other structures.

Gravel roads = gravel roads and roads with an ADT of less than 400 vehicles.

8.11.2.6 Drip Grooves

Continuous drip grooves shall be formed on the underside of bridge decks and shall be detailed as shown below in Figure 8.11.2.6.

Figure 8.11.2.6
Drip groove detail



8.11.2.7 Waterproofing

Delete the first paragraph and replace with:

Unless otherwise Consented to by the Ministry, all bridges, other than on gravel roads, in the South Coast Region shall have a hot rubberized asphalt membrane system for waterproofing and 100 mm thick asphalt overlay on top of the bridge deck. Wick and weep drains shall be detailed at low points and points of blockages to ensure that water between asphalt and the membrane can be discharged. Water shall be discharged to avoid safety and maintenance issues such as avoiding sidewalks, pathways, traffic lanes, and bearing seats.

Buried concrete structures with a soil cover of 1000 mm or less shall receive a hot rubberized asphalt membrane system for waterproofing. Positive drainage shall be provided on the top surfaces of buried structures to avoid ponding of water.

Bridges with no waterproofing membrane located on paved roads shall be protected with an application of linseed oil or as otherwise directed by the Ministry.

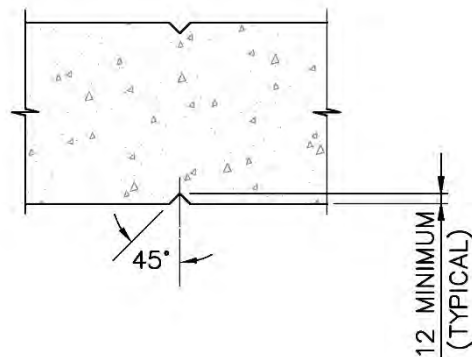
8.12 Control of Cracking

8.12.1 General

Add the following:

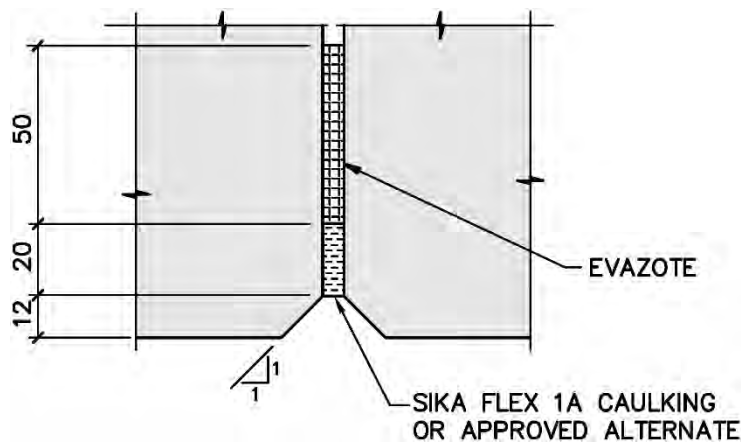
Control joints shall extend around the entire perimeter of the traffic barrier and be evenly spaced throughout the full length of the barrier with spacing not exceeding 3 m as shown below.

Figure 8.12.1(a)
Control joint detail



Concrete traffic barriers shall have a minimum 6 mm wide rotation joint over the supports on continuous spans as shown below.

Figure 8.12.1(b)
Rotation joint detail



8.13 Deformation

8.13.3 Deflections and rotations

8.13.3.3 Total deflection and rotation

Commentary: The Commentary to S6 (S6.1:19) states that long time deflection and rotation may be calculated by using the empirical multipliers given in Table C8.8 which is taken from CPCI (1996). However, Table C8.8 is not an exact copy of the table included in CPCI (1996). The original table or the table in the current edition of the CPCI Handbook (CPCI (2017)) may be used in place of the commentary.

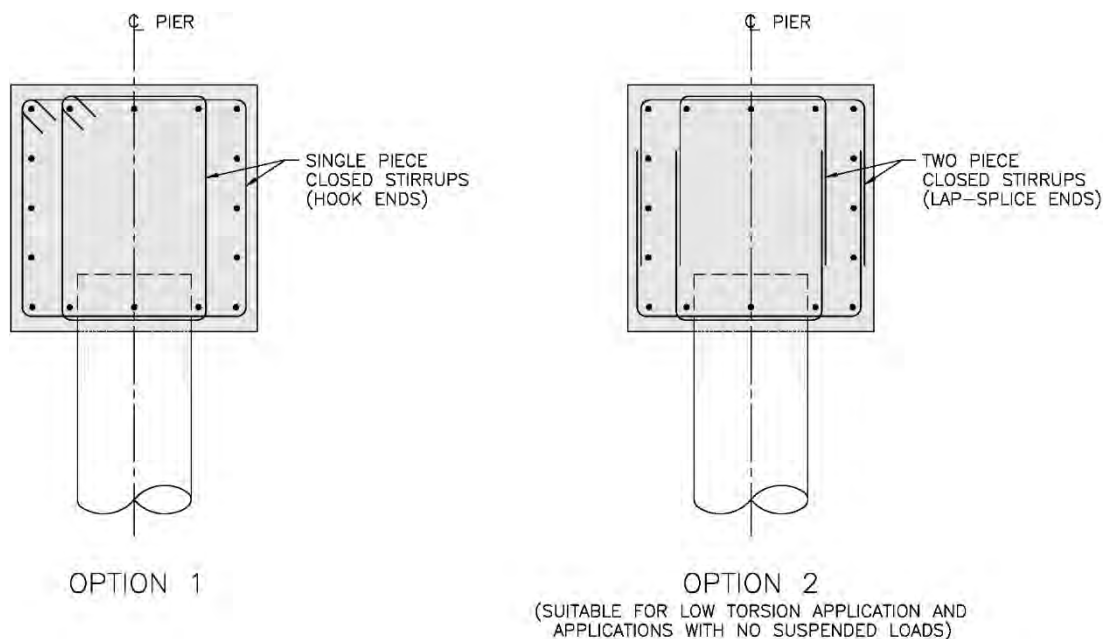
8.14 Details of reinforcement and special detailing requirements

8.14.3 Transverse reinforcement for flexural components

Add the following:

Typical arrangements for transverse reinforcement of pier caps are shown in Figure 8.14.3.

Figure 8.14.3
Typical transverse reinforcement of
pier caps (drip grooves and top surface slope not shown)



Commentary: The typical transverse reinforcement arrangements shown in Figure 8.14.3 alleviate problems encountered with installation of longitudinal reinforcing in situations where piles are installed slightly off alignment. These preferred arrangements facilitate placement of two longitudinal bars in

close proximity to the piles. Identical-size pairs of closed stirrups which lap one another horizontally do not provide as much tolerance for placement of the two longitudinal bars adjacent to the piles.

For diaphragms and other varying depth members, closed stirrups formed from two-piece lap-spliced U-stirrups or U-stirrups with lapped L splice bars as shown in Supplement Figure 8.20.7.1 shall be used (low torsion applications and applications with no suspended loads).

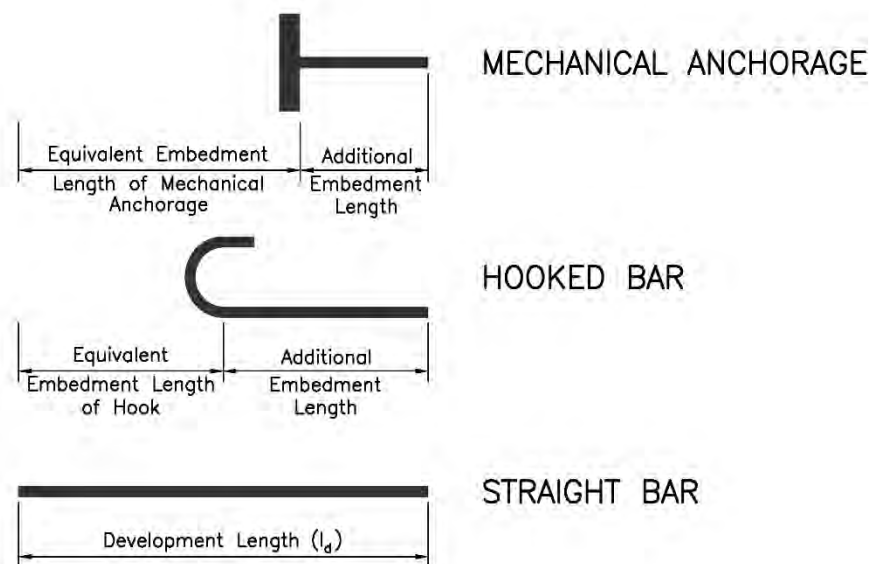
Commentary: Problems are encountered with stirrup sizes in diaphragms when stirrups are either too long or too short depending on the final depth of the haunches. The method of using two-piece U-stirrups of suitable depth allows for minor adjustments and alleviates problems of proper field fit-up when accommodating variable depth of diaphragms.

8.15 Development and splices

8.15.7 Combination development length

Commentary: Figure 8.15.7 below illustrates how the development length, l_d , may consist of a combination of the equivalent embedment length of a hook or mechanical anchorage plus additional embedment length of the reinforcement measured from the point of tangency of the hook.

Figure 8.15.7
Combination development length



8.15.10 Splicing of reinforcement

8.15.10.1 Lap splices

Add the following:

All splices that are critical to the structure shall be indicated on the Plans.

Splicing of transverse reinforcing bars in bridge decks shall be avoided if possible. If such splices are necessary, their location shall be indicated on the Plans.

8.15.10.2 Welded splices

Add the following:

The use of welding to splice reinforcement is not permitted unless Consented to by the Ministry.

8.16 Anchorage zone reinforcement

8.16.7 Anchorage of attachments

Add the following:

Anchor dowel details for the Ministry standard prestressed concrete box stringers shall be designed based on the details similar to those shown in the Ministry Standards and Procedures Manual Volume 3, Ministry Standard Drawings 2978-1 to 2978-24 (latest revision) and the standard reference details for Standard Twin Cell Box Stringers. Similar details may be used, as appropriate, for Ministry Standard Single Cell Box Stringers, Drawing D205.

8.18 Special provisions for deck slabs

Add the following:

Bridge deck heating systems are not permitted.

***Commentary:** Heating of bridge decks in British Columbia has been problematic. Its use has therefore been discontinued.*

8.18.1 Applicability

Add the following:

The design of partial and full depth precast panel decks shall be completed in accordance with Cl. 8.18.3.4 and Cl. 8.18.3.5 of the Supplement except that 8.18.3.4.2 d) only applies when the deck is proportioned with the empirical design method.

8.18.2 Minimum slab thickness

Delete the last sentence and replace with the following:

The slab thickness shall not be less than 225 mm.

8.18.2.4 Allowance for wear

Delete this clause.

8.18.3 Empirical design method

8.18.3.4 Cast-in-place deck slabs on precast panels

8.18.3.4.3 Cast-in-place topping

Commentary: The Ministry has observed cracks at the ends of short discontinuous reinforcement placed above transverse joints in decks with partial depth panels. Use of such details should be carefully considered by the designer in terms of crack control.

8.18.3.5 Full-depth precast panels

Add the following:

Full-depth precast panels may only be used on numbered highways when Consented to by the Ministry.

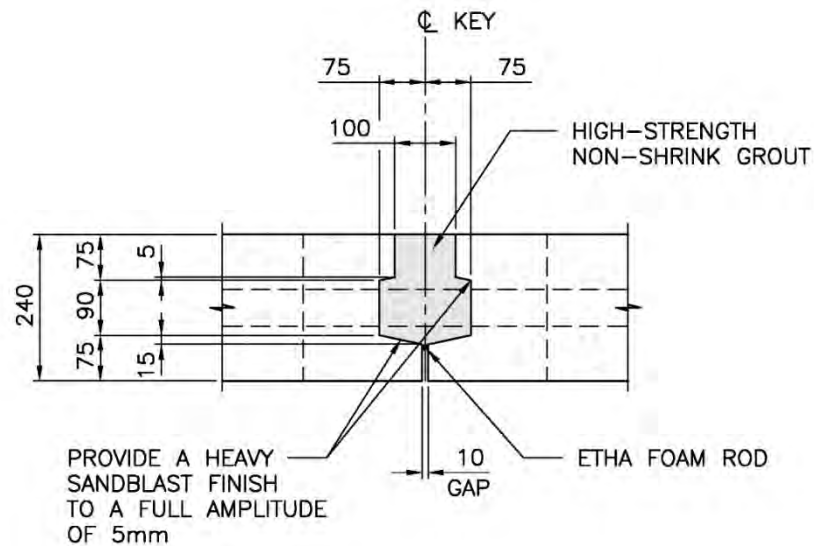
Delete the first sentence and replace with the following:

Design of full-depth precast panels shall satisfy the following conditions in addition to those of Clause 8.18.3.2 and, as applicable, Clause 8.18.3.3:

Delete Item (c) and replace with the following:

- c) at their transverse joints, the panels are joined together by grouted reinforced shear keys and are longitudinally post-tensioned with a minimum effective prestress of 1.7 MPa. The post-tensioning system shall be fully grouted. The transverse joints shall be of a female to female type. Tongue and groove type shear keys and butt joints shall not be used. The shear key shall be detailed to allow for the panel reinforcing to be lapped with hooked ends with reinforcing placed parallel to the shear key. Figure 8.18.3.5(a) details the requirements for minimum shear key size.

Figure 8.18.3.5a)
Full depth precast panel shear key



Alternatively, reinforced concrete shear keys may be used without post-tensioning where Consented to by the Ministry. The shear key design shall account for all force transfer effects through the shear keys. Figures 8.18.3.5(b) and 8.18.3.5(c) give examples for reinforced shear keys.

Figure 8.18.3.5(b)
Full depth precast panel with reinforced shear key over deck support member

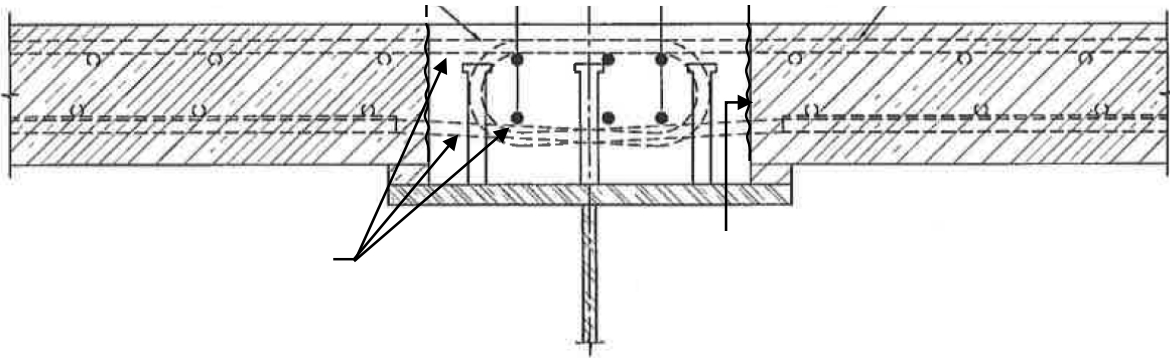
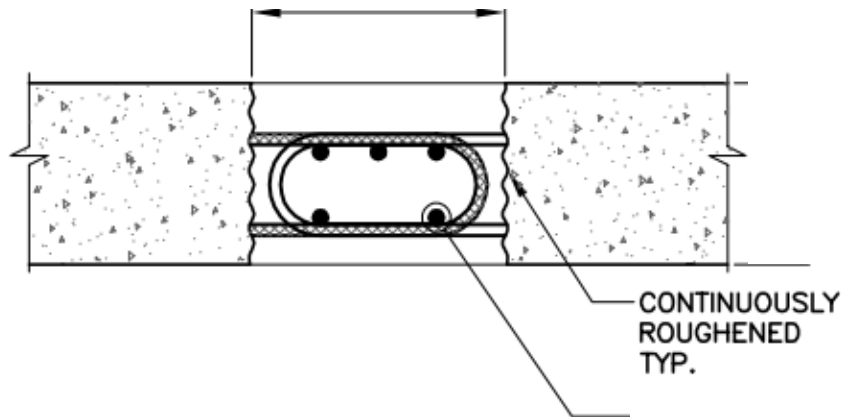


Figure 8.18.3.5(c)
Full depth precast panel with reinforced suspended shear key



Add the following additional items:

- h) a minimum specified gap of 25 mm for grout shall be provided under the panels above the supporting beams, including any splice plates. The gap for grout may be omitted for LVR bridges when Consented to by the Ministry.
- i) the deck slab comprised of full-depth precast panels shall be fully composite with the supporting beams.
- j) when cast-in-place concrete parapets are they shall be continuous across the transverse joints except in the negative moment regions of the supporting beams. The parapets shall be placed after the deck panels are placed and fully grouted.
- k) the deck shall have a waterproofing membrane with a 100 mm thick asphalt wearing surface unless otherwise Consented to by the Ministry. Bare precast concrete decks may be used on low volume road bridges and gravel roads.
- l) stud connectors shall be in accordance with Clause 10.11.8.3.3.

Commentary: Shear keys between precast deck panels may also consist of reinforced concrete joints or, when Consented to by the Ministry, of ultra-high performance fibre-reinforced concrete (UHPC) In these cases adequate force transfer through the joints and reinforcing bar overlap need to be assured.

Further information on UHPFRC shear keys and joints as well as guidance to splice and development lengths can be found in Annex A8.1 in S6:19.

Confinement of stud clusters may be required to obtain the required shear connector strength.

8.18.4 Diaphragms

Add the following sentence to the end of the first paragraph:

Steel diaphragms for concrete girders shall be hot-dipped galvanized and detailed similar to Supplement Figure 8.20.7.3. For monolithic cast-in-place concrete end diaphragms and intermediate diaphragms, consideration shall be given to additional deck reinforcing over the diaphragms to withstand negative moment demands. Refer to Clause 8.20.7 for specific guidance regarding design of concrete diaphragms for concrete girders.

8.19 Composite construction

8.19.1 General

Add the following:

Ministry standard prestressed concrete box girders with a concrete overlay wearing surface with no interface shear-friction reinforcement shall be designed as non-composite, unless otherwise Consented to by the Ministry. For non-composite design, the placement of a concrete overlay wearing surface on top of box girders shall be considered as an additional dead load and shall not be assumed to contribute to any composite properties under live loads.

Composite prestressed concrete box girders shall achieve composite action using interface shear-friction reinforcement between the box girder and the composite topping.

See Clause 8.21 for further details regarding concrete box girders.

8.19.3 Shear

Add the following:

Shear reinforcement in prestressed I-beams shall extend 125 mm above the top of the beam. When the haunch height exceeds 75 mm, additional shear reinforcement (e.g. shear ties matching the spacing of stirrups in the I-beams) and additional longitudinal reinforcing at the haunch corners shall be provided as shown in Supplement Figure 8.19.3 (a).

Additional shear reinforcement and longitudinal reinforcing at the haunch corners shall also be provided above steel girders, as shown in Figure 8.19.3 (b), where haunch heights exceed 75 mm.

Figure 8.19.3 (a)
Additional reinforcement for haunches over 75 mm high (conceptual)

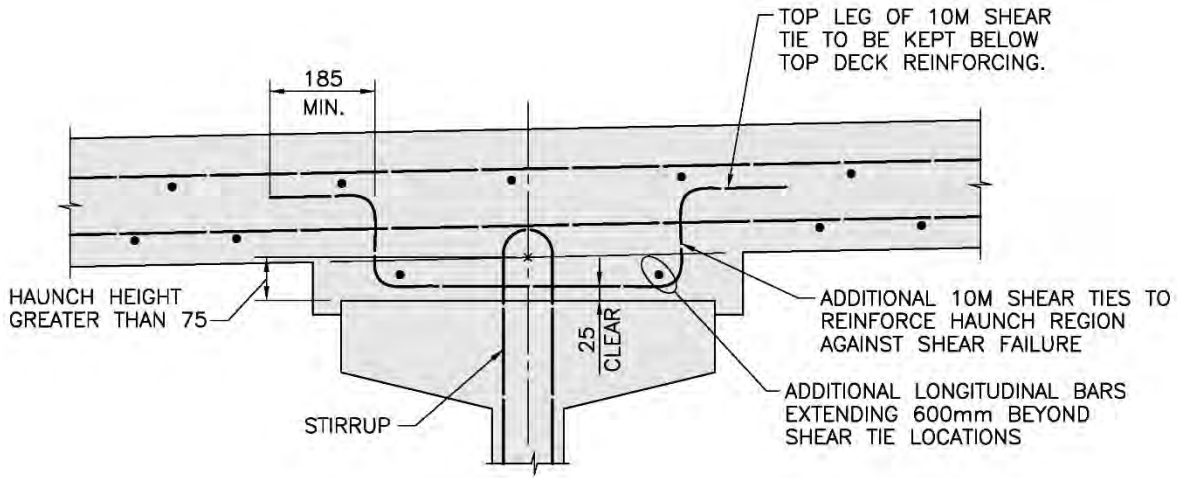
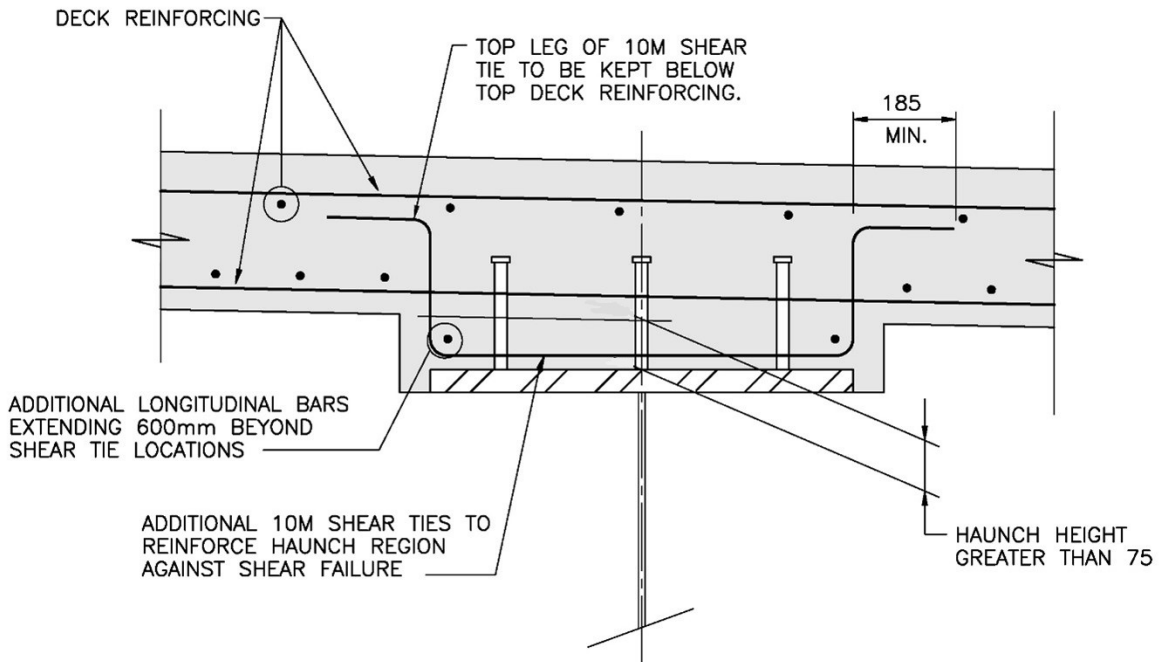


Figure 8.19.3 (b)



Additional reinforcement for haunches over 75 mm high (conceptual)

Commentary: The examples in Figure 8.19.3 (a) and (b) show haunch reinforcement on a conceptual level. The reinforcing requirements for these haunches should be checked during design and the reinforcement in the haunches adjusted as required. Particular attention should be given to situations with deep haunches where the stirrups or shear studs may not protrude into the deck beyond the bottom deck slab reinforcement.

8.20 Concrete girders

8.20.1 General

Add the following:

Prestressed concrete I-girder and box girder skews over 30° shall be avoided where possible. Where skews over 30° are used, sharp corners at ends of girders shall be chamfered as a precaution against breakage.

Box girders shall be skewed in increments of 5°, unless otherwise Consented to by the Ministry.

8.20.3 Flange Thickness for T and Box Girders

8.20.3.2 Bottom Flange

Add the following:

The cross-section dimensions of the Ministry Standard Twin Cell Box Stringers shown on Drawings 2978-1 to 2978-24 (latest revision) shall be considered acceptable for use on Ministry projects.

Commentary: The bottom flange thickness of Ministry standard prestressed concrete box stringers does not comply with the minimum code requirement of 100 mm. No rationale is given in the Code or the Commentary for this minimum requirement.

The current series of standard twin cell boxes have been in use since the late 1970's and have performed extremely well over the years. The increase in cost of fabrication and transportation necessary to update to the cover requirements of S6:19 is not considered to be warranted.

8.20.6 Post-Tensioning Tendons

Add the following:

Unbonded post-tensioning tendons shall not be used.

Commentary: Unbonded tendons have experienced numerous corrosion incidents due to inadequacies in corrosion protection systems, improper installation, or environmental exposure before, during and after construction.

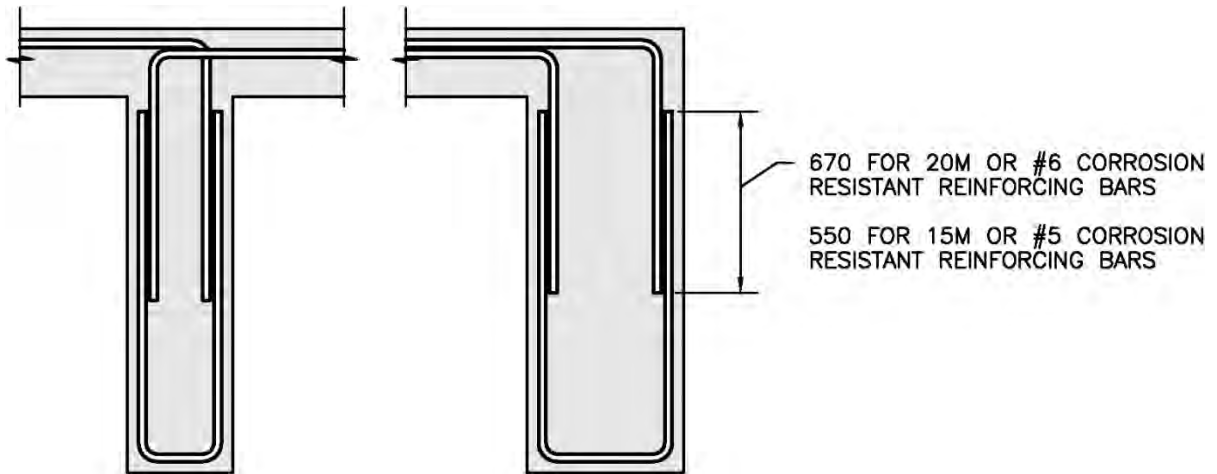
8.20.7 Diaphragms

Delete clause and replace with the following:

Concrete diaphragms shall be provided at abutments and piers. The diaphragms shall be proportioned to support the deck, to transfer loads to the supports and to allow for the future jacking of the girders. Abutment and pier diaphragms shall be oriented parallel to the bridge skew and shall have a minimum thickness of 350 mm. Additional reinforcing shall be placed between longitudinal temperature reinforcement to account for negative moment effects. The minimum added reinforcing shall be 15M bars and shall extend for a distance $S/2$ into the deck slab from the edge of the diaphragm where 'S' is the c/c of stringers. The bars shall have a standard hook at the diaphragm end. Where intermediate concrete diaphragms support the slab, bars shall be added between the longitudinal reinforcing. The bars shall be 15M and be the same bar type as the reinforcing steel in the top mat of the deck and the length shall equal to 'S.'

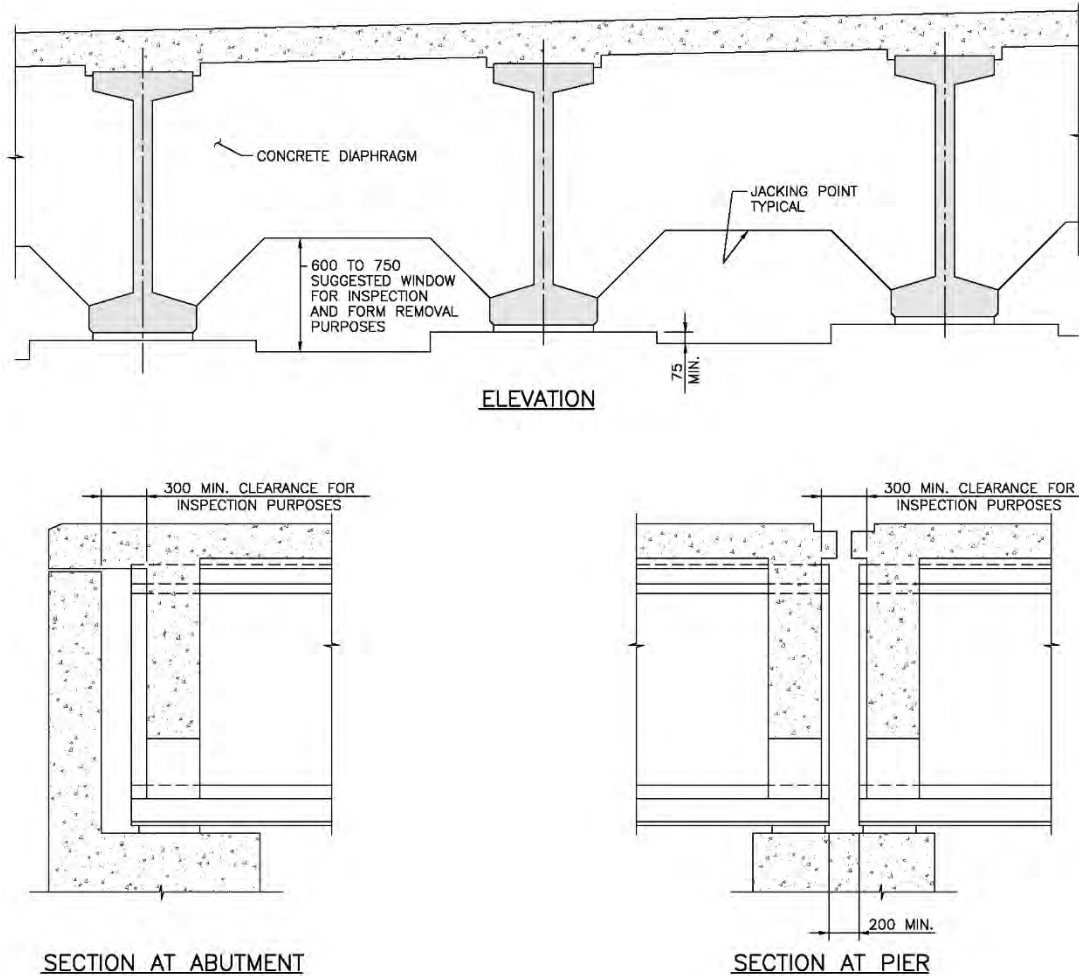
A typical tie arrangement for intermediate and end diaphragms is shown in Figure 8.20.7.1 below.

Figure 8.20.7.1
Typical diaphragm tie arrangement



Diaphragms shall be detailed to provide access for maintenance inspection, as generally outlined in Figure 8.20.7.2 below.

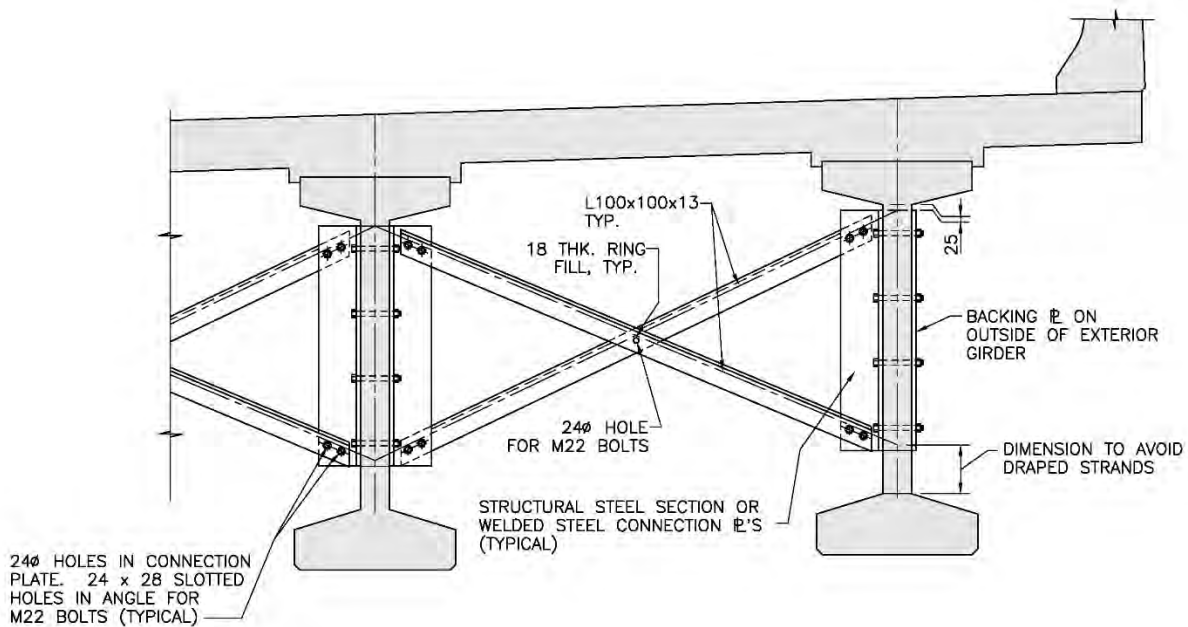
Figure 8.20.7.2
Typical concrete diaphragm arrangement



The hole size for abutment and pier diaphragm reinforcing which passes through the ends of prestressed girders should be 2.5 times the bar diameter.

Unless Consented to by the Ministry, the designer shall provide intermediate diaphragms to improve load distribution and for stability during construction and future rehabilitation. The diaphragms shall be galvanized steel framing with details similar to those in Figure 8.20.7.3 unless analysis dictates the use of a concrete intermediate diaphragm.

Figure 8.20.7.3
Typical steel diaphragm arrangement



8.21 Multi-beam decks

Add the following:

The shear key and reinforcement details shown on Ministry Standard Twin Cell Concrete Box Stringer, Standard Drawings 2978-1 to 2978-24 (latest revision) shall be considered as an approved means for live load shear transfer between multi-beam units in accordance with Clause 8.21(c) of S6:19.

Commentary: Ministry standard box stringers less than 20 m in length without lateral post-tensioning have performed well (no longitudinal cracks or leaks) since they were first introduced in the late 1970's. According to site investigations completed by the Ministry on multi-beam decks with asphalt overlay where transverse post-tensioning was not used, no longitudinal cracking of the asphalt overlay was observed over the shear key areas. The majority of the non-composite box spans investigated were less than 20 m spans.

Standard box stringer bridges up to 30 m may also be used without lateral post-tensioning, provided explicit analysis indicates that the shear key has sufficient live load shear transfer capacity.

In most cases, a composite or non-composite concrete overlay is applied on twin or single cell box beams to provide a wearing surface. The specified and minimum thickness of a reinforced concrete overlay must consider the nominal maximum concrete aggregate size and bar clearance and spacing requirements. Where specified as an alternative to a concrete overlay, or as otherwise Consented to by the Ministry, the top surfaces may be protected with a waterproofing membrane selected from the Ministry's Recognized Products List, and applied in accordance with the manufacturer's instructions with an asphalt overlay of 100 mm placed in two lifts of 50 mm.

Commentary: Figures 8.21 (a) and 8.21 (b) are suggested means of achieving composite action between the structural beam and the composite reinforced concrete deck slab.

Figure 8.21 (a)
Twin cell box beam composite deck slab connection detail

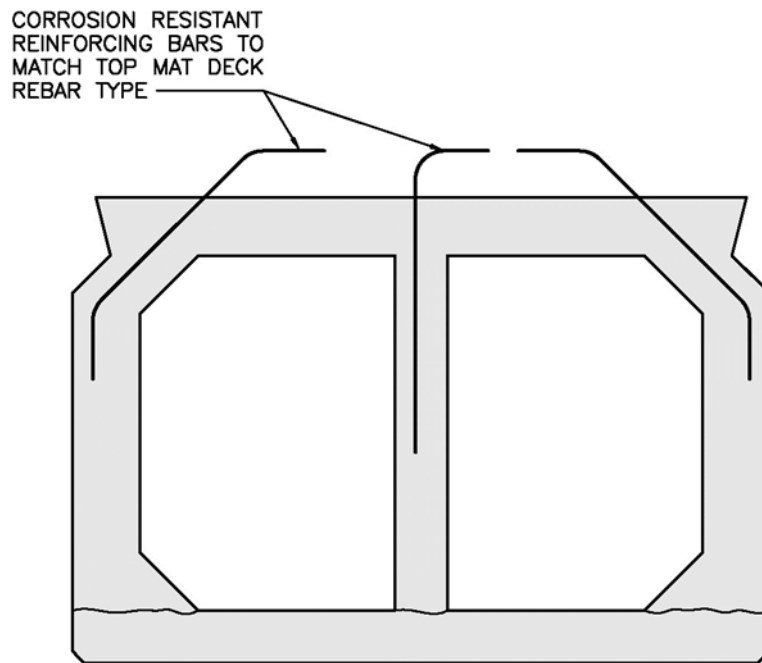
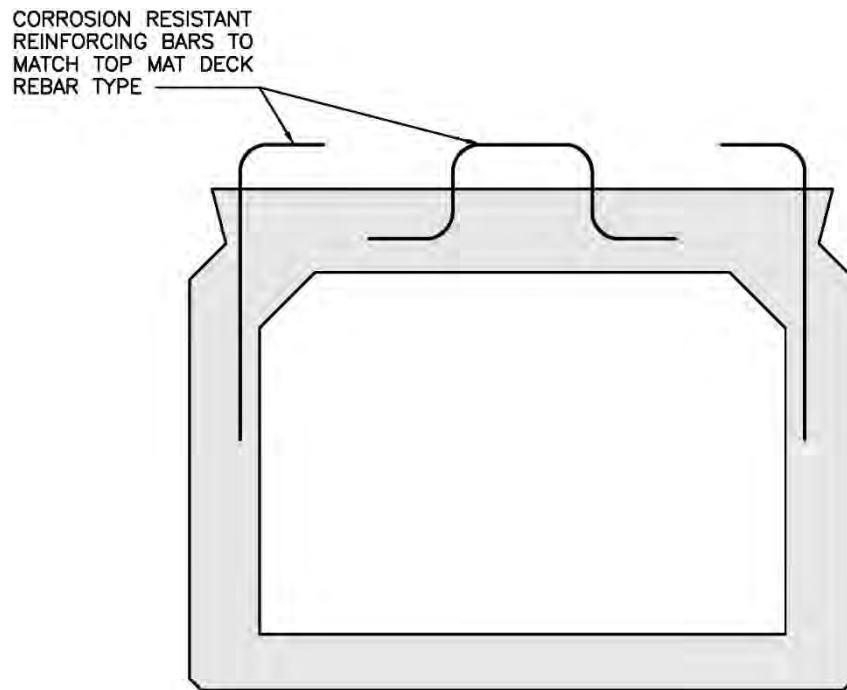


Figure 8.21 (b)
Single cell box beam composite slab connection detail



ANNEX A8.1 Fibre-reinforced concrete (FRC)

A8.1.1 General

Add the following:

FRC shall only be used when Consented to by the Ministry.

9.5 General design 2
 9.5.6 Load-sharing factor 2
9.13 Structural composite lumber 2
 9.13.1 Materials 2

9.5 General design

9.5.6 Load-sharing factor

Add to **Table 9.5 Values of D_e** the following:

Structure	D_e , m
Stringer of Glued-laminated timber stringer bridge	1.75

Commentary: *There is no reference to glue-laminated structures.*

9.13 Structural composite lumber

9.13.1 Materials

Add the following:

Structural Composite Lumber shall not be used unless otherwise Consented to by the Ministry.

Commentary: *Structural composite lumber used in bridges may not meet code requirements because of the high humidity conditions in Coastal British Columbia. The Engineer should contact the structural composite lumber manufacturer and treatment facility to determine if its use will provide adequate durability in coastal British Columbia or in other harsh environments to assure that appropriate durability and warranties are available when structural composite lumber is used.*

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10.4 Materials

10.4.1 General

Delete the second paragraph and replace with the following:

- Coil steel shall not be used unless Consented to by the Ministry.

Commentary: Coil steel undergoes stressing during the rolling and unrolling process that may result in undesirable properties for a given application. It may also be difficult to straighten.

10.4.2 Structural steel

Commentary: The following information is provided as an aid to the designer:

1. *The availability of the required widths and thicknesses of plate should be confirmed early in the design stage, to minimize the amount of shop and field splicing required. Choosing sizes of plates and shapes that are readily available and economical, and that minimize fabrication and erection effort can, to some degree, reduce the cost of the end product.*
Structural steel supplied from the US will likely be supplied in Imperial dimensions. If a large order is placed, mills will produce plates in metric sizes.
2. *Standard metric plate thicknesses are: 6 mm, 9 mm, 13 mm, 14 mm, 16 mm, 19 mm, 22 mm, 25 mm, 32 mm, 38 mm, 44 mm, 51 mm, 57 mm, 64 mm, 70 mm, and 76 mm. (Equivalent imperial plate thicknesses are: ¼", 3/8", ½", 9/16", 5/8", ¾", 7/8", 1", 1-1/4", 1-1/2", 1-3/4", 2", 2-1/4", 2-1/2", 2-3/4", and 3"). Plates thicker than 76 mm (3") are available, but are not common, and therefore should be avoided if possible.*
3. *Standard plate widths are 2440 mm (8'0") and 1830 mm (6'0"). Wider plates may be obtained as a special mill order but long supply times can be expected. Girders more than 8' deep will generally require a longitudinal web splice and, therefore, designers should take into account the added cost associated with the splice when determining the optimum girder depth.*
4. *Provided sufficient quantities are specified (≥100 tonnes) plates and welded wide flanged shapes (WWF) are available in both imperial and metric sizes.*
5. *Rolled shapes are no longer available from Canadian mills. Rolled shapes from US mills are currently available only in imperial sizes. Common metric angle sizes and their Imperial equivalents currently available are: L90x90x8 (L3-1/2"x3-1/2"x 5/16"), L100x100x6 (L4"x4"x1/4"), L100x100x10 (L4"x4"x3/8"), and L125x125x8 (L5"x5"x5/16"). Metric sizes included in steel handbooks are soft conversions of the imperial equivalents.*
6. *Grades of steel used in bridge construction shall preferably be based on their availability. The following sections and grades of steel are usually more readily available than others and their use is recommended wherever possible:*

- a. *Angles and channels, non-weathering: 350W (equivalent to ASTM A572, Grade 50); weathering: ASTM A588, Grade 50A.*
 - b. *Hollow structural sections: 350W or ASTM A500, Grade B. If ASTM A500 is used, the design properties shall be determined from wall thickness equal to 90% of the nominal wall thickness (per CSA S16).*
 - c. *HP Sections: 350W (equivalent to ASTM A572, Grade 50)*
 - d. *Plate: 300W, 350W, 350WT, 350A, 350AT*
 - e. *Structural tees: 350W (equivalent to ASTM A572, Grade 50)*
 - f. *Welded reduced wide flange shapes: 350AT, 350W*
 - g. *Welded wide flange shapes: 350AT, 350W*
 - h. *Wide flange shapes, non-weathering: 350W (equivalent to ASTM A572, Grade 50), weathering ASTM A588, Grade 50.*
 - i. *Anchor rods: ASTM A307, Grade C ($F_y = 250$ MPa, 36 ksi).*
 - j. *Shear studs: (refer to S6:19 Clause 10.4.7)*
7. *Local fabricator experience indicates that rolled sections are usually purchased as conforming to ASTM A572, Grade 50 (non-weathering) or ASTM A588 (weathering steel).*
 8. *Local fabricator experience is that HSS is available as CSA G40.21M, Grade 350W, Class C or ASTM A500, Grade B. Designers are encouraged to design for and specify ASTM A500 This would allow fabricators to use either grade. For ASTM A500, the design properties shall be determined from wall thickness equal to 90% of the nominal wall thickness.*
 9. *The delivery time for welded reduced wide flange and welded wide flange shapes is sufficiently long that fabricators will often fabricate the sections rather than order them from a mill.*
 10. *Higher strength anchor rods such as ASTM A449 or ASTM F1554 (105 ksi) may be used where required.*
 11. *It is recommended that designers not specify one particular grade of shear stud as manufacturers will not guarantee studs to meet one grade.*

10.4.5 Bolts

Add the following after the second paragraph:

In general supply of bolts shall comply with the following:

1. Bolts shall preferably be 22 mm (7/8") in diameter, although larger diameters may be used where they are deemed beneficial.
2. Bolt size and grade should be uniform throughout the design as much as possible.
3. Availability of bolts (standard, size and quantity) should be confirmed prior to start of design.
4. ASTM F3125 Grade A490 bolts, nuts, and washers shall not be used unless Consented to by the Ministry.

5. Tension control bolt-nut washer assemblies (such as ASTM F1852 and ASTM F2280) and direct tension indicating (DTI) washers (such as ASTM F959) shall only be used when Consented to by the Ministry.

Commentary:

1. Bolts are not typically available in Metric sizes, therefore, the designer should confirm the availability of bolt size and type prior to design.
2. In general, one size of bolt should be used on an entire bridge to avoid the need for multiple size wrenches and impact guns, and to avoid the possibility of undersized bolts being inadvertently installed where larger ones were specified.
3. A490 bolts are less ductile than A325 bolts and cannot be galvanized. In unusual situations where A325 bolts cannot be used, A490 bolts may be considered by the Ministry.
4. See the Ministry SS 421.11.03 for coating requirements for bolts.

10.4.10 Galvanizing and metallizing

For steel that is to be hot-dip galvanized, the following restriction is made in addition to the chemical composition (heat analysis) requirements of CSA G40.21:

- Si content; less than 0.03% or within a range of 0.15% to 0.25%
- C content; maximum of 0.25%.
- P content; maximum of 0.05%
- Mn content; maximum of 1.35%

Commentary: These elements are restricted to mitigate their adverse effects on galvanizing.

10.4.13 Pins and rollers

Add the following paragraph:

Pins and rollers shall conform to ASTM A668 and ASTM A108 as appropriate.

10.6 Durability

10.6.3 Corrosion protection

Add the following paragraphs:

Primary superstructure members shall be corrosion-resistant weathering steel unless otherwise Consented to by the Ministry.

Bracing members fabricated from 300W or 350W steel shall be coated for corrosion resistance. For bracing members of these materials, the preferred method of coating shall be galvanizing or metallizing. If galvanizing or metallizing are inappropriate (e.g. for aesthetic reasons), bracing shall be coated in accordance with SS or DBSS 421.

Commentary: Due to the initial and ongoing maintenance cost of painting, it is recommended that corrosion-resistant weathering steel be used where appropriate.

10.6.4 Superstructure components

10.6.4.2 Structural steel

Delete the first paragraph and replace with the following:

For weathering steel structures, all structural steel, including contact surfaces of bolted joints, diaphragms and bracing but excluding surfaces in contact with concrete, shall be coated with a coating system selected from the Ministry's Recognized Products List. The coating shall extend for the larger of the following two distances from locations of deck joints, such as at expansion joints and fixed joints:

- 3000 mm; or
- 1.5 x the structure depth.

In the above, the structure depth shall include the girder, haunch, and slab heights.

In areas of high exposure and for elements that are critical to the structure, the designer may consider metallizing the zone as described above. If the metallized zones will be visible from the outside of the bridge, they shall also be top-coated with paint selected from the Ministry's Recognized Products List to match the colour of the adjacent steel elements.

For structures not using weathering steel, the steel shall be coated with a coating system selected in accordance with SS/DBSS 421 unless Consented to by the Ministry.

In marine environments, or where the steel is likely to be exposed to de-icing chemicals, the steel shall be coated.

The designer shall provide details that avoid situations where water is allowed to pool on girder flanges. Where this cannot be avoided, such areas shall be painted with an immersion-grade coating system.

Bottom flange water deflector plates shall be installed as per Figure 11.6.6.6 (a).

Commentary: *Experience has shown that there is little benefit from specifying corrosion-resistant steel and a complete paint system on the entire bridge. However, there may be situations where good design practice would require both.*

10.6.4.3 Cables, ropes, and strands

Delete the first paragraph and replace with the following:

A method of corrosion protection, as Consented to by the Ministry, shall be used for all wires in the cables and hangers of suspension bridges, stay cables of cable-stayed bridges, arch bridge hangers and other cables, ropes and strands used in bridges.

Commentary: *Corrosion protection systems for cables are advancing rapidly. As such, discussion with the Ministry is required when cables are used. As a minimum, wires will be hot-dip galvanized as per this clause.*

10.7 Design details

10.7.1 General

Add the following:

Design detailing shall address constraint induced fracture.

Commentary: For helpful background information and suggested details regarding the design of steel bridges, designers may refer to “Guidelines for Design Constructability,” AASHTO/NSBA Steel Bridge Collaboration, Document G12.1-2016. In the event of conflict with Canadian Standards, Canadian Standards shall prevail.

The document may be referenced at:

<https://www.aisc.org/globalassets/nsba/aashto-nsba-collab-docs/nsbaqdc-3.pdf>

NSBA is the US-based National Steel Bridge Alliance.

AASHTO LRFD Bridge Design Specifications has design specifications for constraint induced fracture.

Add the following clauses:

10.7.1.1 Flange widths between splices

Unless economic analysis indicates that other arrangements are more cost-effective, flange plate widths should be kept constant between field splices.

Commentary: Flanges for girders are purchased in economical multi-width plates. Where a change in flange thickness occurs, the mill plates are butt welded together. If the flange width is constant for a given shipping length, the plates can be stripped into multiple flanges in one continuous operation. When determining flange widths, the designer should take into account that plate typically comes in 2440 mm (8'-0") and/or 1830 mm (6'-0") widths (depending on availability).

10.7.1.2 Transition of flange thicknesses at butt welds

Transition of flange thickness at butt welds shall be made in accordance with CSA W59, with a slope through the transition zone not greater than 1 in 2.

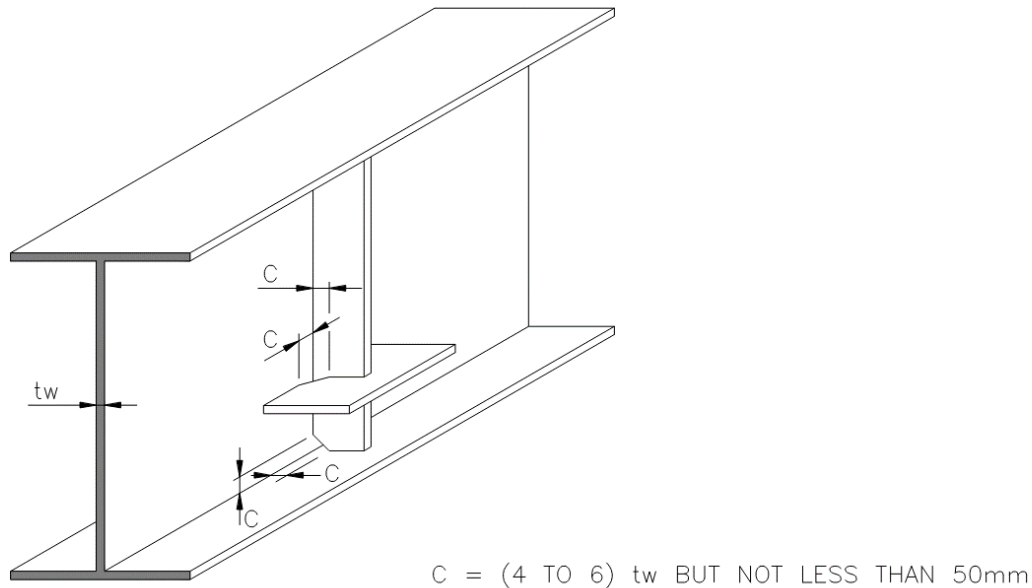
Commentary: A slope of 1 in 2 can be produced by burning followed by grinding in the direction of primary stress. Research indicates that this detail achieves the required fatigue categories. Less steep slopes require more expensive fabrication methods with no significant compensating improvement in fatigue classification.

10.7.1.3 Recommended details

10.7.1.3.1 Coping of stiffeners and gusset plates

As shown in Figure 10.7.1.3.1 for I-girders with vertical webs, copes on details such as stiffeners and gusset plates shall be 4 to 6 times the girder web thickness but not less than 50 mm.

Figure 10.7.1.3.1
Coping of stiffeners of gusset plates



Commentary: Copes as dimensioned above are desirable because they:

- prevent the possibility of intersecting welds;
- reduce the high weld shrinkage strains associated with smaller copes;
- allow drainage, and;
- facilitate access for welding.

For other situations such as the horizontal flange of a box girder with transverse stiffeners, refer to the latest edition of “Bridge Fatigue Guide Design and Details” by J.W. Fisher.

10.7.1.3.2 Gusset plates for lateral bracing

All gusset plates for lateral bracing should be fillet welded and designed in accordance with 10.17.3. As shown in Figure 10.7.1.3.1. The outer corners of the gusset plates should be left square. “Bridge Fatigue Guide, Design and Details” by J.W. Fisher should be consulted when determining the location of bolt holes.

Commentary: The position of lateral bracing gusset plates should allow for access for fabrication and inspection of the gusset plate-to-web connection.

Although this is a preferred detail, under certain circumstances (such as when fatigue stresses govern) a designer may wish to consider a radiused gusset plate or a bolted connection.

10.7.1.3.3 Frames for lateral bracing, cross-frames and diaphragms

Frames (assemblies of bracing elements and connecting plates) should be used for lateral bracing, cross-frames and diaphragms in lieu of angle sections shipped loose to the site. The frames for use between girders should be detailed for shipping and erection as a single unit. A sample arrangement is shown in Figure 10.7.1.3.3.

Frames should be designed for fabrication from one side, eliminating the need for “turning over” during fabrication. Oversized holes in the gusset plates are permitted.

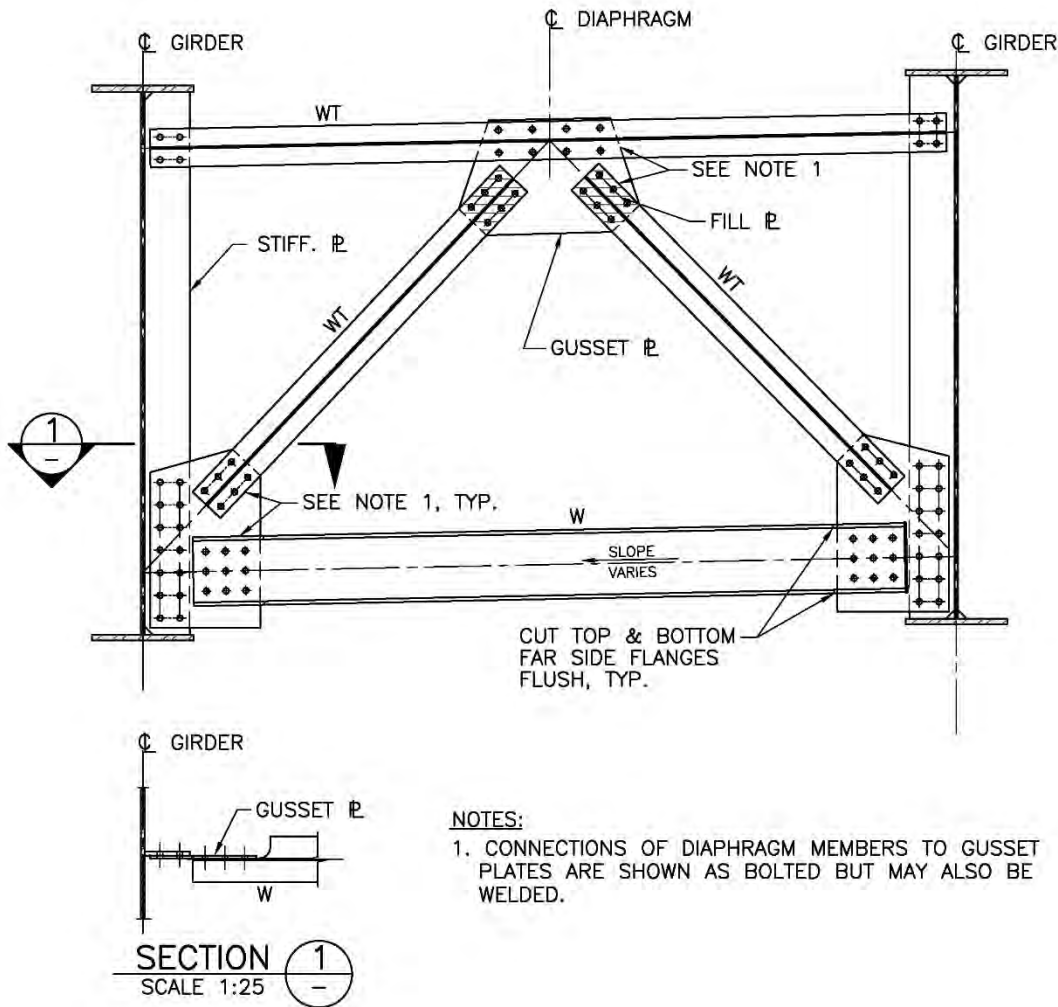
Bracing shall be designed to accommodate construction loading and the final loading on the structure. The designer shall identify any assumptions regarding construction loading on the plans.

Commentary: *Bracing intended for construction purposes which remain in the in-service structure can be subject to significant loads and have undesirable effects, potentially leading to fatigue failures.*

The designer shall account for eccentric force effects for both strength and fatigue arising from the arrangement described above.

The arrangement described above may result in heavy members, stiffeners and connections because of additional stresses from eccentric load paths that must be carefully accounted for in the design.

Figure 10.7.1.3.3
Typical diaphragm



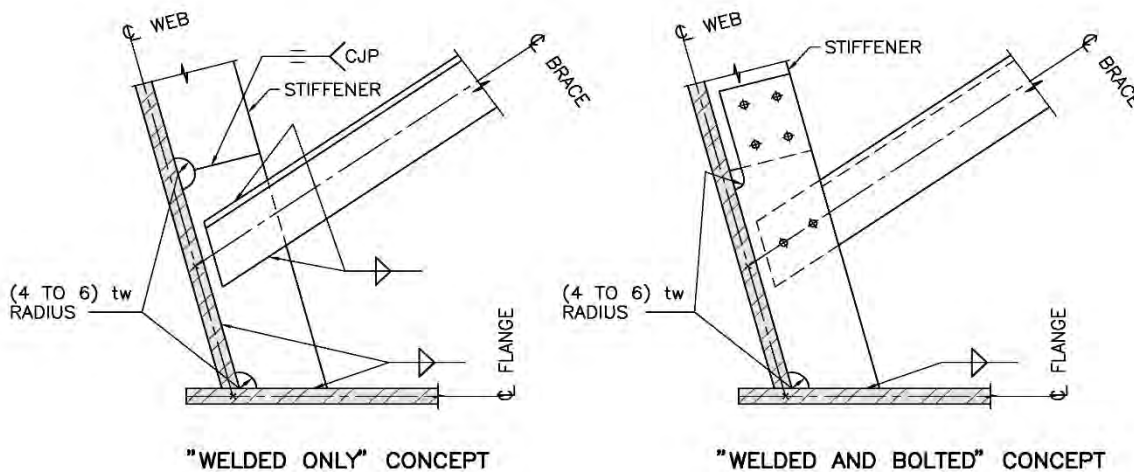
Commentary: Frame brace systems for use between girders should consist of angles or tees shop welded to one side of gusset plates which would be field bolted to the girder stiffeners. Efficient fabrication and erection procedures result when frames can be produced in one jig and when fewer pieces are handled in the field.

10.7.1.3.4 Box girder diaphragm bracing

Unless design requirements dictate otherwise, 100 x 100 x 10 mm angles should be considered as a standard angle size for box girder bracing. If additional interior bracing is required for handling of the girders (in excess of what the contract drawings call for), the fabricator shall propose such on the shop drawings which shall then be subject to approval by the designer and the Consent of the Ministry. Care

shall be exercised to address issues of constructability, account for eccentric load paths, satisfy the Ultimate Limit State and preclude those details that would compromise the Fatigue Limit State requirements. Figure 10.7.1.3.4 suggests two concepts for consideration.

Figure 10.7.1.3.4
Box girder bracing at diaphragm



Commentary: Because of minimum tonnage orders that can be placed with mills, standardization of angle bracing will result in economy. The 100 x 100 x 10 angle is believed to be adequate for the normal range of bridge spans.

10.7.1.3.5 Intermediate diaphragms in shallow girders

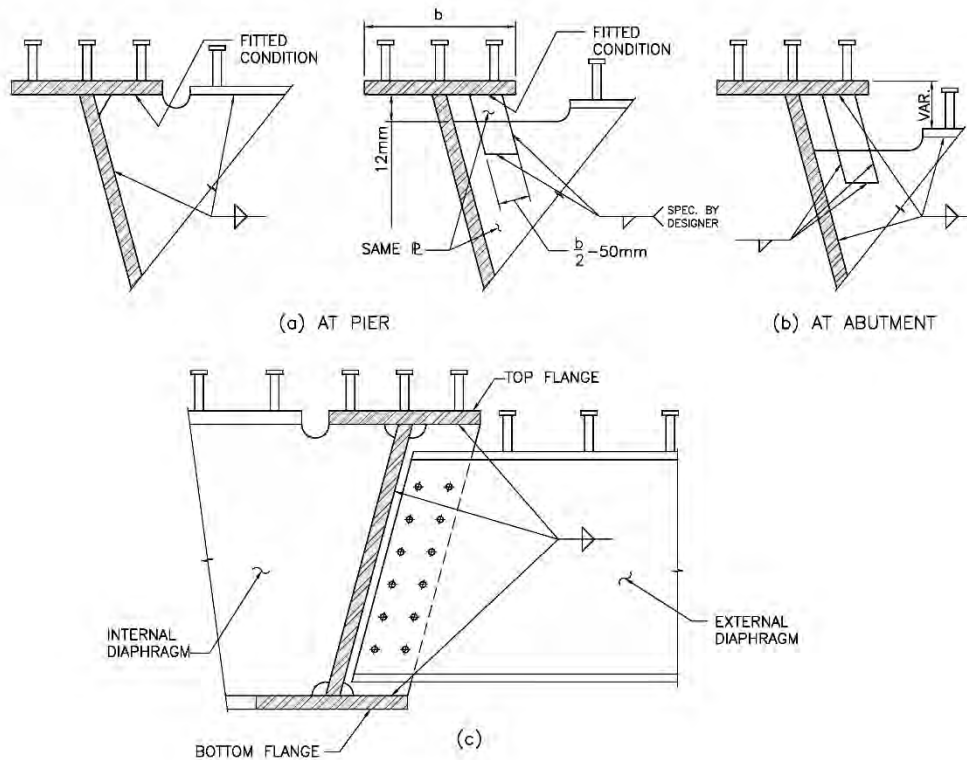
Constant depth intermediate diaphragms, in lieu of frame bracing, are preferred in I-girders bridges up to approximately 1200 mm in girder depth.

Commentary: Diaphragms comprising channel or beam sections would be less expensive in shallow bridges.

10.7.1.3.6 Box girder diaphragms at piers and abutments

Diaphragms at piers should be detailed so that the box girder and diaphragm flanges are not connected (see Figure 10.7.1.3.6 (a)) showing two possible solutions. Also, provisions for jacking within the width of the bottom flange should be provided for by the designer. Diaphragms at abutments are normally of a shallower depth to allow for deck details. In this case, the box girder flanges should be stabilized against rotation (see Figure 10.7.1.3.6 (b)). Diaphragms between box girders at piers and abutments should be of constant depth and bolted to exterior box girder web stiffeners (see Figure 10.7.1.3.6 (c)). Oversized holes in diaphragms or stiffeners are permitted.

Figure 10.7.1.3.6
Box girder diaphragms



Commentary: The details as shown in Figure 10.7.1.3.6, are suggested to meet design and fabrication needs.

10.7.1.3.7 Transitions of box girder flange and web thicknesses

Flange thickness transitions should be made so that a constant depth web plate is maintained. Web thickness transitions should be made to maintain a flush inner box girder face.

Commentary: Flange thickness transitions, made so that a constant web depth is maintained, result in economy. Web thickness transitions made so that a flush inner face is maintained makes for repetition of inner diaphragms which then act as “templates” for maintaining the geometric shape of the box. Different fabricators with different equipment and assembly procedures will have distinct opinions and different preferences and there are no rigid rules that would satisfy all conditions. Note that eccentric transitions produce small local bending effects which can be significant where elastic instability is possible, e.g., in tension plates temporarily subject to compression during construction.

If erection by launching is an option contemplated in the design, the underside of the bottom flange should be kept a constant width to facilitate lateral guiding and the plate thickness transitions should be made into the web to have a flush bottom flange surface in contact with the supports.

10.7.1.3.8 Grinding of butt welds

Grinding of butt welds shall be finished parallel to the direction of primary tensile stress and in accordance with CSA W59.

Butt welds in webs of girders designed for tension in Category B shall be “flush” for a distance of at least 1/3 the web depth from the tension flange.

All other butt welds designed for tension in Category B shall be “flush”.

Butt welds designed for compression only or for stresses in Category C shall be at least “smooth”.

“Flush” is defined as a smooth gradual transition between base and weld metal, involving grinding where necessary to remove all surface lines and to permit RT and UT examination. Weld reinforcement not exceeding 1 mm in height may remain on each surface, unless the weld is part of a faying surface, in which case all reinforcement shall be removed.

“Smooth” is defined for the surface finish of weld reinforcement to provide a sufficiently smooth gradual transition, involving grinding where necessary to remove all surface lines and to permit RT or UT examination. Weld reinforcement not exceeding the following limits may remain on each surface:

- for plate thicknesses ≤ 50 mm, 2 mm
- for plate thicknesses > 50 mm, 3 mm

Commentary: *In webs of girders, butt welds more than approximately 1/3 the girder depth from the tension flange are in a lower stress range. Top flange of composite girder with welded nelson stud is considered as Category C. This results in a less severe fatigue category not requiring the “flush” condition. The designer is responsible for confirming whether more or less stringent limits are warranted.*

Where the contour of the weld is to be “smooth” grinding may be required to permit RT or UT examination of the tension welds. Compression welds may require grinding if the weld reinforcement limits specified above are not met.

10.7.1.3.9 Vertical stiffeners

Bearing stiffeners on plate girder bridges shall be true vertical under full dead load with the requirement noted on the Plans. Intermediate stiffeners may be either true vertical, or perpendicular to fabrication work lines, depending on the fabricator’s practice.

Commentary: *The requirement for bearing stiffeners to be true vertical under full dead load is primarily for aesthetics with the normal pier and abutment designs. Vertical diaphragms would also result at the bearing points which will facilitate the jacking arrangement for bearing maintenance. Some fabricators choose to work from a horizontal work line on the webs of girders and install intermediate stiffeners perpendicular to these work lines with the girder in a relaxed condition. When the dead load acts, the intermediate stiffeners are not vertical, but the difference is slight with no functional loss.*

If all stiffeners (bearing, intermediate and diaphragms) are vertical then modular repetition of the lateral bracing system can be attained which may be desirable for detailing and fabrication.

10.7.1.3.11 Intermediate stiffener to flange connection

In plate girders up to a depth of 2000 mm, in the positive moment regions, the intermediate stiffeners shall be cut short of the tension flange except that stiffeners at lateral bracing, cross-frame, and diaphragm connections may be either fitted, bolted or welded to the tension flange, depending on the strength and fatigue requirements.

In plate girders over a depth of 2000 mm, all intermediate stiffeners should be welded to the compression flange. The stiffeners can be welded, bolted or fitted to the tension flange, depending on the strength and fatigue requirements and economic considerations.

Commentary: *In plate girders over a depth of 2000 mm, racking of the flanges during shipment may result in cracks forming in the web/flange weld if intermediate stiffeners are cut short of the flange. To avoid this problem, the intermediate stiffeners should be fitted, bolted or welded to the tension flange. If the stiffeners are on one side of the web only, fabrication and transportation requirements may dictate some additional means of preventing flange rotation.*

10.7.1.3.12 Stiffener to web connection

All stiffeners shall be welded to the webs of the girders by continuous fillet welds, of the minimum required size.

Commentary: *Continuous welding improves the fatigue performance in a girder by reducing the number of stress raisers. The minimum weld size is specified to reduce residual stresses and web deformations.*

10.7.1.3.13 Intersecting longitudinal and transverse stiffeners

Longitudinal stiffeners shall be located on the opposite side of the girder web to intermediate transverse stiffeners, unless detailing precludes this. Where longitudinal and transverse stiffeners intersect, the longitudinal stiffener should be cut short of the transverse stiffener. However, in tension regions, where fatigue is a governing design criterion, and where longitudinal and transverse stiffeners intersect, the longitudinal stiffener may be made continuous and the transverse stiffener welded to it at the intersection.

Commentary: *Longitudinal stiffeners should be continuous as much as practical, especially in the case of fracture-critical members. The designer may wish to modify the design to avoid the need for longitudinal stiffeners which may result in more material but potentially cheaper fabrication.*

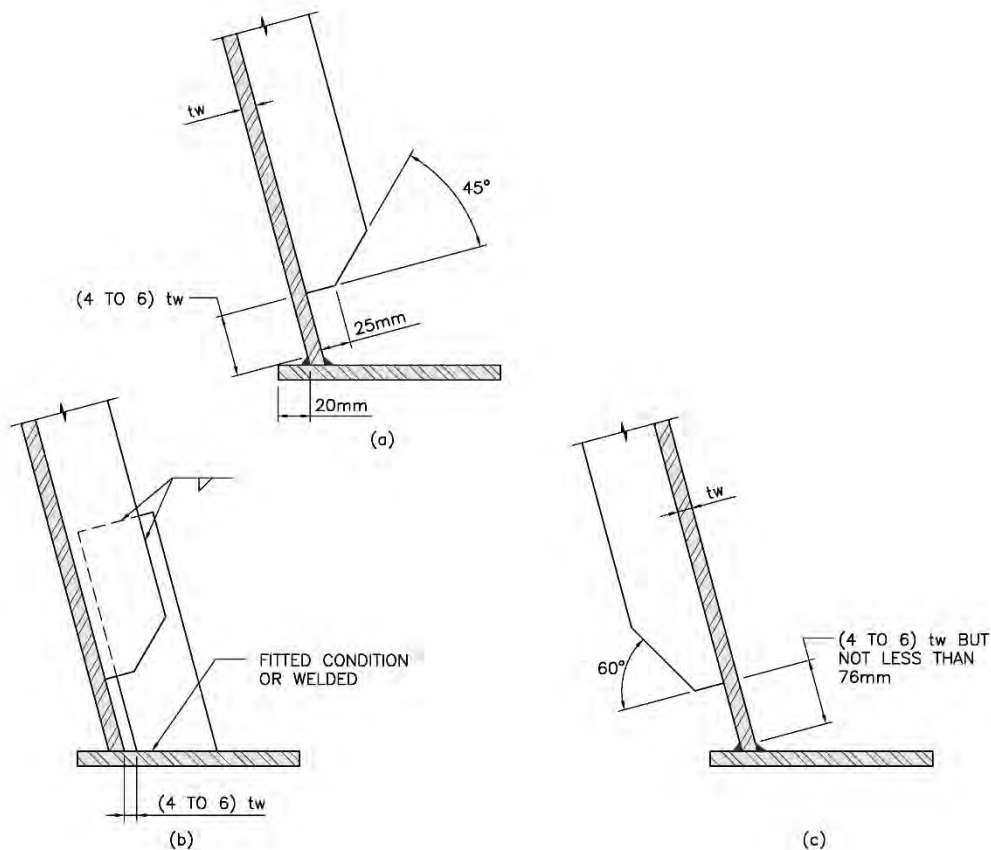
Locating longitudinal and transverse stiffeners on opposite sides of girder webs facilitates fabrication and reduces the number of stress-raisers in the web of the girder.

Where intersection of stiffeners is unavoidable, cutting the longitudinal stiffener in tension regions results in a Category E detail which may be improved by providing a radiused transition if this Category is too severe, or by making the longitudinal stiffener continuous and welding the transverse stiffener to it, resulting in a Category C detail.

10.7.1.3.14 Box girder intermediate web stiffeners

Intermediate web stiffeners on the inner and outer faces of box girders should be cut short of the bottom flange (see Figures 10.7.1.3.14 (a) and 10.7.1.3.14 (c)). If a fitted condition is required due to design, an additional plate may be provided (see Figure 10.7.1.3.14 (b)).

Figure 10.7.1.3.14
Box girder intermediate web stiffeners

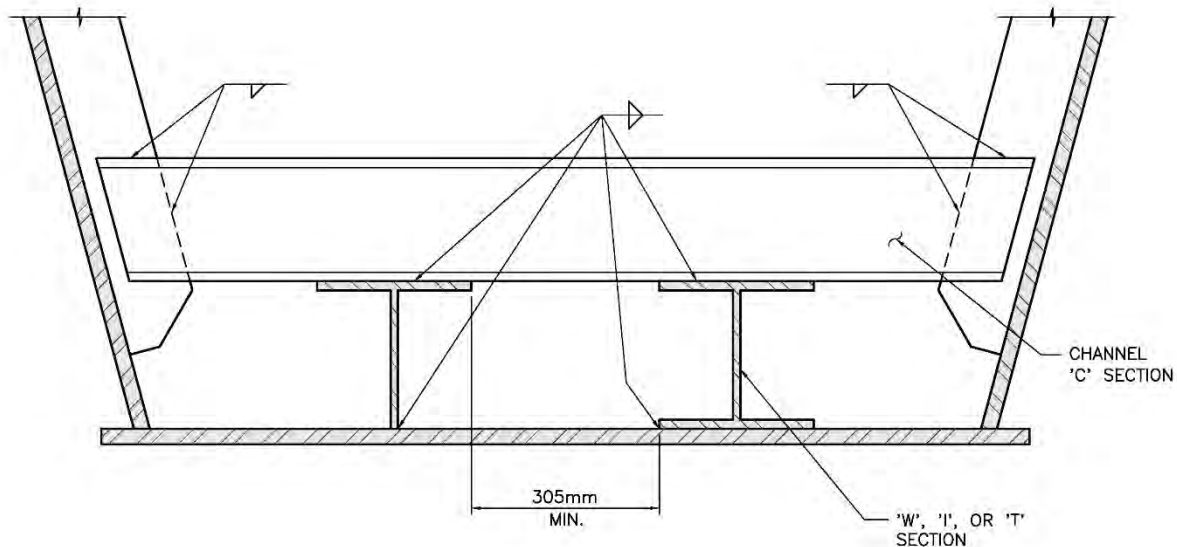


Commentary: In order to allow the use of automatic welding of the web-to-flange joint, the details as shown in Figures 10.7.1.3.14 (a) and 10.7.1.3.14 (c) are essential. The process of fabricating the box girders calls for the web stiffeners to be welded prior to welding the web to the flanges.

10.7.1.3.15 Box girder bottom flange stiffener details

Wide flange "I" or "T" section longitudinal stiffeners shown in Figure 10.7.1.3.15 are preferred over plate stiffeners. The sections should be spaced a minimum of 305 mm between flanges to facilitate automatic welding. Channel sections, welded to the top flange of the longitudinal stiffeners and to the inner web stiffeners, are the preferred arrangement for transverse stiffening.

Figure 10.7.1.3.15
Box girder bottom flange stiffener details



10.7.4 Camber

10.7.4.1 Design

Delete the second paragraph and replace with the following:

Camber information shall be provided by the designer. Camber shall be shown at splice points and at intervals not greater than 2 m.

A camber diagram shall be included in the Plans and shall include elevations for:

- the target finished steel girder grades.
- camber profiles for deflections due to the deck, locked in effects due to construction including formwork, curbs, sidewalks, barriers, railings, wearing surface, creep and shrinkage, and utilities.
- camber profiles for deflections due to steelwork (girders, beams, bracing, diaphragms etc.)

Commentary: Item (c) is required by the steel fabricator. Item (b) is required by the erector to set the girders. Differences between the surveyed profile of the erected steelwork and Item (b) are used to adjust the height of slab haunches over the girders to attain the target finished grade profile.

10.18 Splices and connections

10.18.1 General

Add the following paragraphs:

Field splices shall be bolted connections.

Locations of slip-critical connections shall be shown on the Plans.

Connections for cables (hangers, suspension cables, cable stays, etc.) shall be designed and/or specified so that the ultimate breaking strength of the connection exceeds the maximum guaranteed tensile strength of the cable.

Commentary: *This requirement is included to ensure that failure occurs via yielding of the cable element and not failure of the connection.*

Compression flange splices shall use a bolted connection designed using the forces given in Clause 10.18.1.1.

Commentary: *Splices for compression flanges that rely on bearing between the ends of the flange plates are not permitted. This splice detail has been used on bridges with precast deck panels to avoid splice plates above the top flange and therefore simplify deck panel fabrication and installation. However, during the evaluation of extreme overloads, sections with this splice detail have been found to limit capacity.*

10.18.3 Welds

10.18.3.1 General

Add the following:

The designer shall specify any testing requirements for welds that are required in addition to the testing requirements of SS/DBSS 421.

Commentary: *SS/DBSS 421 generally outlines the expected extent of weld testing for Ministry projects which the designer should be familiar with. The designer is reminded that W59 requires the engineer to specify the type and extent of testing for welds.*

10.19 Anchor rods

10.19.1 General

Delete the second paragraph and replace with the following:

- Anchors shall comply with the following:
- Anchor rods for bearing assemblies shall have a minimum diameter of 25 mm and a minimum embedment length of 300 mm.
- Anchor rods, including nuts and washers, shall be galvanized or metallized;
- Anchor rod nuts shall be secured by spoiling the threads after installation;
- Proprietary anchorage systems may be used only when Consented to by the Ministry;
- Mechanical anchorage systems shall not be used.

A10.1 Construction requirements for structural steel

A10.1.1 General

Add the following:

Construction shall be in accordance with SS/DBSS 421 unless amended by the Supplement or otherwise Consented to by the Ministry.

The Plans shall clearly define all construction requirements.

A10.1.5 Welded construction

A10.1.5.1 General

Add the following:

Field welding of attachments to girders shall only be permitted where Consented to by the Ministry.

***Commentary:** Quality Assurance of field welding can be problematic. Field welding is strongly discouraged but permission may be granted in unique circumstances.*

A10.1.6 Bolted construction

A10.1.6.4 Installation of bolts

Add the following paragraph:

Fully tensioned bolts shall be installed in all bolt holes used for erection.

A10.1.8.2 Non-destructive testing of welds

Add the following:

Ultrasonic Testing (UT) may supplement Radiographic Testing (RT) when Consented to by the Ministry.

***Commentary:** In thicker plate, UT testing may reveal defects not readily apparent from the RT testing.*

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11.4 Common requirements

11.4.1 General

Delete the fourth paragraph and replace with:

All exposed and embedded steel components of joints and bearings shall be protected against corrosion. The corrosion protection system shall either be:

- hot-dip galvanizing in accordance with ASTM A123/A123M, or,
- metalizing to AWS C2.23M/C2.23 with a minimum zinc coating thickness of 0.2 mm, or
- a coating system which is selected from the Ministry's Recognized Product List.

The choice of corrosion system shall be subject to the Consent of the Ministry.

The steel/concrete interface for both joints and bearings shall be detailed such that no rust staining of the concrete occurs.

Add the following to the list in the fifth paragraph:

- k) Traffic noise caused by the deck joint system.
- l) Ride-ability for all bridge users including motorcycles, bicycles and pedestrians.

Commentary: *Ministry experience has shown that bridge maintenance and rehabilitation is most commonly associated with deck joints and bearings. Designers should consider structural forms, such as integral and semi-integral abutments, continuous girders, and fixed pier joints, which either eliminate or minimize the use of deck joints and bearings.*

Where bearing assemblies are required to support structural steel girders fabricated from atmospheric corrosion resisting steel, the use of similar material for bearing plates may be considered.

11.5 Deck joints

11.5.1 General requirements

11.5.1.1 Functional requirements

Add the following:

All deck joints, except finger plate joints, shall be sealed. All deck joints shall have armour.

Unless otherwise Consented to by the Ministry, expansion joints shall be designed as finger plate deck joints when the total movement exceeds 100 mm. When the total movement becomes too large for finger plate joints to meet the functional requirements of the joint, then modular deck joints shall be used when Consented to by the Ministry.

In regions of high seismicity where large relative displacements may occur at deck joints, the joints chosen shall be suitable for the performance requirements.

Commentary: *The Ontario requirements for modular deck joints systems can provide guidance to designers. The Ministry has had past performance issues with modular joints, including fatigue and*

premature deterioration of springs and sliders. Additional considerations are necessary for contract specifications for modular joints. Designers are cautioned that large movement finger joints require special consideration for safe passage of motorcyclists, cyclists and pedestrians. The limit of 100 mm has been developed based upon the Ministry experience with the performance of strip and compression seals.

Add to the fourth paragraph:

Deck joints with a left hand forward skew angles between 32 and 38 degrees shall be avoided by designers. For right hand forward skew angles between 32 and 38 degrees, the designer shall confirm the maintenance practices at the bridge site.

Commentary: *On bridges with large skews there is the possibility that the skew angle could match the angle used on snowplow blades (which is generally about 35 degrees) and this could result in a blade dropping into a deck joint and damaging it.*

In general, the use of deck joints should preferably be limited to skew angles of 30 degrees or less. The joint type should be carefully selected and detailed to accommodate the transverse displacements that are commonly experienced in skewed deck joint applications.

Add to the end of the sixth paragraph:

Cover plates over joints on bicycle paths or pedestrian walkways which are greater than 100 mm in width shall be surfaced with a non-skid protective coating acceptable to the Ministry.

Add the following:

Proprietary joint products shall be Consented to by the Ministry prior to use on a Project.

Gaps between the superstructure end-diaphragm and the substructure directly exposed to backfill shall be waterproofed with a glued EPDM membrane to prevent ingress of moisture from the backfill.

Commentary: *Ministry experience is that peel and stick type membranes have not performed well and should not be used. Properly installed EPDM membrane provides greater longevity than modified bitumen or SBS type membrane. Designers must carefully consider structure articulation and seam locations when detailing waterproofing. For structures with tall abutments and return walls, drainboard and supplementary drains may improve the longevity of the installed EPDM membrane. Similar waterproofing details are provided on the standard reference details for box girder bridges.*

11.5.2 Selection

11.5.2.1 Number of joints

Commentary: *The main weakness in the various forms of deck joints has been the lack of durability and associated maintenance problems. Minimizing or eliminating deck joints should improve overall lifecycle performance. Where feasible, semi-integral or integral abutments should be considered in consultation with the Ministry.*

Damage to deck joints can be attributed to the increase in traffic volumes, especially heavier vehicles. Impact forces caused by vehicles passing over expansion joints combined with poor detailing has resulted in the leakage of surface run-off and de-icing salts onto the substructure and bearings.

11.5.2.3 Types of deck joints

Add the following:

Joints at piers shall only be used when Consented to by the Ministry.

Commentary: *Ministry experience has shown that a significant proportion of bridge maintenance and rehabilitation is attributable to poorly performing deck joints. Designers should select joint types with a reliable track record, and which ensures that wearable components are accessible and maintainable with minimal disruption to traffic. Good design and correct installation are key to good performance. Where feasible, expansion joints should be located at the abutments for accessibility.*

11.5.3 Design

11.5.3.1 Bridge deck movements

11.5.3.1.2 Open deck joints

Delete paragraph and replace with the following:

Only properly detailed finger plate joints, including appropriate drainage details below the joint to accommodate debris and run-off water, Consented to by the Ministry, shall be use as an open deck joint. Openings in finger joints shall not exceed 60 mm in the least direction at SLS. Control of deck drainage is mandatory and shall be detailed in accordance with Clause 11.5.8.

Commentary: *Ministry experience has shown that well designed cantilever finger joints require minimal maintenance. Sliding finger joints are susceptible to debris accumulation and wear of the sliding surface. Consideration should be given to designing the joint system so that it can be removed and replaced in sections.*

Some finger joint configurations can present a problem for bicycles. Designers should consider options to meet the functional requirements.

11.5.3.2 Components

Commentary: *anchors should penetrate the reinforcing cage to achieve the required joint anchorage., The designer should consider compatibility of the anchor spacing and details with the embedded reinforcement to ensure correct fit-up of the joint assembly.*

11.5.3.2.4 Bolts

Delete and replace with the following:

All anchor bolts for bridging plates, joint seals, and joint anchors shall be high-strength bolts fully tensioned as specified. Cast-in-place anchors shall be used for all new construction unless otherwise

consented to by the Ministry. Expansion anchors shall not be used for any joint connection. Drilled-in epoxy anchors may be used when Consented to by the Ministry. Tapered-head countersunk anchor bolts shall only be used when Consented to by the Ministry.

11.5.6 Joint seals

Add the following:

Only deck joint seals made of natural rubber or virgin neoprene shall be used. The use of silicone requires Ministry Consent.

Commentary: Deck joint seals made of tyfoprene and santoprene have been observed to perform poorly and are not allowed. Silicone can have a significant cost premium.

11.5.8 Open joint drainage

Delete and replace with the following:

"Finger" plate deck expansion joints shall have a drainage trough installed beneath. The drainage trough design shall use of a corrosion-resistant material such as high-density polyethylene (HDPE) or stainless steel. The trough shall be robust enough to prevent deflection when fully loaded with wet sand. All steelwork supporting the trough shall be galvanized or metallized after fabrication.

Where HDPE material is used for joint drainage, the material shall be UV-resistant. The design shall accommodate the coefficient of thermal expansion of HDPE which is an order of magnitude greater than steel.

Slopes for drainage troughs shall be maximized and where possible, the drainage trough should be sloped at a minimum of 10%. A 50 mm hose bib connection shall be provided to deck level, at the top end of the trough, to allow easy access and attachment for flushing and cleaning of the drainage trough during maintenance. Adequate space shall be provided to the drain trough and other drainage hardware for inspection and maintenance.

Commentary: Deflection plates may be required between the underside of the finger joint and the top of the drainage trough to guide water into the trough.

11.6 Bridge bearings

11.6.1 General

11.6.1.1 Design considerations

Add the following to the first paragraph:

Elastomeric bearings shall be used whenever possible.

Add the following to the end of the seventh paragraph:

Bearing replacement shall be considered in the design. Jacking points and design jacking loads shall be shown on the Plans.

Sufficient horizontal and vertical space shall be provided between the superstructure and substructure to accommodate the required jacks for replacing the bearings.

Commentary: *A minimum of 150 mm is suggested for typical bridges up to 35 m in span, larger spans may require specific assessment. Ensure that the end diaphragm is detailed for jacking if this is part of the design intent (e.g. that web stiffeners or additional reinforcing are included).*

Connections between girders and sole plates and the bearings and sole plates etc., must use bolts or cap screws on at least one interface to facilitate maintenance and replacement.

Commentary: *Elastomeric bearings accommodate the bi-axial rotational and displacements that are typically required for most bridge bearing applications. By accommodating superstructure displacements with shear strains, elastomeric bearings reduce maintenance requirements. Ministry experience is that correctly designed elastomeric bearings have performed well and are a cost-effective solution.*

The inaccessibility of bearings creates a major problem for their inspection and maintenance. In the past little consideration has been paid to bearing accessibility. A suitable gap should always be provided between the top of the bearing seat and the soffit of the diaphragm, and as many sides of the bearing should be accessible as possible.

The use of concrete shear keys with appropriate rebar detailing may be considered for lateral seismic load restraint. Shear keys can be used in addition to the anchor bolt details. Shear keys are considered to be more cost-effective and require less maintenance than guided bearings. Shorter shear keys or the use of curved faces can mitigate the potential for jamming.

The designer shall ensure compatibility between the various structural elements (shear keys and their allowable gaps, joints, and bearings).

Where practicable, a single line of bearings in lieu of a double row of bearings over the piers may result in a reduction in construction costs.

For seismic load applications the use of a base isolation system in accordance with Section 4 can be considered.

11.6.3 Sliding surfaces

11.6.3.2 Sliding Surface

11.6.3.2.1 PTFE Layer

Replace the last sentence of paragraph 3 with the following:

The lubricant shall be silicone grease, effective to -40°C and comply with U.S. Department of Defense MIL-S-8660C or the Society of Automotive Engineers specification SAE AS8660

11.6.3.4 Attachment

11.6.3.4.1 PTFE layer

Commentary: Sheet polytetrafluorethylene (PTFE) should preferably be confined.

Delete the third sentence and replace with the following:

Sheet PTFE which is not confined must be bonded to a rigid metal surface by a method acceptable to the Ministry.

11.6.4 Spherical bearings

11.6.4.1 General

Add the following:

Spherical bearings shall preferably be installed concave part down to prevent accumulation of water and dirt. When installed concave down, additional forces can be developed at the bearing level, which need to be accommodated by the bearing and adequately transferred to the substructure.

11.6.6.2 Geometric requirements

Delete the second row and replace with:

$L \geq 4h_e$ and $R \geq 2h_e$, with $15 \text{ mm} < h_e < 25 \text{ mm}$

Add the following:

Bearing pressure requirements for continuous strips may be waived where the bearing is used only as a temporary bearing pad.

Commentary: Problems with plain bearings that are too thin or too thick have been observed. Therefore, the allowable thickness has been amended here.

11.6.6.5 Fabrication

11.6.6.5.2 Laminated bearings

Add after first sentence the following:

Steel reinforced elastomeric bearings shall have at least two steel reinforcing plates, and the minimum cover of elastomer for the top and bottom steel reinforcing plates shall be 5 mm. Allowable tolerance on the top and bottom cover shall be 2 mm.

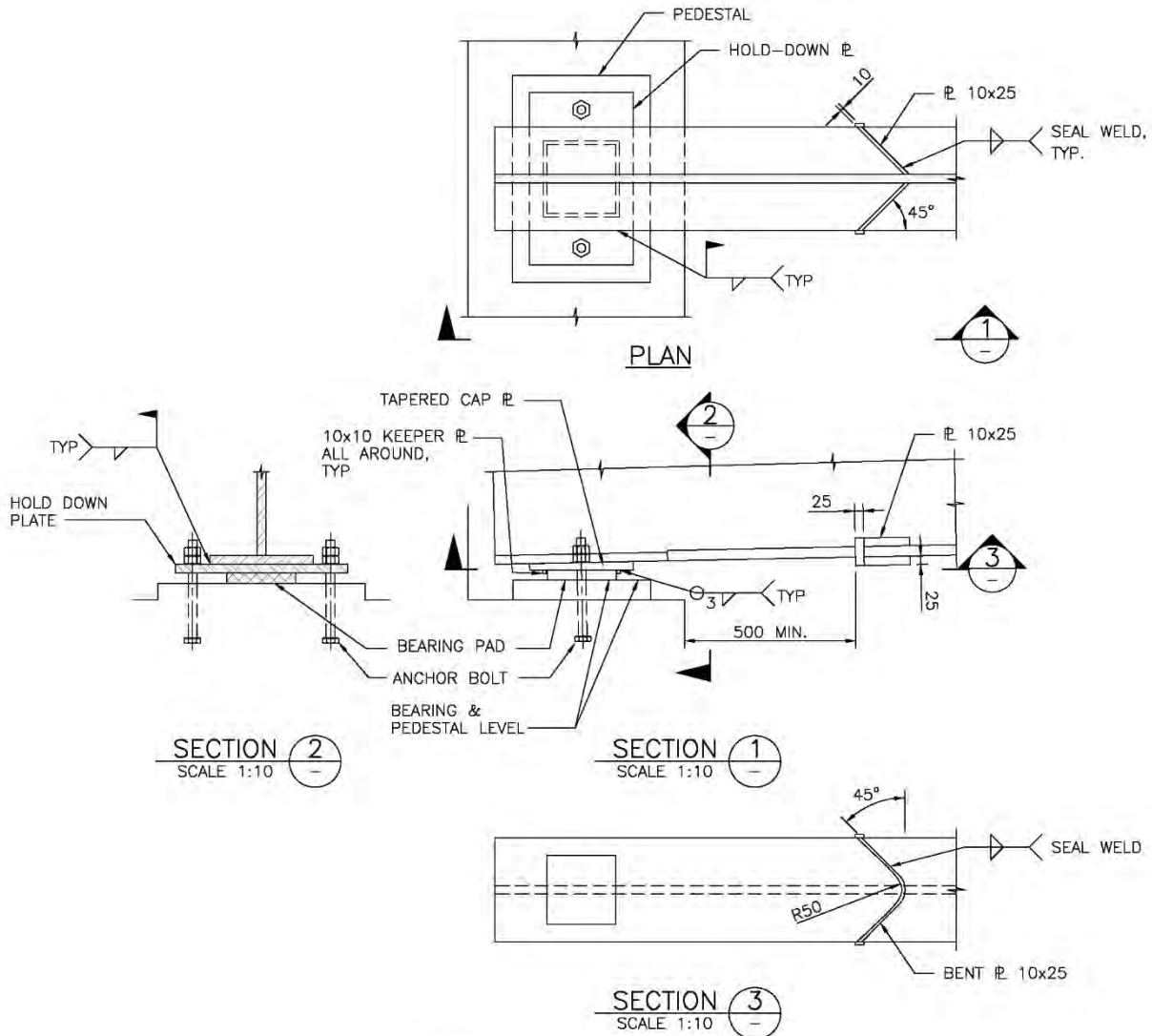
11.6.6.6 Positive attachment

Add the following:

The recommended attachment details for elastomeric bearings under non-seismic loadings shall be as shown in Figures 11.6.6.6 (a) and 11.6.6.6 (b) below.

The holes for anchor bolts in hold-down plates shall be slotted at expansion ends.

Figure 11.6.6 (a)
Bearing hold down details for steel girders



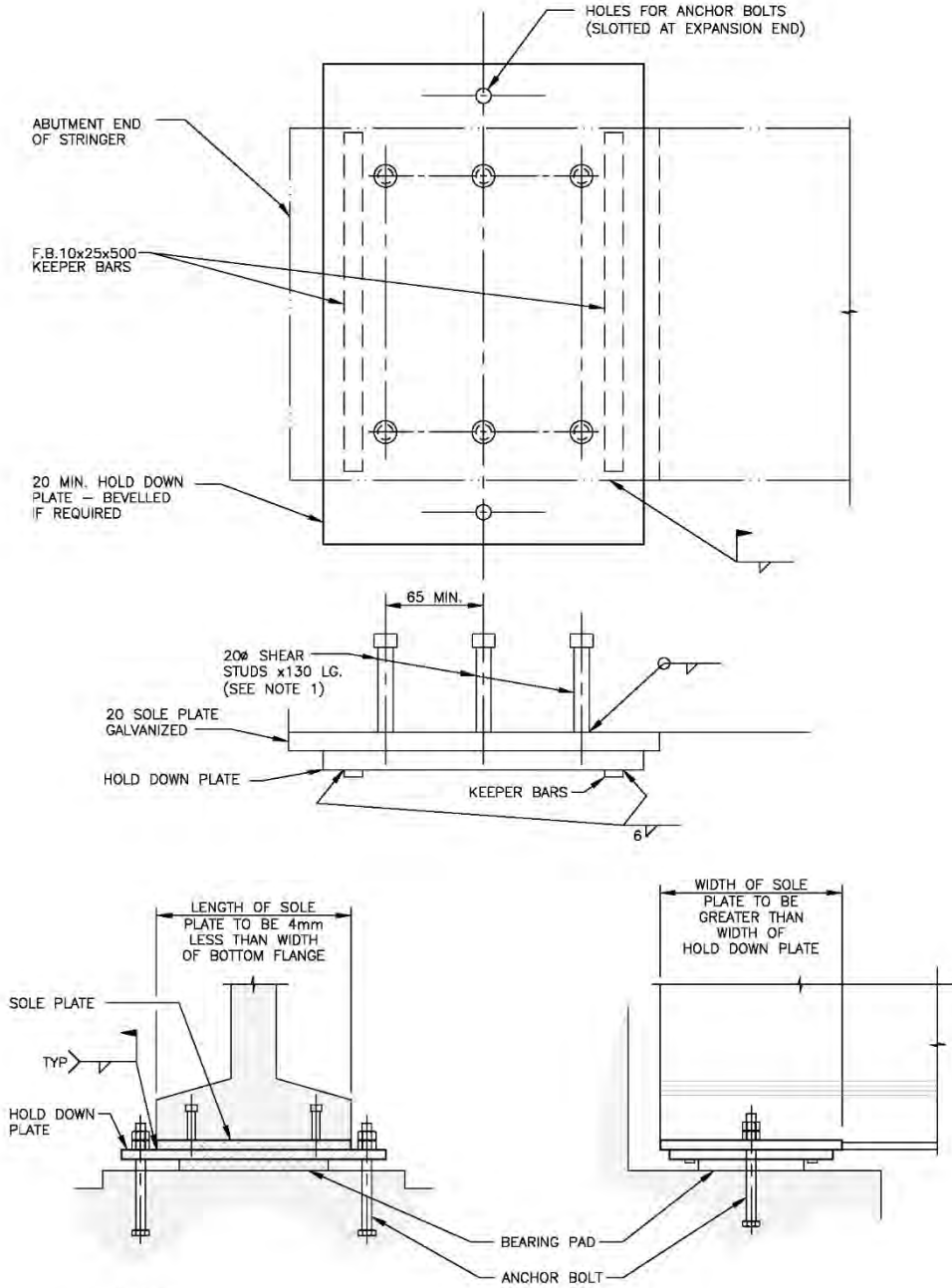
NOTES:

1. HOLES FOR ANCHOR BOLTS IN HOLD-DOWN PLATES SHALL BE SLOTTED AT EXPANSION ENDS.
2. FIELD WELD SHALL BE COATED WITH EITHER AN APPROVED GALVANIZING AGENT OR PAINT PRODUCT AFTER ERECTION.
3. WATER DEFLECTOR REQUIRED ON EXTERIOR GIRDERS, OPTIONAL FOR INTERIOR GIRDERS.

BEARING DETAILS AND BOTTOM FLANGE WATER DEFLECTOR PL

11.6.6.6.(b)

Bearing hold down details for concrete girders



NOTES:

1. LENGTH OF STUDS TO BE ADJUSTED SUCH THAT THE STUD HEAD LIES BETWEEN LAYERS OF STRANDS.
2. GRIND OFF GALVANIZING ON EDGES OF SOLE PLATE AND TOP OF HOLD DOWN PLATE TO ACCOMMODATE WELDING. PAINT WELDS AND EXPOSED STEEL WITH AN APPROVED GALVANIZING AGENT AFTER ERECTION.

11.6.9 Load plates and attachment for bearings

11.6.9.2 Tapered plates

Add the following:

Unless otherwise Consented to by the Ministry, bearings shall be installed level using tapered sole plates to account for differential slopes between the girders and the bearing seat.

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12.4 Barriers

12.4.1 General

Commentary: The CHBDC provides detailed coverage of the “warrants” for a bridge traffic or combination barrier. Using site-specific factors (i.e.: traffic volume, bridge geometry, etc.), an appropriate barrier performance, or “test level” can be analytically determined using the CHBDC methodology. In general, the Ministry’s Supplement to the CHBDC requires no changes to this approach.

However, the CHBDC provides only limited guidance on the design of a bridge traffic or combination barrier. This guidance includes a minimum barrier height requirement and specifies that barrier adequacy shall be determined from crash tests. Specific barrier design requirements are left to individual jurisdictions to establish. Hence, the content of this chapter of the Ministry’s Supplement focuses on bridge barrier design and detailing.

The CHBDC identifies additional factors to be considered in the appraisal of a barrier. These factors are further considered and supplemented as follows:

Deck Drainage: To facilitate deck drainage, some recent projects have incorporated a large drain opening (scuppers) in the barrier face to channel water off of the deck and into an externally mounted discharge pipe. Note that a large drainage opening is already approved for use in roadside applications, as per the Ministry’s Precast Concrete Drainage Barrier (SP941-01.02.05). Such large openings can present a hazard due to snagging of a vehicle’s wheel during impact. The use of a large drainage opening in a bridge traffic or combination barrier shall be avoided where possible. Use of scuppers requires the Consent of the Ministry. (See Supplement Clause 1.8.2.3.3.)

Electrical Conduits and Junction Boxes: Concrete bridge and combination barriers can serve as a convenient location for running electrical conduit over the bridge length. The size and number of conduits should be limited such that their presence does not have an adverse effect on the crash performance of the barrier. Criteria are provided in Supplement Clause 1.7.3.2. The conduit(s) should be located at the base of the barrier, within the rebar cage. The junction boxes to service the conduit should be located in the rear (non-impact) face of the barrier.

Further Barrier Reference: For expanded detail on all bridge barrier topics, a recent and relevant Canadian reference document is the Guide to Bridge Traffic and Combination Barriers (2010, Transportation Association of Canada).

12.4.2 Barrier joints

Add the following:

Barrier joint openings exceeding 100 mm shall be protected by sliding steel plates. All steelwork shall be hot-dipped galvanized in accordance with ASTM A 123M.

Commentary: Barrier joints and the ends of a barrier present a load path discontinuity resulting in a shortened zone for impact load dispersal to the bridge deck. Supplemental barrier anchorage and deck reinforcing is sometimes required in these end zones. Openings in barriers pose a catching and snagging hazard to vehicles.

12.4.3 Traffic barriers

12.4.3.2 Test level

12.4.3.2.1 General

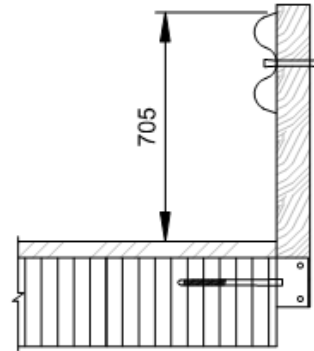
Add the following:

The following bridge traffic barrier “reference concepts” have been accepted by the Ministry for use on highway bridges in B.C. Other bridge traffic barrier concepts may be considered but require prior Consent.

All bridge barrier design shall meet the CHBDC requirements for crash testing. Each of the listed “reference concepts” is known to have met the CHBDC requirements for crash testing. Jurisdictional usage for each listed “reference concept” is included. The Design Engineer shall confirm the applicability of the of the “reference concept” with respect to crash testing, usage and detailing. The test level identified in the table below are based upon testing to NCHRP 350.

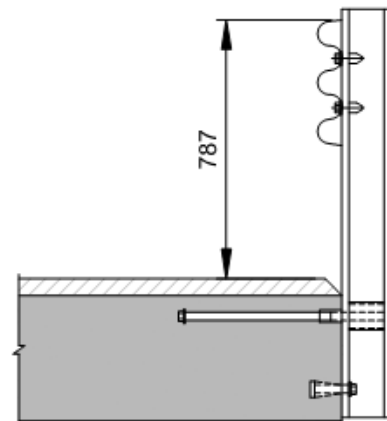
TL-1 W-Beam

Commentary: Details based on USDA Forest Service W-Beam Bridge Rail.



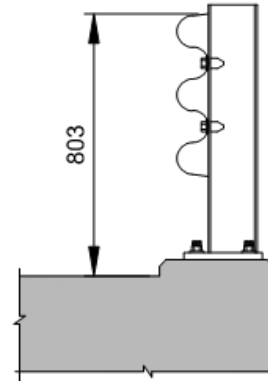
TL-2 Thrie Beam (Side Mounted)

Commentary: Side mounted details based on Oregon Standard Drawing BR233.



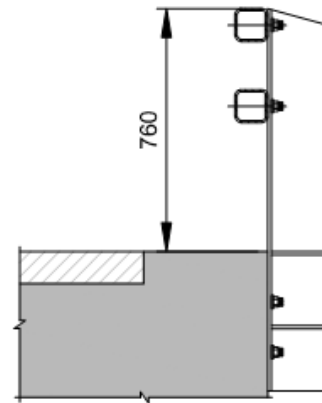
TL-2 Thrie Beam
(Top Mounted)

Commentary: Top mounted details based on Alberta Standard Drawing S-1652-00.



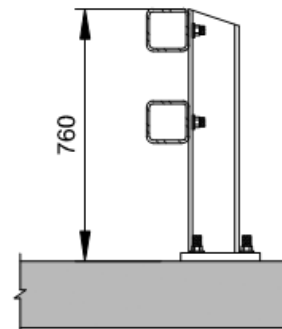
TL-2 Two Rails
(Side Mounted)

Commentary: Side mounted details based on California Type 115 Bridge Rail.



TL-2 Two Rails
(Top Mounted)

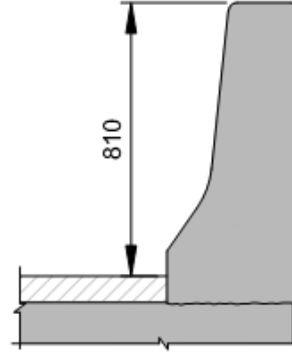
Commentary: Top mounted system based on California Type 115 Bridge Rail, modified for top mounted anchorage. Modified anchorage to be designed in accordance with Clause 12.4.3.5 in the CHBDC.



**TL-4 "F" Shape
(Cast-in-Place Concrete)**

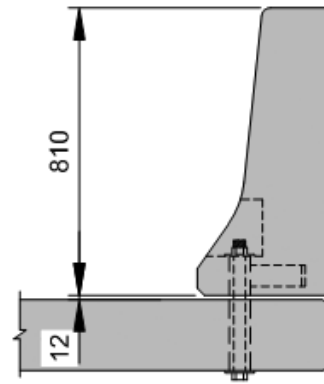
See the Ministry's Standard Bridge Parapet (2874-1) for detailing.

Commentary: Similar systems are used widely in jurisdictions across North America.



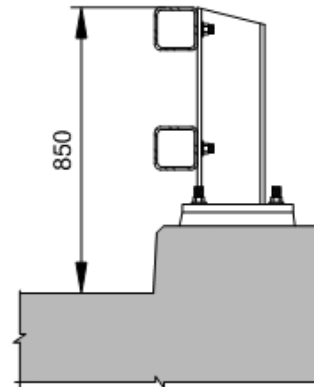
**TL-4 "F" Shape
(Precast Concrete, Bolt-Down)**

Commentary: The Ministry has developed drawings for this "F" Shape barrier system and completed load testing of the anchorage details in the barrier. The anchorage testing confirmed that the CHBDC requirements for anchorage are met. As such, this barrier system is considered acceptable for use when Consented to by the Ministry. When Consented to by the Ministry, these conceptual drawings will be provided by the Ministry, and the designer will need to determine the appropriate connection details to the superstructure.



TL-4 Double Tube on Curb

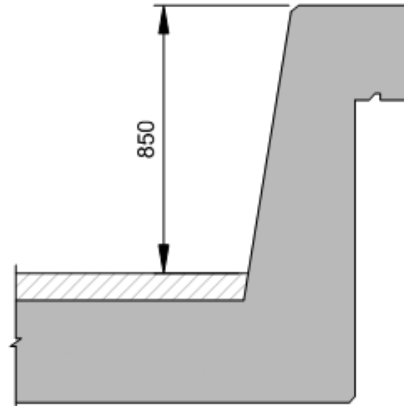
Commentary: Details based on Alberta Standard Drawing S-1642-00. Alternate details in Oregon Standard Drawing BR206.



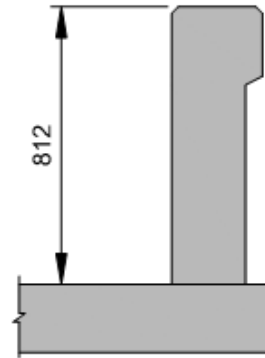
TL-4 Constant Slope

Commentary: Details based on Alberta Standard Drawing S-1650-00. Alternate details in California Type 732 Bridge Rail and Texas SSTR Bridge Rail.

A taller 1070 version of this barrier is approved for TL-5 based on the T80SS Bridge Rail used in Texas.

**TL-4 Vertical Face***

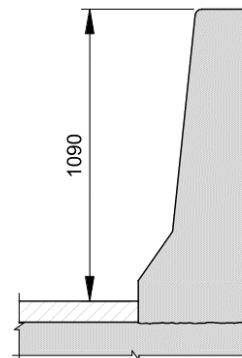
Commentary: Details based on the Texas T221 Bridge Rail.

**TL-5 "F" Shape**

This is an extended version of the Ministry's Standard Bridge Parapet (2874-1).

Barrier, anchorage and deck reinforcing shall be increased in proportion to the loads listed in Table 3.8 of the CHBDC.

Commentary: This barrier also meets the height requirements for a pedestrian barrier.



Commentary: Resources for further information for both the noted reference concepts and other barrier concepts include:

- *Guide to Bridge Traffic and Combination Barriers (2010)*, Transportation Association of Canada
- *TF13 Guide to Bridge Railing Systems*, (on-going project), Task Force 13 (<https://tf13.org/Guides/bridgeRailGuide/>)
- *Roadside Safety Pooled Fund* (on-going project) <https://www.roadsidepooledfund.org/>
- *Standard drawings published on provincial and state web sites*

12.4.3.2.5 Test level for barriers on low volume roads

Delete and replace with the following:

- TL-1 barrier systems may be used on bridges that meet all of the following:
- are classified as a Ministry Low Volume Road Structure,
- have a maximum roadway width of 8.6 m,
- have a maximum deck height above ground or water surface of 5.0 m,
- have a maximum design speed of 80 km/h combined with a maximum ADT of 100 or a maximum design speed of 50 km/h combined with a maximum ADT of 400.

For other bridges that meet the Ministry definition of a Low Volume Road Structure, Test Level 2, 4, or 5, determined in accordance with Clauses 12.4.3.2.3 and 12.4.3.2.4, shall be used unless alternative test levels are Consented to by the Ministry.

Barrier anchorage loads for Test Level 1 shall be determined in accordance with Clause 12.4.3.5. Barrier anchorage loads specified for Test Level 1 in Table 3.7 may be reduced by 20%.

Commentary: See CL. 1.3.3 of this Supplement for the Ministry's definition of a Low Volume Road Structure.

12.4.3.3 Geometry and end treatment details

Delete the third paragraph and replace with:

Protected sidewalks, bicycle and multi-use pathways shall be separated from traffic by a barrier or guide rail. For design speeds ≤ 60 km/h, a raised curb may be used with the curb having a face height of 200 mm and a face slope steeper than one horizontal to three vertical.

Add the following:

Traffic barriers shall be constructed such they are oriented perpendicular to the deck surface. The minimum height of barrier shall be determined for initial construction without consideration of future overlay.

Commentary: Traffic barriers are constructed perpendicular to the deck surface in order that the roadway face of the barrier remains correctly oriented to withstand vehicle impacts which may be inclined due to deck crossfall. This also avoids discontinuities in the barrier faces at bridge ends where parapets meet transition barriers.

Consideration should be given to potential for overdriving of the posted speed limit such as where the bridge near a change in speed limit, near sharp curves or at the base of a downhill grade.

12.4.3.4 Crash test requirements

12.4.3.4.6 Zone of Intrusion

12.4.3.4.6.1 General

Commentary: While still a relatively new topic, further information and delineation of the Zone of Intrusion limits for different test levels can be found here: [Zone of Intrusion and Concrete Barrier Countermeasures \(2010, Stephen Hobbs, Annual Conference of the Transportation Association of Canada\)](#)

12.4.3.5 Anchorages

Commentary: Note that a live load factor of 1.7 shall be applied to the barrier loads specified in Clause 3.8.8.

12.4.4 Pedestrian barriers

12.4.4.1 General

Add the following:

The Ministry's Standard steel sidewalk fence concept shall be used (Standard Drawing 2891-1). The standard steel sidewalk fence shall extend a minimum of three (3) metres beyond the back of ballast wall at bridge abutments or extend a minimum of three (3) metres beyond the ends of return walls, as appropriate.

Debris and/or safety fence shall be installed when directed by the Ministry.

Commentary: The debris or safety fencing should be considered in urban areas for bridges over roadways, or bridges that cross railways to reduce the risk of objects falling from the bridge.

12.4.4.2 Geometry

Pedestrian barriers shall be constructed such that they are oriented plumb.

12.4.5 Bicycle barriers

12.4.5.1 General

Add the following:

The Ministry Standard steel bicycle fence concept shall be used (refer to Standard Drawing 2891-2). The standard steel bicycle fence shall extend a minimum of three (3) metres beyond the back of ballast wall at bridge abutments or extend a minimum of three (3) metres beyond the ends of return walls, as appropriate.

12.4.5.2 Geometry

Add the following:

Bicycle barriers shall be constructed such that they are oriented plumb.

***Commentary:** Alternatives to the Ministry's Standard steel sidewalk or bicycle fence may be considered when debris being thrown from the bridge or people climbing the fence are identified as site-specific issues. Jurisdictions with facilities under Ministry structures, such as railways, may have requirements for protective screening that include height of screen, size of openings and length.*

12.4.6 Combination barriers

12.4.6.1.a Configuration of combination barriers

The configuration of bridge traffic and combination barriers depends on the roadway type and the makeup of its users. In general, the bridge barrier design shall match one of the three following configurations, each described in the appended illustrations.

Configuration #1 - Bridge with No Sidewalk

Configuration #2 - Bridge with Raised Sidewalk

Configuration #3 - Bridge with Sidewalk Separated by a Barrier

***Commentary:** For sides of bridges where there is no sidewalk, Combination Barriers are installed at the outside of the bridge for the safety and protection of pedestrian and/or bicycle traffic on the bridge deck.*

For bridges with sidewalk(s), it is left to the Design Engineer to determine the most suitable type of separation based on anticipated traffic volumes and details of the crossing. In general, concrete parapet type barriers are used to separate the roadway from the sidewalk(s). The sidewalk face of the barrier shall have a smooth surface without snag points.

The installation of Combination Barriers is an additional cost item for bridges having no provision for sidewalks. In remote areas, where pedestrian and bicycle traffic is minimal, Traffic Barriers may possibly be used in lieu of Combination Barriers.

12.4.6.1.b Pedestrian combination barriers

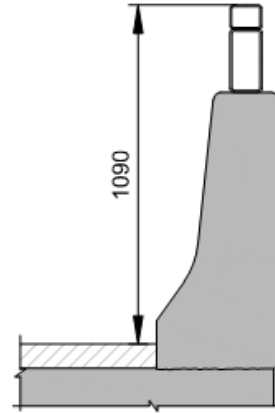
The following pedestrian combination barrier "reference concepts" have been accepted by the Ministry for use on highway bridges in B.C. Other pedestrian combination barrier concepts may be considered but require prior Consent of the Ministry.

All pedestrian combination barrier designs shall meet the CHBDC requirements for crash testing. Each of the listed "reference concepts" is known to have met the CHBDC requirements for crash testing. Jurisdictional usage for each listed "reference concept" is included. The Design Engineer shall confirm the applicability of the of the "reference concept" with respect to crash testing, usage and detailing.

TL-4 “F” Shape with Pedestrian Railing

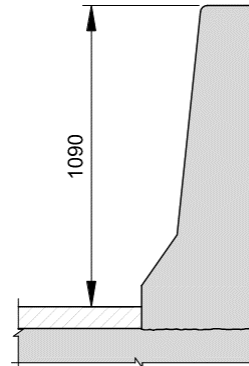
See the Ministry’s Standard Bridge Parapet (2874-1) for barrier detail.

See the Ministry’s Standard Bridge Parapet Steel Railing (2785-2) for railing detail.

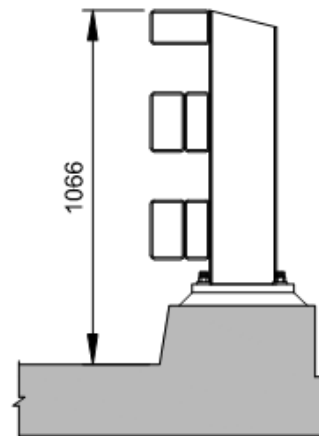
**TL-4 “Tall F” or TL-5 “F” Shape**

This is an extended version of the Ministry’s Standard Bridge Parapet (2874-1).

Commentary: The Ministry’s TL-5 “F” Shape inherently provides pedestrian-height protection. The barrier can be detailed for TL-4 loading, as required.

**TL-4 3-Tube on Curb**

Commentary: Details based on Oregon Standard Drawing BR208.



12.4.6.1.c Bicycle combination barriers

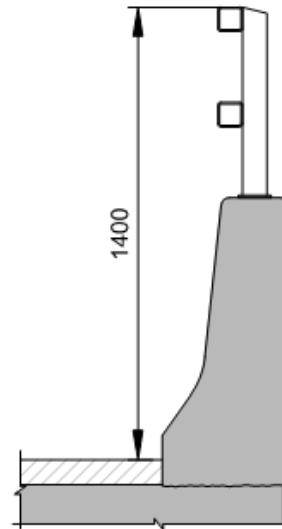
The following bicycle combination barrier “reference concepts” have been accepted by the Ministry for use on highway bridges in B.C. Other bicycle combination barrier concepts may be considered but require prior Consent of the Ministry.

All bicycle combination barrier designs shall meet the CHBDC requirements for crash testing. Each of the listed “reference concepts” is known to have met the CHBDC requirements for crash testing. Jurisdictional usage for each listed “reference concept” is included. The Design Engineer shall confirm the applicability of the of the “reference concept” with respect to crash testing, usage and detailing.

TL-4 “F” Shape with Bicycle Railing

See the Ministry’s Standard Bridge Parapet (2874-1) for barrier detail.

See the Ministry’s Standard Bridge Parapet Steel Bicycle Railing (2785-3) for railing detail.

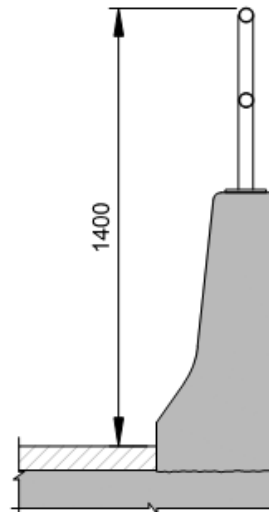


TL-4 “F” Shape with Flush-Post Bicycle Railing

See the Ministry’s Standard Bridge Parapet (2874-1) for barrier detail.

See the Ministry’s Standard Bridge Parapet Steel Bicycle Railing (2785-4) for railing detail.

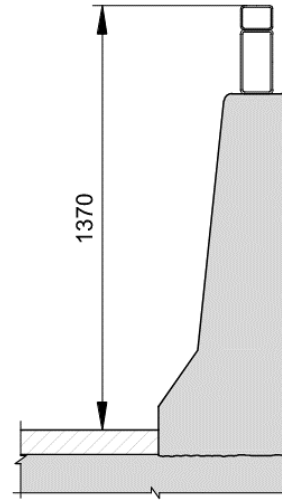
Commentary: A flush post railing is preferred when a railing is required on top of a barrier that separates traffic from a mixed-used sidewalk.



TL-4 “Tall F” or TL-5 “F” Shape with Pedestrian Railing

This is an extended version of the Ministry’s Standard Bridge Parapet (2874-1).

See the Ministry’s Standard Bridge Parapet Steel Railing (2785-2) for railing detail.



12.4.6.2 Geometry

Add the following:

Where combination barriers are installed on sidewalks separated from traffic by raised curbs, the barriers shall be constructed such they are oriented plumb. Otherwise, where combination barriers are installed on the bridge deck, barriers shall be constructed such that they are oriented perpendicular to the deck surface.

Commentary: *Combination barriers installed on bridge decks are constructed perpendicular to the deck surface in order that the roadway face of the barrier remains correctly oriented to withstand vehicle impacts.*

12.5 Highway accessory supports

12.5.5 Design

12.5.5.1 General

Delete the second paragraph.

12.5.5.4 Fatigue limit state

12.5.5.4.1 General

Add the following:

Galloping and truck-induced gust vertical deflections of overhead and cantilever single-arm sign supports and traffic signal arms shall not exceed 200 mm.

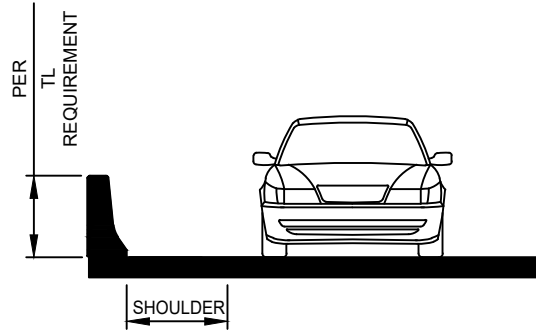
12.5.11 Damping devices

Delete the first paragraph and replace with the following:

The use of damping or energy-absorbing devices shall be considered for highway accessory supports that are subject to significant vibrations or have a horizontal arm length equal to or greater than 12m. Highway accessory supports equipped with damping devices shall not be exempt from meeting the requirements of Clause 12.5.5.4.

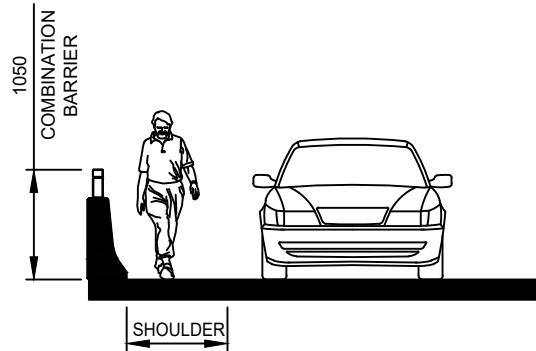
CONFIGURATION #1 - BRIDGE WITH NO SIDEWALK

USAGE: THIS SYSTEM IS SPECIFIED FOR LVR STRUCTURES OR WHERE THE GAP BETWEEN STRUCTURES IS LESS THAN 600 mm. CONSIDERATION SHOULD BE GIVEN TO WORKSAFE BC HEIGHT REQUIREMENTS FOR FALL PROTECTION WHERE APPLICABLE.



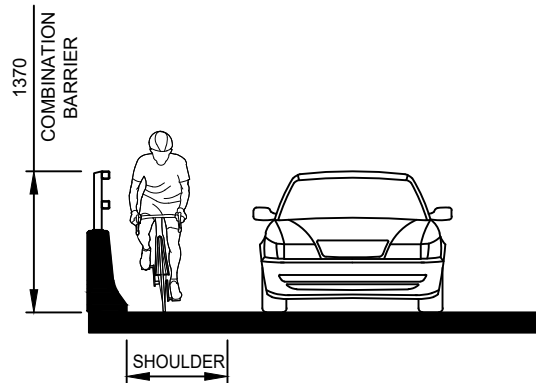
OPTION A
VEHICULAR TRAFFIC ONLY

USAGE: THIS SYSTEM IS THE DEFAULT FOR NON-LVR BRIDGES.



OPTION B
VEHICULAR & PEDESTRIAN TRAFFIC

USAGE: THIS SYSTEM IS SPECIFIED WHERE WARRANTED BY BICYCLE VOLUMES AND/OR WHEN A BICYCLE ROUTE IS IDENTIFIED ACROSS THE BRIDGE.

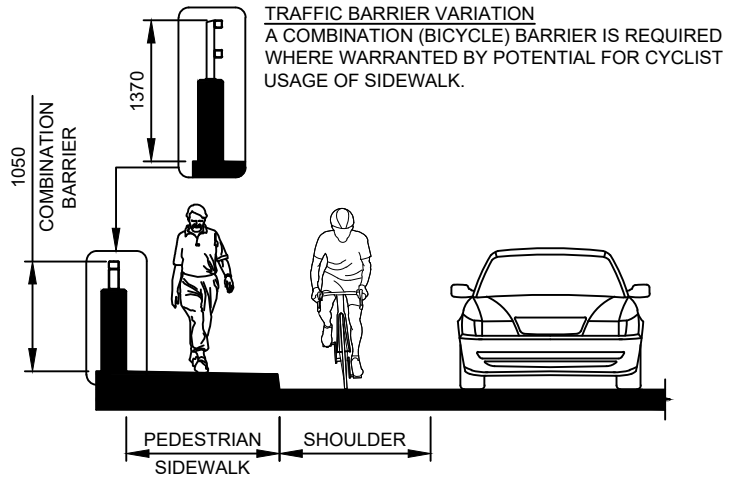


OPTION C
VEHICULAR & CYCLIST TRAFFIC

NOTES:

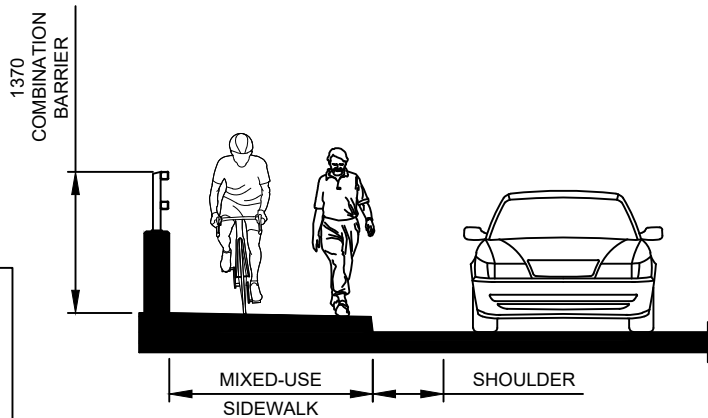
1. DIMENSIONS SHOWN ARE CODE MINIMUMS, SYSTEM SPECIFIED HEIGHTS SHALL BE USED.
2. TRAFFIC BARRIER PROTECTION BASED ON THE MINISTRY'S STANDARD BRIDGE PARAPET. OTHER BARRIERS MAY BE ACCEPTABLE.
3. THE USE OF A TRAFFIC BARRIER (OPTION A) IN LIEU OF A COMBINATION BARRIER MAY BE ACCEPTABLE IN REMOTE AREAS, AS RECOMMENDED BY THE DESIGN ENGINEER AND AS CONSENTED TO BY THE MINISTRY, ON THE BASIS OF THE ANTICIPATED VOLUME OF PEDESTRIAN AND/OR BICYCLE TRAFFIC AND GEOMETRIC DETAILS OF THE CROSSING.
4. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS NOTED OTHERWISE.

CONFIGURATION #2 - BRIDGE WITH RAISED SIDEWALK



USAGE: THIS SYSTEM IS SPECIFIED WHERE:
(A) TRAFFIC SPEEDS ARE LESS THAN OR EQUAL TO 60 km/hr
(B) THE SIDEWALK IS INTENDED FOR PEDESTRIAN USE WITH CYCLISTS USING THE ROADWAY.

OPTION A VEHICULAR & CYCLIST TRAFFIC ON ROADWAY & PEDESTRIAN TRAFFIC ON SIDEWALK



USAGE: THIS SYSTEM IS SPECIFIED WHERE:
(A) TRAFFIC SPEEDS ARE LESS THAN OR EQUAL TO 60 km/hr
(B) WHERE BOTH PEDESTRIANS AND CYCLISTS ARE EXPECTED TO USE THE MIXED-USE SIDEWALK.

OPTION B VEHICULAR TRAFFIC ON ROADWAY & PEDESTRIAN & CYCLIST TRAFFIC ON SIDEWALK

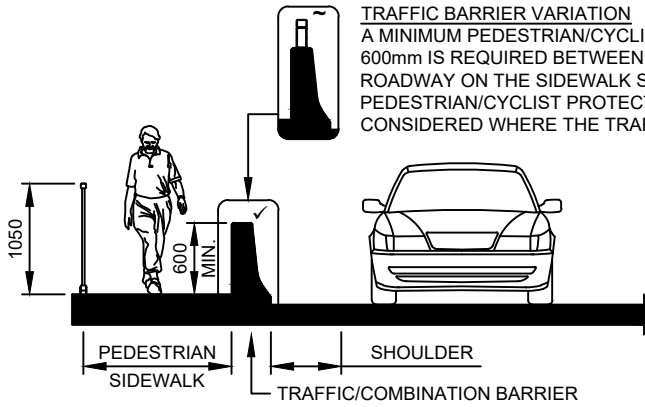
NOTES

1. DIMENSIONS SHOWN ARE CODE MINIMUMS, SYSTEM SPECIFIED HEIGHTS SHALL BE USED.
2. MINISTRY STANDARD BRIDGE PARAPET SHALL NOT BE USED FOR THESE CONFIGURATIONS. A BARRIER SYSTEM WHICH HAS BEEN CRASH TESTED IN A RAISED CURB CONFIGURATION SHALL BE USED.
3. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS NOTED OTHERWISE.
4. CURBS SHOULD BE NO HIGHER THAN 200mm.

CONFIGURATION #3 - BRIDGE WITH SIDEWALK SEPARATED BY A BARRIER

TRAFFIC BARRIER VARIATION

A MINIMUM PEDESTRIAN/CYCLIST HEIGHT PROTECTION OF 600mm IS REQUIRED BETWEEN THE SIDEWALK AND ROADWAY ON THE SIDEWALK SIDE. ADDITIONAL PEDESTRIAN/CYCLIST PROTECTION HEIGHT MAY BE CONSIDERED WHERE THE TRAFFIC SHOULDER IS NARROW.



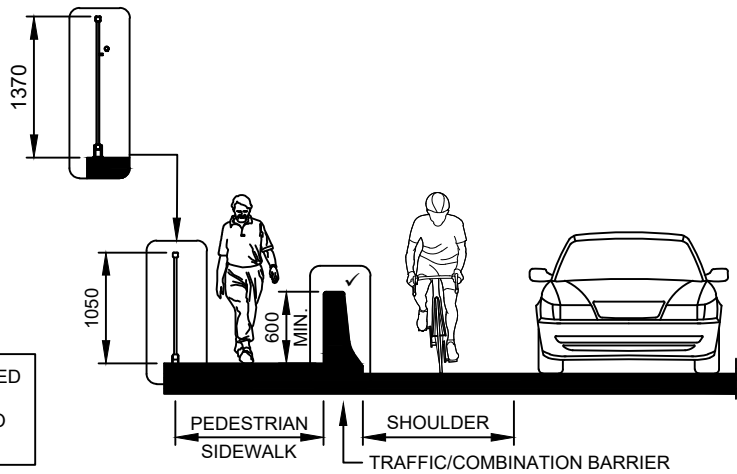
USAGE: THIS SYSTEM IS TYPICALLY SPECIFIED ON HIGH SPEED ROUTES (>60km/hr) WHERE NO CYCLIST TRAFFIC EXPECTED.

OPTION A

VEHICULAR TRAFFIC ON ROADWAY & PEDESTRIAN TRAFFIC ON SIDEWALK

BICYCLE BARRIER VARIATION

WHERE THE POTENTIAL FOR CYCLISTS ON THE SIDEWALK IS HIGH, A BICYCLE BARRIER IS RECOMMENDED ALONG THE EDGE OF THE BRIDGE.



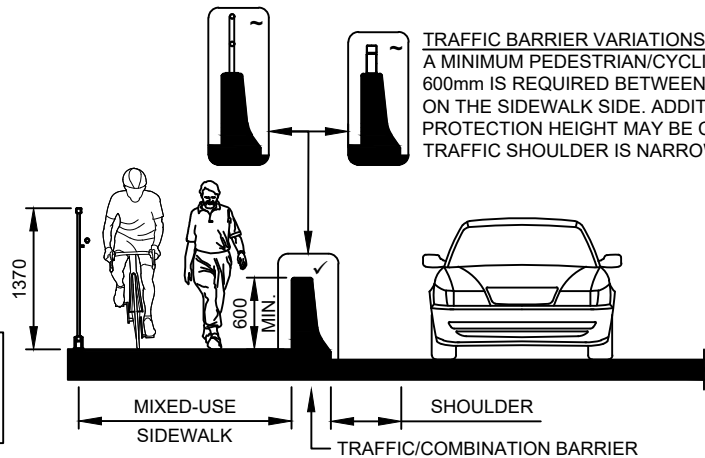
USAGE: THIS SYSTEM IS TYPICALLY SPECIFIED ON HIGH SPEED ROUTES (>60km/hr) WHERE CYCLISTS ARE INTENDED TO USE THE ROADWAY.

OPTION B

VEHICULAR & CYCLIST TRAFFIC ON ROADWAY & PEDESTRIAN TRAFFIC ON SIDEWALK

TRAFFIC BARRIER VARIATIONS

A MINIMUM PEDESTRIAN/CYCLIST HEIGHT PROTECTION OF 600mm IS REQUIRED BETWEEN THE SIDEWALK AND ROADWAY ON THE SIDEWALK SIDE. ADDITIONAL PEDESTRIAN/CYCLIST PROTECTION HEIGHT MAY BE CONSIDERED WHERE THE TRAFFIC SHOULDER IS NARROW.



USAGE: THIS SYSTEM IS TYPICALLY SPECIFIED ON HIGH SPEED ROUTES (>60km/hr) WHERE SIDEWALK IS DEDICATED TO MIXED USE.

OPTION C

VEHICULAR TRAFFIC ON ROADWAY & PEDESTRIAN & CYCLIST TRAFFIC ON SIDEWALK

NOTES:

- DIMENSIONS SHOWN ARE CODE MINIMUMS, SYSTEM SPECIFIED HEIGHTS SHALL BE USED.
- TRAFFIC BARRIER BASED ON THE MINISTRY'S STANDARD BRIDGE PARAPET. OTHER BARRIERS MAY BE ACCEPTABLE. PEDESTRIAN AND BICYCLE BRIDGE BASED ON THE MINISTRY'S STANDARD STEEL SIDEWALK AND BRIDGE FENCE, RESPECTIVELY.
- ALL DIMENSIONS ARE IN MILLIMETRES UNLESS NOTED OTHERWISE.

LEGEND

✓ -ACCEPT-BLE CONFIGUR-TION

~ M-Y BE -N -CCEPT-BLE CONFIGUR-TION

13.1 Scope 2

13.7 Mechanical system design 2

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 13.7.18.4.3 Lubricated plain bearings..... 2

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13.9 Electrical system design 2

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 13.9.11.2 Control logic 3

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 13.10.1 Operation and Maintenance Handbook 3

 13.10.2 Spare Parts 3

13.1 Scope

Movable bridges shall not be used unless Consented to by the Ministry.

13.7 Mechanical system design

13.7.18 Bearings

13.7.18.4 Plain bearings

13.7.18.4.3 Lubricated plain bearings

Add the following:

Use of self-lubricated non-bronze bushings shall be Consented to by the Ministry.

Commentary: Self-lubricated non bronze bushings may be appropriate for some applications.

13.7.18.4.4.2 Non-metallic bearings

Commentary: Self-lubricated bearing materials may be appropriate for some applications. For proprietary bearing materials the coefficients of friction shall be as advised by the suppliers.

13.9 Electrical system design

13.9.2 General requirements for electrical installations

13.9.2.1 Corrosion Protection

Add the following:

External wiring to control panels and consoles shall be of type listed in the Canadian Electrical Code CEC Standard (CSA C22.1), Table 19, for exposed wiring in wet locations. Wireways and trays shall not be used outside the operator's house except with armoured cables. Tray and fittings shall be stainless steel complete with cover (to reduce the problems of birds and their residue). The designer shall detail all wireways such that they do not impose a tripping hazard for the operator.

Commentary: Wires in rigid galvanized steel conduit is the preferred method for bridges. Armoured cable with PVC jacket may be an acceptable alternative.

13.9.11 Electrical control systems

13.9.11.1 Operating sequence and interlocking requirements

Commentary: CCTV systems are suggested to assist the operator in monitoring mechanisms not visible from the operator's cabin.

13.9.11.2 Control logic

13.9.11.2.3 Programmable logic controller (PLC)

Add the following paragraph:

The PLC shall be provided with a communication card installed to allow remote communication monitoring by the Ministry at its Provincial Control Centre.

13.10 Additional Requirements

13.10.1 Operation and Maintenance Handbook

The designer should provide the Operation and Maintenance Handbook, not the Contractor. In addition to the relevant drawings to describe the work, the handbook shall also include:

- A regular schedule of inspection, and lubrication;
- A schedule of operating or testing the bridge. The test operations should occur at regular intervals and should include emergency operating conditions;
- Calibration and set points of all devices; and
- A copy of the testing and commissioning records.

13.10.2 Spare Parts

The design shall include a list of spare parts in the Plans that the Contractor must provide...

Commentary: The list should be reviewed to include PLC and UPS spare parts.

14.1 Definitions 2

14.7 Material strengths..... 2

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 14.7.4.2 Structural steel..... 2

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 14.9.1 Normal traffic..... 2

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14.15 Live load capacity factor 4

 14.15.4 Combined load effects 4

14.17 Bridge posting 5

 14.17.1 General..... 5

14.18 Fatigue..... 5

14.1 Definitions

Add the following:

Supervision – monitoring of the passage of a permit vehicle by a BC registered professional engineer familiar with bridge design or their designate to ensure bridge crossing restrictions of a permit are followed, including weighing of a permit vehicle if required by the overload permit. The engineer shall have the authority to stop further movement of the permit vehicle if in non-compliance with the bridge crossing requirements of a permit. The engineer shall submit a report detailing their observations to the Ministry upon completion of the move, including records of vehicle and axle weight, dimension measurements and any other requirements specified in the bridge crossing conditions of the permit.

14.7 Material strengths

14.7.4 Strengths based on date of construction

14.7.4.2 Structural steel

Commentary: Further information on historical steel grades may be found on the CISC website, specifically at the following URL:

<http://dir.cisc-icca.ca/files/technical/techdocs/historical/obsoletegrades.pdf>

14.9 Transitory loads

14.9.1 Normal traffic

14.9.1.1 General

Delete and replace with:

Unless otherwise consented to by the Ministry, evaluation shall be to the Evaluation Level 1 loading (vehicle trains) described in Clause 14.9.1.2 with W as 625 kN. CL-800 design loading shall not normally be used for evaluation.

Commentary: Loadings that differ from the CL1-W loadings specified in Clause 14.9 may be specified by the Ministry on a project-to-project basis.

14.12 Target reliability index

14.12.2 System behaviour

Add to Item (a), Category S1 the following:

Simply supported girder in a three-girder system.

14.12.3 Element behaviour

Add to Item (a), Category E1 the following:

This can also include timber in bending, compression parallel to grain (slender members) and tension, when element is subject to sudden loss of capacity with little or no warning and no post failure capacity,

Add to Item (b), Category E2 the following:

Timber in bearing, when element is subject sudden loss of capacity with little or no warning and with post failure capacity, i.e. crushing of timber

Add to Item (c), Category E3 the following:

Timber in shear, when element is subject to gradual failure with warning of probable failure, end splits are signs of gradual failure

Commentary: *This section does not give any guidance for timber element behavior.*

14.13.3 Transitory Loads

Add the following:

When permit vehicle loaded lanes are mixed with normal traffic loaded lanes, each lane will be assigned its corresponding different live load factor based on the traffic in the lane. For example, a PS loaded lane will get a PS live load factor (Table 14.8) and the other lanes will get normal traffic live load factors (Table 14.7).

Alternatively, if using the simplified method of analysis, then the permit vehicle shall be used in all lanes with the permit vehicle live load factor in all lanes. The engineer shall ascertain that this is a conservative approach.

14.14 Resistance

14.14.1 General

14.14.1.6 Shear in concrete beams

14.14.1.6.1 General

Delete and replace with the following:

Concrete beams shall have their shear resistance calculated in accordance with Clause 8.9.3 with the exception that the factored sectional shear force and factored bending moment used to calculate longitudinal strain of the member, ϵ_x in Clause 8.9.3.8 is given by:

$$V_f = \alpha_D V_{DL} + F (\alpha_L V_{LL})$$

$$M_f = \alpha_D M_{DL} + F (\alpha_L M_{LL})$$

where, a value for F is first assumed, and the calculations repeated, iterating the value of F , until V_r from Clause 8.9.3.3 converges to the value of V_f given above. The value of F at convergence is the live load capacity factor. All other aspects of Clause 8.9.3.8 remain unchanged, except as modified in Clauses 14.14.1.6.2 and 14.14.1.6.3.

Commentary: The shear design provisions of Clause 8.9.3.8 are based on the Modified Compression Field Theory (MCFT). Simplifications were made to the theory to create a suitable procedure for the design of new concrete beams. According to the MCFT, the shear resistance of a concrete member depends on the longitudinal strain ϵ_x of the member. The longitudinal strain in turn depends on a number of factors such as the amount of longitudinal reinforcement and the applied loads including the applied shear force. Thus according to MCFT, the shear resistance of a concrete member depends on the applied shear force at failure. Iteration (trial and error) is therefore generally needed to determine the shear resistance of a member according to MCFT. A simplification in Clause 8.9.3.8 that avoids iteration is the longitudinal strain ϵ_x being calculated from the design forces rather than the forces at shear failure. This is a reasonable assumption for design as the shear resistance is adjusted through the selection of stirrup quantity and concrete section properties to be approximately equal to (slightly greater than) the design shear force V_f .

The simplifying assumptions described above for design cannot be used for determining the ultimate shear resistance for evaluation. The sectional shear force V_f , the corresponding bending moment M_f , as well as any applied axial force N_f used in Clause 8.9.3.8 to determine longitudinal strain ϵ_x , which in turn is used to determine shear resistance, must be the sectional forces that result from the total bridge loading that causes shear failure. Thus evaluating the shear resistance of existing concrete beams using Clause 8.9.3 requires trial and error.

One method of doing these calculations is to include the Live Load Capacity Factor (F) in the equations for calculating V_f and M_f and iterate the value of F until V_r equals V_f .

14.14.1.7 Wood

14.14.1.7.2 Shear

Replace k_{sv} with k_{zv}

Add the following after the equation:

The shear resistance in glued-laminated members shall be taken from Clause 9.7, with the size effect factor, k_{zv} , taken as follows:

$$k_{zv} = \frac{75}{\sqrt{d}} \frac{1}{1 + \frac{2d}{d}} \leq 1.3$$

Delete the third paragraph and replace with:

For solid sawn members that are not older than five years, or where the end split length has not been measured, a shall be assumed to be $0.33d$ for Select Structural Grade and $0.75d$ for Grade 1.

For glued-laminated beam that are not older than five years, or where the end split length has not been measured, a shall be assumed to be $1.0d$.

14.15 Live load capacity factor

14.15.4 Combined load effects

Add to the first paragraph:

Combined shear and moment in steel plate girders with slender webs relying on tension field action to carry shear (refer to Clause 10.10.5.2) shall be calculated by successive iteration or another suitable method.

Add the following paragraph:

Interaction formulas for combined load effects shall be based on factored material strengths which include the resistance adjustment factor U of Clause 14.14.3.

14.17 Bridge posting

14.17.1 General

Replace the first paragraph with the following:

Posting requirements for a bridge evaluated as being deficient shall be determined by the responsible Ministry bridge engineer.

***Commentary:** Ministry posting requirements and standards vary from those specified in Clause 14.17.*

14.18 Fatigue

Add the following:

For fatigue in riveted connections, the stress Category "D" shall be used in determining the allowable range of stress in tension or reversal for base metal at the net section of riveted connections.

***Commentary:** This category will be useful during the evaluation and rehabilitation of existing riveted bridge structures.*

15.1 Scope 2

15.3 General requirements 2

 15.3.2 Limit states 2

 15.3.11 Seismic upgrading 2

15.5 Rehabilitation loads and load factors 2

 15.5.1 Loads 2

 15.5.1.3 Rehabilitation design live loads 2

 15.5.1.3.2 Normal traffic 2

15.1 Scope

Add the following:

The scope of the investigation and rehabilitation of existing structures shall be defined or Consented to by the Ministry.

15.3 General requirements

15.3.2 Limit states

Add the following:

Members evaluated in accordance with Section 14 that have adequate live load capacity do not need to be rehabilitated.

15.3.11 Seismic upgrading

Delete and replace with the following:

Seismic upgrading of the bridge shall be carried out in accordance with the Ministry's Supplement to CHBDC-S6:19 Section 4.

15.5 Rehabilitation loads and load factors

15.5.1 Loads

15.5.1.3 Rehabilitation design live loads

15.5.1.3.2 Normal traffic

Delete the four paragraphs and replace with:

CL1-625 shall be used for the rehabilitation design of bridges that are to carry unrestricted normal traffic after rehabilitation. Design loading in excess of CL1-625 shall be Consented to by Ministry.

For restricted normal traffic, the Ministry will define the suitable fraction of the CL1-625 live load or alternative live load model to be used, which may use Table 15.1.

Bridges rehabilitated for live loads less than CL1-625 shall be posted for the actual restricted loads in accordance with Section 14.

Commentary: *It may be unrealistic to achieve CL-800 loading for rehabilitation of existing bridges and should only be considered in consultation with the Ministry.*

16.1 Scope 2
 16.1.5 Uses requiring consent 2
16.4 Durability 2
 16.4.7 Allowance for wear in deck slabs 2
16.7 Externally restrained deck slabs 2
 16.7.1 General 2
 16.7.3 Cast-in-place deck slabs on stay-in-place formwork 2
16.8 Concrete beams, slabs and columns 2
 16.8.1 General 2
 16.8.10 Cast-in-place deck slabs with FRP stay-in-place structure forms 3

16.1 Scope

Add the following:

16.1.5 Uses requiring consent

Fibre-reinforced polymer (FRP) products shall not be used unless Consented to by the Ministry.

16.4 Durability

16.4.7 Allowance for wear in deck slabs

Delete and replace with:

No allowance for wear is required.

16.7 Externally restrained deck slabs

16.7.1 General

Delete item (c) and replace with:

- (c) The total thickness of the deck slab, t , is at least 175 mm and at least $s/15$.

Delete item (e) and replace with:

- (e) The deck slab is confined transversely by straps in accordance with the applicable provisions of Clause 16.7.2, 16.7.3 or 16.7.4.

Commentary: *The Ministry does not permit stay-in-place formwork. Partial depth concrete panels are not considered stay-in-place formwork.*

16.7.3 Cast-in-place deck slabs on stay-in-place formwork

Delete and replace with:

Stay-in-place formwork is not permitted.

Commentary: *The Ministry does not permit stay-in-place formwork.*

16.8 Concrete beams, slabs and columns

16.8.1 General

Delete the last paragraph and replace with the following:

FRP stay-in-place structural forms are not permitted for cast-in-place deck slabs. FRP stay-in-place structural forms for other components shall only be used when Consented to by the Ministry.

16.8.10 Cast-in-place deck slabs with FRP stay-in-place structure forms

Delete and replace with the following:

FRP stay-in-place structural forms are not permitted for cast-in-place deck slabs.

Note – There are no supplemental clauses for Section 17 of S6:19.