



**BRITISH
COLUMBIA**
Ministry of
Transportation
and Infrastructure



British Columbia Ministry of Transportation and Infrastructure

Bridge Standards and Procedures Manual

VOLUME 1
Supplement to CHBDC S6-14



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Preface

The BC Ministry of Transportation and Infrastructure (BC MoTI) Supplement to CSA S6-14 is to be read and utilized in conjunction with the CSA S6-14 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 S6-14 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-S6-14 (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-S6-14 (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-S6-14” (Supplement to CHBDC S6-14) 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 S6-14 and the CHBDC, the Supplement to CHBDC S6-14 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, on the one hand, and the Supplement to CHBDC S6-14 or the CHBDC, on the other hand, the Project specific Contracts and Terms of Reference shall take precedence over the Supplement to CHBDC S6-14 or the CHBDC, as the case may be.

1.3**Definitions****1.3.2****General administrative definitions**

Add the following administrative definitions:

Engineering Association: means the Association of Professional Engineers and Geoscientists of B.C. (APEGBC.)

Ministry: means the BC Ministry of Transportation and Infrastructure (BC MoTI); the use of “consented to by the Ministry” shall mean consented to by the Ministry engineer who has the authority, responsibility and technical expertise to provide consent as allowed herein.

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 Infrastructure.

1.3.3 General technical definitions

Add the following technical definitions:

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-S6-14.

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 S6-14 uses the term SS, then the corresponding DBSS section shall apply to Design-Build projects.)

Embankment: means earth slopes with or without a foundation unit.

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 S6-14 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 vehicles per day. 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.

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 for bridge work which are accepted by 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

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 S6-14 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

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:

Exemptions from the Supplement to CHBDC S6-14, including for the purpose of application of codes other than S6-14, 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) Induced current cathodic protection system;
- c) Modular deck joints;
- d) Bridge deck heating systems;
- e) Timber components;
- f) Proprietary composite steel/concrete girders;
- g) Full depth precast deck panels;
- 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

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 and creep, 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 S6-14, 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 S6-14 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:

Sidewalks and cycle paths 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 not flatter 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>

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 underpasses, 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 metres. 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 and to avoid pounding during seismic events. For gaps greater than 0.6 m and up to 3 m between adjacent structures, fall arrest provisions shall be provided to prevent people from errantly falling through the gap.

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 unless otherwise consented to by the Ministry. Contact information is as follows:

*Rail/Navigable Waters Specialist
PO Box 9850, Stn Prov Govt
Victoria BC V8W9T5
Phone: 250-387-7733*

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 unless otherwise consented to by the Ministry. Contact information is as follows:

*Rail/Navigable Waters Specialist
PO Box 9850, Stn Prov Govt
Victoria BC V8W9T5
Phone: 250-387-7733*

Add the following clause:

1.5.2.3 Pedestrian/cycle bridges

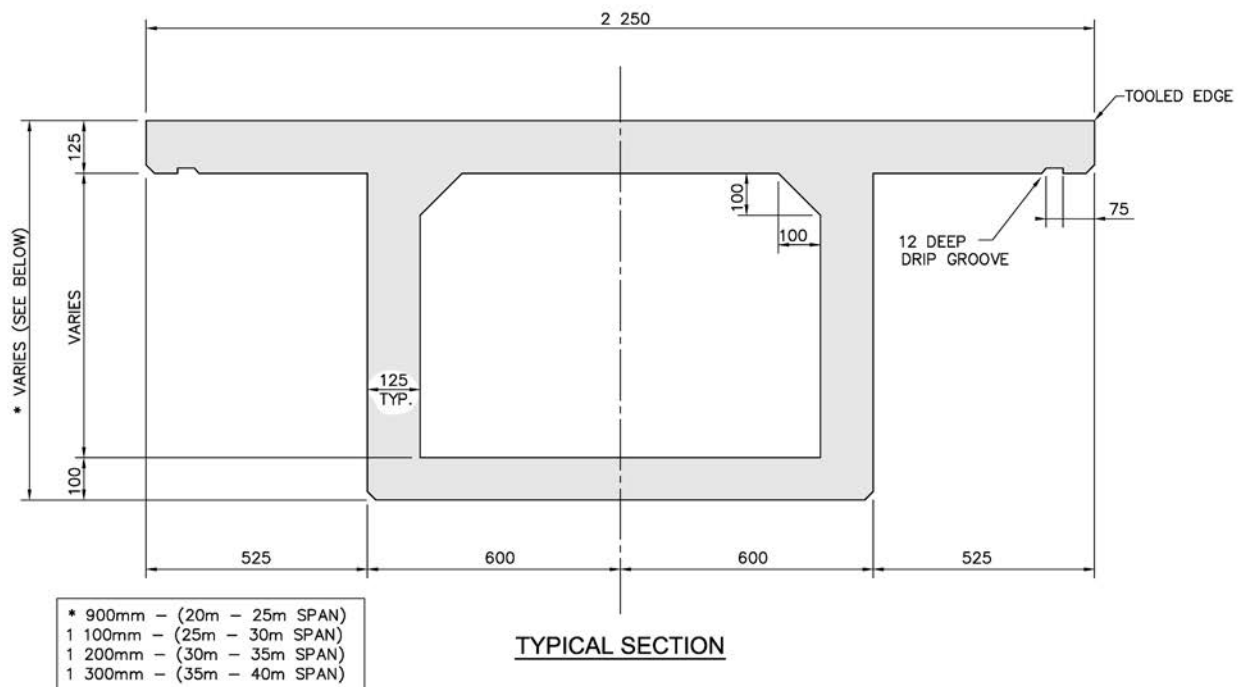
A maximum gradient of 1:12 shall be used for wheelchair traffic on ramps.
The clear distance between the railings shall comply with Clause 1.5.2.1 but shall not be less than 2.0 m.

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.

Figure 1.5.2.3



1.6 Barriers**1.6.1 Superstructure barriers**

Add the following paragraph:

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

1.6.2 Roadside substructure barriers

Add the following to the end of the second paragraph:

When barrier is placed with less than 125 mm clearance to a structural component, the structural component shall be designed for full impacts loads.

1.7 Auxiliary components**1.7.2 Approach slabs**

Delete clause and replace with the following:

The inclusion of approach slabs on paved roads shall be based on site-specific conditions as directed by the Ministry. Approach slabs, if required, shall be 6 m in length, located at least 100 mm below finished grade, anchored to the abutment ballast wall 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 BCL-625 loading or Special Truck and Special lane loading if applicable, whichever produces the maximum effect. The slab shall be assumed to be unsupported over its full length from the abutment to leading edge to account for future long-term settlement.

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

Approach slabs shall be provided for bridges on Numbered Routes where a total settlement greater than 50 mm is anticipated between the abutment and the roadway fill, unless otherwise directed by the Ministry.

Approach slabs shall be provided as follows:

- all Lifeline bridges
- all Major Route and Other bridges in Seismic Performance Categories 2 and 3.

Approach slabs are not required for low-volume road structures.

1.7.3 Utilities on bridges

1.7.3.1 General

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:

<http://www.th.gov.bc.ca/permits/Utility%20Permit%20Manual.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 three 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. The conduit(s) should be located at the base of the barrier, within the rebar cage. The junction boxes to service the conduit should, in most cases, be located in the rear (non-impact) face of the barrier.

1.8 Durability and maintenance**1.8.2 Bridge deck drainage****1.8.2.1 General**

Commentary: *In general the following objectives relate to bridge deck drainage:*

- *Water shall not pond on decks;*
- *Deck drainage inlets should be avoided when possible.*

Deck drainage inlets may be avoided in bridges with the following characteristics, subject to analysis regarding rainfall intensity and volume:

- *Two lanes or less;*
- *Minimum 2% crossfall;*
- *Minimum 1% longitudinal grade;*
- *Less than 120 m in length.*

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 may 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.

Delete the last paragraph and replace with the following:

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

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 paragraphs:

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.*

Delete the first sentence in the second paragraph and replace with the following:

Steel drain pipes shall be hot-dipped galvanized steel pipe and straight to facilitate cleaning.

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 rip-rap

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

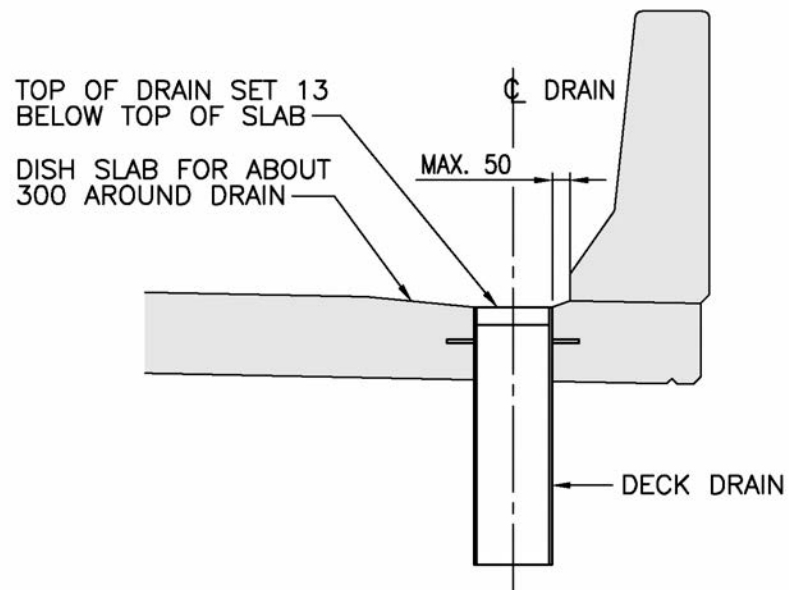
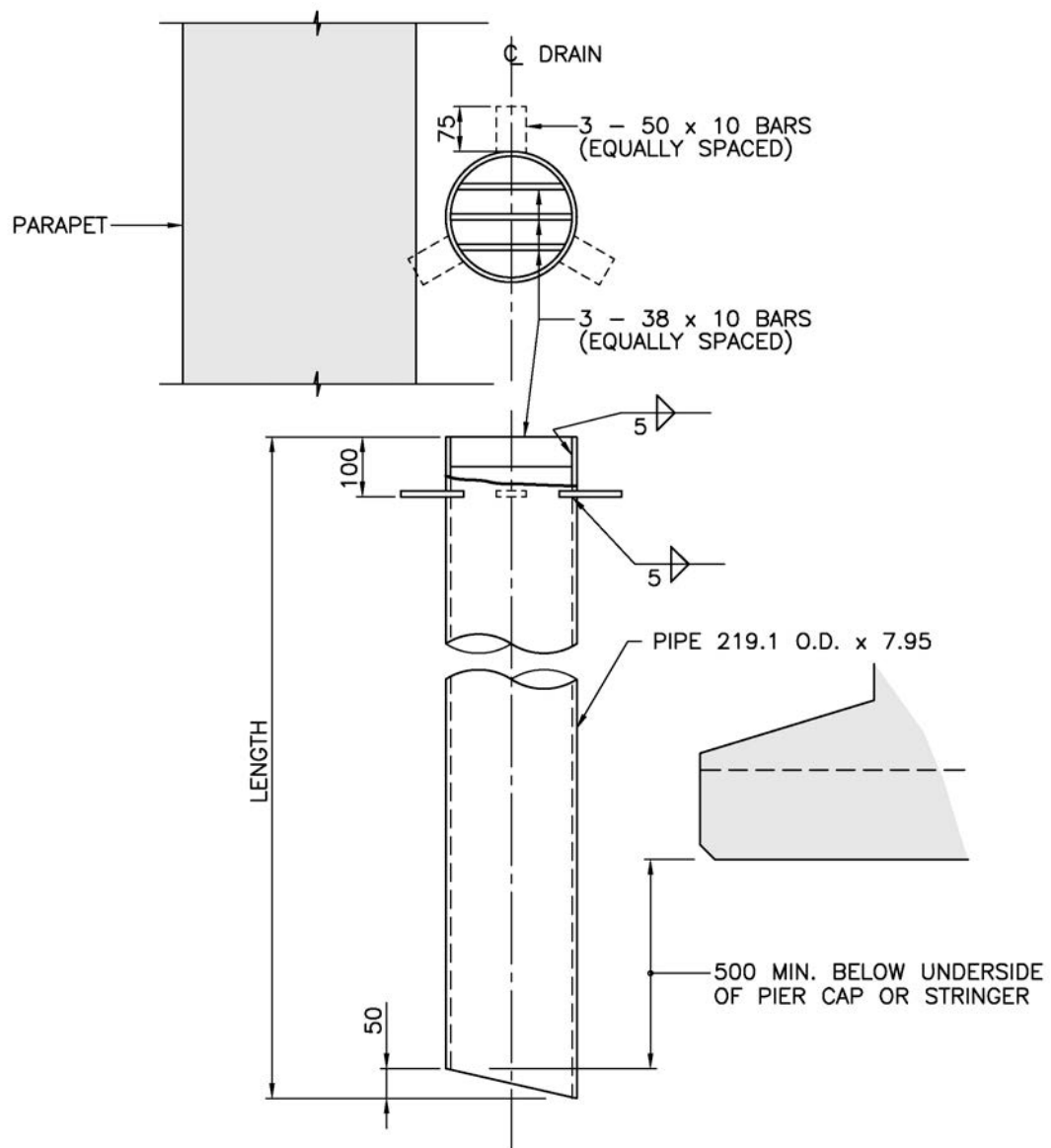


Figure 1.8.2.3.3b
Deck drain fabrication detail



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.

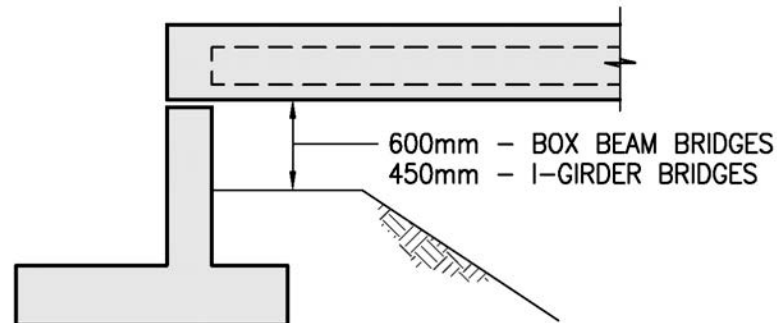
The following minimum clearances shall be maintained between the top of the finished fill in front of the abutment and the underside of the superstructure to facilitate the inspection:

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

Box Beam Bridges	600 mm
------------------	--------

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.

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 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. A minimum vertical clearance of 150 mm is suggested. For steel girders the web stiffeners of the end diaphragm must be located accordingly.

Connections between bearings and girder sole plates shall be bolted and not welded.

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). For buried structures and culverts:

- inlet control 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.

1.9.1.2 Normal design flood

Delete the first paragraph and replace with the following:

The design flood shall be the maximum instantaneous discharge with return periods as follows:

Bridges, retaining walls and river training & channel control works	200-year
Buried structures and culverts ≥ 3 m span	200-year
Low-Volume Road – bridges, buried structures, culverts, retaining walls and river training & channel control works	100-year

In all cases the hydraulic design shall meet the requirements the Water Sustainability Regulation and the 1 in 200 year maximum daily flow or the hydraulic capacity of the stream channel shall also 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.*

For information on maps refer to:

http://www.env.gov.bc.ca/wsd/data_searches/fpm/reports/index.html

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*
- *Hydrotechnical issues*

For additional information on Low Volume Road bridges, refer to: Guidelines for Design and Construction of Bridges on Low-Volume Roads – by Engineering Branch, Ministry of Transportation.

1.9.1.3

Check flood

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 Approved.

- (a) For drainage areas greater than 25 km², the recommended design flow calculation methods are:

- Station Frequency Analysis
- Regional Frequency Analysis

Commentary:

Annual peak 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 drainage areas less than 25 km², design flows can be estimated using the Soil Conservation Service (SCS) Unit Hydrograph 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, regional frequency analysis, etc.) and confirmed with an on-site inspection of stream channel capacity.

Commentary: *For information on the SCS Method, refer to TAC Guide to Bridge Hydraulics, Section 3.4.3 (June 2001).*

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

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, (June 2007).*

1.9.4 Estimation of scour

Add the paragraph as 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 the first paragraph 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 paragraph

For Low Volume Road Bridges, abutments and piers subject to potential scour 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.

For all other bridges, abutments and piers subject to potential scour 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 an Approved Risk Assessment, acceptable to the Ministry, is carried out to justify their use.*

The Risk Assessment in this situation entails a documented design rationale by the responsible engineer with input from the hydrotechnical engineer, 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. The risk assessment signed by the responsible engineer shall include a cost comparison between the proposed spread footing foundation and a piled foundation as well as the analysis of the risks and the recommended solution.

1.9.5.2.2 Protection of spread footings

Add the following paragraph:

Riprap and MSE walls shall not be used for protecting the bottom of spread footings against scour 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 quality rock, weathering, installation details and maintenance. The risk assessment 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 S6-14 commentary). Placing spread footings at streambed level or lower makes them less vulnerable.

1.9.5.5 Protective aprons

Replace the second paragraph with the following:

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

Add the following paragraph:

Riprap shall conform to SS 205. The gradation of the class of riprap shall be in accordance to Table 205-A.

1.9.6 Backwater

1.9.6.1 General

HEC-RAS numerical analysis is approved 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 debris as well as accommodating sediment bed load at the site for the Q200 design flood and for the Q100 design flood on low-volume roads.

Commentary: 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. For debris torrents/flows, the required clearance can potentially become excessively large and may require a risk assessment to justify the additional cost. For navigable waters, the Navigation Protection 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 revetment

Add the following paragraphs:

Riprap shall be used for protecting the bank slopes and bridge end fills of abutments, in conformance with SS 205. Toe protection shall be provided to prevent undermining of slope revetments in accordance with the TAC Guide to Bridge Hydraulics. The revetment shall be wrapped around the bridge end fills and both ends shall be keyed into the bank slopes.

Commentary: Top of slope revetment should be placed a minimum of 0.6 m above the design high-water level. The dimension 0.6 m is consistent with the guidelines set by BC FLNRO for the riprap design and construction for dikes - Riprap Design and Construction Guide.

1.9.11.2 Culvert end treatment

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.

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

1.9.11.6.6 Soil-steel structures

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. Collar walls are required where there is a possibility of uplift, piping or undermining.

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2.3 Design for durability**2.3.2 Durability requirements****2.3.2.5 Bridge joints****2.3.2.5.1 Expansion and/or fixed joints in decks**

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

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.*

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

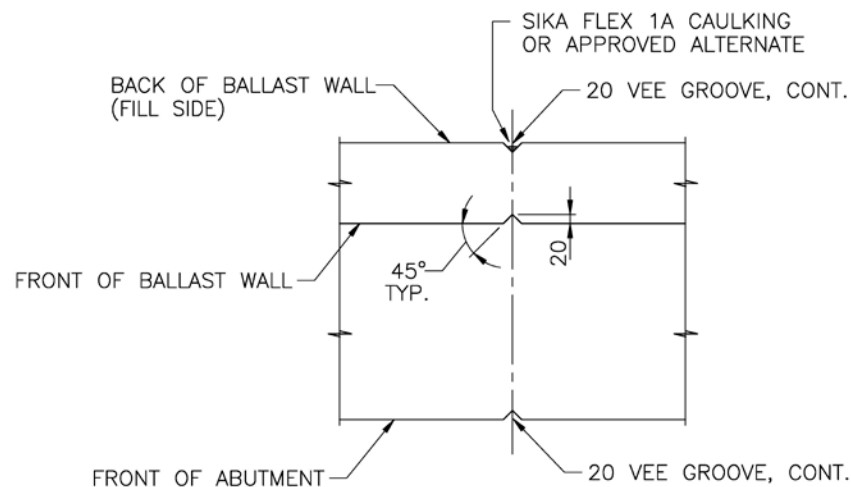
http://www2.gov.bc.ca/assets/gov/driving-and-transportation/transportation-infrastructure/engineering-standards-and-guidelines/recognized-products-list/recognized_products_list.pdf

2.3.2.5.2 Joints in abutments, retaining walls, and buried structures

Add the following and Figure 2.3.2.5.2 after the first paragraph:

Typical details for concrete control joints are shown in Figure 2.3.2.5.2.

**Figure 2.3.2.5.2
Typical control joint**

**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.

2.3.2.6 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.

2.3.2.7 Utilities

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:

<http://www.th.gov.bc.ca/permits/Utility%20Permit%20Manual.pdf>

2.4 Aluminum

2.4.2 Detailing for durability

2.4.2.2 Inert separators

Aluminum railing post surfaces in contact with concrete shall be coated with an alkali resistant bituminous paint, and anchor bolt projections and washers shall be coated with an aluminum impregnated caulking.

2.7 Waterproofing membranes

Add the following paragraphs after the first paragraph:

Unless otherwise consented to by the Ministry, all new bridge decks in the South Coast Region shall have waterproofing membrane and asphalt overlay.

On bridge decks with a waterproofing membrane, the asphalt overlay thickness shall be 100 mm.

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:

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

2.8 Backfill material

Add the following paragraphs and Table:

Backfill for structures shall be Bridge End Fill 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.

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

2.9**Soil and rock anchors**

Add the following paragraph:

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

2.10**Other materials**

Add the following paragraph:

An acceptable premolded 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.

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3.3 Definitions

Delete **Short Span** and replace it with:

Short span – shall be as defined in Clause 14.13.3.1

Add the following definition:

Supervision – monitoring of the passage of an overload by a BC registered professional engineer familiar with bridge design to ensure bridge crossing restrictions in an overload permit are followed by the permit vehicle. Monitoring of the weighing of a permit vehicle is also to be performed if called for in the overload permit. The engineer shall have the authority to stop further movement of the permit vehicle if it is not in compliance with permit requirements. Records of vehicle weight and dimension measurements and of each bridge crossing by the permit vehicle shall be kept by the engineer and a report detailing these observations sent to the Ministry on completion of the move.

3.3 Abbreviations and symbols**3.3.1 Abbreviations**

Add the following abbreviation:

BCL – British Columbia Loading

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	D	E	P	L*	K	W	V	S	EQ	F	A	H
Ultimate Limit States‡												
ULS Combination 5A***	α_D	α_E	α_P	λ	0	0	0	0	1.00	0	0	0

*** For long spans in Seismic Performance Categories 2 and 3, either continuous or semi-continuous for live load, with any one span or combination of spans greater than 200 meters in length. λ shall be equal to 0.50 unless otherwise consented to by the Ministry,

Commentary: For long-span bridges classified as lifeline bridges in accordance with Clause 4.4.2, partial live load shall be included in ULS Combination 5A. Effects of live load on bridge inertia mass for dynamic analysis need not to be considered for this special load case.

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.

For long-span lifeline bridges, presence of partial live load during a major seismic event shall be considered. Application of Turkstra's rule for combining uncorrelated loads indicates that 50% of live load is reasonable for a wide range of values of average daily truck traffic (ADTT). This issue has been considered for the first time in the third edition of the AASHTO LRFD Bridge Design Specifications, 2004.

The maximum (1.25) and minimum (0.8) values of load factor for dead load, α_D , are intended to account for, in an indirect way, the effects of vertical accelerations. If these effects are considered explicitly by using a vertical design spectrum, the load factor for dead load, α_D , should be taken as 1.0.

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

Load	Live load factor	
	Special loads travelling on bridge without supervision meeting crossing restrictions in Clause 3.8.3.2.3	
	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 S6-14 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/m ³
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

Add the following paragraph:

Where the code uses the term “CL-W” loading, this shall be modified to “BCL-625” loading.

BCL-625 design loading described in Figures 3.2(a) and 3.3(a) is the designated live load unless Approved otherwise.

3.8.3.1.2 CL-W Truck

Delete the third paragraph and replace with the following:

A BCL-625 Truck, as specified in Figure 3.2(a) shall be used.

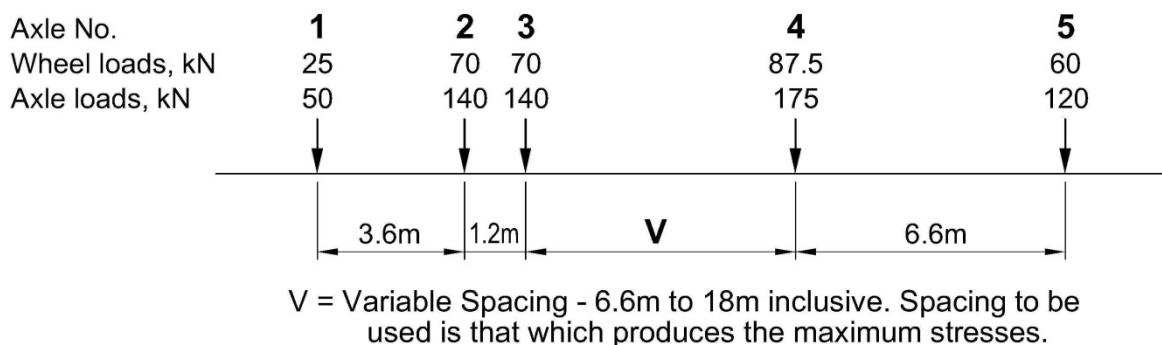
Note: *The total load of the BCL-625 Truck is 625 kN, but the axle loads and distribution differs from that shown in Figure 3.2.*

Delete the fourth paragraph and replace with the following:

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

Figure 3.2(a)

BCL-625 Truck



Commentary: *Bridges designed to BCL-625 Live Load will have adequate load capacity for 85 tonne Class Permit Vehicles and 6 Axle Mobile Cranes with boom in cradle to travel with other normal traffic. CL-625 Loading is inadequate on short spans for Cranes and on medium length continuous spans in moment for 85 tonne Class Permit Vehicles.*

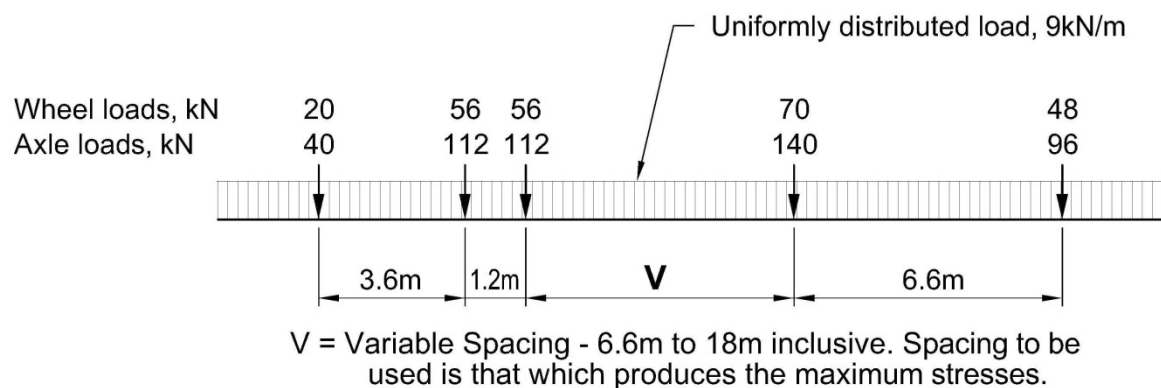
3.8.3.1.3 CL-W Lane Load

Delete the second paragraph and replace with the following:

A BCL-625 Lane Load as detailed in Figure 3.3(a) shall be used.

Figure 3.3(a)

BCL-625 Lane Load

**3.8.3.2 Special loads**

Add the following Clause in sequence:

3.8.3.2.3 Geographically Specific Special Loads

In addition to BCL-625 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

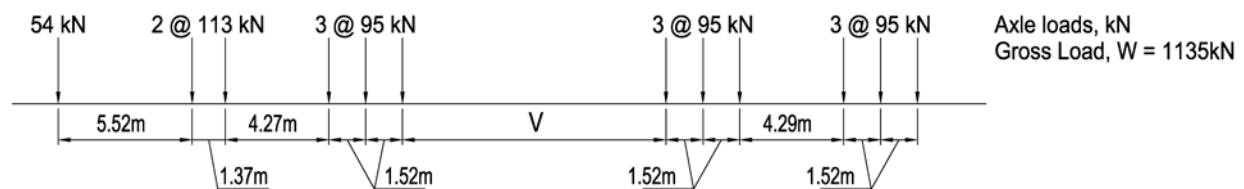
- Highway 2 from the BC/Alberta border to the junction with Dangerous Goods Route
- Highway 52
- Highway 29S from Chetwynd to Highway 52
- Highway 97 from Hasler north to Mile 83.5 on the Alaska Highway/Highway 97
- Highway 49
- Highway 29N from Charlie Lake to Canyon Dr #520R
- Highway 77
- Dangerous 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).

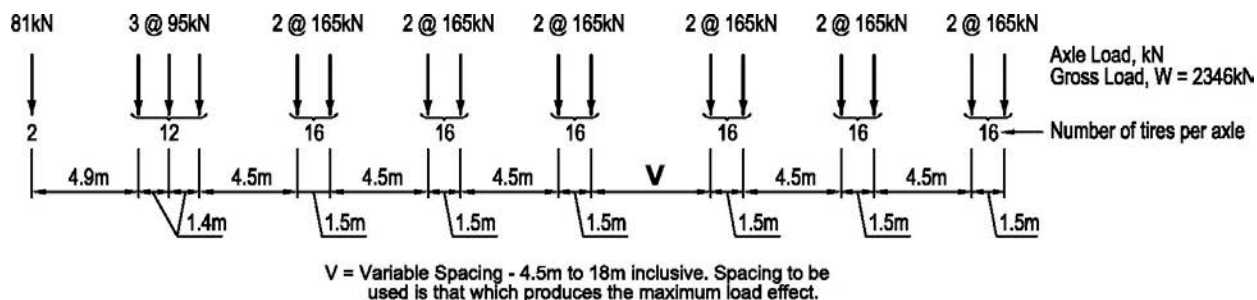
Figure 3.8.3.2.3 i
EPLL1



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

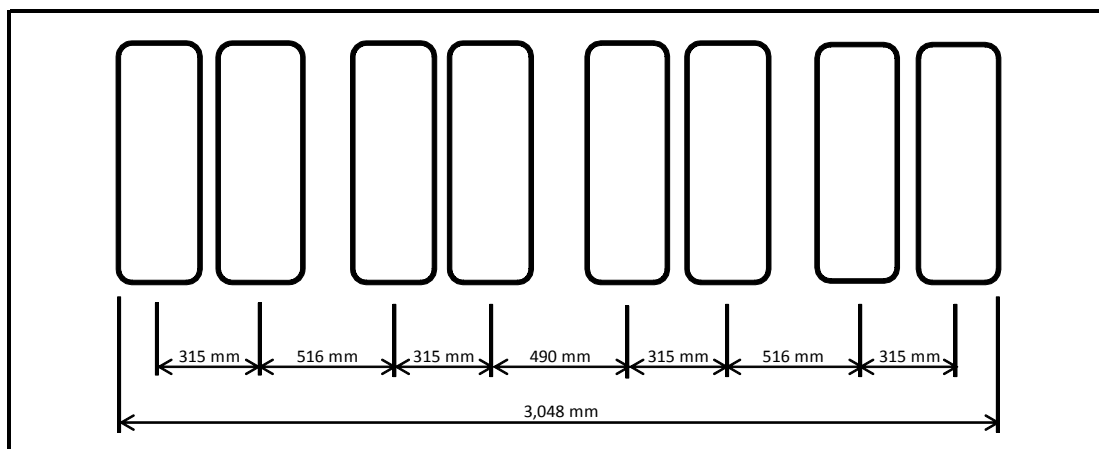
Transverse wheel spacings and the clearance envelope for EPLL1 truck load shall be similar to those indicated for the CL-W truck in Figure 3.2 of CHBDC.

Figure 3.8.3.2.3 ii
EPLL2



For the EPLL2 truck, transverse wheel spacings for 16 tire tandems 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 Axles of EPLL2



Commentary: The extraordinary vehicle configurations described in this section are based on recent 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:

- (c) For the FLS, the traffic load shall be one BCL-625 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 BCL-625 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

Note: the axle numbers for the BCL-625 Truck are shown in Figure 3.2(a)

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.

Note: the axle numbers for the BCL-625 Truck are shown in Figure 3.2(a)

3.8.8 Barrier loads**3.8.8.1 Traffic Barriers**

Delete the second sentence and replace with the following:

These loads shall be used only for the design of traffic barrier anchorages, decks and other structural components supporting the barrier.

3.14 Vessel collisions**3.14.2 Bridge classification**

Add the following paragraph:

The Ministry shall determine the bridge classification for vessel collision design purposes.

3.16 Construction load and loads on temporary structures**3.16.1 General**

Insert the following paragraph:

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.

A3.3 Vessel collision**A3.3.2 Design vessel selection****A3.3.2.1 General**

Replace the first sentence with the following:

Method II shall be used for “Class I” bridges, unless the Ministry determines that there is insufficient data to determine reliable probabilistic values. Method I or Method II may be used for “Class II” bridges.

Commentary: *The Ministry does not collect data on vessel type and passage frequency or collision frequency.*

A3.3.3.2 Probability of aberrancy

Replace the first sentence with the following:

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.*

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4.1**Scope**

Add the following:

Commentary: *Chapter 4 Seismic Design of S6-14 has made a major shift in philosophy toward performance-based design, which is in keeping with current BC practice for bridges.*

4.2**Definitions****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.

Traditionally the capacity design approach involved an explicit selection of a plastic mechanism as the lateral load resisting system with 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 beyond the minima specified by the code. 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. Elastic forces calculated from static or dynamic analyses may be acceptable in the design of the lateral load-resisting system within S6-14 and this Supplement, but such forces do not constitute 'capacity protection' within a capacity design approach. See specific requirements under Clause 4.4.10.4.

Capacity-protected element - *the critical structural component that is being protected from damage by using the limited and controlled structural capacity of ductile elements within the lateral load-resisting system.*

Probable resistance: *The combined effects of probable resistances (overstrength factor >1.0, see Clauses 4.4.10.4.2 and 4.4.10.4.3) with expected material properties (see Clause 4.7.2) can be considered as equivalent to the over-strength capacity of structural components as described in previous codes. The term "over-strength" is not used in S6-14 but is conceptually important in a capacity design approach.*

Static Pushover analysis - an inelastic static analysis involving a step-by-step force-deformation procedure in order to identify the local and global inelastic behaviour and failure modes of the lateral load resisting system

Pushover analyses are used to determine both capacity design demands and to assess structural behaviour and damage at each stage of inelastic deformation of the lateral load resisting system. Section capacities can account for degradation with increasing ductility demands, and the local deformations and strains allow for damage and performance assessments at all specified earthquake levels.

Add the following definitions:

Extended pile bent – 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. Where “pile bent” is used in this chapter it may be interpreted as an extended pile bent.

Seismic performance category (SPC): A category assigned to a bridge that affects the requirements for design approach (FBD or PBD), analysis (See Clause 4.4.4 and Table 4.10) and detailing.

Sign structures – Structures supporting signs for road direction, tolling equipment or messages that span or cantilever over a roadway.

4.3 Abbreviation and Symbols

4.3.2 Symbols

Commentary:

P_f within a capacity design approach can account for plastic behaviour in the lateral resisting system.

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 specified by the Ministry.*

4.4.3 Seismic hazard**4.4.3.1 General**

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

Spectral values shall be adjusted to reflect local site conditions in accordance with Clause 4.4.3 to give the design spectral values. Design spectral values may also be obtained using site response analysis with consent of the Ministry. The spectra from site response analysis shall not be less than 80% of the code based spectra.

Delete the 4th paragraph.

4.4.3.2 Site properties

Commentary: Update No. 1 to S6-14 was published in April 2016 and shall apply.

4.4.3.3 Site coefficients

Commentary: Update No. 1 to S6-14 was published in April 2016 and shall apply.

4.4.3.6 Time-history input motions

Time history input motions used in the design are subject to the consent of the Ministry.

4.4.4 Seismic Performance Category

Change Table 4.10 SPC from 2 to 1 for Row 1 for Lifeline Bridges (See modified Table 4.10 following).

Table 4.10
Seismic performance category based on 2475 year return period spectral values
(See clause 4.10.3)

For $T < 0.5$ s	For $T > 0.5$ s	Seismic Performance Category	
		Lifeline bridges	Major-route and other bridges
$S(0.2) < 0.20$	$S(1.0) < 0.10$	1	1
$0.20 \leq S(0.2) < 0.35$	$0.10 \leq S(1.0) < 0.30$	3	2
$S(0.2) \geq 0.35$	$S(1.0) \geq 0.30$	3	3

Add note to Table 4.10 as follows:

For lifeline bridges in SPC 1, detailing of structural elements shall adopt requirements for SPC 2 as a minimum.

Commentary: *As published by CSA, and considering also Table 4.11, all Lifeline bridges in BC (and Canada) regardless of seismic hazard would require explicit demonstration of seismic performance through PBD. Values for $S(0.2) < 0.2$ and for $S(1.0) < 0.1$ are considered unduly low for many bridges to benefit from the analyses and methods of PBD methods. At low levels of seismic hazard, a bridge's seismic performance would have little or no post-elastic behaviour, such that the bridge design focus should not be on plastic design methods. The Ministry may require PBD on specific projects.*

4.4.5 Analysis and design approach

4.4.5.1 General

Delete the reference to "Clause 4.4.3.5" in the last sentence and replace with "Clause 4.4.10.2".

Add the following sentence:

Sign structures in seismic performance categories 2 and 3 require that seismic performance be demonstrated for a no-collapse requirement at a 2% in 50-year hazard (2,475 year return period). *See also Clause 4.4.6.1.*

Commentary: *Collapse prevention for sign bridges should be demonstrated using displacement-based approaches and considering local plastic behaviour and buckling to demonstrate performance. Applicable clauses within and cross-referenced from Clause 4.8.4 should be applied.*

4.4.5.2 Single-span bridges

4.4.5.2.1 Analysis requirements

Replace the first sentence with

In Seismic Performance Categories 2 and 3, all bridges except single span bridges having a skew angle less than 20° and a maximum subtended angle of 30° shall be analyzed and designed to address the seismic behaviour resulting from the geometric irregularities.

4.4.5.3 Multi-span bridges**4.4.5.3.1 Analysis requirements and design approach**

For Table 4.11: Change the last sentence in the title of the Table to:

(See Clause 4.4.7 for FBD requirements).

Replace Table 4.12 with the following:

Seismic Performance Category	Lifeline Bridges		Major Route Bridges		Other Bridges	
	Irregular	Regular	Irregular	Regular	Irregular	Regular
1	No seismic analysis required					
2	EDA, ISPA and NTHA	EDA and ISPA	EDA and ISPA	ESA	EDA	ESA
3	EDA and ISPA NTHA	EDA and ISPA	EDA and ISPA	EDA	EDA	ESA

Add the following:

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

Elastic Static Analysis (ESA): These analyses may be carried out on structural model(s) without rigorous treatment of soil-structure interaction but should include effects of foundation flexibility important to the global structural response. The seismic demand may be based on free-field ground surface or near-surface (as appropriate to the foundation system) response spectrum established using either code factors or wave propagation (1D or 2D) analysis as consented to by the Ministry. Where the benefits of site-specific site response analyses are sought, site characterization consistent with “a high degree of site understanding” shall be undertaken, and ground motions that represent the site and hazard shall be determined.

For Class F sites, the inertial loads shall be established based on the Geological Survey of Canada (GSC) response spectra adjusted for site conditions as per the shear wave average velocity classification, or using spectra from site response analysis.

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. The seismic demand shall be based on site-specific free-field response spectrum or time-history records computed at an elevation determined by the structural and geotechnical engineers. The applicable free-field response spectrum shall be established using either code factors or wave propagation (1D or 2D) analysis as consented to by the Ministry utilizing equivalent linear or non-linear method of analysis. Where the benefits of site-specific site response analyses are sought, site characterization consistent with “a high degree of site understanding” shall be undertaken, and ground motions that represent the site and hazard shall be determined.

For Class F sites, the inertial loads shall be established based on the Geological Survey of Canada (GSC) response spectra adjusted for site conditions as per the shear wave average velocity classification, or using spectra from site response analysis.

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 4.6.4.

Where applicable(e.g. liquefaction-induced lateral spreading or settlements), the effects of kinematic loading from inelastic ground deformations on the structure shall be evaluated and combined with the displacement and other effects of inertial loading using the combinations described below:

- 100% kinematic demands
- 100% inertial demands
- 50% inertial demands + 100% kinematic demands

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.

For Class F sites, the inertial loads shall be established based on the GSC response spectra adjusted for site conditions as per the V_s classification, or using spectra from site response analysis.

Non-linear Time-history Analysis (NTHA): These analyses shall be carried out on a full or partial model of the bridge system incorporating the non-linear behaviour of foundation soils and foundation elements. Computer software used for this purpose shall have the capability to incorporate non-linear soil effects, pre- and post-earthquake stress-strain-strength characteristics of soils, and non-linear structure effects. These analyses shall be either 2D or 3D. Unless otherwise specified by the Ministry, analyses shall be carried out for all input ground motions defined in Clause 4.4.3.6.

Commentary: *Tables 4.12 and 4.13 apply to structural analyses including appropriate modelling for important soil-structure interaction effects in all analysis types. They do not refer to site response analyses used for seismic hazard considering soil behaviours.*

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

Kinematic demands include the effects of liquefaction-induced ground deformations, for example lateral spreading or support settlements. The combinations provided are intended for sites where kinematic demands are induced by liquefaction which reduces the soils ability to transmit ground motions to the structure. Where this is not the case, then a special assessment is required to develop an appropriate combination of inertial plus the applicable kinematic effects. In lieu of an explicit effective stress and non-linear coupled approach to these combined effects, some allowance for concurrent effects is appropriate.

Multiple support inputs are difficult to predict in British Columbia owing to the limited information on known faults. These effects may also provide a net reduction in structural response. Project specific seismic specifications will be provided for important or major bridges when needed.

4.4.6 Performance-based Design

4.4.6.1 General

Replace third paragraph with:

Lifeline bridges in SPC 2 and 3 shall require independent peer review unless stated otherwise in project specifications.

4.4.6.2 Performance levels

Replace Table 4.15 with the following:

Seismic Ground Motion Probability of exceedance in 50 Years (return period)	Lifeline Bridges		Major-Route Bridges		Other Bridges	
	Service	Damage	Service	Damage	Service	Damage
10% (475 years)	Immediate*	Minimal*	Immediate	Minimal	Service Limited*	Repairable*
5% (975 years)	Immediate	Minimal	Service Limited*	Repairable*	Service Disruption*	Extensive*
2% (2475 years)	Service Limited	Repairable	Service Disruption	Extensive	Life Safety	Probable replacement

* Optional performance levels unless required by the Ministry.

Commentary: S6-14 mandates a higher seismic hazard (2475 year levels) than S6-06. This change is consistent with hazard levels in NBCC 2015. S6-14 also introduced PBD. Damage levels as tied to service expectations are believed to have been unduly conservative for modern, well-detailed columns in ductile substructures. This was true in particular for the “none” and “minimal” damage descriptions. Accordingly, adjustments to performance requirements in Table 4.15 (above) and for damage descriptors in Table 4.16 (below) are adopted.

4.4.6.3 Performance criteria**Table 4.16:**

Replace description for “Minimal Damage” to:

Minimal Damage

- General: Bridge shall sustain minor damage that does not affect the 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 yield (see Clause 10.5.3.3). Local or global buckling shall not occur.
- Connections: 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 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.
- 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.

Replace description for “Repairable Damage” to:

Repairable Damage

- General: The bridge may experience inelastic behaviour, however primary members shall be repairable in place and shall be capable of supporting the dead load plus live load corresponding to the service performance criteria during repairs.
- Concrete Structures: Tensile rebar strains shall not exceed 0.025.
- Steel Structures: 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.
- Residual displacements including settlement, translation or rotation of the structure or supports, including abutments, retaining and wing walls shall not compromise the service and repair requirements of the bridge.
- 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 prevent span loss in aftershocks. Damage to restrainer supporting elements such as end diaphragms or substructure shall not require bridge closure to repair.
- Ground deformations shall be mitigated such that permanent foundation offsets are small and repair objectives specified above can be met. Foundation offsets shall be limited such that repairs can bring the structure back to the original operational capacity.

Commentary:

The requirements for demonstration of aftershock capacity have been deleted at this time since there are no generally accepted methodologies for this type of assessment.

In general, superstructures, ductile substructures, restrainers and foundations designed to S6-14 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 which retains essentially its full capacity after the design event and is capable of sustaining multiple additional cycles of seismic loading.

ATC-49 provides guidance in determining performance limits for pile foundations.

Replace description for “Extensive Damage” to:

Extensive Damage

- 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 the dead load plus 1 lane of live load in each direction (to account for emergency vehicles), 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: 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 emergency traffic. Bearings and joints may require replacement.
- Restrainers: Restraining systems might suffer damage 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.

Commentary: *The requirements for demonstration of aftershock capacity have been deleted at this time since there are no generally accepted methodologies for this type of assessment.*

In general, superstructures, ductile substructures, restrainers and foundations designed to S6-14 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 which retains essentially its full capacity after the design event and is capable of sustaining multiple additional cycles of seismic loading.

ATC-49 provides guidance in determining performance limits for pile foundations.

Replace description for “Probable Replacement” to:

Probable Replacement:

- 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.
- Extensive distortion of beams and column panels may occur.
- Members shall be capable of supporting the dead plus 30% live loads, excluding impact, but 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 nor prevent evacuation.

4.4.6.4 Performance Criteria for Walls, Slopes and Embankments

The following seismic performance criteria shall be met for the design of retaining walls, slopes and embankments:

Category	Retaining Walls	Slopes and Embankments
<i>Lifeline</i>	• No collapse of retaining wall during and following 975-year ground motion • 100% of lanes in close proximity to the bridge are available for use following 975-year ground motion • 50% of lanes away from the bridge are available for use following 975-year ground motion • Permanent wall lateral deformations shall be such that the service level performance and damage level performance requirements for Structures are met.	• 100% of lanes in close proximity to the bridge are available for use following 975-year ground motion • 50% of lanes away from the bridge are available for use following 975-year ground motion

<i>Major-route</i>	• No collapse of retaining wall during and following 975-year ground motion • Permanent wall lateral deformations shall be such that the service level performance and damage level performance requirements for Structures are met.	• Factor of Safety against slope failure under static loading as per Table 6.2b. • Pseudo-static factor of safety against slope failure = 1.1 under 975-year ground motion
<i>Other</i>	• No collapse of retaining wall during and following 475-year ground motion • Permanent wall lateral deformations shall be such that the service level performance and damage level performance requirements for Structures are met.	• Factor of Safety against slope failure under static loading as per Table 6.2b. • Pseudo-static factor of safety against slope failure = 1.1 under 475-year ground motion

Note: As a minimum, the distance defined as “close proximity to the bridge” shall be taken as the length of an approach embankment equal to a horizontal distance that is twice the height of the embankment or retaining wall, as the case may be. This distance may be altered by the Ministry in project-specific requirements.

4.4.7 Force-based Design

4.4.7.1 General

Add the following paragraph:

For regular bridges of slab, beam-girder, or box girder construction, a detailed analysis of earthquake effects on superstructure components is not required. However, lateral analysis and related design of cross-frames or diaphragms between girders at the abutments and piers, and of bearings, bracing connections and connections between the superstructure and substructure are required.

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****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. Displacement and force demands are commonly calculated and assessed in each direction separately.

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° 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**

In second paragraph, delete the sentence: "These restraint forces need not apply if the requirements of clauses 4.4.3.5 are satisfied."

Add the following:

For bridges without transverse seismic shear restraint, the transverse support length from the edge of the girder to the transverse face of pier or abutment shall be in accordance with the "N" dimension from Clause 4.4.10.5.

Commentary: *Clause 4.4.10.1 refers specifically to connection forces between superstructure and substructure, when structural connections are used (e.g. through bearings or separate restrainers). They are prescriptive for that purpose alone. They apply when seat lengths are less than prescribed as a means to prevent loss of span failures. Neither these connection forces nor the seat lengths prescribed in Clause 4.4.10.5 are applicable to integral or semi-integral abutment bridges in which the superstructure – integral with the ballast wall or the entire abutment – is restrained from movements by soil pressures during earthquakes. In lieu of analyses and calculations to demonstrate that shorter seat lengths are sufficient for integral or semi-integral abutments, the seat lengths of Clause 4.4.10.5 shall be used.*

These connection forces are also not intended to be combined with seismic soil pressure forces on abutments, nor with self-inertia forces from massive concrete abutments. This issue has been investigated recently as part of the AASHTO LRFD code, with a disposition consistent with the above

commentary. See also Clauses 4.4.10.7 regarding hold-down forces and Clause 4.6.5 regarding seismic forces on abutments and retaining walls..

4.4.10.4 Seismic Performance Category 3

4.4.10.4.2 Modified seismic design forces for force based design

Delete the second paragraph and replace with the following:

Seismic design forces for capacity-protected elements shall be designed to have factored resistances equal to or greater than the maximum force effect that can be developed by the ductile substructure element(s) attaining their *probable* resistances, as part of an identified plastic mechanism or other predictable mechanism attaining their probable resistance.

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 seismic forces of 1.25 times the elastic seismic forces (i.e. $R = 1.0$ and $I_E = 1.0$).

Commentary: *Based on S6-14 for the design of capacity-protected elements, the margin of resistance compared to demands of the chosen ductile substructure mechanism, is summarized as:*

$$\phi R_{nominal} > \phi_{probable} \times \{D_{expected}\}$$

where

ϕ concrete, rebar or steel resistance factors in Chapters 8 and 10

$R_{nominal}$ section resistance using specified grades for material strengths

$\phi_{probable}$ Phi factor greater than unity as described in Clause 4.4.10.4.3

$D_{expected}$ Demand calculated using a ductile plastic mechanism (or other predictable and acceptable lateral load-resisting system) using expected material properties as defined in Clause 4.7.2.

This margin is also to be used for capacity design checks following a performance-based design approach.

Elastic forces may be smaller than those derived from plastic mechanisms but design to such elastic forces is not considered to produce “capacity protection” and may produce a small margin against unexpected brittle failure modes. Components designed elastically require additional conservatism to ensure that brittle failures or collapse would not occur at demand levels marginally greater than the adopted seismic hazard. The 1.25 factor is generally consistent with the approach specified for connection force design in Clause 4.4.10.4.2.

4.4.10.4.3 Yielding mechanisms and design forces in ductile substructures

Delete the third paragraph and its related clauses (i.e. (a) and b)) and replace with the following:

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

(a) Shear Force – the shear corresponding to inelastic hinging of the column as determined from static plastic analysis considering the probable flexural resistance of the member and its effective height. For flared columns and columns attached to partial height walls, the top and bottom flares and the height of the walls shall be considered in determining the effective column height. If the column foundation is significantly below ground level, consideration shall be given to the possibility of the hinge forming above the foundation due to soil confinement. This is acceptable provided the inelastic hinges are at reasonably accessible and repairable locations.

(b) Axial Force – the axial force corresponding to inelastic hinging of the column in a ductile substructure at its probable resistance.

For cases where elastic design forces are significantly lower than forces derived from capacity design principles, then for capacity-protected elements in accordance with Clause 4.4.10.4.2, shear and axial design forces for ductile substructure elements shall be taken as the unreduced elastic design forces increased by 1.25 times and in accordance with Clause 4.4.9 (i.e $R=1.0$ and $I_E=1.0$.)

Commentary: The Ministry considers “reasonably accessible” to mean less than 2 metres below ground or below mean water or tide level.

4.4.10.7 Hold-down devices

Replace this clause with:

Bridges in Seismic Performance Categories 2 & 3 shall be vertically restrained unless otherwise consented to by the Ministry. Hold-down devices shall be provided to resist a minimum uplift force of $0.3D$ or the net uplift force that exists resulting from the tributary dead load (D) multiplied by $(F_a S_a (0.2) - 1.0)$ whichever is greater. The hold-down devices shall consist of anchored vertical bars and must be of reinforcing steel of Grade 400W, 500W or steel having similar or better rupture strains and ratios of ultimate stress to yield stress.

Where design and detailing explicitly accounts for uplift effects in bridges using seismic isolation systems, supplementary uplift restraint as described in this clause is not required.

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. Integral or semi-integral abutment bridges, or bridges with structurally integral superstructure-to-substructure connections, would be

considered to be held down at the relevant supports if the required capacity were demonstrated.

4.5.3.5 Static pushover analysis

Add to this clause

The static pushover analysis must be taken to the deformation necessary to identify the full plastic mechanism, expected ultimate displacement capacity, and ultimate failure mode. Displacement demands must 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.

Foundation flexibility must be considered within pushover models to obtain a realistic pattern of hinges and their related deformations.

Commentary: *Static push-over 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 push-over 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.

4.6 Foundations

4.6.2 Analysis methods

Add to first paragraph:

The analysis shall address local site effects, including slope and basin effects where applicable, and effects from or on adjacent infrastructure.

4.6.3 Geotechnical resistance factor

4.6.3.1 Performance-based design

Delete last sentence and replace with:

The consequence factor shall be 1.00.

Commentary: *S6-14 provides resistance factors only for "essentially elastic" performance, for capacity design and for "life safety" performance. It is not the intent that the length or number of piles be increased by forcing the use of*

static resistance factors for intermediate damage states. For these intermediate damage states, performance based design shall apply.

For preliminary design for axial resistance of deep foundations, one approach could be to modify the static values (Table 6.2) at different seismic performance levels such as follows:

Immediate/Minimal = Static loading value + .1

Service Limited/Repairable = Static loading value + .2

Service Disruption/Extensive = Static loading value + .25

Life Safety/Probable replacement = 1.0

For example, for compression of deep foundations with a Static Pile Test for low/typical/high degree of understanding, resistance factors would be:

<i>Degree of Understanding (See Section 6.5.3.2)</i>	<i>Resistance Factor from Table 6.2</i>	<i>Minimal Damage</i>	<i>Repairable Damage</i>	<i>Extensive Damage</i>	<i>Probable Replacement</i>
<i>Low</i>	<i>.5</i>	<i>.6</i>	<i>.7</i>	<i>.75</i>	<i>1.0</i>
<i>Typical</i>	<i>.6</i>	<i>.7</i>	<i>.8</i>	<i>.85</i>	<i>1.0</i>
<i>High</i>	<i>.7</i>	<i>.8</i>	<i>.9</i>	<i>.95</i>	<i>1.0</i>

4.6.5 Seismic forces on abutments and retaining walls

Add the following commentary:

Commentary: *This clause considers lateral forces from soils and abutment inertia. It does not require these be combined with forces from superstructure connections specified in Clause 4.4.10.1 and from Clauses 4.4.10.6, 4.4.10.7 and 4.4.10.8. Superstructure connection forces and combinations were recently updated in AASHTO, 2015, and have reverted to not combining superstructure connection forces with substructure seismic soil pressures. Such effects may combine for short durations and may be either detrimental or beneficial to sub-structure components, but are unlikely to increase abutment displacements. For pile-supported abutments where it is foreseeable that combined effects may lead to unacceptable pile hinging or to brittle failure modes in the piles or connections, superstructure and substructure combined effects should be considered. For integral or semi-integral abutments these connection forces may be neglected.*

4.6.6 Liquefaction of foundation soils

4.6.6.1 Liquefaction potential of foundation soils

Add the following:

The maximum earthquake magnitude for liquefaction analysis shall be based on deaggregation of seismic hazard.

For *Major-route* Bridges: A minimum of seven single-component horizontal ground motion time-histories shall be used. Input ground motion time-histories developed for bridges in the vicinity of the site are permitted with uniform scaling to the site-specific peak horizontal ground acceleration. The mean response quantity shall be used for design.

For *Other* Bridges: A minimum of three single-component horizontal ground motion time-histories shall be used. Input ground motion time-histories developed for bridges in the vicinity of the site are permitted with uniform scaling to the site-specific peak horizontal ground acceleration. The maximum response quantity shall be used for design.

Commentary: Refer also to Clause 4.4.3.6 for Lifeline bridges. For “Major route” bridges using seven records, this is considered sufficient to adopt the mean response quantities for design. For “Other” bridge using three records, the maximum response quantity is appropriate. If seven records are used as described for the Major route bridges then the average response quantity may be used for Other bridges.

4.6.8 Fill settlement and approach slabs

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

Approach slabs shall be provided in accordance with Clause 1.7.2.

Commentary: Project specific design criteria developed by the Ministry may specify settlement slabs (6 m long, measured normal to the abutment) as part of the structural and seismic design criteria. In general approach slabs improve post-seismic performance and vehicle access. For the seismic design of bridges identified in this Supplement under Clause 1.7.2, the role of approach slabs shall emphasize fill settlement. The portion (length) of approach slabs structurally spanning over gaps between end piers or abutments and approach fills shall not be considered as mitigating against fill settlements for post-earthquake bridge access.

4.7 Concrete structures

4.7.4 Seismic performance category 2

Delete the second sentence and replace with the following: resume

The transverse reinforcement at the top and bottom of a column and in potential plastic hinge zones of beams, columns 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****4.7.5.2.3 Flexural resistance**

Delete this Sentence.

4.7.5.2.4 Column shear and transverse reinforcement

Replace Clause 4.7.5.2.4 with the following:

The factored shear force, V_f , on each principal axis of each column and concrete pile bent shall be as specified in Clause 4.4.10.4.3.

In lieu of more detailed analysis and design of concrete columns using the commentary below, for columns designed as capacity-protected elements within a ductile substructure, the amount of transverse reinforcement shall not be less than that determined in accordance with Clause 8.9.3, modified by sub-clause (a) below.

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

- (a) Shear reinforcement shall be designed in accordance with the requirements of Clause 8.9.3 with $\beta = 0.10$ and $\theta = 45^\circ$. The transverse reinforcement shall consist of hoops, seismic crossties or spirals.
- (b) The plastic hinge region shall be assumed to extend down from the soffit of girders or cap beams at the top of columns, and up from the top of foundation at the bottom of columns, a distance taken as the greatest of:
 - (i) The maximum cross-sectional dimension of the column;
 - (ii) one-sixth of the clear height of the column;
 - (iii) 450 mm;
 - (iv) The length over which the moment exceeds 80% of the maximum moment.
- (c) For tall columns or piers or those having high axial loads, rational analysis that considers potential plastic hinging mechanisms shall be performed to determine the location and extent of plastic hinge regions.

The plastic hinge region at the top of a concrete extended pile bent shall be taken as that specified for columns. In the region near the bottom of an extended pile bent the plastic hinge region shall be considered to extend from a low point of three times the maximum cross-section dimension below the calculated point of maximum moment, taking into account soil-pile interaction, to an upper point at a distance of not less than the maximum cross-section dimension, and not less than 500 mm, above the ground line.

Commentary:

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

• **Detailed analysis and design of concrete columns methodology:**

For typical reinforced concrete columns used in bridges in British Columbia, the shear provisions contained in Clause 8.9.3 are unduly conservative and can impede the design of an economic and seismically desirable ductile substructure. In particular, the need for increased column dimensions to meet V_c provisions within 8.9.3 can make it impractical and uneconomic to design capacity-protected footings, pile caps or cap beams.

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 contained in either:

- *Displacement-based Seismic Design of Structures, Priestley, Calvi and Kowalsky, IUSS Press, 2007.*
- *Seismic Design and Retrofit of Bridges, Priestley, Seible and Calvi, Wiley Interscience, 1996.*
- *Bridge Design Practice, Caltrans, 2015 (or latest edition).*

Care is required in the application of equations from references. An implementation example using appropriate resistance factors and material strengths for use with S6-14 is provided below (from Displacement-based Seismic Design of Structures)

$$\phi V_{ns} = \phi_c V_c + \phi_s (V_s + V_p)$$

$$V_c = v_c * 0.8 A_g$$

Where

$$v_c = \alpha \beta \lambda (f'_c)^{0.5}$$

$$\alpha = (3 - M/(VD)) \text{ but no less than } 1.0 \text{ nor greater than } 1.5$$

$$\beta = 0.5 + 20 \rho_l \text{ but no greater than } 1.0$$

$$\lambda = \text{factor for degradation in } V_c \text{ with increasing curvature ductility.}$$

$$= 0.25 \text{ (MPa) for curvature ductilities less than } 3$$

$$= 0.04 \text{ (MPa) for curvature ductilities greater than } 15$$

$$= \text{varies between using linear interpolation, between curvature ductilities of } 3 \text{ to } 15$$

$$= \text{For columns in biaxial bending, similar to above but varying from } 0.25 \text{ to } 0.04 \text{ for curvature ductilities between } 1 \text{ and } 15.$$

$$V_s = \pi/(2 s) \{A_v f_{ye} (D - c - c_o) \cot(\theta)\} \text{ (for round columns. For rectangular columns delete } \pi/2 \text{ term and modify } A_v \text{ as described below)}$$

Where

$$s = \text{spiral spacing}$$

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

$$f_y = \text{hoop steel nominal yield stress}$$

$$D = \text{Column diameter (out to out)}$$

$$c = \text{depth from extreme compression fibre to neutral axis under the loading considered}$$

$$c_o = \text{cover to centre of the peripheral spiral cage}$$

$$\text{Spirals or ties crossed by crack with } \cot \theta \text{ measured from vertical, using } \theta = 35^\circ \text{ for design}$$

$$V_p = 0.85 P \tan \alpha$$

Where

P = axial load from bridge weight plus plastic mechanism effects

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

- **Plastic Hinge Zones in Tall Columns:**

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

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

4.7.5.2.5 Transverse reinforcement for confinement at plastic hinge regions

Delete phi factors from all equations in this clause.

4.7.5.2.7 Splices

Add the following at the end of the third paragraph:

Welded splices will not be allowed unless consented to by the Ministry.

4.7.5.4 Column connections

Delete the second paragraph and replace with the following:

For lifeline and major route bridges in seismic performance category 3, the design of column connections, including member proportions, details, and reinforcement, shall be based on beam-column joint design methodologies as described in either:

- Seismic Design and Retrofit of Bridges, Priestley and Calvi (1996).
- Caltrans Seismic Design Criteria (latest version, currently 2012)
- ATC-49 Section 8.8.4

Joints shall be designed as capacity-protected elements as described in this Supplement. For bridges in seismic performance category 2, or for “other ‘bridges’ in seismic performance category 3, in lieu of a detailed beam-column joint design, column transverse reinforcement as specified in Clause 4.7.5.2.5 shall be continued full depth through the joint region.

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 Chapter 8 of CHBDC.

4.7.6 Piles

4.7.6.4 Seismic performance category 3

4.7.6.4.1 General

Add the following paragraph:

For bridges in seismic performance category 3 and where plastic hinging may reasonably be expected to form, concrete piles shall be designed and detailed as ductile components to ensure performance similar to concrete columns designed to Section 4.7.

4.8 Steel structures

4.8.3 Sway stability effects

Add the following:

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

4.8.4.4.5 Buckling restrained braced frames

Change “R = 5” to “R = 4” at end of the sentence for consistency with Table 4.17.

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

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 S6-14 as modified in this Supplement:

S6-14 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 S6-14 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 will be evaluated based on performance-based principles and criteria from Clause 4.4.6.3 based on 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

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 previous baseline hazard for seismic evaluation of existing bridges 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

The service and damage criteria for sites without liquefaction (site class A to E), shall be in accordance to Table 4.15, unless otherwise specified or consented to by the Ministry.

The service and damage criteria for sites with liquefaction (site class F), shall be specified by the Ministry on a project by project basis. The performance criteria as a minimum shall be "Life Safety" and "Probable Replacement".

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

Clauses 4.11.5.1 and 4.11.5.2 will not be used for seismic assessment or retrofit of Ministry bridges.

Commentary: *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 or time-history methods 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 sentences to end of Clause:

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.*

4.11.7 Minimum support length

Replace last sentence with:

Alternately, longitudinal restrainers complying with Clause 4.4.10.6 shall be provided, or structurally integral superstructure to sub-structure connections having sufficient capacity to be capacity-protected elements may be relied on.

4.11.9 Required response modification factor for force-based design approach

Delete clause.

4.11.10 Response modification factor for existing substructure elements

Response modification factors shall not be used in lieu of explicit displacement-based methods.

Delete the words “modification factors” from sub-clause (a).

Commentary: *Sub-clauses (a) and (b) as modified above remain applicable.*

4.11.11 Evaluation acceptance criteria

Delete second paragraph.

4.11.12 Bridge access

Modify sentence by deleting “... for Major-Route bridges located in Seismic performance category 3.”

Commentary: *Damage to embankments and abutments shall be evaluated.*

4.11.13 Liquefaction of foundation soils

Delete first sentence and replace with “The potential for liquefaction of the foundation soils shall be evaluated as required to determine performance.”

Delete sub-clauses (a) and (b).

4.11.14 Soil-structure interaction

Delete entire Clause and replace with the following Clause

Soil-foundation-structure interaction shall be assessed in accordance with Clause 4.6.4

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 of 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 having a 10% and a 2% in 50 year probability of exceedance.
- Description of 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.
- Define 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 [Load Patch 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 high demand vulnerable components, for the current structure, that could affect use and 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 and 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

Delete entire clause and replace with:

4.12.1 General

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

The level and type of retrofit to be implemented shall consider the existing seismic resistance of the bridge and the type of modifications to the structure and substructure that will allow the bridge to meet the performance objectives specified by the Ministry. Analytical studies shall be carried out and experimental studies may be used to determine retrofit alternatives for the bridge.

Commentary: *The performance levels, type and staging of seismic retrofit to be implemented shall consider:*

- *The seismic hazard of the bridge location.*
- *The importance of the bridge to the transportation network considering post-disaster response and recovery, and longer term local and regional economic recovery.*
- *The existing seismic resistance and resilience of the bridge, considering the bridge form and materials, and the severity and consequences of assessed seismic vulnerabilities.*
- *The expected remaining in-service life of the bridge.*
- *The nature and timing of other bridge renewal measures identified and planned.*

- *The costs and benefits of implementing seismic upgrades.*
- *The type of modifications to the structure, substructure and foundation soils that will allow the bridge to meet the performance objectives specified by the Ministry.*

Analytical studies for the structure and soils to demonstrate performance using deformations shall be carried out for the design of seismic upgrades. Experimental studies may be used to aid in the assessment and design of retrofit alternatives or works for the bridge. Material testing shall be done where appropriate to either assess the bridge performance or to design upgrading works.

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.*
- *Where appropriate, the Province will assess and target retrofit levels for existing bridges to a 2% in 50 year performance level. The Ministry's seismic upgrading program started in the 1990's targeted collapse prevention for a seismic hazard having a 10% probability of exceedance in 50 years. For critical bridges post-seismic usage by emergency vehicles was also specified. Currently, all major Lower Mainland Lifeline Bridges have been upgraded or replaced (or slated for replacement), and many other important bridges have been assessed and seismically upgraded. The remaining older, deficient bridges not yet upgraded have since expended an additional two decades of their remaining service lives. As such, the remaining economic lives of some of these older bridges will be significantly shorter than the life of a new bridge. Where economically feasible, retrofit to a 2% in 50 year level should be adopted for bridges reasonably expected to remain in service for 20 years or more. These may include large or important bridges which are expensive to replace, bridges having their economic lifespans extended through renewal measures, or other bridges designated by the Ministry. For other bridges in the retrofit program, seismic hazard levels and performance requirements used previously, general a collapse prevention or risk mitigation to a 10% hazard level, may provide appropriate levels of protection.*

- *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.*

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.

Add the following:

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 testhole/testpit 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.

5.5.7	Rigid frame and integral abutment types	2
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5.5.7 Rigid frame and integral abutment types

Add the following paragraphs:

Analysis of these structures must take account of the zone of soil/structure interaction behind the abutments, specifically the lateral soil pressure build-up and settlements that will occur in this zone as a result of thermal cycling.

Movement calculations shall consider temperature, creep, and long-term pre-stress shortening in determining potential movements at the abutment.

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.

Commentary: *Some suitable 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

Limit States Design shall be used for foundations, embankments, slopes and geotechnical systems.

6.1 Definitions

Add the following:

Embankment – earth slopes with or without a foundation unit.

6.4.1 Limit States

Serviceability Limit State SLS Combination 1, given in Table 3.1, shall be used for global (overall) stability of embankments, slopes and fills.

6.7 Geotechnical report

6.7.3 Design information

Replace the last sentence and replace with the following:

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

6.9 Geotechnical Resistance**6.9.1 General**

Add the following to this section:

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	Design based on SPT blow counts and soil sample descriptions from boreholes representative of conditions at project site.	Design based on SPT blow counts and soil sample descriptions from boreholes representative of conditions at each bridge pier and abutment.	- Design based on CPT data representative of conditions at each bridge pier and abutment. OR - 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	- 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 or CPT data, and - Test pile size and toe condition may not be the same as production piles.	- 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 shall be similar to the production pile, but toe condition shall be the same as production piles, and - Results extrapolated to other bridge piers by consideration of borehole or CPT data. OR - Design based on a single pile test with single level high capacity, sacrificial loading unit embedded in the foundation unit (O-Cells unless consented to by the Ministry) instrumented with force measurement, and - Test pile size shall be similar to the production pile, but boring method shall be the same as the production pile, and - Results extrapolated to other bridge piers by consideration of borehole or CPT data.	- Design based on a single test pile for bridge pier as per ASTM D1143, if bridge piers are separated less than 500 metres, and - Design based on two test piles for bridge pier as per ASTM D1143, if bridge piers are separated more than 500 metres, 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 size shall be similar to the production pile, but toe condition shall be the same as production piles, and - Results extrapolated to other bridge piers by consideration of borehole or CPT data. OR - Design based on one test pile with two-level high capacity, sacrificial loading unit embedded in the foundation unit (O-Cells unless consented to by the Ministry) if bridge piers are separated less than 500 m, and - Design based on two test piles with two-level high capacity, sacrificial loading unit embedded in the

			foundation unit (O-Cells unless consented to by the Ministry) if bridge piers are separated more than 500 m , and • Test pile size shall be similar to the production pile, but boring method shall be the same as the production pile, and • Results extrapolated to other bridge piers by consideration of borehole or CPT data.
Dynamic Analysis	• Wave equation analysis (WEAP unless otherwise consented to by the Ministry) performed before construction for multiple driving systems OR • Wave equation analysis (WEAP unless otherwise consented to by the Ministry) performed using pile driving blow count data from previous installations at the site.	• 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.	• 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	• 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. OR • 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 or CPT data, and • Test pile size and toe condition may not be the same as production piles.	• 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.	• 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 or CPT data to define the ground conditions, and • Have consistent driving energy delivered from the driving system with measured blow rate data for diesel hammer, or using 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	Design based on SPT blow counts and soil sample descriptions from boreholes representative of conditions at project site.	Design based on SPT blow counts and soil sample descriptions from boreholes representative of conditions at each bridge pier and abutment.	- Design based on CPT data representative of conditions at each bridge pier and abutment. OR - 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	- Design based on a single test pile for bridge pier as per ASTM D1143, 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.	- Design based on a single test pile for a bridge pier as per ASTM D1143, and - Test pile instrumented to measure toe and shaft capacity. Force applied at pile head above ground, and - Test pile size may not be the same as the production pile, but length shall be the same as production piles, and - Results extrapolated to other bridge piers by consideration of borehole or CPT data. OR - Design based on a single pile test with single level high capacity, sacrificial loading unit embedded in the foundation unit (O-Cells unless consented to by the Ministry) instrumented with force measurement, and - Test pile size and boring method should be the same as the production pile, and - Results extrapolated to other bridge piers by consideration of borehole or CPT data.	- Design based on a single test pile for bridge pier as per ASTM D3689, if bridge piers are separated less than 500 metres, and - Design based on two test piles for bridge pier as per ASTM D3689, if bridge piers are separated more than 500 metres, and - Test pile size and length same as production piles, and - Results extrapolated to other bridge piers by consideration of borehole or CPT data. OR - Design based on one test pile with two-level high capacity, sacrificial loading unit embedded in the foundation unit (O-Cells unless consented to by the Ministry) instrumented with force measurements, if bridge piers are separated less than 500 metres, and - Design based on two test piles with two-level high capacity, sacrificial loading unit embedded in the foundation unit (O-Cells unless consented to by the Ministry) instrumented with force measurements, if bridge piers are separated more than 500 metres, and - Test pile size and boring method should be the same as the production pile, and - Results extrapolated to other bridge piers by consideration of borehole or CPT data.

Note: Pile relaxation must be considered when using pile driving blow count or PDA data in certain very stiff soils or weak rock. Restrike data may 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.

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

The geotechnical resistance factors given in Table 6.2 for Global Stability of Embankments 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
(to be used in conjunction with Table 6.2)

Degree of Understanding	Low			Typical			High		
Resistance Factors for Global Stability – Permanent from S6-14	0.60			0.65			0.70		
Resistance Factors for Global Stability – Temporary from S6-14	0.70			0.75			0.80		
Consequence Factor from S6-14	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 factor of safety on global stability of embankments using the currently available computer software programs.

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:

Table 6.2c
Benchmarks for Degree of Understanding for Embankments

Degree of Understanding	Low Understanding	Typical Understanding	High Understanding
Global Stability	• 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 specification and published parameters.	• 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.	• 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.

Commentary: For low volume road bridges, modifications to the resistance factors may be considered with the consent of the Ministry.

6.10 Shallow foundations**6.10.3 Pressure distribution****6.10.3.4 Eccentricity limit**

Delete and replace with the following:

In the absence of detailed analysis, at the ultimate limit state for soil or rock, the eccentricity of the resultant of the factored loads at the ULS acting on the foundation, as shown in Figure 6.4, shall not exceed 0.30 times the dimension of the footing in the direction of eccentricity being considered for non-seismic load combinations, nor 0.40 times the dimension of the footing in the direction of eccentricity being considered for seismic load combinations.

Commentary: *This seismic requirement is in the Code Commentary. A study of some typical representative abutment and retaining wall configurations with typical bridge loading indicates that the Eccentricity Limits approach yields wall geometry requirements reasonably close to the traditional Working Stress design approach requiring a Safety Factor of 2.0 against overturning.*

6.15 Mechanically stabilized earth (MSE) structures**6.15.2 Design****6.15.2.1 General**

Add the following to this section:

The requirements of the AASHTO LRFD Bridge Design Specifications (7th Edition, 2014) including interim revisions and FHWA-NHI-10-024 and -025 shall be used for items not covered in this Supplement or S6-14.

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 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.

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

Two stage MSE walls shall only be used where consented to by the Ministry. The designer shall liaise with MSE wall suppliers 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.

a. Mechanically Stabilized Earth (MSE) Walls at Bridge Abutments and the associated Abutment Wing Walls

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.

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

Any portion of an MSE wall within a horizontal distance away from an abutment footing or pile cap equal to the height of the abutment wall shall also be considered as an abutment wall.

The minimum soil reinforcement length for walls shall be 70% of the distance from the top of the leveling pad to the bridge road surface. The reinforcement length shall be uniform throughout the entire height of the wall.

Geotechnical design, including global stability and subsurface liquefaction may require longer reinforcement than specified above.

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.

Uneven reinforcing lengths may be used when intact rock must be removed to accommodate the soil reinforcing.

MSE walls with wire mesh facing, dry cast concrete block facing, or rock stack facing shall only be used with consent from 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 walls systems and add these requirements to the Plans.

Add the following Clause:

6.18

Lightweight fills

All lightweight fills shall be adequately protected in terms of wheel loads, ground water, road salts, weather and fire resistance, 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 Foundation 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 fill to the 200-year flood level shall be addressed in the design of lightweight fills, regardless of any flood protection provided for the area in which the 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:

6.19

Retaining Walls

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

Design issues not addressed by S6-14 shall meet the requirements of AASHTO LRFD Bridge Design Specifications (7th Edition, 2014) including interim revisions.

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.

Walls with steel anchors, tie-backs, MSE soil reinforcing and/or soil nails, shall include additional full length anchors, tie-backs, soil reinforcing and/or soil nails installed in the walls to allow for future extraction for long term inspection and testing. The number of additional elements provided for each wall shall be equal to 2% of the number required by design but not less than 2 additional elements per wall.

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7.1**Scope**

Buried structures with span smaller than or equal to 3 m may also be designed to S6-14 Section 7, but the designer shall pay due regard to empirical methods and solutions that have a proven record of success for small diameter culverts.

Commentary: *The CHBDC Commentary (C7.1 Scope and C7.6 Soil-metal structures) indicates that the provisions of Section 7 apply only to buried structures with span (D_n) greater than 3 m, but the CHBDC provides only very minimal design guidance for smaller structures.*

For buried structures, consideration should be given to increasing the size and durability of the structure and/or providing additional measures 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*
- *Hydrotechnical issues*

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);
- 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.

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

7.5

Structural design

7.5.2

Load factors

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.

7.5.5 Seismic requirements**7.5.5.4 Seismic design of concrete structures**

Delete and replace with the following:

For concrete buried structures, the effects of earthquake loading shall be computed in accordance with Clauses 7.8.4.1 and 7.8.4.4 (as modified herein).

Commentary: *Horizontal earthquake loads should be considered for large span buried structures.*

7.6 Soil-metal structures**7.6.2 Structural materials****7.6.2.1 Structural metal plate**

The use of aluminum plates and components must satisfy the minimum protective measures requirements of S6-14 Clause 2.4.

7.6.3 Design criteria**7.6.3.1.1 General**

Delete all and replace with the following:

The thrust, T_f , in the conduit wall due to factored live loads and dead loads shall be calculated for ULS load combination 1 of Table 3.1, according to the following equation:

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

The dynamic load allowance, DLA , is obtained from Clause 3.8.4.5.2.

The dead and live load thrusts, T_D and T_L , respectively, shall be obtained as follows;

- a) For soil-metal structures with a span of less than or equal to 10 m, T_D and T_L shall be calculated in accordance with Clauses 7.6.3.1.2 and 7.6.3.1.3, respectively;
- b) For soil-metal structures with a span of more than 10 m, T_D and T_L shall be computed using a finite difference, or finite element, soil-structure interaction analysis method. The thrust expressions in Clauses 7.6.3.1.2 and 7.6.3.1.3, respectively, shall be used as an

additional check to clarify the results of the finite difference, or finite element, method;

- c) Designers of deeply buried soil-metal structures may use the S6-14 methodology or, if consented to by the Ministry, may use an alternate finite difference or finite element soil-structure interaction analysis method to determine the dead and live load thrusts.

Commentary: S6-14 does not place any limitations on the applicability of Section 7 for soil-metal structures with large spans, or for those deeply buried. Recent load rating studies indicate that the S6-14 design formulae may not be conservative for all large span soil-metal structures. Conversely, the same load rating studies show that the S6-14 design formulae for deeply buried, soil-metal structures to be overly conservative. S6-14 does not place an upper limit on the applicability of Section 7 for deeply buried soil-metal structures.

7.6.3.1.2 Dead loads

- (d) "H" is measured vertically from crown of structure to finished grade, reference Figure 7.3.

Commentary: The depth of cover or height of overfill, "H", for the various configurations of single and double corrugation soil-metal and box structures is shown on Figure 7.3.

7.6.3.1.3 Live loads

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

(c)(i) within the span length, position as many axles of the BCL-625 Truck or Trucks (and/or Special Truck if specified) at the road surface above the conduit as would give the maximum total load;

7.6.3.4 Connection strength

Commentary: Designers are advised that values of unfactored seam strength for bolted steel plates, S_s , for standard corrugation profile with bolted connections are shown in Commentary Figure C7.4.

There is currently no reference to the values of unfactored seam strength for bolted aluminum plates.

7.6.4 Additional design requirements**7.6.4.1 Minimum depth of cover**

Commentary: *Notwithstanding conduit wall design by any other approved method, it is recommended that minimum cover should conform to the criteria in this Clause.*

7.6.4.3 Durability

The design life for Soil-Metal Structures, based on corrosion allowance calculations, shall be 100 years.

Design shall be in accordance with the Corrugated Steel Pipe Institute (CSPI) Technical Bulletins:

- Performance Guideline for Corrugated Steel Pipe Culverts (300mm to 3,600mm Diameter) – August 2013
- Performance Guideline for Buried Steel Structures - February 2012

Commentary: *The S6-14 Section 7 Commentary suggests that an expected design life of up to 100 years is achievable, and presents sample values for corrosion loss.*

The specified coating thickness for soil-metal buried structures shall be “total both sides”, per ASTM A444 and CSA G401-M. The minimum galvanic coating thickness for all soil-metal buried structures shall be 610g/m² total both sides of plate. For culverts subject to heavy abrasion or corrosive products, additional protection shall be provided. Options including concrete liners, thicker galvanic coating, polymer laminated coating and asphalt coating shall be considered. The effects of corrosive run-off or abrasive stream flows shall be accounted for in the design. Abrasive stream flows should be avoided wherever possible by appropriate hydraulic mitigation.

Commentary: *SS 320 stipulates galvanized steel sheet to ASTM A444 or CSA G401-M, both of which refer to coating thickness “total both sides”, which is standard industry practice. Some culverts are more vulnerable to streambed abrasion than corrosion, per se. Some installations may be vulnerable to corrosive run-off (salts or fertilizers).*

For non-saturated soil conditions, the “AASHTO corrosion loss model” for zinc-coated steel structures, as presented in S6-14 Commentary Table C7.2, shall be used. The designer shall consider whether the culvert’s structural backfill might become saturated in high groundwater conditions.

For saturated soil conditions, a recognized corrosion loss model, which relates soil/water “pH” values to corrosion losses, shall be used.

Portions of culverts that have both the interior and exterior faces exposed to soil and/or water (e.g. stream inside culvert) shall include corrosion loss allowances for both faces.

Commentary: The “AASHTO” method is the industry standard for non-saturated conditions throughout North America. The S6-14 Section 7 Commentary presents two sets of values for Non-Saturated Loss Rates (i.e. UBC 1995 & AASHTO 1993) in Table C7.2, and a single set of values for Saturated Loss Rates (i.e. UBC 1995) in Table C7.3. Practical experience suggests that some of these corrosion loss results are too conservative in typical applications.

7.6.5 Construction

7.6.5.6 Structural backfill

7.6.5.6.2 Material for structural backfill

Structural backfill shall meet the requirements for Bridge End Fill accordance with SS 201.40 unless otherwise consented to by the Ministry.

7.6.6 Special features

Where stiffener ribs are used to bolster structure strength, the combined plate/rib section properties shall be calculated in a cumulative (not composite) manner.

Commentary: AASHTO allows section properties for composite SPCSP plate/rib sections to be calculated on the basis of “integral action”; this terminology is not explicit, but may imply composite action. S6-14 requires section properties for composite SPCSP plate/rib sections to be calculated in a cumulative (not composite) manner, which is conservative.

7.7 Metal box structures

The additional geometric limitations provided in AASHTO Standard Specifications for Highway Bridges (2014) Tables 12.9.4.1-1 and 12.9.4.1-2 shall be applied; e.g., maximum radius at crown and minimum radius at haunch.

Unless consented to by the Ministry, soil-structure interaction shall not be considered for metal box structures larger than 8.0 m span, or 3.2 m rise.

7.7.3 Design criteria**7.7.3.1.3 Live loads**

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

where A_L is the weight of a single axle of the BCL-625 Truck (or Special Truck if specified) for $D_h < 3.6$ m, or the combined weight of the two closely spaced axles of the BCL-625 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 7.6

7.7.3.2 Design criteria for connections

Commentary: Designers are advised that values of unfactored seam strength of bolted steel plates, S_s , for standard corrugation profile with bolted connections are shown in S6-06 Commentary Figure C7.4.

Values of unfactored seam flexural strength, for steel or aluminum plates, are not presented in the S6-14, or in the AASHTO Standard Specifications for Highway Bridges (2014).

7.7.4 Additional design considerations**7.7.4.2 Durability**

The design life and durability requirements for Metal box structures shall be the same as stipulated for soil-metal structures in Supplement Clause 7.6.4.3 above.

7.7.5 Construction**7.7.5.1.2 Material for structural backfill**

Structural backfill shall meet the requirements for Bridge End Fill accordance with SS 201.40 unless otherwise consented to by the Ministry.

7.8 Reinforced concrete buried structures

Commentary: It is recommended that engineering judgment be used, on a case-by case basis, to determine whether Section 7.8 or Section 8 (Concrete Structures) is more applicable for large reinforced concrete buried structures.

The analysis and design provisions of Section 7.8 appear to focus on medium sized precast concrete pipe or box structures. These provisions may not be appropriate for large reinforced concrete buried structures (e.g. tunnels for transit systems or highway underpasses, typically over 6m in span). For example, the simplistic vertical and lateral earth pressure distributions stipulated by Clauses 7.8.5.3.2 and 7.8.5.3.3 may not be appropriate for large structures.

7.8.1 Standards for structural components

For top slabs of concrete culverts which are within 600 mm of the roadway surface, shall be treated with a waterproofing membrane.

7.8.3 Installation criteria**7.8.3.1 Backfill soils**

Backfill material shall meet the requirements for Bridge End Fill in accordance with SS 201.40.

7.8.4 Loads and load combinations**7.8.4.4 Earthquake loads**

For concrete buried structures with span (D_h) greater than 3m, the effects of earthquake loading shall be computed in accordance with Section 4, Seismic design. Seismic lateral soil pressures on each side of the buried structure shall be determined by a recognized analysis method, such as the Mononobe-Okabe expressions or Woods' procedure. Alternately, the effects of seismic soil loading may be computed using a finite difference, or finite element, soil-structure interaction analysis method. Regardless of the analysis method used, the structure shall be designed for the maximum seismic soil loading on one side, and the corresponding minimum seismic soil loading on the other side. Where appropriate, the seismic design shall include the effects from hydrodynamic mass. The potential for, and effects of, seismic soil liquefaction shall also be investigated.

Commentary: Both horizontal and vertical earthquake loads are to be addressed.

7.8.5.2.2 Earth Load

Commentary: For further information refer to AASHTO Section 12.10.2.1

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8.4 Materials**8.4.1 Concrete****8.4.1.2 Concrete strength**

Insert after first sentence:

The specified concrete strength for Ministry standard prestressed I girders and box girders shall not exceed 55 MPa at 28 days or 37.5 MPa at release unless otherwise consented to by the Ministry.

8.4.2 Reinforcing bars and deformed wire**8.4.2.1 Reinforcing bars**

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.

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

8.4.2.1.1 Specification

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

Low carbon/chromium reinforcing steel shall meet the requirements of ASTM A1035 - Types CS, CM and CL. The minimum yield strength based on the 0.2% offset method shall be equal to 690 MPa.

Other reinforcing bar types are permitted for use where 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 metric bars and a conversion table for allowable substitutions with imperial 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 metric and imperial sizes, while low carbon/chromium reinforcing steel meeting the requirements of ASTM A1035 is currently only available in imperial sizes. Currently, ASTM A1035 Types CS, CM and CL compliant materials are produced for and sold by MMFX Steel for North America.*

8.4.2.1.3 Yield strength

Grade 400W reinforcing bars shall be specified for flexural reinforcement in plastic hinge regions.

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

Low carbon/chromium reinforcing steel may have a yield strength larger than 420 MPa and a stress-strain curve differing from Grade 400 reinforcing steel. These differences shall be taken into account in the design, in particular with respect to assumption for moment redistribution or seismic design considerations.

8.6 Design Considerations**8.6.1 General**

Connection details for precast concrete abutment components shall be designed to minimize the potential for cracking in the concrete at the connections.

Commentary: *The ministry has observed concrete cracking near welded connections at precast concrete abutment 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 designer is cautioned that the losses tabulated in Table C8.2 may be unconservative for prestressed girders where the span to depth ratio pushes the capacity limit of the section.*

8.8 Flexure and axial loads**8.8.1 General**

Section 4 shall apply for seismic design and detailing.

Commentary: *As per Clause 8.17 of S6-14, seismic design and detailing shall meet the requirements of Section 4 (of S6-14).*

8.8.4.5 Maximum reinforcement

The requirement of this clause may be waived by the design engineer provided it is established to the satisfaction of the Ministry that the consequences of reinforcement not yielding are acceptable.

8.9 Shear and torsion**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. Acceptable approaches can be found in the latest CSA A23.3 Standard or AASHTO LRFD Bridge Design Specifications.*

8.11 Durability**8.11.2 Protective measures****8.11.2.1 Concrete Quality**

Supply of concrete including methods and testing shall comply with the quality requirements of SS 211, SS 413, SS 415, and SS 933, and DBSS 211, DBSS 413, DBSS 415, and DBSS 933. Where there are any discrepancies between the SS or DBSS and CSA S6-14, the SS or DBSS will take precedence.

8.11.2.1.1 General

Delete the entire clause and replace with the following:

For the structural elements listed below, concrete mix design parameters shall be determined in consultation with the Ministry and shall comply with the requirements given in the following table 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 cementing materials ratio shall be 0.45 unless otherwise consented to by the Ministry.

Table 8.4 is deleted and replaced with the following:

Table 8.4
Maximum water to cementing materials ratio

(See Clause 8.11.2.1.1.)

Classification	Minimum Compressive Strength at 28 days (MPa)	Nominal Maximum Size of Coarse Aggregate (mm)	Air Content (%)	Slump (mm)	Maximum W/C Ratio by Mass
Deck Concrete: Deck Slab, Approach Slab, Parapet and Median Barrier					
• Standard ⁽¹⁾⁽²⁾⁽³⁾	35	28 ⁽⁴⁾	5 ± 1	50 ± 20	0.38
• With Silica Fume	35	28 ⁽⁴⁾	6 ± 1	80 ± 20 ⁽⁵⁾	0.38
• With Class F or C1 Flyash ⁽¹⁾⁽⁶⁾	35	28 ⁽⁴⁾	6 ± 1	50 ± 20	0.38
Substructure Concrete: Piers, Abutments, Retaining Walls, Footings, Pipe Pile In-fills, Working Floors					
• Standard ⁽¹⁾⁽⁶⁾	30	28	5 ± 1	50 ± 20	0.45
Keyways between Box Stringers:					
• Standard ⁽¹⁾⁽²⁾⁽⁷⁾	35	14 ⁽⁸⁾	5 ± 1	20 ± 10	0.38
Concrete Slope Pavement:					
• Standard ⁽⁴⁾	30	20	5 ± 1	30 ± 20	0.45
Deck Overlay Concrete:					
1. High Density ⁽¹⁾	35	20 ⁽⁹⁾	5 ± 1	30 ± 20	0.38
2. Silica Fume Modified	45	14 ⁽⁸⁾	6 ± 1	60 ± 20 ⁽⁵⁾	0.38

Notes:

(1) Superplasticizers or high range water reducers shall not be used.

(2) No supplemental cementing materials shall be used in this concrete (i.e. silica fume, fly ash, etc.).

(3) Cement shall be Type GU and total cement content shall not exceed 380 kg/m³.

(4) The maximum proportion of aggregate passing the 5 mm screen shall be 35% of the total mass of aggregate.

(5) Silica fume application rates shall be 8% maximum by mass of Portland Cement. Slump specification is based on superplasticized concrete.

(6) The addition of fly ash shall not exceed 15% by mass of Portland Cement.

(7) Cement shall be Type GU and total cement content shall not be less than 400 kg/m³.

(8) The maximum proportion of aggregate passing the 5 mm screen shall be 42% of the total mass of aggregate.

(9) The maximum proportion of aggregate passing the 5 mm screen shall be 38% of the total mass of aggregate.

The gradation of the 28 mm nominal size aggregate shall conform to Table 211-B in SS 211 or DBSS 211 unless noted otherwise in this clause.

Semi-lightweight concrete shall not be used in any bridge component.

Commentary: *Superplasticizers may be considered, subject to the consent of 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.*

8.11.2.1.3 Concrete placement

Delete the entire clause and replace with 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: *A deck casting sequence is required in order to minimize the potential for deck cracking due to improper concrete placement sequencing.*

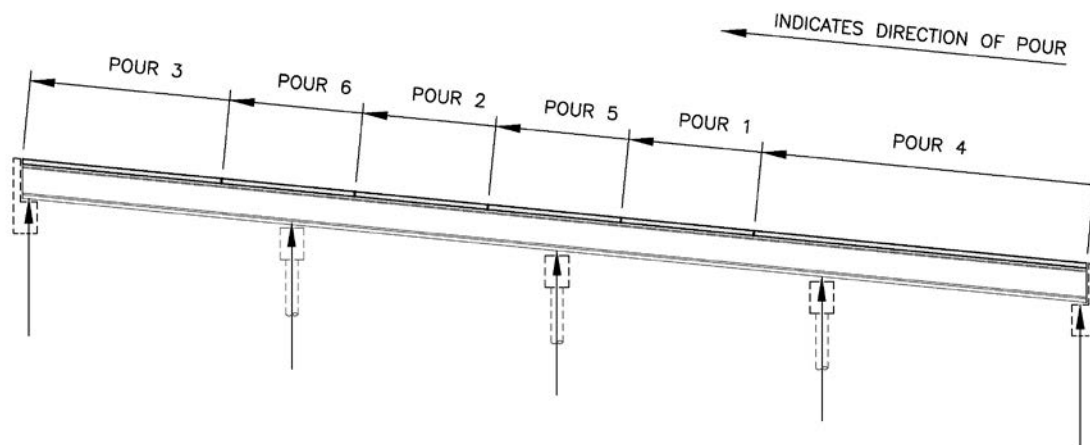
Several factors limit the quantity of concrete which can be placed in one continuous operation. Special consideration shall be given if the continuous placement exceeds a volume of 200 cubic metres or if the bridge deck exceeds four lanes in width.

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

The following is 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.

Figure C8.11.2.1.3
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.

8.11.2.1.6 Slip-form construction

Extruded concrete barriers shall not be used.

8.11.2.1.7 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.

Surface finishes shall be in accordance with Table 8.11.2.1.7 and shall be specified in the Special Provisions.

**Table 8.11.2.1.7
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⁾	413.31.02.05
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 and DBSS 419.33.

(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 give consideration for future removal of graffiti. Such consideration may include the application of anti-graffiti coatings

8.11.2.2 Concrete cover and tolerances

The soffits of deck slabs cantilevered from the exterior girder shall be considered under Environmental exposure class, De-icing chemicals; while the soffits of deck slabs intermediate to the exterior girders may be considered under Environmental exposure class, No de-icing chemicals as detailed in Table 8.5.

All references to “minimum cover” in S6-14 shall be replaced with “minimum specified cover”.

The designer shall check the cover and tolerance in S6-14 and the cover and tolerances in SS 412 and DBSS 412 and on the Ministry standard drawings and adjust the project specifications as required to meet the minimum cover allowed in S6-14, except that the values given in Table 8.5 below shall govern. Minimum cover is hereby defined as the specified cover minus the tolerance.

Table 8.5 in S6-14 shall be amended as follows:

Table 8.5
Minimum concrete covers and tolerances
(See Clause 8.11.2.2.)

			Concrete Covers and Tolerances	
Environmental exposure	Component	Reinforcement/ steel ducts	Cast-in-place concrete (mm)	Precast concrete (mm)
All	(3) Top surfaces of Structural components Add: Bridge Decks and Approach Slabs	Reinforcing Steel Pretensioning strands	⁽¹⁾ +6 -0 —	⁽¹⁾ +6 -0 100 ±5
All	(10) Precast T, I and box girders Add: Ministry Standard Precast Box Girders	Reinforcing steel - Top surfaces - Vertical surfaces - Soffits - Inside surfaces Pretensioning strands - Top surfaces - Vertical surfaces - Soffits - Inside surfaces	— — — — — — — — — —	⁽¹⁾ +10 -5 40 +10 -5 30 +10 -5 30 +10 -5 200 ±5 50 ±5 40 ±5 35 ±5
	Add: Ministry Standard Precast I-Beams	Reinforcing steel - Top surfaces - Vertical surfaces - Soffits Pretensioning strands - Top surfaces - Vertical surfaces - Soffits	— — — — — — — —	30 +10 -5 30 +10 -5 30 +10 -5 100 ±5 40 ±5 40 ±5

Notes:

(1) for specified cover and reinforcing steel type, see Table 8.11.2.3.2 in Section 8.11.2.3.2

Delete Note ‡ under Table 8.5.

Commentary: *An additional 10 mm of concrete cover shall not be provided for concrete decks.*

Cover requirements are governed by the following principles:

1. *Minimum cover, i.e. the specified cover minus the tolerance, shall not be less than the cover specified in S6-14 minus the tolerance given in S6-14.*
2. *The use of cover and tolerance requirements of Table 8.5 of S6-14 is appropriate except for the changes as shown in Table 8.5 above.*
3. *Cover and tolerances in SS 412 and DBSS 412 are acceptable where they meet the minimum cover in item 1 above even though the specified cover may be less than in S6-14.*

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 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. As an example, consideration shall be given to the cover requirements on mechanical splices.

8.11.2.3 Corrosion protection for reinforcement, ducts and metallic components

8.11.2.3.1 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 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 covered.

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.

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

- increasing the minimum bend diameter to meet the requirements for epoxy coated steel as provided in SS Table 412-B
 - 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. 15 M bars would typically be stress relieved for 1 hour at 620 degrees Celsius.)

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 shall take into consideration the greater bend diameter for the galvanized reinforcing steel.

8.11.2.3.2 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 8.11.2.3.2 below.

Table 8.11.2.3.2
Corrosion protection for top mat reinforcing steel for bridge
decks, parapets, curbs and approach slabs

Top mat rebar type⁽¹⁾	Minimum deck thickness (mm)	Specified Top Cover (mm)	Where used⁽²⁾⁽³⁾
Stainless steel plus deck membrane	225	50	SCR – main roads
Low carbon/chromium to ASTM A1035 Type CS (or stainless steel also allowed) plus deck membrane	225	50	SCR – other roads
Stainless steel	225	60	SIR/NR – main roads
Low carbon/chromium to ASTM A1035 Type CS (or stainless steel also allowed)	225	70	SIR/NR – other roads
Black, epoxy coated, galvanized, ASTM A1035 Type CL (or other as consented to by the Ministry)	225	70	SCR/SIR/NR – gravel roads

Notes:

(1) Rebar type in accordance with Clause 8.4.2.1.1.

(2) SCR: South Coast Region, SIR: Southern Interior Region, NR: Northern Region.

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

Other roads = includes all other structures.

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

For cable supported structures where the deck system is under compression, then 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, plain steel reinforcing bars may be used. For pedestrian bridges of 3 m and wider, corrosion protection of deck steel shall be in accordance with “SIR/NR - other roads” in Table 8.11.2.3.3.

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:*

http://www.th.gov.bc.ca/publications/planning/Provincial%20Highways/Functional_Class_Map.pdf.

Black steel is generally used for bottom mat reinforcing.

8.11.2.3.3 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 8.11.2.3.3. 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 8.11.2.3.3
Corrosion protection for components subject to de-icing chemicals

Corrosion resistant material type⁽¹⁾	Where used ⁽²⁾⁽³⁾
Stainless steel	SCR /SIR/NR – main roads
Low carbon/chromium ASTM A1035 Type CS or CM (or stainless steel also allowed)	SCR /SIR/NR – other roads
Black, epoxy coated, galvanized, ASTM A1035 Type CL (or other as consented to by the Ministry)	SCR/SIR/NR – gravel roads

(1) Rebar type in accordance with Clause 8.4.2.1.1.

(2) SCR: South Coast Region, SIR: Southern Interior Region, NR: Northern Region.

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

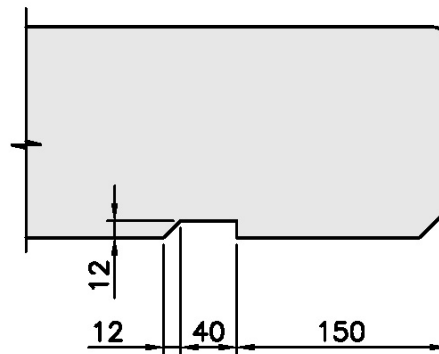
Other roads = includes all other structures.

Gravel roads = gravel roads and roads with an AADT 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 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.

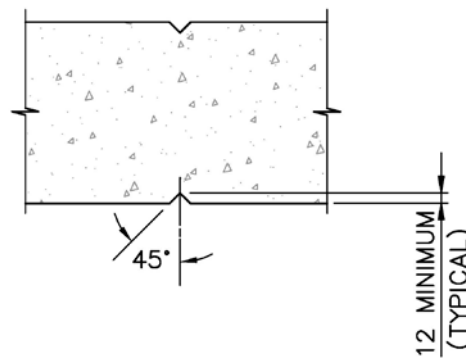
Buried concrete structures with a soil cover of 1000 mm or less shall receive a hot rubberized asphalt membrane system for waterproofing. Positive drainage shall be provided on the top surfaces of buried structures to avoid ponding of water.

Bridges located in the Southern Interior Region and the Northern Region 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**

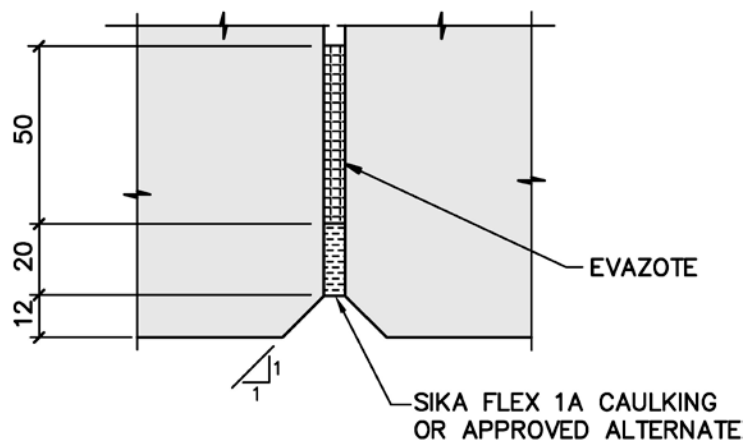
Control joints shall extend around the entire perimeter of the traffic barrier and be evenly spaced throughout the full length of the barrier with spacing not exceeding 3 m as shown below.

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



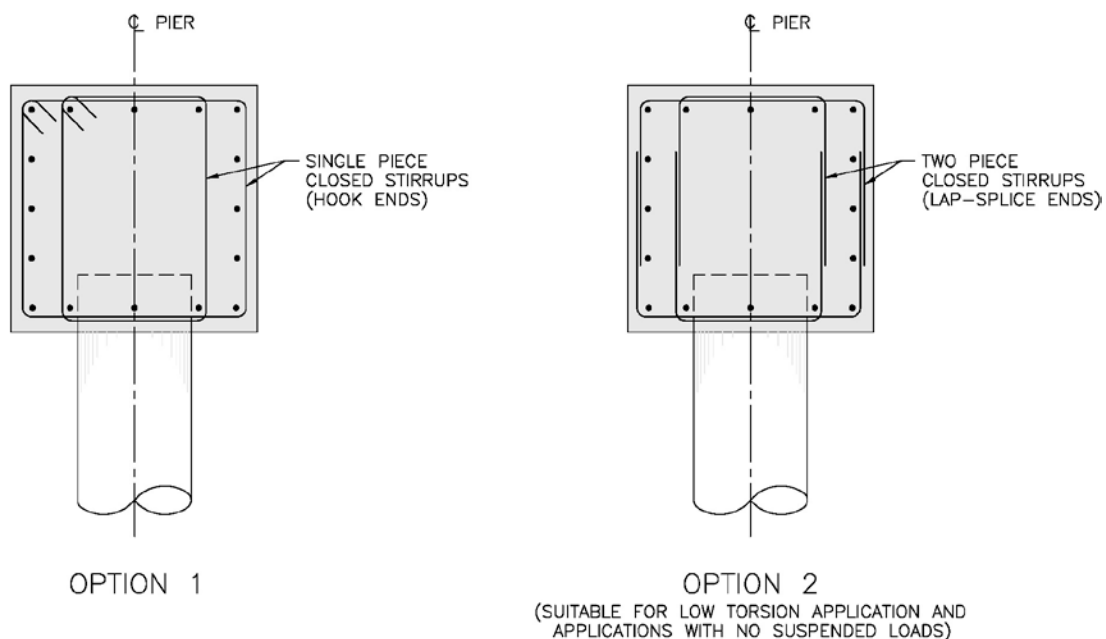
8.13 Deformation**8.13.3 Deflections and rotations****8.13.3.3 Total deflection and rotation**

Commentary: The Commentary to S6 (S6.1-14) 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 (2008)) 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**

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

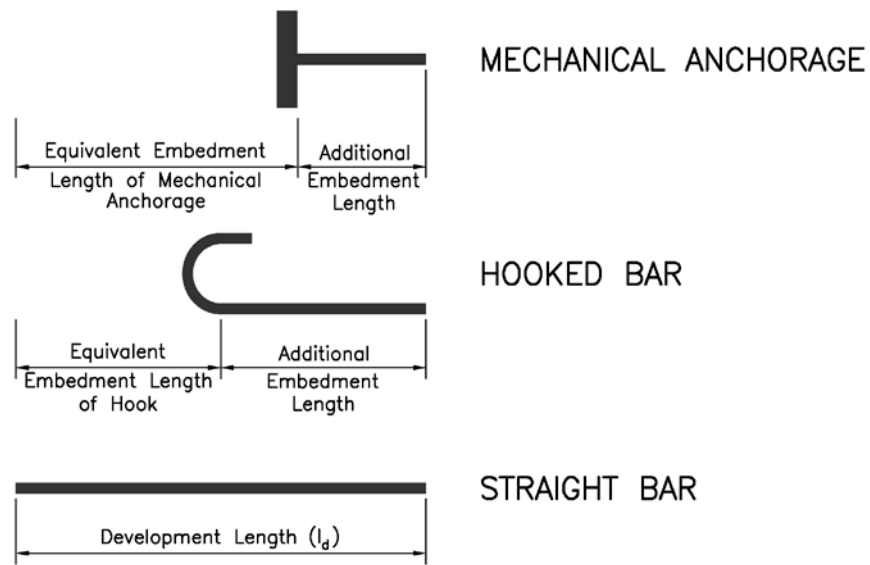
Development and splices

8.15.6

Combination development length

Commentary: Figure 8.15.6 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 8.15.6
Combination development length



8.15.9 Splicing of reinforcement

8.15.9.1 Lap splices

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.

8.15.9.2 Welded slices

Delete clause and replace with 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**

Dowel holes for Ministry standard prestressed concrete box stringers shall be detailed as shown in the Ministry Standards and Procedures Manual Volume 3, Ministry Standard Drawings 2978-1 to 2978-24 (latest revision) 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

Bridge deck heating systems are not permitted.

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

8.18.2 Minimum slab thickness

Delete the last sentence and replace with the following:

The slab thickness shall not be less than 225 mm.

8.18.3 Allowance for wear

Delete this clause.

8.18.4 Empirical design method**8.18.4.4 Full-depth precast panels**

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:

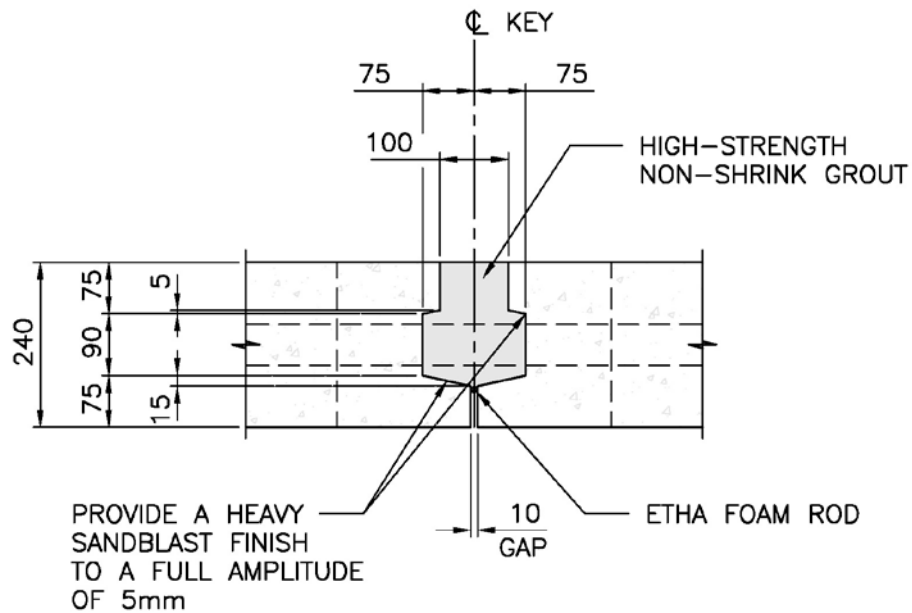
Regardless if the empirical design method or flexural design method is chosen by the engineer, design of full-depth precast panels shall satisfy the following conditions in addition to those of Clause 8.18.4.1 and, as applicable, Clause 8.18.4.2:

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 8.18.4.4(a) details the requirements for minimum shear key size.

Figure 8.18.4.4(a)
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 8.18.4.4(b) and 8.18.4.4(c) give examples for reinforced shear keys.

Figure 8.18.4.4(b)
Full depth precast panel with reinforced shear key over deck support member

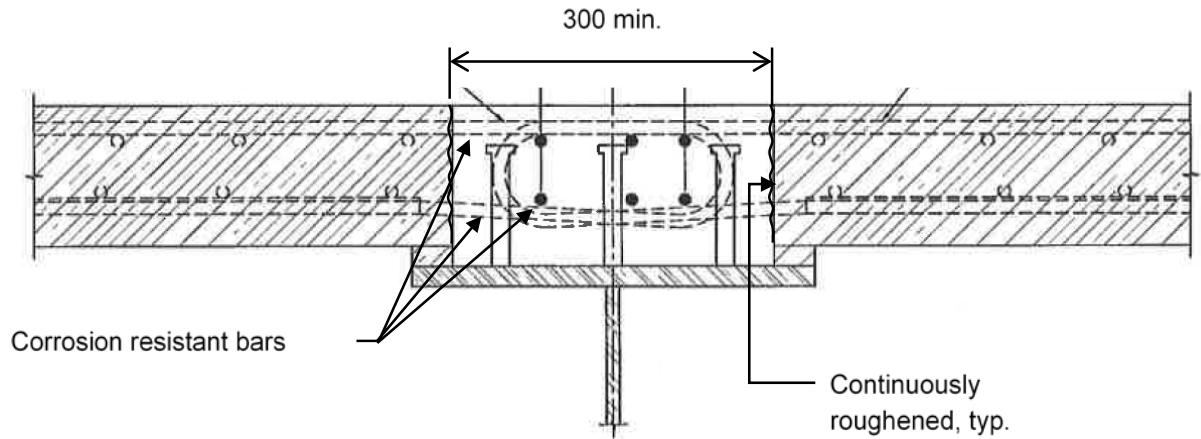
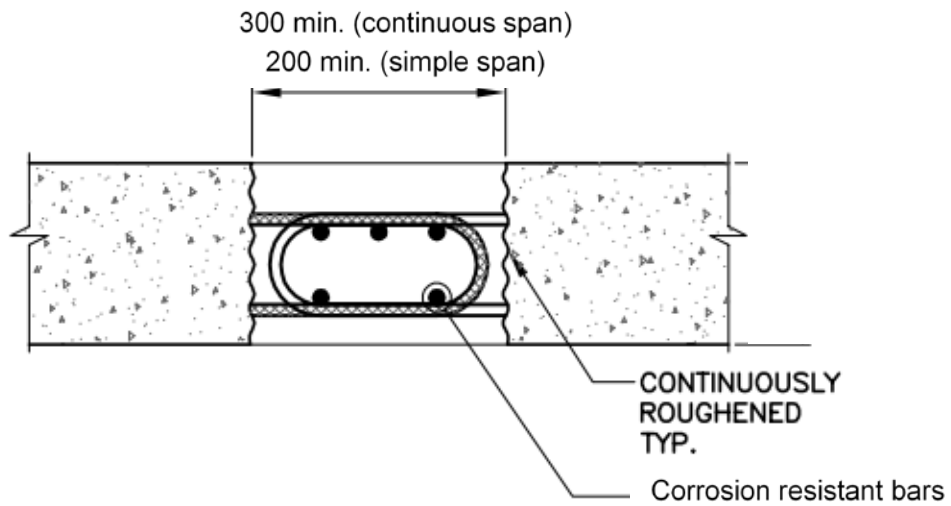


Figure 8.18.4.4(c)
Full depth precast panel with reinforced suspended shear key



Add the following additional items:

- (h) a minimum specified gap of 25 mm shall be provided under the panels above the supporting beams, including any splice plates.
- (i) the deck slab comprised of full-depth precast panels shall be fully composite with the supporting beams.
- (j) cast-in-place concrete parapets shall be used for the bridge barriers on numbered routes unless consented to by the Ministry. The parapets shall be continuous across the transverse joints except in the negative moment regions of the supporting beams. The parapets shall be placed after the longitudinal post-tensioning is complete 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.
- (l) Stud connectors shall be in accordance with Section 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 (UHPFRC, also abbreviated 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:

Graybeal, B., (2014) Design and Construction of Field-Cast UHPC Connections, FHWA Publication No. FHWA-HRT-14-084, Turner-Fairbank Research Center, McLean, VA, October.

Graybeal, B., Yuan J. (2014). Bond Behavior of Reinforcing Steel in Ultra-High Performance Concrete [TechBrief], FHWA Publication No. FHWA-HRT-14-089, Turner-Fairbank Research Center, McLean, VA, November.

Confinement of stud clusters may be required to obtain the required shear connector strength.

8.18.5 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

Ministry standard prestressed concrete box girders with a concrete overlay wearing surface shall be designed as non-composite. 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.

Non-standard composite prestressed concrete box girders shall achieve composite action through the use of mechanical anchorage between the box girder and the composite topping.

8.19.3 Shear

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)

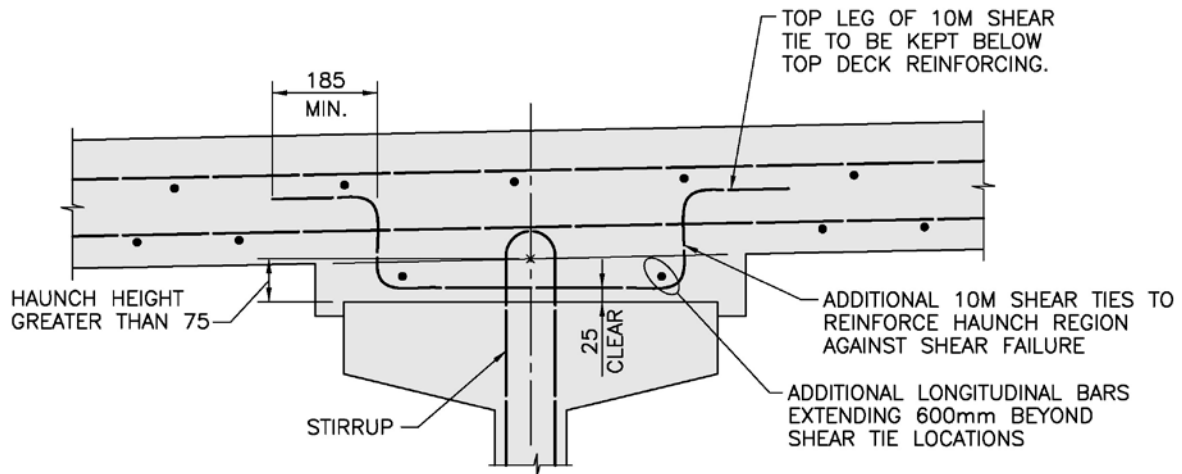
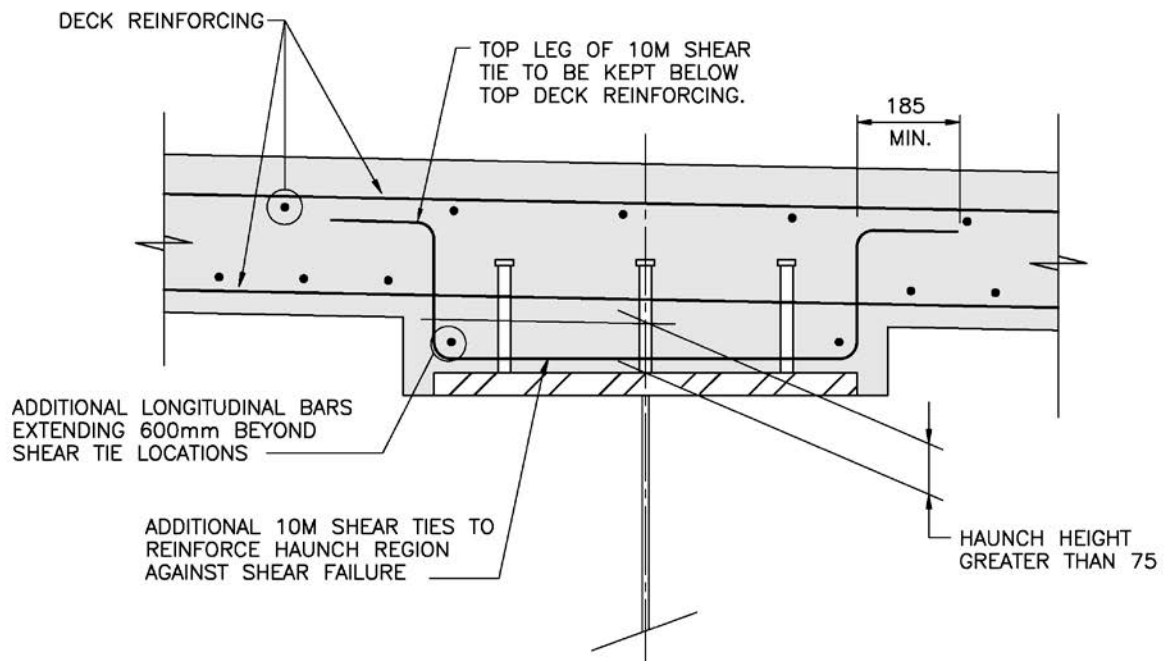


Figure 8.19.3 (b)
Additional reinforcement for haunches over 75 mm high
(conceptual)



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**

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°.

8.20.3**Flange Thickness for T and Box Girders****8.20.3.2****Bottom Flange**

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 S6-14 is not considered to be warranted.

8.20.6**Post-Tensioning Tendons**

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**

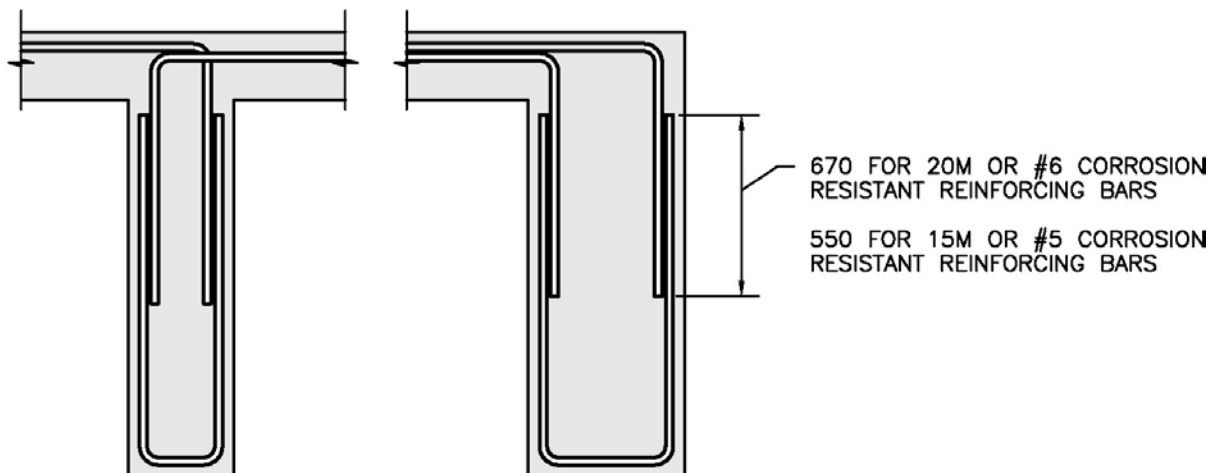
Delete clause and replace with the following:

Concrete diaphragms shall be provided at abutments and piers to support the deck and transfer loads to the supports. Abutment, pier and intermediate

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.'

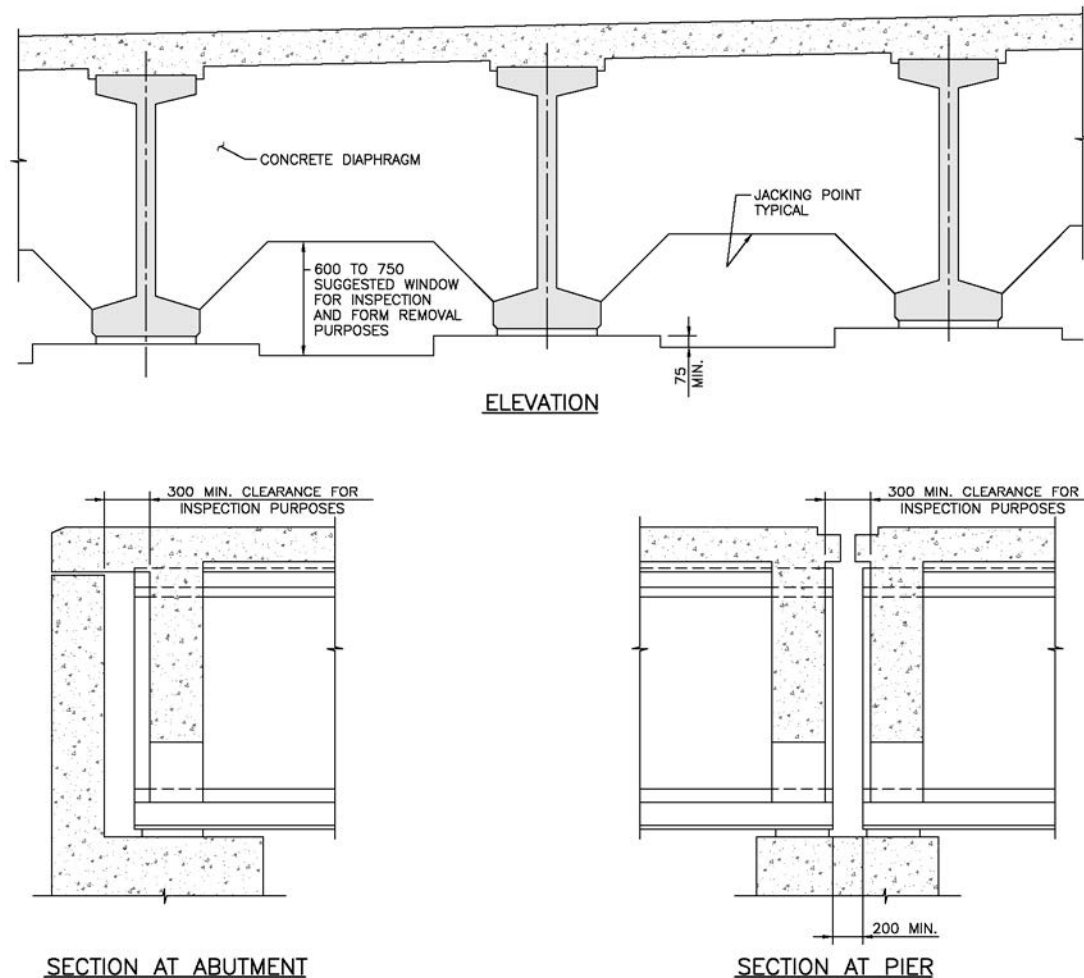
A typical tie arrangement for intermediate and end diaphragms is shown in Figure 8.20.7.1 below.

Figure 8.20.7.1
Typical diaphragm tie arrangement



Abutment and pier diaphragms shall be designed to transfer loads to the supports and to facilitate future jacking. Diaphragms shall be detailed to provide access for maintenance inspection, as generally outlined in Figure 8.20.7.2 below.

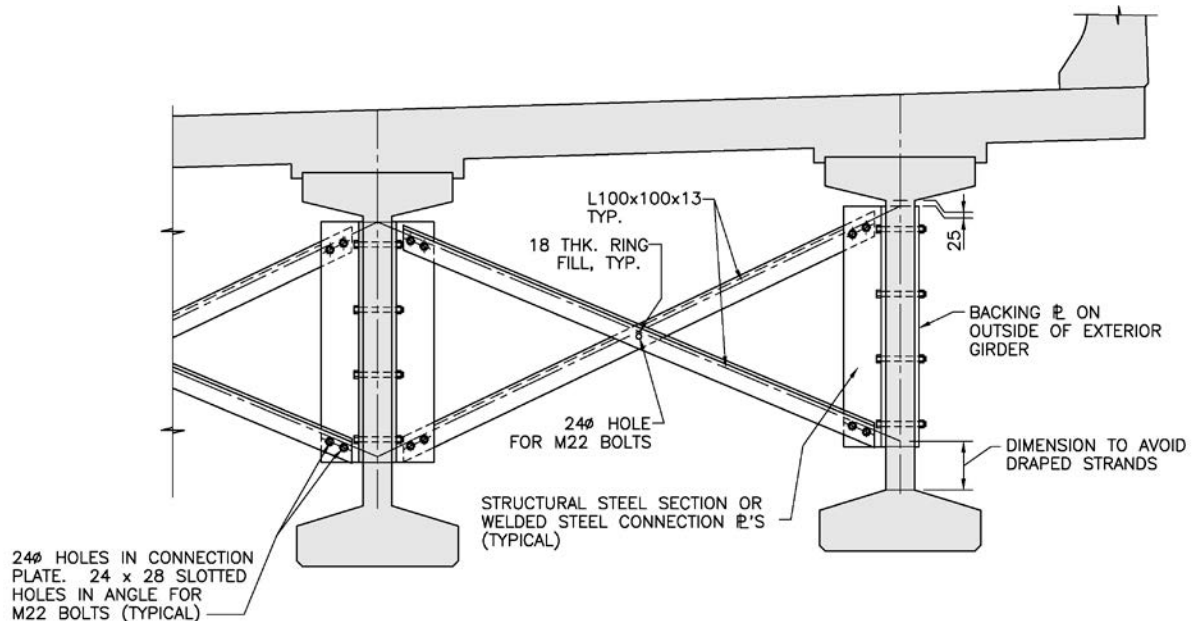
Figure 8.20.7.2
Typical concrete diaphragm arrangement



The hole size for abutment and pier diaphragm reinforcing which passes through the ends of prestressed girders shall be 2.5 times the bar diameter.

Unless specifically 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

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 S6-14.

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 reinforced concrete overlay is applied as a wearing course topping on twin or single cell box beams. 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.

Mechanical anchorage is required between precast beams and a composite reinforced concrete deck slab to achieve composite action.

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)
Double cell box beam composite deck slab connection detail

CORROSION RESISTANT
REINFORCING BARS TO
MATCH TOP MAT DECK
REBAR TYPE

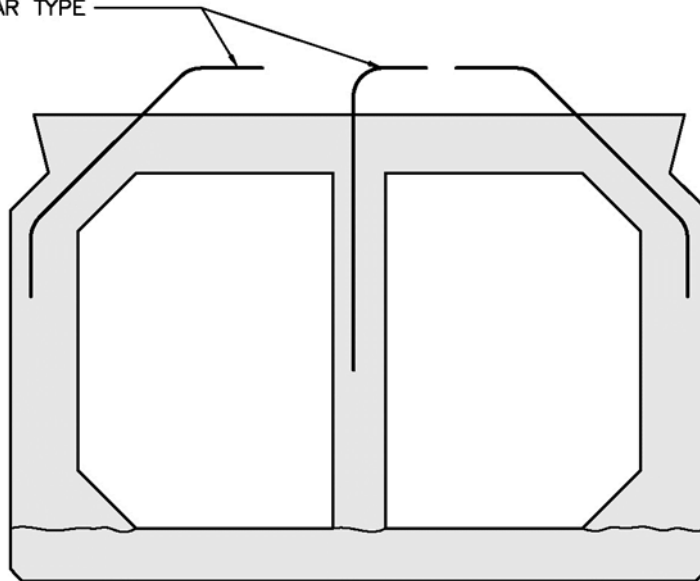
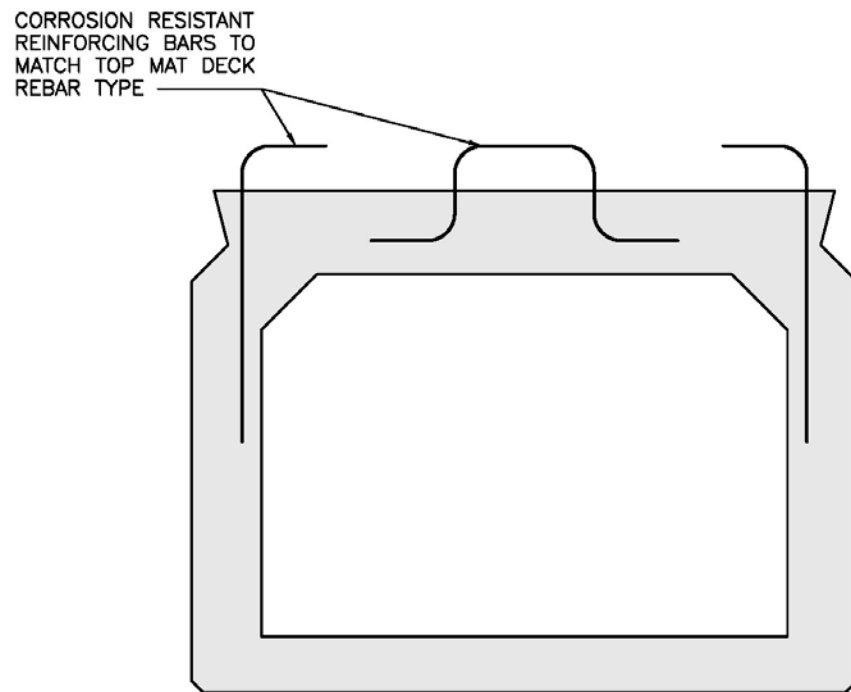


Figure 8.21 (b)
Single cell box beam/overlay connection detail



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9.5.6	Load-sharing factor	2
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9.5 General Design**9.5.6 Load-sharing factor**

Add to **Table 9.3 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

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.*

Parallel Strand Lumber (PSL) has been used in Ontario bridges and may be appropriate in certain applications. Locally available PSL is manufactured from untreated Douglas Fir and is currently available from a single source only. Treatment has to be applied locally by a third party which may void the manufacturer's warranty.

Locally available laminated veneer lumber is manufactured from untreated Douglas Fir which does not easily accept preservative treatment and is not recommended for use in Coastal British Columbia.

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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 Approved.

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:*
 - *Angles and channels, non-weathering: 350W (equivalent to ASTM A572, Grade 50); weathering: ASTM A588, Grade 50A.*
 - *Hollow structural sections: 350W or ASTM A500, Type B*
 - *HP Sections: 350W (equivalent to ASTM A572, Grade 50)*
 - *Plate: 300W, 350W, 350WT, 350A, 350AT*
 - *Structural tees: 350W (equivalent to ASTM A572, Grade 50)*
 - *Welded reduced wide flange shapes: 350AT, 350W*
 - *Welded wide flange shapes: 350AT, 350W*
 - *Wide flange shapes, non-weathering: 350W (equivalent to ASTM A572, Grade 50), weathering ASTM A588, Grade 50.*
 - *Anchor rods: ASTM A307, Grade C (Fy = 250 MPa, 36 ksi).*
 - *Shear studs: (refer to S6-14 clause 10.4.7)*
7. *Canadian mills no longer produce rolled sections. As such, rolled sections will likely be produced by American mills that will have primary designations to ASTM specifications, with possible CAN/CSA equivalency.*
8. *Local fabricator experience indicates that rolled sections are usually purchased as conforming to ASTM A572, Grade 50 (non-weathering) or ASTM A588 (weathering steel).*
9. *Local fabricator experience is that HSS is available as CSA G40.21M, Grade 350W, Class C or ASTM A500, Type B. Designers are encouraged to specify ASTM A500 because the thickness tolerances are more liberal for this grade (see CISC Bulletin dated Nov. 5, 1996). This would allow fabricators to use either grade.*
10. *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.*

11. *Higher strength anchor rods such as ASTM A449 or ASTM F1554 (105 ksi) may be used where required.*
12. *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 Standard 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 Approved.

Commentary:

1. *Bolts may not be available in Metric sizes without ordering an entire lot, 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 can not 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.*

10.4.7 Stud shear connectors

Shear stud connectors shall be comply with ASTM A108 (Grades 1015, 1018 or 1020).

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 CAN/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 with a paint system selected from the Ministry's Recognized Products List.

Commentary: *Due to the 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 bridges constructed of weathering steel, unless the entire structure is coated, the colour of the finish coat shall match the expected colour of the final oxidized surfaces. The colour proposed shall be subject to review by the Ministry.

For structures not using weathering steel, the steel shall be coated with a coating system selected from the Ministry's Recognized Products List and in accordance with SS 421 and SS 422.

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.*

In specifying the top coat colour of the protective coating at the ends of the bridge and under deck joints, the designer shall consider how other weathering steel bridges in the area or in similar environments have weathered over time and match the coating to the expected final oxidized colour.

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.*

10.6.5 Other components

Piling shall be sized for a corrosion allowance of at least 3 mm over the life of the structure unless a detailed corrosion analysis is undertaken. Coated piling shall not be allowed.

Commentary: *It is the Ministry's experience that coated piling has not been found to be successful. Therefore, a sacrificial thickness shall be added to the thickness required to meet structural demands. The 3 mm allowance is intended for fresh water applications. This sacrificial thickness shall be increased as required for more aggressive environments.*

10.7 Design details

10.7.1 General

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 G12.1-2003. In the event of conflict with Canadian Standards, Canadian Standards shall prevail.*

The document may be referenced at:

<http://www.aisc.org/contentNSBA.aspx?id=20130>

NSBA is the US-based National Steel Bridge Alliance.

AASHTO LRFD Bridge Design Specifications has design specifications for constraint induced fracture.

The following Clauses shall be added to Section 10.7.1:

10.7.1.1 Flange widths between splices

Unless economic analysis indicates that other arrangements are more cost-effective, it is preferred that flange plate widths 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

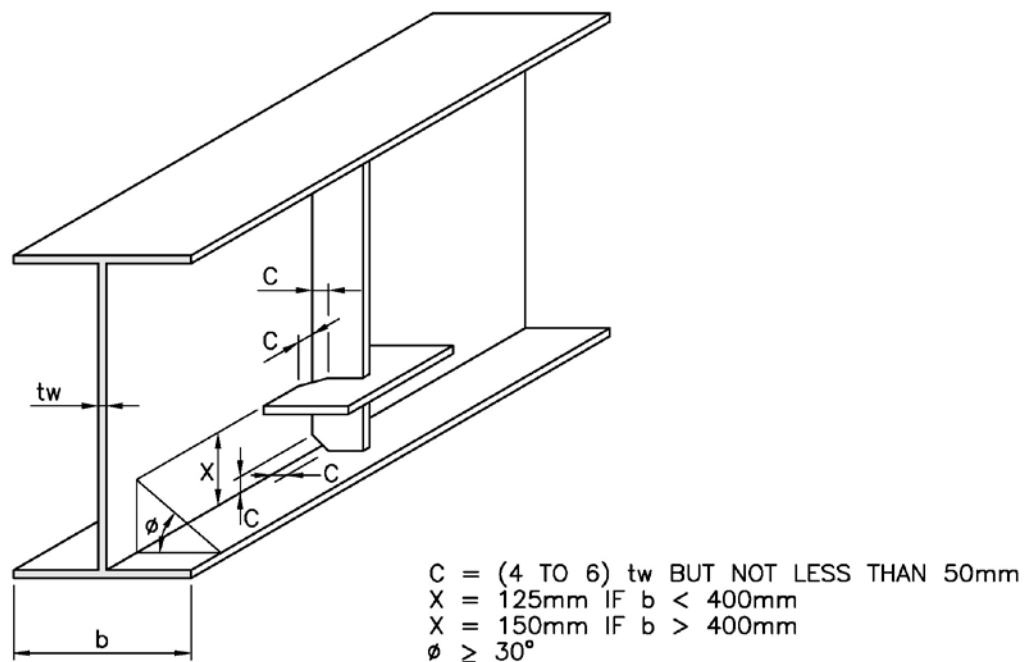
Transition of flange thickness at butt welds shall be made in accordance with CSA Standard W59-Latest Edition, with a slope through the transition zone not greater than 1 in 2.

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 and location 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.*

At end diaphragms, copes are not permitted.

Commentary: *This generally dictates the need for a drain at the diaphragm. 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. As shown in Figure 10.7.1.3.1, they should be located a distance of 125 mm from the bottom flange for flange widths up to 400 mm or 150 mm from the bottom flange for flange widths over 400 mm; but the angle between the flange and a line connecting the flange tip and the gusset plate-to-web connection shall not be less than 30 degrees. 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: *Two factors have been taken into consideration in determining the position of lateral bracing gusset plates.*

- Access for fabricating and inspecting the gusset plate-to-web connection; and
- The improved fatigue performance which results when the gusset plate is moved away from the flange into a lower stress region.

Although this is the 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.

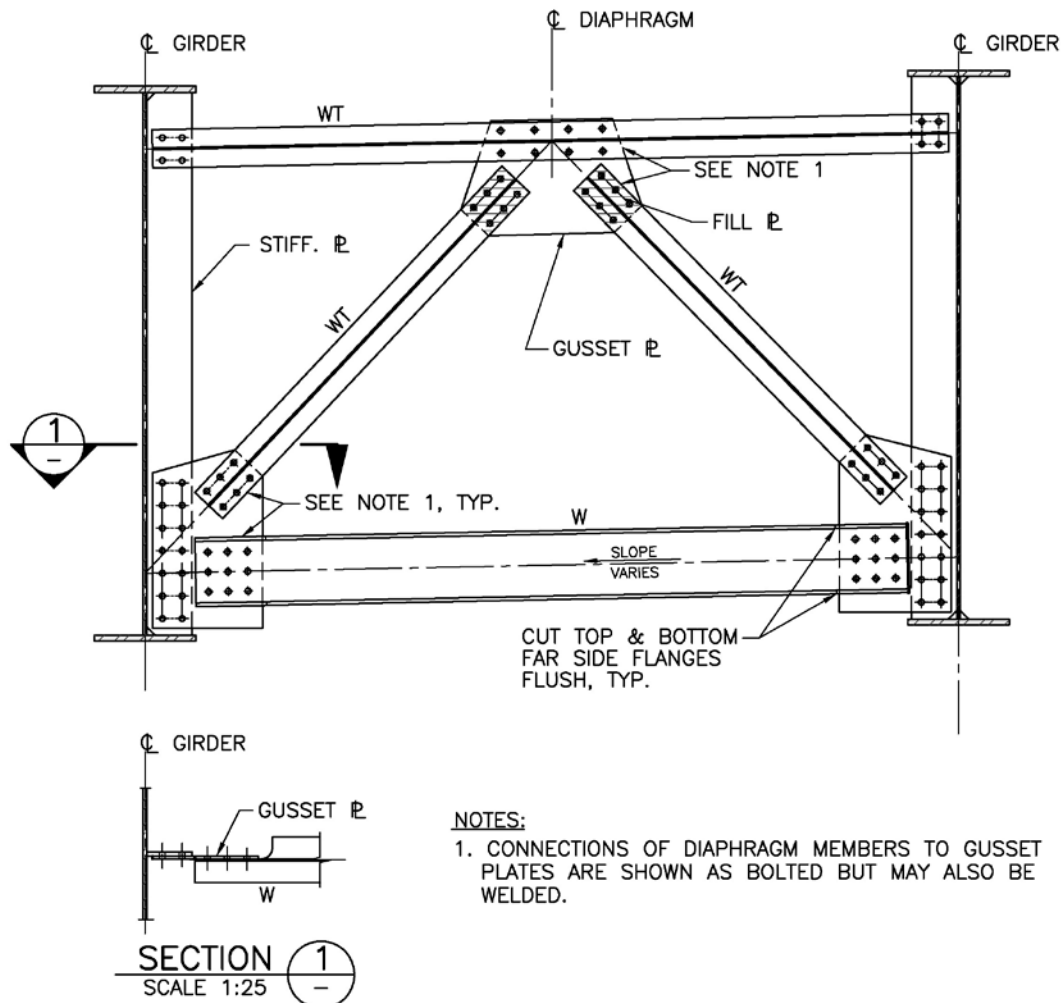
Frames should be designed for fabrication from one side, eliminating the need for “turning over” during fabrication. Oversized holes in the gusset plates are permitted.

Bracing shall be designed to accommodate both construction loading and the final loading on the structure. The designer shall identify any assumptions regarding construction loading on the drawings.

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

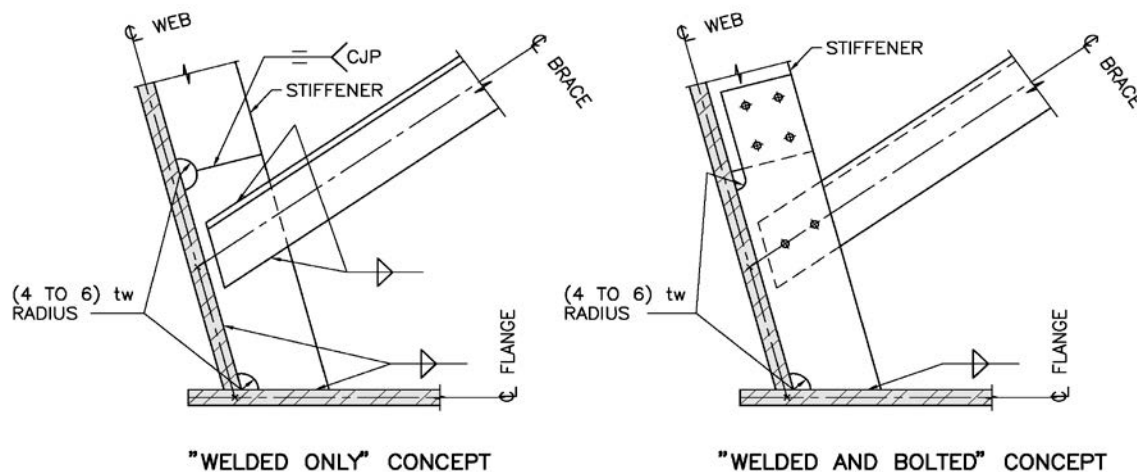


Commentary: Frame brace systems for use between girders should consist of angles or tees shop welded to one side of gusset plates which would be field bolted to the girder stiffeners. Efficient fabrication and erection procedures result when frames can be produced in one jig and when fewer pieces are 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. 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.

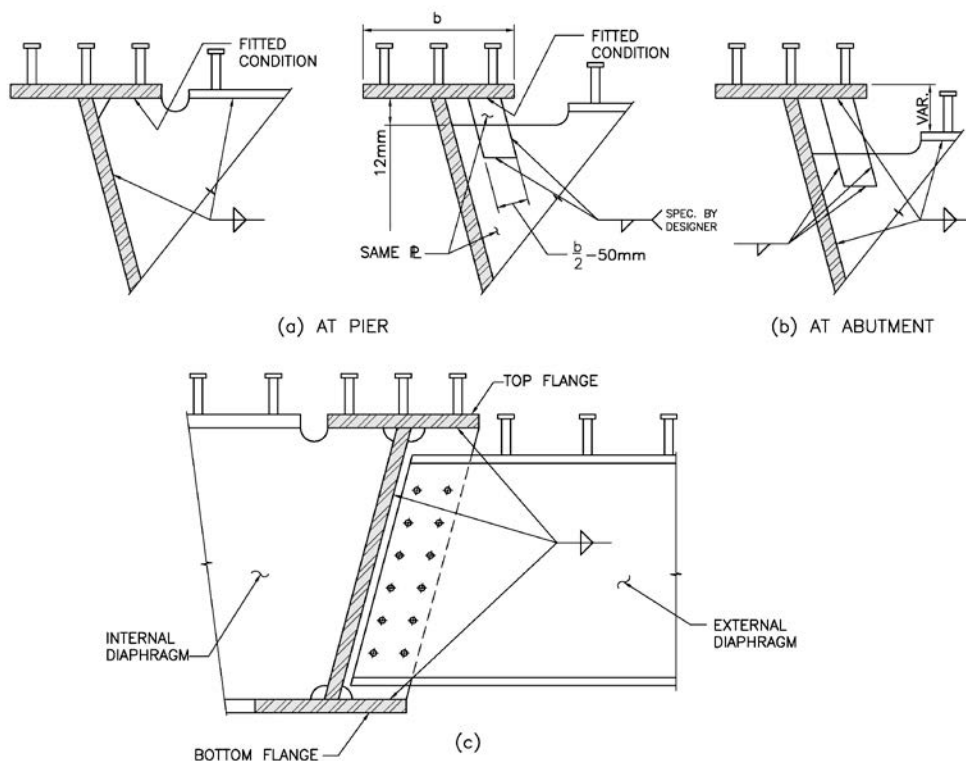
Commentary: Diaphragms comprising channel or beam sections would be less expensive in shallow bridges.

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 really 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. 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 contract documents. Intermediate stiffeners may be either true vertical, or perpendicular to fabrication work lines, depending on the fabricator's practice.

Commentary: *The recommendation 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.

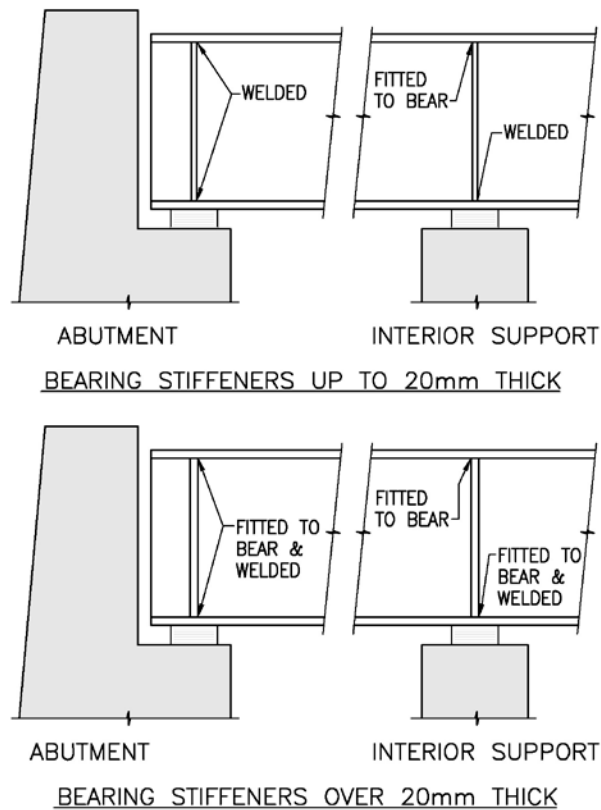
10.7.1.3.10 Bearing stiffener to flange connection

As shown in Figure 10.7.1.3.10, bearing stiffeners up to 20 mm thick may be welded to both flanges at abutments, and fitted to the tension flange and welded to the compression flange at interior supports. The size of weld shall be specified on the contract drawings. Bearing stiffeners over 20 mm thick shall be fitted and welded to both flanges at abutments and shall be fitted to both flanges and welded to the compression flange at interior supports.

Care shall be exercised in the design and during fabrication to mitigate distortions of the bottom flange from welding of the bearing stiffeners so as to ensure a flat surface for the bearing.

Bearing stiffeners at diaphragm locations shall either be welded or bolted.

Figure 10.7.1.3.10
Bearing stiffener to flange connections



Commentary: The load in bearing stiffeners over 20 mm thick would normally be too great to be carried by the stiffener to flange welds; thus fitting to bear is recommended. Welds may be used for load transfer in thinner bearing stiffeners but fitting to bear is not excluded.

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 negative moment regions, all intermediate stiffeners should be fitted to bear on the tension flange and welded to the compression flange.

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.*

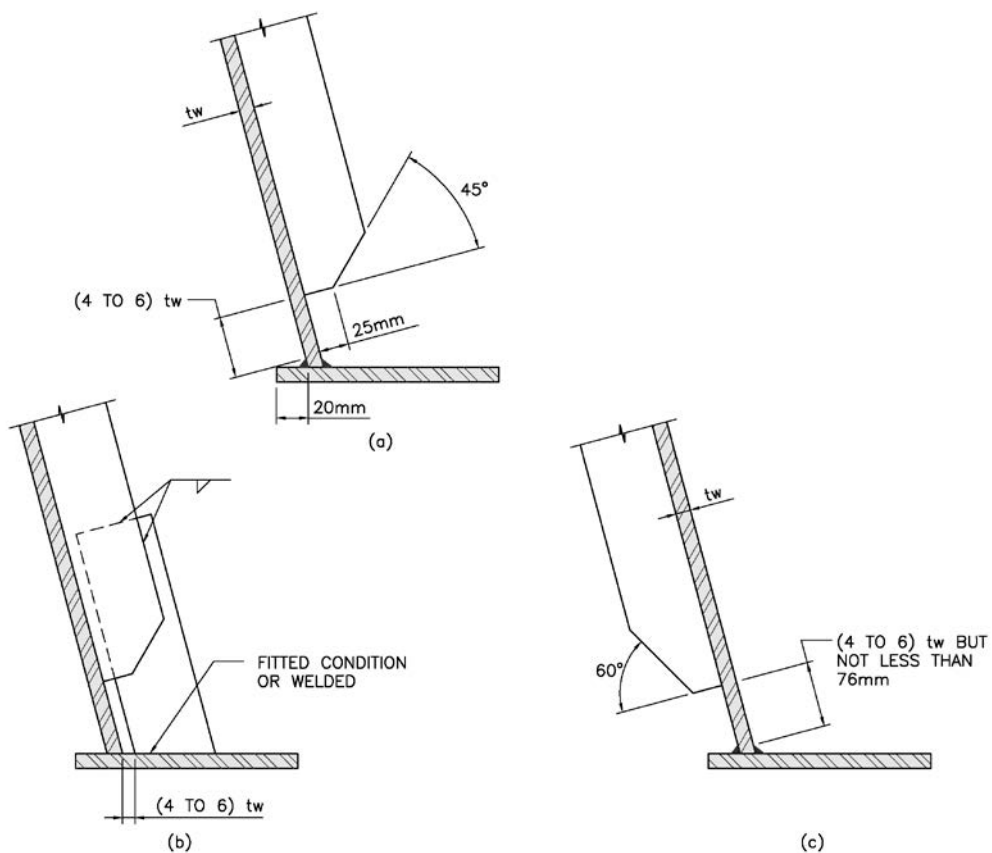
Locating longitudinal and transverse stiffeners on opposite sides of girder webs facilitates fabrication and reduces the number of stress-raisers in the web of the girder.

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



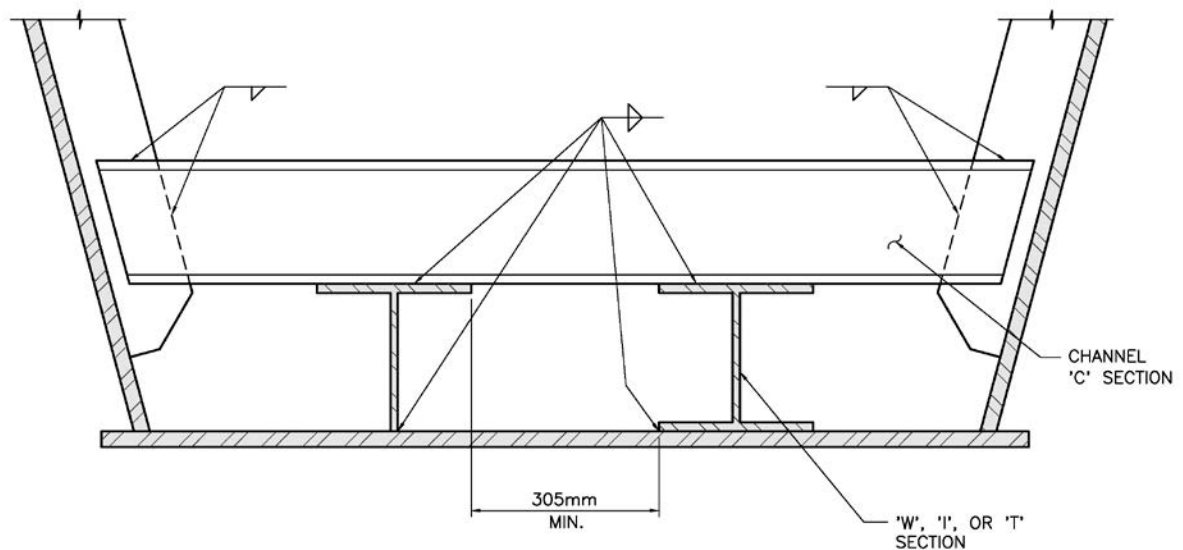
Commentary: In order to allow the use of automatic welding of the web-to-flange joint, the details as shown in Figures 10.7.1.3.14 (a) and 10.7.1.3.14 (c) are essential. The process of fabricating the box girders calls for the web stiffeners to be welded prior to welding the web to the flanges.

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:

- (a) the target finished steel girder grades.
- (b) camber profiles for deflections due to the deck, curbs, sidewalks, barriers, railings, wearing surface, creep and shrinkage, and utilities.
- (c) 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 Section 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

The matching electrode classifications for ASTM A709 steels shall be as specified in Table 10.10B.

The designer shall specify any testing requirements for welds that are required in addition to the testing requirements of SS 421.

Commentary: *SS 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 with the consent of the Ministry;
- Mechanical anchorage systems shall not be used.

Commentary:

Based on inspection of existing bridges, it is prudent to galvanize anchor rods and their components that are not embedded in the concrete and are exposed to damage from corrosion.

10.23 Fracture control

10.23.3 Fracture toughness

10.23.3.3 Fracture-critical members

Delete Table 10.13 title and replace with “**Impact test temperatures and Charpy impact energy requirements for fracture-critical members**”

Commentary: *The second sentence refers to Table 10.13 for impact energy requirements for fracture-critical members however, Table 13 title refers to primary tension members.*

A10.1 Construction requirements for structural steel**A10.1.1 General**

Construction shall be in accordance with SS 421 unless amended by the Supplement or otherwise Approved. SS 421 shall take precedence over CHBDC S6-14 where there is a conflict between these documents.

The Plans shall clearly define all construction requirements.

A10.1.5 Welded construction**A10.1.5.1 General**

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

Ultrasonic Testing (UT) may supplement Radiographic Testing (RT) subject to Approval by the Ministry and acceptance by the designer.

Commentary: *In thicker plate, UT testing may reveal defects not readably 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 0.3 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 and ride-ability caused by the deck joint system.

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 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**

All deck joints, except finger joints, shall be sealed. Unless otherwise consented to by the Ministry, expansion joints shall be designed as "finger" plate deck joints when the total movement is in excess of 100 mm. This shall not apply to bridges in regions of high seismicity.

Commentary: *In regions of high seismicity where large relative displacements may occur at deck joints, the joints chosen shall be suitable for the expected displacements.*

Add to the end of the fourth 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 which is acceptable to the Ministry.

Add to the fifth paragraph:

Deck joints with skew angles between 32 and 38 degrees shall be avoided by designers.

Commentary: *On bridges with large skews there is the possibility that the skew angle could match the angle used on snow plow 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 to accommodate the transverse displacements that are commonly experienced in skewed deck joint applications.

Proprietary joint products must either be listed in the Ministry's Recognized Products List or be consented to by the Ministry prior to use on a Project.

Water ingress into the abutment wall backfill or onto the substructure from the superstructure above shall be prevented. Joints between the superstructure end-diaphragm and the substructure shall be waterproofed with a material selected from the Ministry's Recognized Products List.

Modular deck joints may be used only when consented to by the Ministry.

Commentary: *Ministry experience is that modular joints are expensive and that a significant number of these joints have been replaced with finger joints after 20 to 30 years of service. Others have experienced maintenance problems that are costly to repair. Ministry consent is required on a project specific basis for their use.*

11.5.1.2

Design loads

Delete the third paragraph and replace with the following:

Except for modular joint systems, a horizontal load of 60 kN per metre length of the joint shall be applied as a braking load in the direction of traffic

movement at the roadway surface, in combination with forces that result from movement of the joint, to produce maximum force effects. For modular joint systems the horizontal load shall be developed in consultation with the Ministry with the recommended load consented to by the Ministry.

11.5.1.5 Maintenance

Commentary: *When open joint drainage is used, access to the drain trough and other drainage hardware should be provided for inspection and maintenance.*

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

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. 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 consented to by the Ministry will be allowed for use as an open deck joint. No other type of open deck joint will be allowed unless consented to by the Ministry. 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.

Bicycle tires present a problem for finger joints. Designers should consider measures to accommodate cyclists on the highway shoulders and in pathways.

11.5.3.2 Components

Commentary: To engage with a reinforced concrete substrate, anchors should penetrate the reinforcing cage sufficiently to achieve the required joint anchorage. In detailing the joint anchorage, the designer should consider compatibility of the anchor spacing and details with the embedded reinforcement. This will help 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 permitted on any joint connection. Drilled-in epoxy anchors will be permitted with the consent of the Ministry. The use of tapered-head countersunk anchor bolts requires Ministry consent.

11.5.6 Joint seals

Only deck joint seals made of natural rubber or virgin neoprene shall be used.

Commentary: Deck joint seals made of tyfoprene and santoprene have been observed to perform poorly and are not allowed. The use of silicone requires Ministry consent as it is only available at 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 consider the use of a corrosion-resistant plastic such as high density poly ethylene (HDPE). 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.

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

Add the following to the first paragraph:

Elastomeric bearings shall be used whenever possible.

Add to the end of the seventh paragraph the following:

Bearing replacement procedures shall be shown on the Plans, including jacking locations and jacking loads.

Enough space, both vertically and horizontally, must be provided between the superstructure and substructure to accommodate the required jacks for replacing the bearings. While it is difficult to establish a vertical clearance for all situations, a minimum vertical clearance of 150 mm is suggested. For steel girder bridges the web stiffeners of the diaphragms must be located accordingly.

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.

Proprietary products must be listed in the Ministry's Recognized Products List or consented to by the Ministry prior to use.

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.

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

11.6.3.4 Attachment

11.6.3.4.1 PTFE layer

Commentary: *Sheet polytetrafluorethylene (PTFE) should preferably be confined in a recess in a rigid metal backing plate for one half of its thickness. Sheet PTFE may be bonded or unbonded however, unbonded PTFE offers the advantage of ease of replacement.*

Delete the third sentence and replace with the following:

Sheet PTFE which is not confined must be bonded by an Approved method to a rigid metal surface.

11.6.4 Spherical bearings

11.6.4.1 General

Spherical bearings shall be installed concave part down to prevent accumulation of water and dirt.

11.6.6 Elastomeric bearings

Commentary: *See Clause C11.6.6 at the end of this Section for additional commentary on the design of elastomeric bearings.*

11.6.6.1 General

The design of unreinforced and steel reinforced elastomeric bearings for compressive deformation shall account for the different deformation responses in all layers of elastomer.

11.6.6.2 Materials**11.6.6.2.2 Elastomers**

Commentary: Table 11.5 in S6-14, *Physical Properties of Polyisoprene and Polychloroprene*, lists requirements for the physical properties of polyisoprene and polychloroprene but does not provide properties required for design, e.g., shear modulus and the relationship between compression stress, shape and compression strain. For design purposes AASHTO LRFD Bridge Design Specifications refer to shear modulus which is the most important physical property of the elastomer in bridge bearings. The designer is responsible for incorporating appropriate properties with the bearing design.

11.6.6.3 Geometric requirements

Contrary to part (a), h_e shall be less than 25 mm and greater than 15 mm.

The shape factor must always be checked.

Commentary: Problems with plain bearings that are too thin or too thick have been observed. Therefore, the allowable thickness has been amended here.

The geometric requirements for laminated bearings are conservative and may reduce efficiency of the bearings as part of a seismic base isolation system (i.e. the bearings may be too stiff for seismic isolation if the geometric requirements are satisfied). The geometric requirements may be relaxed as long as stability of the bearings under different load combinations is checked explicitly and verified by testing in accordance with Clause 4.10 of S6-14.

The bearing pressure requirements for continuous strips may be waived where the bearing is used as a temporary bearing pad.

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 and along the edges shall be 5 mm. Allowable tolerances on the cover amount shall be + 3 mm, - 0 mm.

Commentary: *It is recommended that a minimum cover of 5mm be specified with a tolerance of + 3 mm and – 0 mm on this amount.*

11.6.6.6

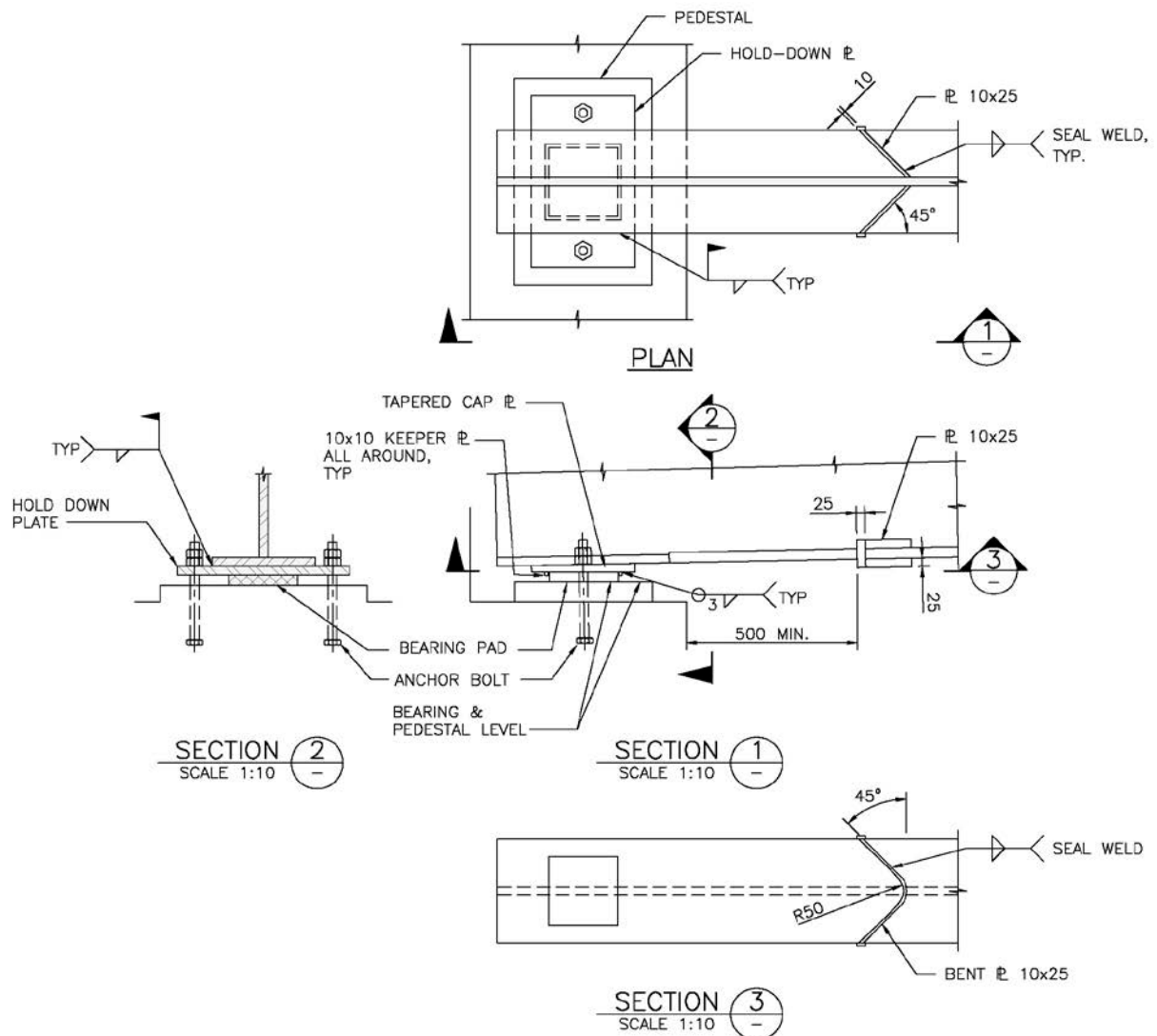
Positive attachment

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 11.6.6.6 (a)
Bearing hold down details for steel girders

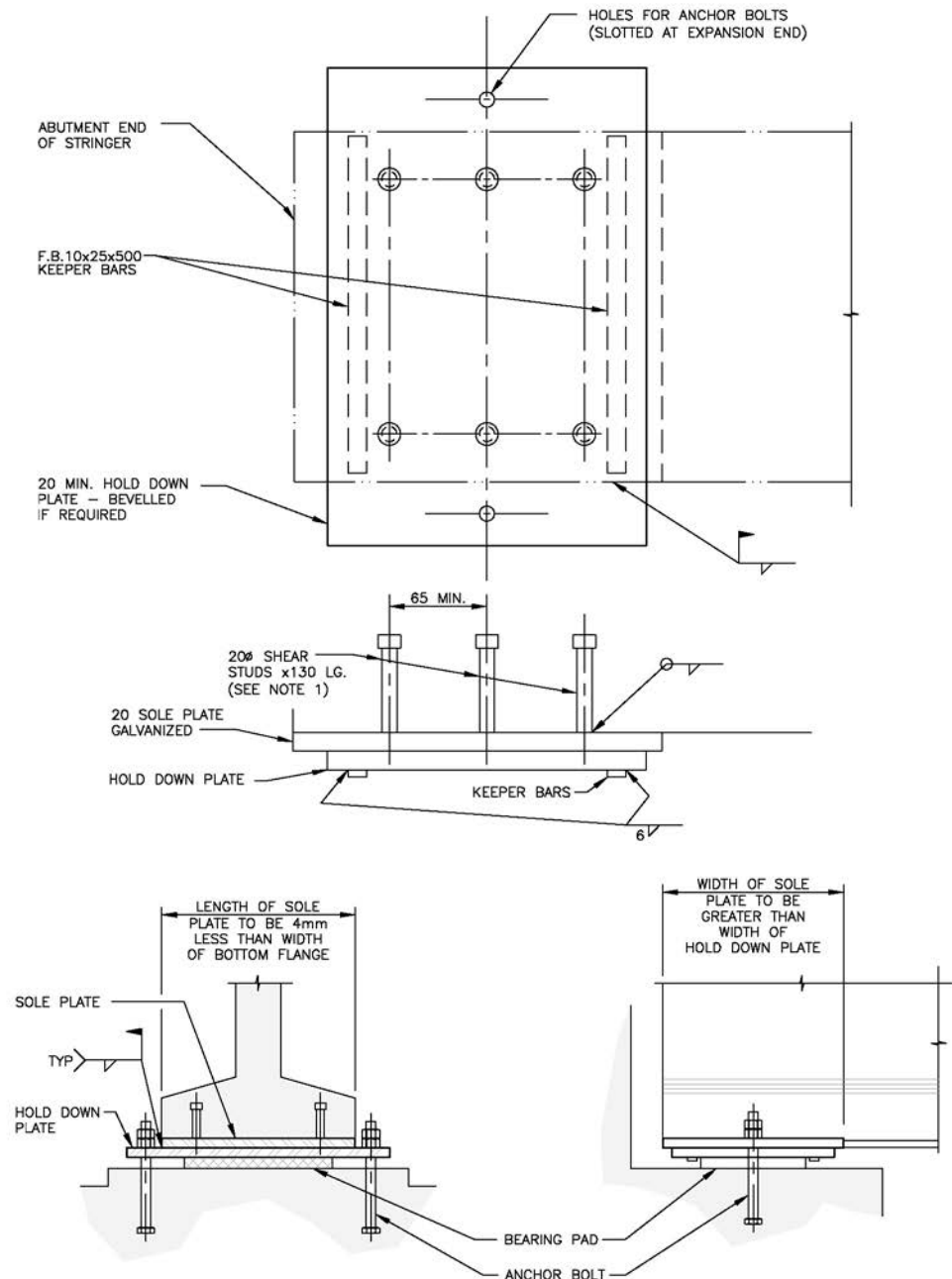


NOTES:

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

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.

11.6.6.7 Bearing Pressure

Add the following:

The bearing pressure requirements for laminated bearings may be relaxed if the laminated bearings are used as part of a seismic base isolation system. However, the strain requirements for the laminated bearings under different load combinations shall be satisfied and verified by analysis and testing in accordance with Clause 4.10.

Commentary: *In Clause 4.10, design of elastomeric bearings for seismic base isolation is based on a strain approach. The equivalent shear strains in the rubber due to different load combinations are limited to the allowable values. The strain based design typically results in bearing sizes somewhat less conservative than those based on the bearing pressure requirements. This will increase efficiency of the bearings for seismic isolation.*

11.6.10 Load plates and attachment for bearings**11.6.10.2 Tapered plates**

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.

Commentary on elastomeric bearings**C11.6.6 Elastomeric bearings****C11.6.6.8 Design procedure****C11.6.6.8.a Preamble**

The following information is based on the AASHTO LRFD Bridge Design Specifications and is intended to provide assistance to designers for design of elastomeric bearings. The information is presented in the following format:

- selection of design properties for elastomer,
- calculation of compressive deformations,
- determination of horizontal shear forces; and
- bearing testing.

C11.6.6.8.b Elastomeric properties

If the elastomer is specified by hardness on the Shore A scale, a range of shear modulus, G , shall be considered to represent the variations found in practice as given in the following table (reproduced from Table 14.7.6.2-1 of the AASHTO LRFD Bridge Design Specifications):

Table C11.6.6.8b
Shear Modulus, $G^{(1)}$

	Hardness (Shore A)	
	50	60
Shear Modulus @ 23 °C (MPa)	0.66-0.90	0.90-1.38
Creep deflection @ 25 years divided by instantaneous deflection	0.25	0.35

Notes:

(1) Reference Table 14.7.6.2-1, AASHTO LRFD Bridge Design Specifications

The shear modulus shall be taken as the least favourable value from the range in design.

If the elastomer is specified explicitly by its shear modulus, that value shall be used in design and shall be verified by shear test using the apparatus and procedure described in Annex A of ASTM D4014 (see Clause 18.2.5.3 of AASHTO LRFD Bridge Construction Specifications). The shear modulus

obtained from testing shall fall within 15 percent of the value specified in the contract documents.

C11.6.6.8c Shape factor

The shape factor of an elastomeric layer shall be taken as the plan area of the layer divided by the area of perimeter free to bulge. For rectangular bearings without holes, the shape factor of a layer may be taken as:

$$S_i = \frac{LW}{2h_{ri}(L + W)} \quad (\text{Equation [1]})$$

Where:

L = length of a rectangular elastomeric bearing (parallel to longitudinal bridge axis) (mm);

W = width of the bearing in the transverse direction (mm); and

h_{ri} = thickness of i^{th} elastomeric layer in a laminated bearing (mm).

For circular bearings without holes, the shape factor of a layer may be taken as:

$$S_i = \frac{D}{4h_{ri}} \quad (\text{Equation [2]})$$

Where:

D = diameter of a circular elastomeric bearing.

If holes are present, their effect shall be accounted for when calculating the shape factor because they reduce the loaded area and increase the area free to bulge. Suitable shape factor formulae for an elastomeric layer with holes are:

For rectangular bearings:

$$S_i = \frac{LW - \sum \frac{\pi}{4} d^2}{h_{ri}(2L + 2W + \sum \pi d)} \quad (\text{Equation [3]})$$

For circular bearings:

$$S_i = \frac{D^2 - \sum d^2}{4h_{ri}(D + \sum d)} \quad (\text{Equation [4]})$$

Where:

d = the diameter of the hole or holes in the bearing (mm).

C11.6.6.8.d Vertical compressive deformation

Instantaneous vertical compression deformation

If the elastomer is specified by hardness, the average total instantaneous vertical compressive deformation of a laminated bearing shall be taken as:

$$\delta = \sum \varepsilon_i h_{ri} \quad (\text{Equation [5]})$$

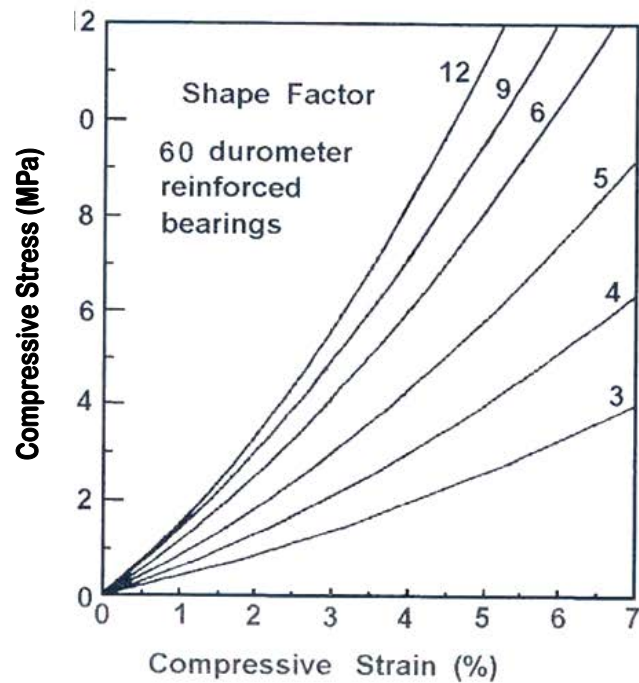
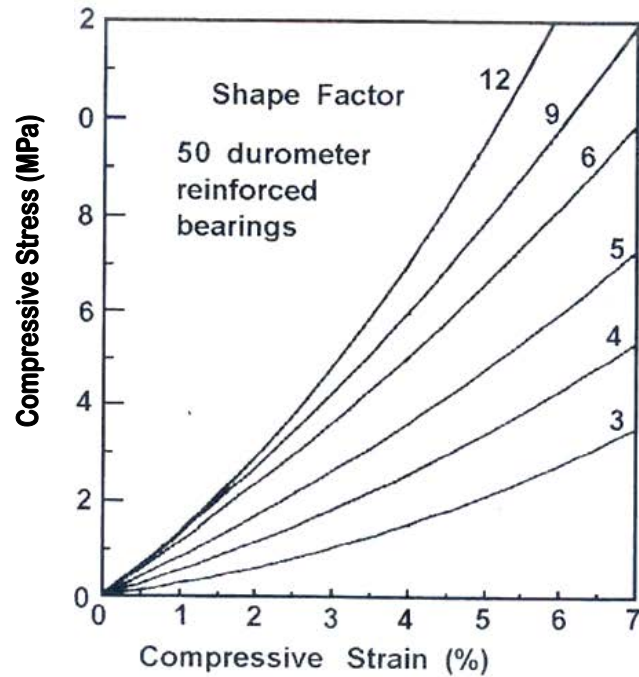
Where:

ε_i = instantaneous compressive strain in i^{th} elastomer layer of a laminated bearing;

h_{ri} = thickness of i^{th} elastomeric layer in a laminated bearing (mm).

In the absence of material specific data from testing, the following figure (reproduced from Figure C14.7.6.3.3-1 of the AASHTO LRFD Bridge Design Specifications) may be used to estimate vertical compressive strain of an elastomeric layer in a laminated bearing:

Figure 1
Vertical compressive stress-strain curves for elastomeric layer
(reproduced from Figure C14.7.6.3.3-1 of the
AASHTO LFRD Bridge Design Specifications)



If material-specific data from testing are available, the average total instantaneous vertical compressive deformation of a laminated bearing may be estimated as follows:

$$\delta = \sum \delta_i \quad (\text{Equation [6]})$$

Where:

δ_i is the vertical compressive deformation of i^{th} elastomeric layer and given by

$$\delta_i = \frac{\sigma_c h_{ri}}{E_0(1 + 2kS_i^2)} = \frac{\sigma_c h_{ri}}{4G(1 + 2kS_i^2)} \quad (\text{Equation [7]})$$

Where:

σ_c = average compressive pressure at SLS (MPa);

h_{ri} = thickness of i^{th} elastomeric layer in a laminated bearing (mm);

S_i = shape factor of i^{th} elastomeric layer in a laminated bearing;

E_0 = elastic modulus of elastomer typically taken as $4G$ (MPa);

G = shear modulus of elastomer (MPa); and

k = elastomer material coefficient for compressive deflection.

In the absence of test data, the compressive deflection of a plain elastomeric bearing may be estimated as 3 times the deflection estimated for steel-reinforced bearings of the same shape factor (Figure 1 and Equation 7) in accordance with Clause 14.7.6.3.3 of AASHTO LRFD Bridge Design Specifications.

Creep vertical compressive deformation

The effects of creep of the elastomer shall be added to the instantaneous deflection when considering long-term deflections. In the absence of material-specific data, the values given in Supplement Table C11.6.6.8b may be used.

C11.6.6.8.e Horizontal forces

The factored horizontal force due to shear deformation of an elastomeric bearing shall be taken as:

$$H_u = GA \frac{\Delta_u}{h_{rt}} \quad (\text{Equation [8]})$$

Where:

G = shear modulus of the elastomer (MPa);

A = plan area of the elastomeric bearing (mm²);

Δ_u = factored shear deformation (mm); and

h_{rt} = total elastomeric thickness (mm).

If an elastomer is specified by its hardness, the upper bound value of shear modulus in the range shall be used in estimating the horizontal force transmitted from the bearing to the substructure. The effects of cold temperature on shear modulus shall also be considered. Unless material-specific data from testing are available, the effects of cold temperature may be considered in accordance with Clause 14.7.5.2 of the AASHTO LRFD Bridge Design Specifications. The horizontal force resulting from shear deformation of the elastomer shall be considered in the design of the substructure unless a low friction sliding surface is provided. If the horizontal force transmitted is governed by the sliding surface, a conservative estimate of the friction force shall be considered (see Clause 14.7.5.2 of AASHTO LRFD).

C11.6.6.8.f Bearing testing

The elastomeric bearings shall be tested in accordance with the requirements specified in the Ministry of Transportation and Infrastructure Template Special Provisions: Appendix - Supply, Fabrication and Installation of Bearing Assemblies.

A copy of the Special Provisions and Appendix - Supply, Fabrication and Installation of Bearing Assemblies is available at:

http://www.th.gov.bc.ca/publications/eng_publications/bridge/bridge_standards.htm#provisions

C11.6.6.8g Commentary

The above information provides additional design aids for elastomeric bearings, particularly for selection of design properties for elastomer,

calculation of vertical compressive deformation in the elastomer, and horizontal shear force resulting from shear deformation in the elastomer. This information is based on the design provisions of the AASHTO LRFD Bridge Design Specifications.

The design provisions for elastomeric bearings in the AASHTO LRFD Bridge Design Specifications are almost identical to those in the AASHTO Standard Specifications. In AASHTO, it is recognized that shear modulus, G , of the elastomer is the most important material property for design. Hardness has been widely used in the past because the test for it is quick and simple. However, hardness is at best an approximate indicator of the engineering properties of the elastomer and correlates only loosely with shear modulus. Therefore, AASHTO allows two ways of specifying material properties for elastomer. One method is to specify hardness on the Shore A scale, and a range of shear modulus values corresponding to the specified hardness should be considered to cover the expected variations found in practice. The shear modulus shall be taken as the least favorable value from the range in design, e.g. lower bound shear modulus for calculating vertical compressive deformation of the elastomer and upper bound shear modulus for estimating horizontal shear force transmitted by the bearing to the substructure. The other method is to specify the shear modulus explicitly. In this case, shear tests using the apparatus and procedure described in Annex A of ASTM D4014 shall be conducted to verify that the shear modulus values obtained from testing fall within 15 percent of the value specified.

Equations [1] and [2] are the shape factors for rectangular and circular bearings without holes. The shape factor of an elastomeric layer is the loaded area of the bearing in plan divided by the area of the layer which is free to bulge, and is an approximate measure of this bulging restraint. The shape factor, S , is an important design parameter for elastomeric bearings because the vertical compressive strength and stiffness of the bearing are approximately proportional to S and S^2 . Holes are discouraged in reinforced elastomeric bearings. If holes are present, Equations [3] and [4] should be used to calculate the shape factors for rectangular and circular bearings.

Figure 1 is reproduced from Figure C14.7.6.3.3-1 of the AASHTO LRFD Bridge Design Specifications. The figure shows vertical compressive stress-strain curves for elastomeric layers with different values of shape factor for 50 or 60 durometer reinforced elastomeric bearings. These curves are based on the lower bound value of shear modulus for a given hardness.

Equation [7] is commonly used to calculate instantaneous vertical compressive deformation of an elastomeric layer in a laminated bearing (see Goodco catalogues, papers on elastomeric bearing design, and AASHTO Guide Specifications for Seismic Isolation Design). The material constants used in the equation should be verified by testing, or lower bound values should be used if hardness is specified for the elastomer.

Unreinforced elastomeric pads frequently slip at the loaded surfaces under applied compressive load resulting in a significant increase in the compressive deflection. This is accounted for by applying a factor of 3 to the deflection estimated for steel-reinforced bearings of the same shape factor.

If the elastomer is specified by hardness, the upper bound value of its shear modulus should be used in estimating the horizontal force transmitted from the bearing to the substructure. Shear modulus increases as the elastomer cools, but the extent of stiffening depends on the elastomer compound, temperature, and time duration. It is, therefore, important to specify a material with low-temperature properties that are appropriate for the bridge site. The effects of cold temperature on shear modulus should be considered in estimating the horizontal force transmitted from the bearing to the substructure. Unless material-specific data are available from testing, such effects may be considered in accordance with Clause 14.7.5.2 of the AASHTO LRFD Bridge Design Specifications. The upper bound horizontal force resulting from bearing shear deformation shall be considered in design of the substructure unless a low friction sliding surface is provided. If the horizontal force transmitted is governed by the sliding surface, a conservative estimate of the friction force shall be used.

Quality control test shall be conducted on all elastomeric bearings.

CSA-S6-14 does not include any testing provisions for elastomeric bearings, except for elastomeric bearings used as part of a seismic base isolation system.

The AASHTO LRFD Bridge Construction Specifications specify both short-term and long-term compression proof load tests for elastomeric bearings. Short-term compression proof load test is required for every bearing where the bearing is loaded in compression to 150% of its rated service load. The load is held for 5 minutes, removed, then reapplied for a second period of 5 minutes. The bearing is then examined visually when under the second loading. Long-term compression proof load test is required only for one random sample from each lot of bearings. The long-term compression test is similar to the short-term test except that the second load is maintained for 15 hours.

In the current Ministry Special Provisions template, a compression load test is required for every laminated bearing. The compression test specified in the Ministry's 'Appendix – Supply, Fabrication and Installation of Bearing Assemblies' as part of the Special Provisions template is somewhat different from that specified in AASHTO. The compression test specifies sequences of loading and unloading in increments and requires measurement of not only axial load (average pressure) but also axial deformation at different steps. Therefore, this test is more involved than the compression tests required in AASHTO, but it provides additional information on bearing axial stiffness. The

time required for this test will be longer than the AASHTO short-term compression test, but significantly shorter than the AASHTO long-term compression test. Previous experience indicates that any bulging suggesting poor laminate bond will show up almost immediately after application of the vertical load, and the test requirement in the Ministry Special Provisions template would be adequate.

The advantages of short-term compression testing can be seen from the following figures:

Figure C11.6.6.8.f.1
Splitting along a bulge (above the number 50)



Figure C11.6.6.8.f.2
“Roll out” of the bottom of the bearing along the right face,
possibly because the thickness of the lowest layer of
rubber was too thick



Figure C11.6.6.8.f.3

**Loss of bonding between two layers of rubber. Note the
coin inserted into a crack**



Figure C11.6.6.8.f.4

**Evidence from the bulges that the top plate is bent along the
right face**



Figure C11.6.6.8.f.5
Loss of bond between two rubber layers



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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. Through the use of 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:

Barrier Attachments: Attachments on top of barriers (i.e.: poles, railings) can present a snag hazard to an impacting vehicle. This snag can adversely affect the impacting vehicle’s trajectory, as well as creating a potential compartment intrusion and/or debris hazard. Two strategies to mitigate this hazard are to offset the attachment behind the barrier face, outside the barrier’s “Zone of Intrusion” (ZOI), or to increase the barrier height to reduce the impacting vehicle’s “ride-up”, effectively reducing the barrier’s ZOI.

Where possible, all barrier attachments shall be removed from the barrier’s ZOI. Otherwise, the snag risks can still be minimized by increasing the setback as much as possible and grouping several accessories on a single attachment.

While still a relatively new topic, further information and delineation of the ZOI 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)

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 Section 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 Section 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, in most cases, 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

Barrier joints with openings greater than 100 mm shall be protected by sliding steel plates to prevent catchment of vehicles. All steelwork shall be protected from corrosion with hot-dipped galvanizing 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.

12.4.3 Traffic barriers

12.4.3.2 Test level

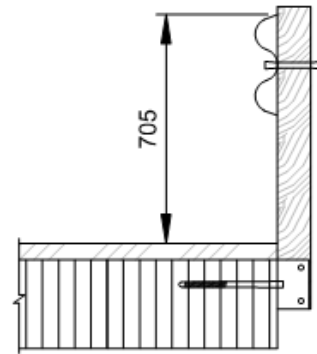
12.4.3.2.1 General

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 Approval.

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.

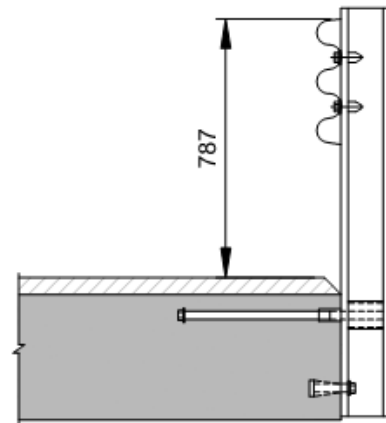
TL-1 W-Beam

Commentary: Details based on
USDA Forest Service W-Beam Bridge
Rail.



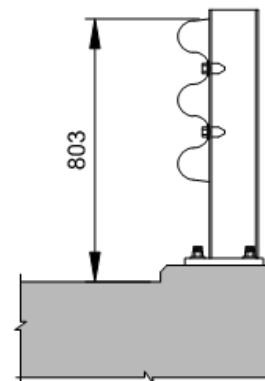
TL-2 Thrie Beam
(Side Mounted)

Commentary: Side mounted details
based on Oregon Standard Drawing
BR233.



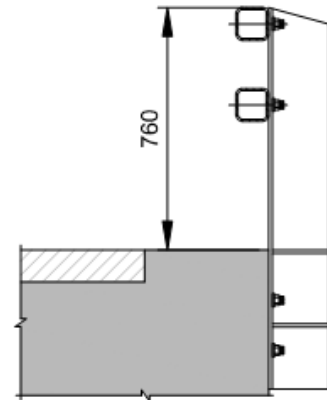
TL-2 Thrie Beam
(Top Mounted)

Commentary: Top mounted details
based on Alberta Standard Drawing
S-1652-00.



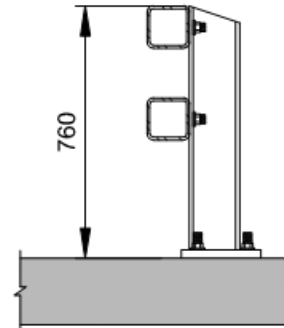
TL-2 Two Rails
(Side Mounted)

Commentary: Side mounted details based on California Type 115 Bridge Rail.



TL-2 Two Rails
(Top Mounted)

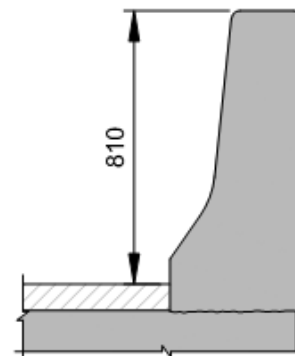
Commentary: Top mounted system based on California Type 115 Bridge Rail, modified for top mounted anchorage. Modified anchorage to be designed in accordance with Clause 12.4.3.5 in the CHBDC.



TL-4 "F" Shape
(Cast-in-Place Concrete)

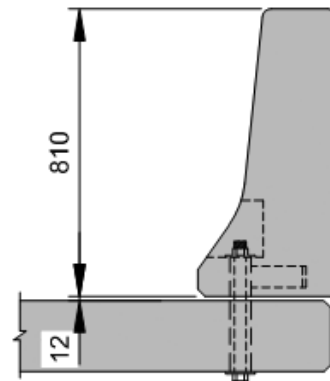
See the Ministry's Standard Bridge Parapet (2874-1) for detailing.

Commentary: Similar systems are used widely in jurisdictions across North America.

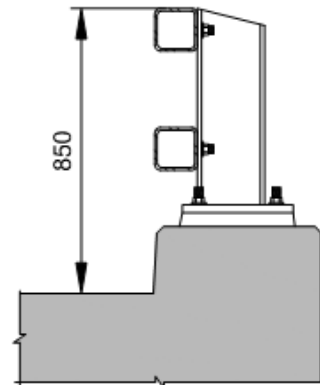


TL-4 “F” Shape
(Precast Concrete, Bolt-Down)

Commentary: The Ministry has developed drawings for this “F” Shape barrier system and completed load testing of the anchorage details. The anchorage testing confirmed that the CHBDC requirements for anchorage are met. As such, this barrier system is considered acceptable for use as consented to by the Ministry. When consented to by the Ministry, these drawings will be provided by the Ministry for use.

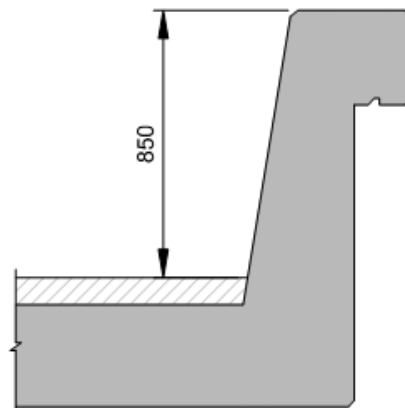
**TL-4 Double Tube on Curb**

Commentary: Details based on Alberta Standard Drawing S-1642-00. Alternate details in Oregon Standard Drawing BR206.

**TL-4 Constant Slope**

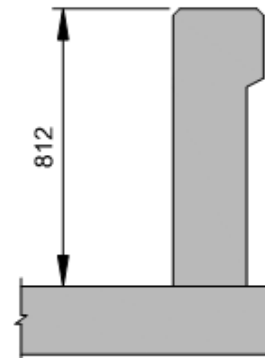
Commentary: Details based on Alberta Standard Drawing S-1650-00. Alternate details in California Type 70 Bridge Rail and Texas SSTR Bridge Rail.

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

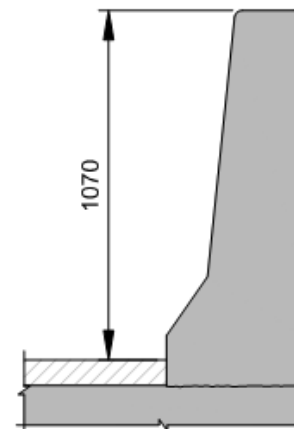
Commentary: Details based on the Texas T221 Bridge Rail.



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
- Online Guide to Bridge Railings, (on-going project), AASHTO-ARTBA-
AGC Task Force 13 (<http://guides.roadsafellc.com/bridgeRailGuide/>)
- 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:

For bridges that meet the Ministry definition of a Low Volume Road Structure and that also meet the following criteria, then a TL-1 barrier system may be used:

- Bridges on a road with a maximum roadway width of 8.6 m, a maximum deck height above ground or water surface of 5.0 m, and either a maximum design speed of 80 km/h combined with a maximum AADT of 100 or a maximum design speed of 50 km/h combined with a maximum AADT 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 Approved.

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

Traffic barriers shall be constructed such they are oriented perpendicular to the deck surface.

In Table 12.8 - Minimum barrier heights, change height H to 0.81 m for traffic barrier type TL-4.

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.

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**

The Ministry's Standard steel sidewalk fence 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.

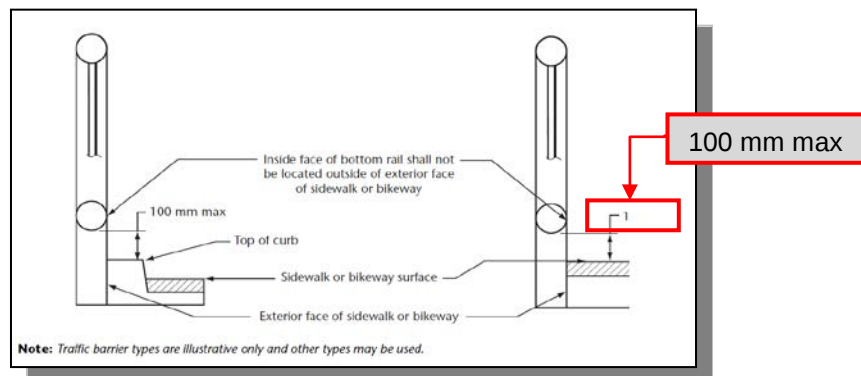
Commentary: *The debris and/or safety fencing should be considered in urban areas for bridges over roadways to reduce the risk of objects falling from the bridge on to the roadway below.*

12.4.4.2 Geometry

Pedestrian barriers shall be constructed such that they are oriented plumb.

Figure 12.2 – Geometry of side-mounted pedestrian and bicycle barriers

Revise as follows:

**12.4.5 Bicycle barriers****12.4.5.1 General**

The Ministry Standard steel bicycle fence 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

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

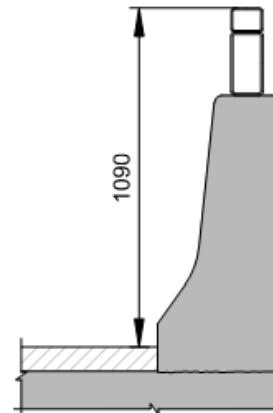
The following pedestrian combination barrier "reference concepts" have been accepted by the Ministry for use on highway bridges in B.C. Other pedestrian combination barrier concepts may be considered but require prior Approval.

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.

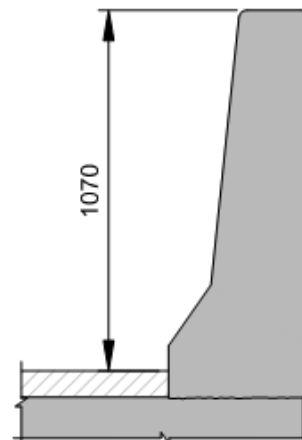
See the Ministry's Standard Bridge Parapet Steel Railing (2785-2) for railing detail.



TL-4 “Tall F” or TL-5 “F” Shape

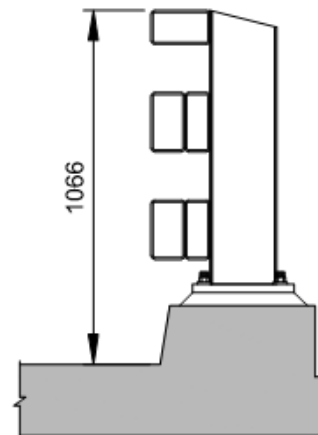
This is an extended version of the Ministry's Standard Bridge Parapet (2874-1).

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

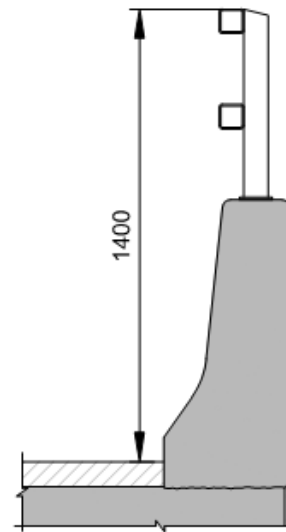
The following bicycle combination barrier “reference concepts” have been accepted by the Ministry for use on highway bridges in B.C. Other bicycle combination barrier concepts may be considered but require prior Approval.

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.

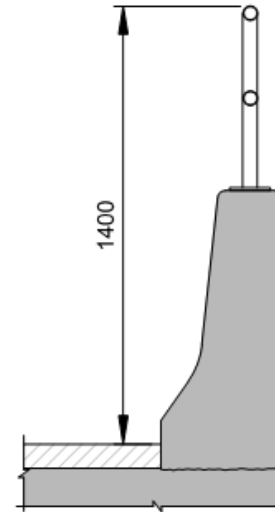
See the Ministry's Standard Bridge Parapet Steel Bicycle Railing (2785-3) for railing detail.



TL-4 “F” Shape with Flush-Post Bicycle Railing

See the Ministry's Standard Bridge Parapet (2874-1) for barrier detail.

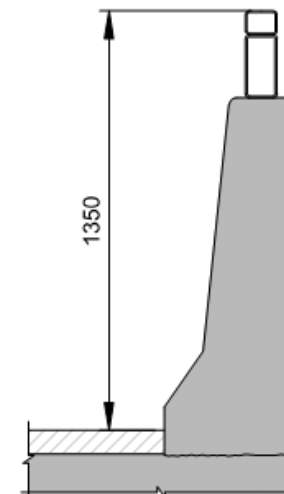
Commentary: A flush post railing is preferred when a railing is required on top of a barrier that separates traffic from a mixed-used sidewalk.

**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.

Commentary: A protection height of 1350mm is 20mm below the CHBDC minimum requirement for bicycle protection. This reduction in protection height is acceptable for this reference concept only.

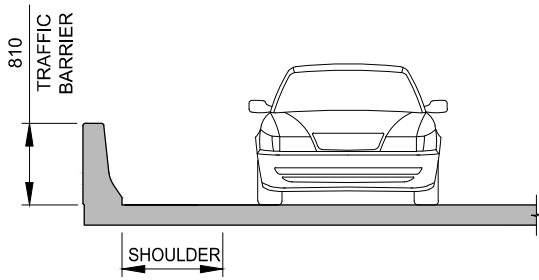
**12.4.6.2****Geometry**

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.

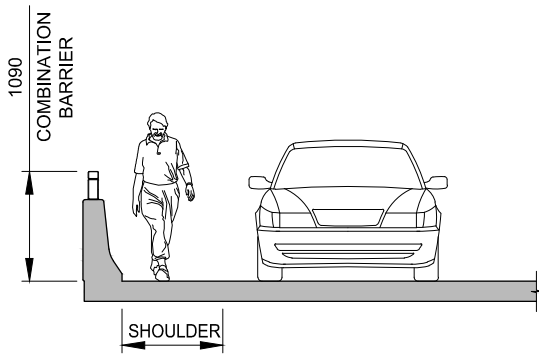
CONFIGURATION #1 - BRIDGE WITH NO SIDEWALK

USAGE: THIS SYSTEM IS SPECIFIED WHERE PEDESTRIAN USE IS RARE AND GENERALLY ASSOCIATED WITH MAINTENANCE PERSONNEL OR VEHICLE BREAKDOWN.



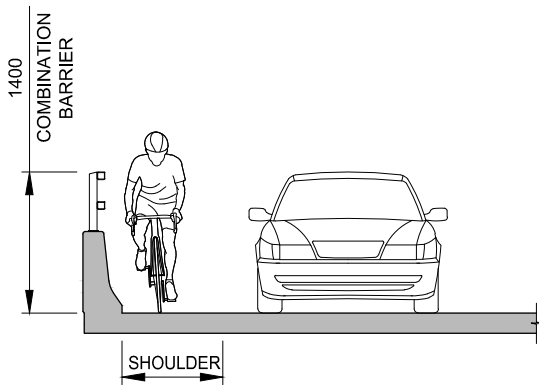
OPTION A
VEHICULAR TRAFFIC ONLY

USAGE: THIS SYSTEM IS SPECIFIED WHERE WARRANTED BY PEDESTRIAN USAGE.



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.

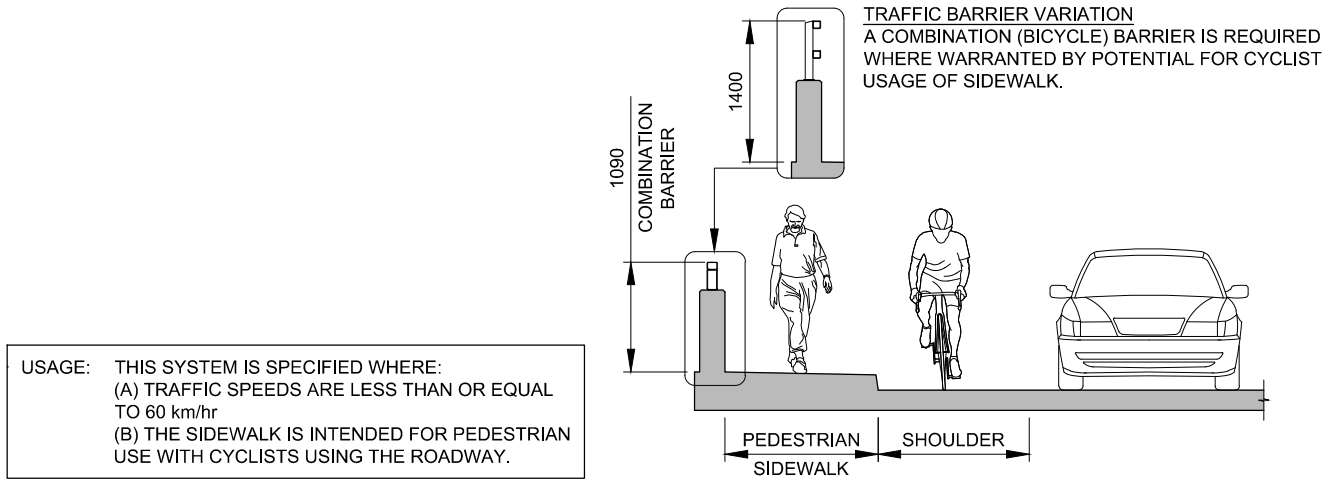


OPTION C
VEHICULAR & CYCLIST TRAFFIC

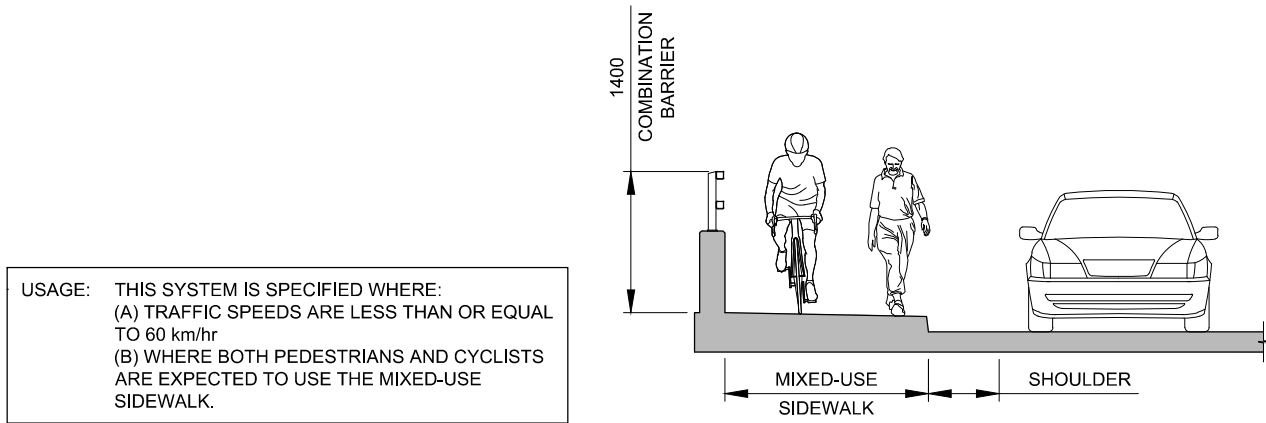
NOTES:

1. TRAFFIC BARRIER PROTECTION BASED ON THE MINISTRY’S STANDARD BRIDGE PARAPET. OTHER BARRIERS MAY BE ACCEPTABLE.
2. 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.
3. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS NOTED OTHERWISE.

CONFIGURATION #2 - BRIDGE WITH RAISED SIDEWALK



OPTION A
VEHICULAR & CYCLIST TRAFFIC ON ROADWAY & PEDESTRIAN TRAFFIC ON SIDEWALK



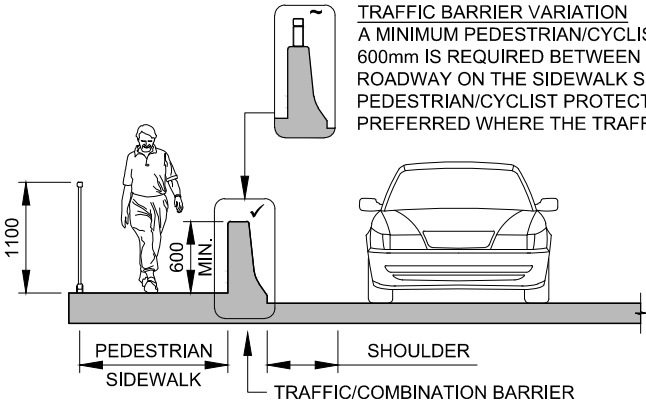
OPTION B
VEHICULAR TRAFFIC ON ROADWAY & PEDESTRIAN & CYCLIST TRAFFIC ON SIDEWALK

NOTES:

1. TRAFFIC BARRIER HAS VERTICAL FACE SINCE THE MINISTRY'S STANDARD BRIDGE PARAPET IS NOT DESIGNED TO BE MOUNTED ON A CURB. OTHER BARRIERS MAY BE ACCEPTABLE.
2. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS NOTED OTHERWISE.
3. CURBS SHOULD BE NO HIGHER THAN 200mm.

CONFIGURATION #3 - BRIDGE WITH SIDEWALK SEPARATED BY A BARRIER

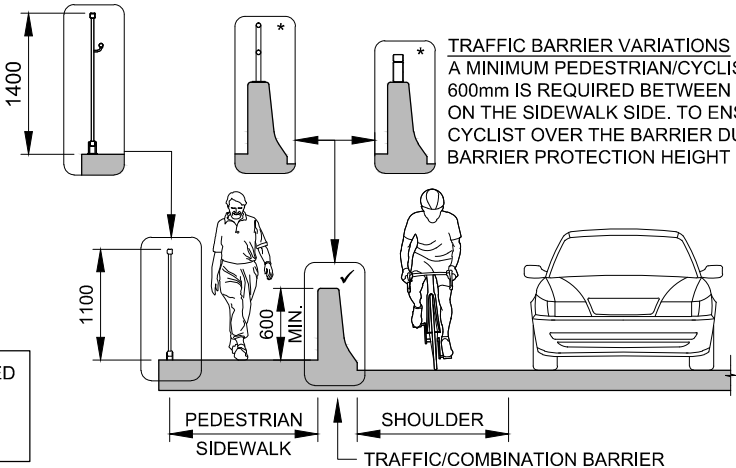
TRAFFIC BARRIER VARIATION
A MINIMUM PEDESTRIAN/CYCLIST HEIGHT PROTECTION OF 600mm IS REQUIRED BETWEEN THE SIDEWALK AND ROADWAY ON THE SIDEWALK SIDE. ADDITIONAL PEDESTRIAN/CYCLIST PROTECTION HEIGHT MAY BE PREFERRED WHERE THE TRAFFIC SHOULDER IS NARROW.



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

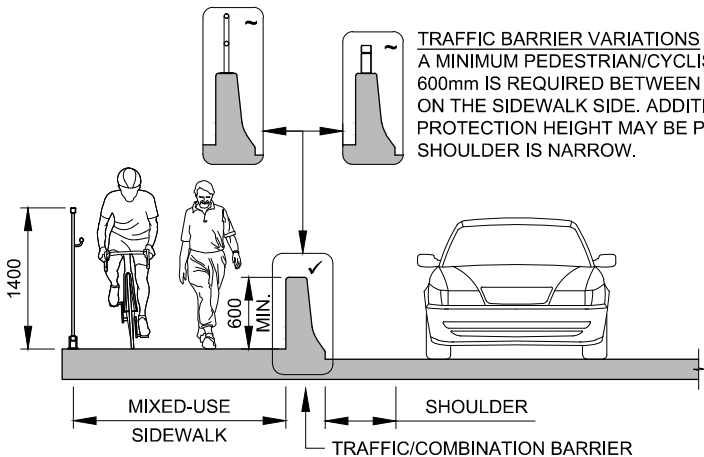
BICYCLE BARRIER VARIATION
WHERE THE POTENTIAL FOR CYCLISTS ON THE SIDEWALK IS HIGH, A BICYCLE BARRIER IS RECOMMENDED ALONG THE EDGE OF THE BRIDGE.



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 PREFERRED 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:
1. 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.
 2. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS NOTED OTHERWISE.

LEGEND	
✓	ACCEPTABLE CONFIGURATION
*	GENERALLY NOT AN ADVISABLE CONFIGURATION
~	MAY BE AN ACCEPTABLE CONFIGURATION

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13.6.2	Access for routine maintenance	2
13.6.7	Hydraulic cylinder connections.....	2
13.7	Mechanical system design	2
13.7.18	Bearings.....	2
13.7.18.4.3	Lubricated plain bearings	2
13.7.18.4.4.2	Non-metallic bearings	2
13.9	Electrical system design	2
13.9.2	General requirements for electrical installations	2
13.9.11	Electrical control systems	3
13.9.11.1	Operating sequence and interlocking requirements	3
13.9.11.2.3	Programmable logic controller (PLC)	3
Additional Requirements		3
	Operation and Maintenance Handbook	3
	Spare Parts	3

13.1 Scope

Movable bridges shall not be used unless Approved.

13.6 Structural analysis and design**13.6.2 Access for routine maintenance**

Commentary: The installation of elevators in tower-drive vertical lift bridges shall be considered for heights greater than 15 metres. This is to allow movement of maintenance materials to the hoisting equipment easily and effectively.

13.6.7 Hydraulic cylinder connections

Commentary: The design philosophy is that the hydraulic cylinder is supposed to be the weakest link, not the structural attachments to the bridge.

13.7 Mechanical system design**13.7.18 Bearings****13.7.18.4.3 Lubricated plain bearings**

Commentary: Self-lubricated non bronze bushings may be appropriate for some applications; however, their use is subject to consent by the Ministry.

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**

Commentary: Wires in rigid galvanized steel conduit is the preferred wiring method for bridges. Armoured cable with PVC jacket may be an acceptable alternative. External wiring to control panels and consoles shall be of type listed in CEC Standard (CAN/CSA 22.1), 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.

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.*

13.9.11.2.3 Programmable logic controller (PLC)

***Commentary:** The PLC shall be provided with a communication card installed to allow remote communication monitoring by the Ministry at its Provincial Control Centre.*

Additional Requirements**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.

Spare Parts

The list of spare parts that the Contractor must provide shall be included in the Contract Documents prepared by the designer on behalf of the Ministry. The list should be reviewed to include PLC and UPS spare parts.

14.2	DEFINITIONS	2
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14.9.1	Normal traffic	2
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14.2**Definitions**

Add the following definition:

Supervision – monitoring of the passage of an overload by a BC registered professional engineer familiar with bridge design to ensure bridge crossing restrictions in an overload permit are followed by the permit vehicle. Monitoring of the weighing of a permit vehicle is also to be performed if called for in the overload permit. The engineer shall have the authority to stop further movement of the permit vehicle if it is not in compliance with permit requirements. Records of vehicle weight and dimension measurements and of each bridge crossing by the permit vehicle shall be kept by the engineer and a report detailing these observations sent to the Ministry on completion of the move.

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://www.cisc-icca.ca/solutions-centre/technical-resources/technical-resources/historical-info>

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. The BCL-625 design loading shall not normally be used for evaluation.

Commentary: Loadings that differ from the CL1-W loadings specified in Section 14.9 may be specified by the Ministry on a project-to-project basis.

14.12 Target reliability index**14.12.1 General**

Commentary: Table 14.6 is based on the assumption that for PC traffic the number of people on and or under the bridge at the same time as the permit vehicle is to be minimized and preferably 3 or less people on the bridge.

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.*

Steel in tension at net section shall remain in Category E1 but, for evaluations, the new resistance adjustment factor specified under Clause 14.14.2 shall be applied to the axial tensile resistances determined in accordance with Clauses 10.8.2(b) and 10.8.2(c).

The axial tensile resistances for effective net sectional areas, A_{ne} and A'_{ne} , specified in Clause 10.8.2(b) and (c) contain a 0.85 reduction factor to account for the reduced warning of failure that may be provided if fracture occurs on the net section prior to yielding of the component on the gross section. The provisions of Clause 14.12.3 address the same issue by effectively increasing the factored loadings on components that provide little or no warning of failure.

The intent of both these provisions was to individually provide an additional margin of safety against this type of failure. Applying both of these provisions for evaluations results in the component being penalized twice for the same behaviour. To remove this double penalty, a new resistance adjustment factor has been developed to remove the reduction in the component resistance while maintaining the increased factored loadings. The new resistance adjustment factor is specified under Clause 14.14.2.

14.13.3 Transitory Loads

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 14.13) and the other lanes will get normal traffic live load factors (Table 14.9).

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

14.14.1.6 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:

$$\begin{aligned}V_f &= \alpha_D V_{DL} + F (\alpha_L V_{LL}) \\M_f &= \alpha_D M_{DL} + F (\alpha_L M_{LL})\end{aligned}$$

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

The size factor (k_{sv}) given in Clause 14.14.1.7.2, shall be applied to both sawn timber and glue-laminated beams. The value of longitudinal shear (f_{vu}) for glue laminated beams shall be taken from Table 9.15.

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 14.14.2.

14.17 Bridge posting**14.17.1 General**

Replace the third sentence of 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

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.*

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15.3.8	Seismic Upgrading	2
15.4	Special Considerations	2
15.6	Rehabilitation loads and load factors	2
15.6.1	Loads	2
15.6.1.3	Rehabilitation design live loads	2
15.6.1.3.2	Normal traffic	2
15.8	Resistance.....	3
15.8.1	Existing members	3
15.8.1.1	General.....	3

15.1 Scope

The scope of the investigation and rehabilitation of existing structures shall be defined or consented to by the Ministry.

Members evaluated in accordance with Section 14 that have adequate live load capacity do not need to be rehabilitated.

15.3 General requirements

15.3.8 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-S6-14 Section 4.

15.4 Special Considerations

Add the following:

- (u) Constraint induced fracture and fatigue
- (v) Rehabilitation to restore resistance compared to upgrading to current code requirements

15.6 Rehabilitation loads and load factors

15.6.1 Loads

15.6.1.3 Rehabilitation design live loads

15.6.1.3.2 Normal traffic

Delete the first paragraph and replace with:

The BCL-625 loading specified in the Ministry's Supplement to CHBDC-S6-14 Clause 3.8.3.1.2 shall be used for the rehabilitation design of bridges that are to carry unrestricted normal traffic after rehabilitation.

15.8**Resistance****15.8.1****Existing members****15.8.1.1****General**

Delete the first sentence and replace with the following:

The factored resistances of existing members, including existing members strengthened with new material, shall be determined in accordance with Clauses 14.14.1 and 14.14.2., except for steel in tension on the net section. For steel in tension on the net section, resistances shall be calculated in accordance with Clauses 10.8.2(b) and 10.8.2(c) and without applying the resistance adjustment factor from Table 14.15.

Commentary for steel in tension on the net section: *The axial tensile resistances for effective net sectional areas, A_{ne} and A'_{ne} , specified in Clause 10.8.2(b) and (c) contain a 0.85 reduction factor to account for the reduced warning of failure that may be provided if fracture occurs on the net section prior to yielding of the component on the gross section. The provisions of Clause 14.12.3 address the same issue by effectively increasing the factored loadings on components that provide little or no warning of failure.*

The intent of both these provisions was to individually provide an additional margin of safety against this type of failure. Applying both of these provisions for evaluations results in the component being penalized twice for the same behavior. To remove this double penalty, a new resistance adjustment factor has been developed to remove the reduction in the component resistance while maintaining the increased factored loadings. The new resistance adjustment factor is specified under Clause 14.14.2.

For rehabilitation, load factors from Clause 3 are specified rather than the load factors from Clause 14, therefore the resistance adjustment factor specified under Clause 14.14.2 does not apply.

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16.1 Scope**16.1.4 Uses requiring Approval**

Fibre Reinforced Polymer (FRP) products shall not be used unless consented to by the Ministry.

The following uses require Approval:

- Any fibre or matrix not listed in 16.1.2 or 16.1.3

16.4 Durability**16.4.3 Fibres in FRC**

Replace the second sentence with the following:

The use of other fibres shall not be considered by the Ministry.

16.4.6 Allowance for wear in deck slabs

Delete and replace with:

The requirement for an additional thickness of 10mm shall be waived by the Ministry.

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 allow stay-in-place formwork*

16.7.2 Full-depth cast-in-place deck slabs

Delete Item (a) and replace with the following:

The top flanges of all adjacent supporting beams shall be connected by straps that are perpendicular to the supporting beams and either connected directly to the tops of the flanges, as in the case of the welded steel straps shown in

Figure 16.6, or connected indirectly, as in the case of the partially studded straps shown in Figure 16.7.

Commentary: *The Ministry does not allow stay-in-place formwork and therefore stay in place formwork is not an Approved transverse confining system.*

16.7.3 Cast-in-place deck slabs on stay-in-place formwork

The clause is deleted in its entirety.

Commentary: *The Ministry does not allow stay-in-place formwork.*

16.7.4 Full-depth precast concrete deck slabs

The clause is deleted in its entirety.

16.11 Rehabilitation of existing concrete structures with FRP

16.11.3 Shear rehabilitation with externally bonded FRP systems

16.11.3.2 Factored shear resistance

Replace formula for factored shear resistance provided by the FRP shear reinforcement (V_{FRP}) with the following:

$$V_{FRP} = \frac{\phi_{FRP} E_{FRP} \epsilon_{FRP} A_{FRP} d_{FRP} (\cot\theta + \cot\alpha) \sin\alpha}{S_{FRP}}$$

Note – There are no supplemental clauses for Section 17 of S6-14.