Preface

The BC Ministry of Transportation Supplement to CAN/CSA S6-06 is to be read and utilized in conjunction with the CAN/CSA S6-06 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 CAN/CSA S6-06 Canadian Highway Bridge Design Code.
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# Supplement to CHBDC S6-06

## Section 1 - General

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1.1 Scope

1.1.1 Scope of Code

The Canadian Highway Bridge Design Code, CAN/CSA-S6-06 (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, CAN/CSA-S6-06 (CHBDC) shall apply for the design and construction of Ministry bridges and other Ministry structure types that are referenced in the scope of CHBDC.

The “BC Ministry of Transportation Supplement to the Canadian Highway Bridge Design Code, CAN/CSA-S6-06” (Supplement to CHBDC S6-06) shall also apply for the 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 S6-06 and the CHBDC, the Supplement to CHBDC S6-06 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 S6-06 or the CHBDC, on the other hand, the Project specific Contracts and Terms of Reference shall take precedence over the Supplement to CHBDC S6-06 or the CHBDC, as the case may be.

1.3 Definitions

1.3.2 General administrative definitions

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

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.

1.3.3 General 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 Geometric Design Guide for Canadian Roads.

CHBDC: means the Canadian Highway Bridge Design Code CAN/CSA-S6-06.

Design-Build Standard Specifications (DBSS): means the BC Ministry of Transportation 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.

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 S6-06 and includes a Provincial public undertaking, within the meaning of the Transportation Act, S.B.C. 2004, c. 44.

Low Volume Road Structure (LVR): 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 500 vehicles per day.

Ministry: means the BC Ministry of Transportation or a bridge engineer employed by the Ministry of Transportation who has the authority, responsibility and technical expertise to affect changes to the Supplement to S6-06 as allowed herein.

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 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 link is as follows:


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.

SPZ: means Seismic Performance Zone

Standard Specifications (SS): means the BC Ministry of Transportation 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 bridge and highway construction projects unless otherwise specified.

S6-00: means the Canadian Highway Bridge Design Code CAN/CSA-S6-00

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

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.

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

1.4 General requirements

1.4.1 Approval

Exemptions from the Supplement to CHBDC S6-06, including for the purpose of application of codes other than S6-06, 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) MSE walls with dry cast concrete block facings;

i) Walls with wire facings for median walls and upslope retaining walls visible to road traffic;

j) MSE walls with polymeric reinforcement used as abutment walls or wing walls;

k) FRP products;

l) Polymer composite based structural products;

m) Welded shear keys for box beams; and,

n) Discontinuous spans between substructure elements

1.4.2 Design

1.4.2.3 Design life

For any calculations which are time dependent (including 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.

1.4.2.8 Aesthetics

General guidelines for bridge aesthetics are set out in the Ministry’s Manual of Aesthetic Design Practice.

1.4.4 Construction

The Ministry SS, DBSS and SP for bridge construction take precedence over this section of S6-06.
1.4.4.3 Construction methods

Commentary: Reference the Ministry Bridge Standards and Procedures Manual - Volume 2 Procedures and Directions, for guidelines associated with transportation of bridge girders in BC.

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 S6-06 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.

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

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

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 Regional Highway Designer or designate. In the case of bridge structures that are greater than 300 m in length, consideration may be given to reducing the stipulated shoulder width on the structure.

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

<table>
<thead>
<tr>
<th>Type of Traffic</th>
<th>Direction</th>
<th>Minimum Width (metres)</th>
</tr>
</thead>
<tbody>
<tr>
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<td>Bi-directional</td>
<td>1.5&lt;sup&gt;1&lt;/sup&gt;</td>
</tr>
<tr>
<td>Pedestrian Only</td>
<td>Bi-directional</td>
<td>1.8&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td>Pedestrian and Cycle</td>
<td>Uni-directional</td>
<td>2.5&lt;sup&gt;3&lt;/sup&gt;</td>
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<tr>
<td>Pedestrian and Cycle</td>
<td>Bi-directional</td>
<td>3.5&lt;sup&gt;3&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

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 with Approval.

1.5.2.2 Clearances

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 meters. If the pathway is designated for shared equestrian use, the clearance shall be increased to 3.5 metres.

Long-term settlement of supports, superstructure deflection and 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.
1.5.2.3 **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.

*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**

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1.6 **Barriers**

1.6.1 **Superstructure barriers**

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

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. A clear cover of 70 mm shall be used for the top reinforcing bars.

Approach slabs shall be designed as a one-way slab in the longitudinal direction to support CL-W loading. 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 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 for structures in Seismic Performance Zones 3 and 4.

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.
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 desirable 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.

The last paragraph is deleted and replaced 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 facia can lead to freeze-thaw durability problems in colder climates.

### 1.8.2.2 Deck finish

Concrete bridge decks shall be textured by tining in accordance with SS 413.31.02.05. The tining shall create transverse grooves 3 mm wide by 1.5 mm to 3 mm deep at 20 mm centre to centre spacing. Concrete bridge decks which receive a waterproof membrane and asphalt topping shall have a smooth float finish. Sidewalks shall receive a transverse broom finish.

### 1.8.2.3 Drainage systems

#### 1.8.2.3.1 General

This clause is amended such that the maximum encroachment on to the traffic lanes shall be limited to 1.2 m. Future settlement shall be considered when locating drains.

#### 1.8.2.3.3 Downspouts and downpipes

The use of scuppers requires Ministry approval.

**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.

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

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.

**Commentary:** Support brackets may be required for girders and steel trusses deeper than 2.3 m.

Add to the fifth paragraph:
The position and length of discharge pipes shall be such that water falling at an angle of 45° to the vertical does not touch any part of the structure.

Typical downspout details are shown in the following figures:

**Figure 1.8.2.3.3a**

Deck drain setting detail
Figure 1.8.2.3.3b

Deck drain fabrication detail

- 3 - 50 x 10 bars (equally spaced)
- 3 - 38 x 10 bars (equally spaced)
- Pipe 219.1 O.D. x 7.95
- 500 min. below underside of pier cap or stringer
1.8.2.5 Runoff and discharge from deck

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 washouts at the ends of the wingwalls.

1.8.3 Maintenance

1.8.3.1 Inspection and maintenance access

1.8.3.1.1 General

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

I-Girder Bridges (Steel or Prestressed Concrete) 450 mm
Box Beam Bridges 600 mm

Reference Clause 8.20.7 for end diaphragm details to facilitate inspection and maintenance.

Figure 1.8.3.1

Abutment berm detail
1.8.3.1.2 Removal of formwork

All other formwork shall be removed.

Partial depth precast panels acting compositely with the concrete deck shall not be considered as formwork.

1.8.3.1.3 Superstructure accessibility

Unless otherwise directed by the Ministry, access to steel girders for inspection purposes shall be incorporated into the design with devices to enable inspectors to walk along both faces of all girders and tie-off safely in accordance with Work Safe BC Occupational Health and Safety Regulations (OHS). Tie-off devices should be galvanized and designed such that the devices require a minimum level of maintenance and inspection. Tie-off devices shall be located 1.5 metres above the bottom flange with no slack permitted over its length.

1.8.3.1.5 Access to primary component voids

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 dead load and full 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).

1.9.1.2  Normal design flood

Delete and replace the first paragraph with the following:

The return period for the design flood is as follows:

<table>
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<th>Return Period</th>
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<tr>
<td>Bridges</td>
<td>200-year</td>
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<td>Buried Structures</td>
<td>200-year</td>
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<tr>
<td>Culverts (≥3m Span)</td>
<td>200-year</td>
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<tr>
<td>Low-Volume Road</td>
<td>100-year</td>
</tr>
</tbody>
</table>

**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 and air photos, refer to:


Low-volume roads shall be considered as side roads with an average daily traffic ADT (for a period of high use) total in both directions, not exceeding 500 vehicles per day. The service function of low-volume roads is usually oriented towards local rural roads, recreational roads, and resource development roads. Numbered Routes shall not be considered as low-volume roads for hydraulics design purposes.

For additional information, 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

Delete paragraphs since these are not applicable to the Ministry.
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
   - Rational Method Analysis

Commentary: The most commonly used distributions to describe extreme flows are:
   - Extreme Value Type 1 (Gumbel)
   - Three Parameter Lognormal
   - Log Pearson Type 3

The Ministry generally uses the Log Pearson Type 3 distribution. 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 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

1.9.4.1 Scour calculations

Hec-Ras numerical analysis is approved for the computation of general and local scour based on the $D_{50}$ and $D_{90}$ streambed particle sizes.

Commentary: The sieve analysis is used for determining the streambed particle sizes, where:

\[
D_{50} = \text{Bed material particle size in a mixture of which 50\% are smaller.}
\]

\[
D_{90} = \text{Bed material particle size in a mixture of which 90\% are smaller.}
\]

1.9.4.2 Soils data

If the Hec-Ras numerical analysis is used, the $D_{50}$ and $D_{90}$ streambed particle sizes shall be determined.

Commentary: The sieve analysis is used for determining the streambed particle sizes, where:

\[
D_{50} = \text{Bed material particle size in a mixture of which 50\% are smaller.}
\]

\[
D_{90} = \text{Bed material particle size in a mixture of which 90\% are smaller.}
\]

1.9.5 Protection against scour

1.9.5.2 Spread footings

Abutments and piers subject to potential scour shall have piled foundations, unless otherwise Approved.

Commentary: Use of spread footings for abutments and piers may be considered acceptable on low-volume roads or in other special circumstances, provided a risk review acceptable to the Ministry is carried out to satisfy the use.

1.9.5.2.2 Protection of spread footings

Riprap and MSE walls shall not be considered as an “Approved means” for protecting the bottom of spread footings against scour.

Commentary: The use of riprap may be considered as an “Approved means” on low-volume road bridges, if consented to by the Ministry.
1.9.5.5 Protective aprons

Riprap shall conform to the clauses in Section 205, of the Ministry Standard Specifications for Highway Construction. The gradation of the class of riprap shall be in accordance to Table 205-A of those specifications.

The class of riprap used shall be based on the design chart available in the BC Ministry of Transportation, Supplement to TAC Geometric Design Guide, (June 2007), Section 1030, Figure 1030A.

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.

Commentary: Clearances shall be increased for crossings subject to ice flows, debris flows and debris torrents. For waters Transport Canada declares to be navigable, a vertical clearance capable of allowing passage of the largest air draft vessel at the 100-year flood level or the HHWLT (Higher High Water, Large Tide) shall be provided. 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 by Transport Canada 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. 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 Coordinator.

For additional information, refer to Volume 2 Procedures and Directions.
1.9.9 Channel erosion control

1.9.9.3 Slope revetment

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.

The riprap design chart is available in the BC Ministry of Transportation, Supplement to TAC Geometric Design Guide (June 2007), Section 1030, Figure 1030A.

1.9.11.2 Culvert end treatment

Cut-off walls shall be used at both ends of the culvert, 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 are required at both ends for closed-bottom type soil-metal structures. Collar walls are required at both ends for open-bottom type soil-metal structures.
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  2.3.2.6 Drainage ....................................................................................................................... 3
  2.3.2.7 Utilities ......................................................................................................................... 3

2.4 Aluminum .................................................................................................................................. 3
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    2.4.2.2 Inert separators ........................................................................................................... 3

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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

Joints shall be designed such that they can be easily accessed for flushing, maintenance, inspection and repair.

Commentary: Joint seals shall be assessed for serviceability throughout the full temperature range at the site. Only products listed in the Ministry Recognized Product List, or as Approved otherwise, shall be incorporated into the work. Refer to http://www.gov.bc.ca and click sequentially on “Ministry and Organizations”, “Transportation”, “Report and Publications”, “Engineering Publications”, “Geotechnical and Pavement Engineering Publications”, and “Recognized Product List”.

2.3.2.5.2 Joints in abutments, retaining walls, and buried structures

Typical details for concrete control joints are shown in Figure 2.4.2.5.2.

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.

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

Unless otherwise consented to by the Ministry, all bridges in the South Coast Region shall have waterproofing membrane and 100 thick asphalt overlay on top of the bridge deck in accordance with SS419.

Only products listed in the Ministry Recognized Products List, or as Approved otherwise, shall be incorporated into the work.


2.8 Backfill material

A drainage course shall be provided along the backside of all foundation walls located in cut providing positive drainage through 100 mm weep holes provided a minimum 3.0 m spacing along the footing line. Drains are not required for abutments located on compacted standard granular bridge end fills.
The gradation of drainage course material shall be as follows:

<table>
<thead>
<tr>
<th>Sieve Size (mm)</th>
<th>Passing Per Nominal Maximum Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>100</td>
</tr>
<tr>
<td>20</td>
<td>0 - 100</td>
</tr>
<tr>
<td>10</td>
<td>0</td>
</tr>
</tbody>
</table>

2.9 **Soil and rock anchors**

Soil and rock anchors permanently incorporated into the structure shall be provided with a double corrosion protection system.

2.10 **Other materials**

Premoulded joint fillers on bridge structures shall consist of minimum 25 thick Evazote 50, or equal as consented to by the Ministry, and shall be applied in accordance with the manufacturer’s instructions.
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3.6 Dead loads ................................................................................................................... 2
3.8 Live loads .................................................................................................................... 3
3.8.3 CL-W loading ............................................................................................................. 3
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3.8.3.2 CL-W Truck ........................................................................................................ 3
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3.5 Load factors and load combinations

3.5.1 General

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

<table>
<thead>
<tr>
<th>Loads</th>
<th>Permanent Loads</th>
<th>Transitory Loads</th>
<th>Exceptional Loads</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D</td>
<td>E</td>
<td>P</td>
</tr>
<tr>
<td>Ultimate Limit States‡</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ULS Combination 5A***</td>
<td>(\alpha_D)</td>
<td>(\alpha_E)</td>
<td>(\alpha_P)</td>
</tr>
</tbody>
</table>

*** For long spans in Seismic performance zones 3 and 4, either continuous or semi-continuous for live load, with any one span or combination of spans greater than 200 metres in length. \(\lambda\) shall be equal to 0.50 unless consented to otherwise 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, \(\alpha_D\), 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, \(\alpha_D\), 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, \(\alpha_D\), should be taken as 1.0.

3.6 Dead loads

Dead loads shall include an allowance for a future 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 minimum dead load for design shall include the design asphalt thickness or 100 mm of asphalt, whichever is greater.

Add to **Table 3.3 Unit material weights** the following:

<table>
<thead>
<tr>
<th>Material</th>
<th>Unit Weight, kN/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood</td>
<td></td>
</tr>
<tr>
<td>Untreated Douglas Fir</td>
<td>5.4</td>
</tr>
<tr>
<td>Creosote treated sawn timber and glulam, &gt;114 mm</td>
<td>6.6</td>
</tr>
<tr>
<td>Creosote treated truss chords, &lt; 114 mm</td>
<td>7.0</td>
</tr>
</tbody>
</table>

**Commentary:** There is no reference to treated timber dead weight for Douglas Fir.

### 3.8 Live loads

#### 3.8.3 CL-W loading

##### 3.8.3.1 General

BCL-625 is the designated live load for all BC bridges unless Approved otherwise.

##### 3.8.3.2 CL-W Truck

Delete the third paragraph and replace with the following:

In BC, a BCL-625 Truck, as detailed in Figure 3.2(a) shall be used.

**Note:** The total load of the BCL-625 Truck is 625 kN, but the axle load 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.
**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 traffic. CL-625 Loading is inadequate in short spans for Cranes and medium length continuous spans in moment for 85 tonne Class Permit Vehicles.

### 3.8.3.3 CL-W Lane Load

Delete the second paragraph and replace with the following:

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

---

<table>
<thead>
<tr>
<th>Axle No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wheel loads, kN</td>
<td>25</td>
<td>70</td>
<td>70</td>
<td>87.5</td>
<td>60</td>
</tr>
<tr>
<td>Axle loads, kN</td>
<td>50</td>
<td>140</td>
<td>140</td>
<td>175</td>
<td>120</td>
</tr>
</tbody>
</table>

V = Variable Spacing - 6.6m to 18m inclusive. Spacing to be used is that which produces the maximum stresses.
**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 traffic. CL-625 Loading is inadequate in short spans for Cranes and medium length continuous spans in moment for 85 tonne Class Permit Vehicles.

### 3.8.4 Application

#### 3.8.4.5 Dynamic load allowance

#### 3.8.4.5.1 General

The use of dynamic load allowance factors other than specified requires written Approval.

### 3.13 Earthquake effects

Delete the second sentence and replace with the following:

The designer is to obtain specific site acceleration values, as referenced in Clause 4.4.3 of this Supplement.

### 3.14 Vessel collisions

#### 3.14.2 Bridge classification

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

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 x 10^{-4} for ships and 1.2 x 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 2007 and are based on analysis of historical data for high use waterways.
# Section 4 Seismic design

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4.2 Definitions

*Pile bent* - Gravity and lateral load resisting substructure comprising piles that extend above grade, without an at-grade pile cap, connecting directly to a pier cap beam supporting the bridge superstructure.

4.4 Earthquake effects

4.4.1 General

Delete the third paragraph and replace with the following:

Design of lateral load resisting substructures in Seismic Performance Zone (SPZ) 3 and 4 shall use capacity design principles.

Earthquake load effects in Ductile Substructure Elements shall be determined from the inelastic action of members with which they connect.

Elastic design forces may be used for:

- structures in Seismic Performance Zone (SPZ) 1 or 2;
- bridges with seismic base isolation.
- abutments walls in the strong direction, and;
- wall piers in the strong direction.

**Commentary:** While clause 4.4.8.1 requires design and detailing for all lateral load-resisting sub-structures, it is recognized that stiff substructures may have significantly greater strength than required for seismic loads. In such cases elastic design forces may be appropriate. Detailing requirements referenced in Table 4.5 remain as in S6-06.

For base isolated bridges the substructure and other elements in the seismic load path are to be designed as capacity protected elements, with force and deformation demands from the isolation devices being scaled up, analogous to over-strength demands on ductile substructures, with appropriate margins to avoid unintended failure modes.

4.4.2 Importance categories

Add the following paragraph immediately before the last paragraph:

Lifeline bridges in SPZ 3 and SPZ 4 shall be explicitly designed to ensure the above performance requirements are met for the 10% in 50 year and 5% in 50 year seismic events unless directed otherwise by the Ministry.
Add the following sentences to the end of this clause:

The Ministry will designate the Importance Category for each bridge.

Structures in regions having PHA < 0.08, and which are classified as "lifeline" within the project specific requirements, shall be designed as if they are "emergency route" bridges in SPZ 2.

**Commentary:** It is appropriate to design lifeline structures in areas of low seismicity with an Importance Factor (I) of 1.5 instead of 3.

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

The following relationship is used to relate probabilities of exceedance and return periods:

\[ R = \left[ 1 - (1 - p)^{1/t} \right]^{-1} \]

Where:

- \( R \) = return period
- \( p \) = probability of exceedance in period \( t \)
- \( t \) = duration consistent with \( p \) (e.g. 1 year for an annual probability of exceedance)

Dividing the period \( t \) by the annual probability of exceedance provides an approximation of the return period.

### 4.4.3 Zonal acceleration ratio

Delete the first paragraph and replace with the following:

The zonal acceleration ratio, \( A \), to be used in the application of these provisions shall be determined from the most current site specific 10% in 50 year PHA values obtained from the Geological Survey of Canada (GSC) either directly, through their Pacific Geoscience Centre (PGC) in Sidney, BC or from their on-line web page at:


**Commentary:** The GSC site may refer to the PGA rather than the PHA. The Pacific Geoscience Centre in Sidney, B.C. can be contacted at:

[www.pgc.nrcan.gc.ca/index_e.html](http://www.pgc.nrcan.gc.ca/index_e.html)

Phone: (250) 363-6500  Fax: (250) 363-6565.
Delete Table 4.1 – Seismic performance zones and replace with the following:

**Table 4.1**

**Seismic performance zones**

(see Clauses 4.4.3, 4.4.4, 4.6.6, 4.10.2, 4.10.3, and 4.10.6.2.1)

<table>
<thead>
<tr>
<th>PHA or A, g, for 10% probability of exceedance in 50 years</th>
<th>Lifeline bridges (see Clause 4.4.2)</th>
<th>Emergency-route and other bridges (see Clause 4.4.2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00 ≤ A &lt; 0.04</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>0.04 ≤ A &lt; 0.08</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>0.08 ≤ A &lt; 0.11</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>0.11 ≤ A &lt; 0.16</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>0.16 ≤ A &lt; 0.23</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>0.23 ≤ A &lt; 0.32</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>0.32 or greater</td>
<td>4</td>
<td>4</td>
</tr>
</tbody>
</table>

**4.4.4 Seismic performance zones**

Delete the first paragraph and replace with the following:

Bridges shall be assigned to one of the four seismic performance zones in accordance with Table 4.1 using the zonal acceleration ratio, A, obtained from the site specific values obtained from Clause 4.4.3.
4.4.5 Analysis for earthquake loads

4.4.5.1 General

Delete the second paragraph and replace with the following:

For modal methods of analysis specified in Clause 4.4.5.3, the elastic design spectrum shall be that given by the equations in Clause 4.4.7.

4.4.6 Site effects

Commentary: Soil profile classifications are relatively broad and generic in S6-06, and Clause 4.4.6.6 allows for engineering judgment. Additional guidance may be found in technical references supporting the proposed NBCC2005 code, ATC-32, and ATC – 49. Comparison of soil classifications considering soil types, thicknesses, and shear wave velocities are useful.

4.4.8 Response modification factors

4.4.8.1 General

Commentary: This clause outlines the use of R factors for the design of ductile substructures and provides simplifying assumptions for the design of superstructures having concrete decks.

Table 4.5 includes R factors for ‘pile bents’ (see definitions in the MoT Supplement, Clause 4.2). The Commentary to S6-06 clarifies that the given R-factors should be acceptable for inelastic hinges that form in "reasonably accessible" locations, described as, “…… less than two metres below ground or mean water or tide level”. This is regarded by the Ministry as a reasonable guideline for “reasonably accessible”.

For the purposes of this clause, R factors identified for “pile bents” may also be applied to ductile piles, i.e. appropriately detailed steel, concrete or composite piles, used as part of integral abutments.

S6-06 and the Commentary are clear that R factors are used to modify bending moments in ductile sub-structure elements. S6-06 Clause 4.4.10 relates to the design forces and detailing as part of a ‘capacity design’ approach, and allows axial loads from either the elastic analyses or as found from the plastic mechanism. For the initial sizing of yielding ductile substructure elements, typically the columns, neither document is explicit on the appropriate axial loads to adopt. The designer must use engineering judgment. Some commonly considered options for axial loads for this purpose include:

i) Those obtained from the elastic analyses, unreduced by the R factor. A designer might assume the seismic axial loads should be taken as either positive or negative to achieve the most conservative design.
ii) Those consistent with the plastic mechanism at probable member resistances. Axial loads in columns of multi-column bents will vary, and the maximum and / or minimum axial load may be inferred.

iii) Those consistent with the plastic mechanism at nominal member resistances.

iv) Those consistent with the plastic mechanism at factored member resistances.

v) Those associated with dead loads only, i.e. changes in axial loads from seismic demands being neglected during column sizing.

It is less important which axial loads are adopted for member sizing than for the subsequent derivation of demands on capacity-protected elements. Minor variations in overall system ductility capacity may be expected for the range of assumptions noted above. Axial loads based on (i) or (ii) above could result in unnecessary conservatism. Care should be taken to ensure that the benefits of capacity design are not made economically or otherwise impractical as a result of sizing columns using unreasonably conservative assumptions regarding column axial loads.

4.4.10 Design forces and support lengths

4.4.10.4 Seismic performance zones 3 and 4

4.4.10.4.2 Modified seismic design forces

Delete the second paragraph and replace with the following:

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 resistance.

4.4.10.4.3 Yielding mechanisms and design forces in ductile substructures

Delete the third paragraph and replace with the following:

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

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

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

For cases of structures where elastic design forces may be used for capacity-protected elements in accordance with Clause 4.4.10.4.2, shear and axial design forces for ductile substructure elements may be taken as the unreduced elastic design forces in accordance with Clause 4.4.9 with \( R = 1.0 \) and \( I = 1.0 \).

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

4.5 Analysis

4.5.1 General

Sway effects shall be considered where appropriate in all bridge substructures.

**Commentary:** Guidance on when and how to incorporate P-Delta effects can be found in ATC – 32 Clause 3.21.15.

4.5.3 Multi-span bridges

4.5.3.4 Time-history method

Delete the first paragraph and replace with the following:

The time-histories of input acceleration used to describe the earthquake loads shall be selected by the designer and subject to Approval. Three or more sets of time history records shall be used, each set comprising three orthogonal records. The design response quantities will be taken as the maximum from the three analyses. If five or more record sets are used, the design quantities may be taken as the mean from the five or more analyses. If site specific time-histories are used, then they shall include the site soil profile effects and be modified by the importance factor, \( I \).

**Commentary:** Time history methods are not required for the design of most new highway bridges in B.C. Where time history methods are proposed, the design benefits should be clearly outlined, and the number and characteristics of the records should be developed in consultation with the Ministry. The above shall be fully described in a project specific design criteria developed by the designer.
### 4.5.3.5 Static pushover analysis

**Commentary:** Static push-over analyses are used to define the sequence of development of inelastic action in ductile structures, to develop member design forces for 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.

*Guidance is available in Priestley and Calvi, SSRP91/03 (UC San Diego), and ATC – 32, ATC - 49. The use of push-over analyses should also be considered to confirm the expected performance of important new or existing bridges under long return period events.*

### 4.6 Foundations

#### 4.6.5 Soil-structure interaction

Delete and replace with the following:

Soil-structure interaction analysis is required for lifeline and emergency route bridges in SPZ 2 and for all bridges in SPZ 3 and SPZ 4. For bridge designs that include soil-structure interaction, geotechnical input shall be obtained.

Dynamic soil-structure interaction shall be performed for retaining walls supporting 5 m or more of soil. Analysis software shall be used that is capable of taking into consideration non-linear soil and structure behavior and the input ground motions.

Upper and lower bound values shall be considered in soil-structure interaction analysis to account for uncertainties in soil properties and analysis methodologies.

**Commentary:** Soil-structure interaction should be included unless the merit or values of such analyses are expected to be minor. Among the potential benefits from such analyses would be an improved estimate of seismic deformations, a reduction of effective seismic input motion, and improved estimates of demand distributions among piers and abutments.

#### 4.6.6 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.
4.7 Concrete structures

4.7.3 Seismic performance zone 2

Delete the second sentence and replace with the following:

The transverse reinforcement at potential plastic hinge zones of beams and columns shall be as specified in Clauses 4.7.4.2.5 and 4.7.4.2.6.

4.7.4 Seismic performance zones 3 and 4

4.7.4.2 Column requirements

4.7.4.2.4 Column shear and transverse reinforcement

Delete Item (b) and replace with the following:

(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; or
iv) The length over which the moment exceeds 70% of the maximum moment

For tall piers, rational analysis, which considers potential plastic hinging mechanisms, shall be performed to determine the location and extent of plastic hinge regions.

4.7.4.2.7 Splices

Delete the second paragraph and replace with the following:

Lap splices in longitudinal reinforcement shall not be permitted in plastic hinge regions. The plastic hinge region shall be as defined in Clause 4.7.4.2.4 (b). Where practical, such lap splices shall be located within the centre half of column height. The splice length shall not be less than the greater of 60 bar diameters or 400 mm. The centre-to-centre spacing of the transverse reinforcement over the length of the splice shall not exceed the smaller of 0.25 times the minimum cross-section dimensions of the component or 100 mm.

Commentary: Splices should be limited to the centre half of columns where standard bar lengths can be accommodated without adding unnecessary extra splice cost.
Welded splices are not allowed unless consented to by the Ministry. Mechanical connection splices in accordance with Clause 8.4.4.4 may be used if not more than alternate bars in each layer of longitudinal reinforcement are spliced at a section and the distance between splices of adjacent bars is greater than the larger of 600 mm or $40d_b$ measured along the longitudinal axis of the column.

**4.7.4.4 Column connections**

Delete the second paragraph and replace with the following:

For lifeline and emergency route bridges in SPZ 3 and SPZ 4, the design of column connections, including member proportions, details, and reinforcement, shall be based on beam-column joint design methodologies as described in either:

- ATC-32 Section 8.34
- Caltrans Seismic Design Criteria (latest version, currently 1999)
- ATC-49 Section 8.8.4

For bridges in SPZ 2, or for “other ‘bridges’ in SPZ 3 and SPZ 4, column transverse reinforcement as specified in Clause 4.7.4.2.5, shall be continued full depth through the adjoining component, unless designed as specified above.

**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.

**4.7.5 Requirement for piles**

**4.7.5.4 Seismic Performance Zones 3 and 4**

**4.7.5.4.1 General**

For bridges in SPZ 3 and 4 and where plastic hinging may reasonably be expected to form, concrete piles shall be designed and detailed as ductile components so as to ensure performance similar to concrete columns designed to Section 4.7.
4.8 Steel structures

4.8.3 Sway stability effects

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

4.10 Seismic base isolation

4.10.1 General

For designs using base isolation, the Designer shall submit to the Ministry for review and acceptance, a seismic design criteria document outlining methodology, including key aspects and assumptions upon which the design is based. This shall include bearing types, properties, potential suppliers, recommended test requirements and acceptance criteria. Information on soil profiles, design response spectra, firm ground and soft soil time history records as required, and how displacements are accommodated at expansion joints, shall also be provided.

Testing of the isolation systems shall be in accordance with the 1999 edition (including 2000 interim) of the AASHTO Guide Specifications for Seismic Isolation Design.

**Commentary:** Clause 4.10 “Seismic Base Isolation” of S6-06 is mainly based on the 1991 edition of the AASHTO Guide Specifications for Seismic Isolation Design. Significant changes have been made in the 1999 edition (including 2000 interim edition) of the AASHTO Guide Specifications for Seismic Isolation Design. The testing requirements in the 1999 AASHTO Guide Specifications are more stringent than those in the 1991 edition. Therefore, the more stringent testing requirements of the 1999 AASHTO Guide Specifications are adopted here.

In the 1999 AASHTO Guide Specifications, three types of tests are clearly identified and required: (a) system characterization tests; (b) prototype tests; and (c) quality control tests. For example, for quality control tests of elastomeric bearings, Clause 4.10 of S6-06 (or the 1991 AASHTO Guide Specifications) requires combined compression and shear tests on 20% of the bearings whereas the 1999 AASHTO Guide Specifications requires such testing on all bearings.

The more stringent testing requirements in the 1999 AASHTO Guide Specifications are intended to ensure that all fabricated isolation bearings meet the specified design properties, and the isolated systems will perform as designed in the event of a major earthquake. After test set-up, the additional cost of testing all bearings versus 20% of the bearings for the combined compression and shear test would not be significant. This is because the combined compression and shear test for each bearing is relatively fast. Both
S6-06 (or the 1991 AASHTO Guide Specifications) and the 1999 AASHTO Guide Specifications require a sustained proof load test on each bearing for 1.5 times the maximum dead load plus live load as part of the quality control tests.

S6-06 (or the 1991 AASHTO Guide Specifications) requires the duration of 12 hours for each sustained proof load test whereas the duration is reduced to 5 minutes in the 1999 AASHTO Guide Specifications. Previous experience indicates that any bulging suggesting poor laminate bond will show up almost immediately after application of the vertical load, and the requirement for a 12 hour duration test is not necessary.

Bearing suppliers and contractors like this trade off between reduction in time for the sustained proof load test and increase in number of bearings for the combined compression and shear test. This is because the time required for quality control test is reduced significantly.

### 4.10.4 Site Effects and Site Coefficient

Delete the asterisked sentence under Table 4.10.4 and replace with the following:

Site specific studies shall be performed for bridges for which isolation systems are proposed on Type IV soils.

**Commentary:** Site specific spectra for soft soils may show that isolation is not effective. A realistic assessment of non-linear deformations of the isolated system, and the potential for unintended inelastic deformations in sub-structures, requires realistic soil spectra and analysis of soil-structure interaction.

### 4.10.6 Analysis procedures

#### 4.10.6.1 General

Foundation flexibility and other relevant soil-structure interaction effects shall be considered in analyses, and shall be included for structures founded on Soil Profile Types III and IV.

### 4.10.7 Clearance and design displacement of seismic and other loads

Allowance shall be made for thermal deformation demands in combination with seismic isolation deformation demands on joint, bearing and railing details unless otherwise Approved. 40% of the thermal deformation demands shall be combined with deformation demands from the base isolation system.
4.11 Seismic evaluation of existing bridges


4.12 Seismic rehabilitation

Clause 4.12 and all subsections shall be deleted. Seismic rehabilitation (retrofit) design shall be in accordance with the Ministry’s Bridge Standards and Procedures Manual – Volume 4, “Seismic Retrofit Design Criteria.”
5.5 Requirements for specific bridge types

5.5.5 Rigid frame types and integral abutment types

5.5.5.2 Integral structures

5.7 Live load

5.7.1 Simplified methods of analysis

5.7.1.1 Conditions for use of simplified analysis
5.5 Requirements for specific bridge types

5.5.5 Rigid frame types and integral abutment types

5.5.5.2 Integral structures

Design 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.

Integral abutments shall not be constructed on spread footings founded on or keyed into rock.

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

The maximum skew angle for integral abutment designs shall be 30°. Skew angles greater than this shall preclude the use of integral abutment bridge construction.

Design 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].
- 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

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.
5.7 Live load

5.7.1 Simplified methods of analysis

5.7.1.1 Conditions for use of simplified analysis

Add to Item (j):

Bridges comprised of twin cell Ministry standard concrete box stringers are categorized as multi-spine bridges that sufficiently meet the conditions for use of the simplified analysis approach.

Add to Table 5.7.1.1 Group (2) Multi-spine Bridges:

Twin cell Ministry standard concrete box stringers are defined as shear-connected beam bridges with clauses 5.7.1.3, 5.7.1.5 and 5.7.1.8 therefore being applicable.
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6.6 Resistance and deformation ........................................................................................................ 2
  6.6.2 Ultimate limit state........................................................................................................ 2
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6.7 Shallow foundations............................................................................................................... 3
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6.9 Lateral and vertical pressures............................................................................................... 4
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6.1 Scope

The Ministry is now using Limit States Design for foundation design versus the Working Stress Design method that has been utilized until 2006.

**Commentary:** Limit States Design provisions for foundation design and geotechnical work in CAN/CSA-S6-06 have not been rigorously tested in actual bridge designs in BC. The Ministry urges designers to use caution in applying these new provisions if they result in designs that substantially deviate from the solution provided by traditional working stress methods.

6.6 Resistance and deformation

6.6.2 Ultimate limit state

6.6.2.1 Procedures

Delete the Deep foundations - piles section of Table 61 Geotechnical resistance factors and replace with the following:

**Deep foundations - Piles**

<table>
<thead>
<tr>
<th>Method / Loading / Instrumentation</th>
<th>Resistance Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dynamic Penetration Tests including SPT, BPT and DCPT</td>
<td></td>
</tr>
<tr>
<td>Compression</td>
<td>0.35</td>
</tr>
<tr>
<td>Tension</td>
<td>0.25</td>
</tr>
<tr>
<td>CPT and BPT or SPT with Dynamic Monitoring and CAPWAP</td>
<td></td>
</tr>
<tr>
<td>Compression</td>
<td>0.45</td>
</tr>
<tr>
<td>Tension</td>
<td>0.35</td>
</tr>
<tr>
<td>Dynamic Monitoring with PDA and CAPWAP; Static Test</td>
<td></td>
</tr>
<tr>
<td>Compression</td>
<td>0.50</td>
</tr>
<tr>
<td>Static Load Test</td>
<td></td>
</tr>
<tr>
<td>Direction test/application reversed</td>
<td>0.40</td>
</tr>
<tr>
<td>Direction test/application same</td>
<td>0.60</td>
</tr>
<tr>
<td>Static Load Test with separate toe and shaft instrumentation</td>
<td>0.70</td>
</tr>
<tr>
<td>Horizontal Passive Resistance</td>
<td>0.50</td>
</tr>
</tbody>
</table>

(\* Use of the 0.70 resistance factor, is subject to submission of adequate details which describe the instrumentation and location and number of tests, all of which shall be to the acceptance of the Ministry prior to the Ministry granting Approval. The separate toe and shaft instrumentation shall be sufficient to determine the load in the toe and the load in the shaft.)

Designs shall be based on information available at the time of design and higher resistance factors may not be used based on the intent to do load testing or dynamic monitoring during construction.
6.7 Shallow foundations

6.7.3 Pressure distribution

6.7.3.4 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 6.7.3.4, 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 walls 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.

6.8 Deep foundations

6.8.5 Factored geotechnical axial resistance

Add the following Clause:

6.8.5.7 Pile load distribution at ULS combinations 5 and 8

Capacity design principles will be applied to the design of piles and pile caps for seismic and ship impact loads, based on assumptions for the potential upper and lower bounds of pile capacities. The number of piles and their arrangement will be based on the lower bound assumption that the piles will settle and redistribute loads to adjacent rows when piles reach their factored geotechnical resistance. The pile cap designs will be based on the upper bound assumption that the piles will not settle and will be capable of enough capacity to develop a linear elastic load distribution. For design of piles and pile caps the following shall be considered for ULS Combinations 5 and 8:

(a) Design of piles and their arrangement will be based on the assumption that when the piles reach their factored geotechnical compressive and/or tensile resistance, they will redistribute load to adjacent rows of piles within the group, and develop a “plastic” axial load distribution.

(b) Demands for the design of the pile cap elements shall be based on the assumption that the piles can develop sufficient geotechnical
resistance to develop a linear elastic axial force distribution as required to resist the axial loads and moments.

**Commentary:** Pile design for seismic and ship impact loads is not addressed separately in S6-06. Previous Working Stress method used increased allowable loads and current AASHTO LRFD methods use higher resistance factors for these load combinations. Without special treatment for these load combinations, S6-06 mandates overly conservative designs. AASHTO and ATC 49 allow for design using ultimate pile capacities (resistance factor = 1.0). However in the U.S. this is often accompanied by additional design criteria which provide a higher level of protection for structures. These can include consideration of two levels of earthquake, with a higher magnitude - lower probability earthquake, as well as specifying elastic behaviour of piled foundations. Additionally ATC 49 requires somewhat arbitrary limits on uplift of piles which can govern pile group design, and neither of these codes are specific on whether plastic or linear load distribution should be considered. Recent parametric studies indicate that using ultimate pile capacities for these load combinations may result in substantially weaker piled foundation designs than previous working stress methods.

The methodology described in this proposed clause is consistent with the approach taken for shallow foundations in S6-06 Clause 6.7.3, and results in designs reasonably close to previous Working Stress methods. Iterative methods may be required to determine plastic axial load distributions in the piles. See Figure C6.8.5.7 (a) and (b).

### 6.9 Lateral and vertical pressures

#### 6.9.2 Lateral pressure

**6.9.2.1 General**

(e) The design of integral abutments shall take into account lateral earth pressure build-up and settlements in the zone of soil behind the abutments.

**Commentary:** Refer to Clause 5.5.4.3 for published reference documents for design of integral abutments.

**6.9.2.2 Calculated pressure**

Seismic lateral earth pressures shall be calculated in accordance with Clause 4.6.4.
6.12 MSE structures

6.12.2 Design

6.12.2.1 General


For MSE bridge abutment walls and associated wing walls, the minimum soil reinforcement length provided for the portion of the MSE walls below the bridge abutments 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.

For MSE retaining walls, (other than bridge abutments and associated wing walls) uneven reinforcing lengths may be used when intact rock must be removed to accommodate the 70% required above. The design shall meet the requirements of FHWA-NH1-00-043, “Mechanically Stabilized Earth Walls and Construction Guidelines”, March 2001, Section 5.3.

MSE bridge abutment walls and associated wing walls shall have precast reinforced concrete facing panels, and shall use inextensible soil reinforcing.

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

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.

MSE walls with wire mesh facing, dry cast concrete block facing, or concrete block facing shall only be used in locations as consented to by the Ministry.

Add the following clause:

6.14 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 S6-06 shall meet the requirements of AASHTO Standard Specifications for Highway Bridges, Seventeenth Edition, 2002, including interim revisions.

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.
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    7.5.5.4 Seismic design of concrete structures .............................................................. 3
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7.1 Scope

Buried structures with span smaller than, or equal to, 3 m may also be designed to S6-06 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 (Dh) greater than 3 m, but the CHBDC does not provide design guidance for smaller structures.

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, Hc, at intermediate stages of construction;
- Construction Live Loading assumed in the design (corresponding to Hc);
- 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.

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);
- Assumed Resistivity of Soil Materials;
• "pH" Range for Groundwater and/or Streamflow, as appropriate;
• Seam Strength at Critical Locations;
• Conduit Rise, \( D_v \) and Span, \( D_h \);
• Radius at Crown, \( R_c \);
• Radius at Spring-line, \( R_s \); and,
• Radius at Base, \( R_b \).

**Commentary:** 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 S6-06 Clause 2.4.2.

7.6.3 **Design criteria**

7.6.3.1.1 General

Delete and replace with the following:

The thrust, $T_r$, 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:

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

Where 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) For deeply buried soil-metal structures, the S6-06 expressions for $T_D$ and $T_L$ may be too conservative. S6-06 does not place an upper limit on the applicability of Section 7 for deeply buried soil-metal structures. Designers of deeply buried soil-metal structures may use the S6-06 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:** S6-06 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 S6-06 design formulae may not be conservative for all large span soil-metal structures. Conversely, the same load rating studies show that the S6-06 design formulae for deeply buried, soil-metal structures to be overly conservative.
7.6.3.1.2 Dead loads

(d) “H” is measured vertically from crown of structure to finished grade, reference Figure 7.2.

**Commentary:** The depth of cover or height of overfill, “H”, is missing on Figure 7.2.

7.6.3.4 Connection strength

Designers are advised that values of unfactored seam strength, $S_s$, for standard corrugation profile with bolted connections are shown in Commentary Figure C74.

7.6.4 Additional design requirements

7.6.4.1 Minimum depth of cover

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.

**Commentary:** The S6-06 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 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”, as presented in S6-06 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 (i.e. not necessarily the conservative UBC’95 model).

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 S6-06 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

Commentary: Refer to SS 303 Culverts, for backfill materials and compaction requirements, where applicable.

### 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 Clause 12.7.2.2 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. S6-06 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 (2002) Table 12.8.2A 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.
Commentary: The 8.0 m span limit, and the 3.2 m rise limit, for metal box structures are based on limitations in the original research. S6-06 Commentary indicates that recent (1998) test data, from as-built large-span structures, may allow the beneficial effects of soil-structure interaction to be taken into account for larger metal box structures.

7.7.3 Design criteria

7.7.3.2 Design criteria for connections

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

Commentary: Values of unfactored seam flexural strength are not presented in the S6-06, or in the AASHTO Standard Specifications for Highway Bridges (2002) Clauses 12.4.2 and 12.6.2.

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

Commentary: Refer to SS 303 Culverts, for backfill materials and compaction requirements, where applicable.

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.

7.8.4 Loads and load combinations

7.8.4.4 Earthquake loads

For concrete buried structures with span \((D_h)\) less than or equal to 3 m, the effects of earthquake loading shall be computed in accordance with Clauses 7.8.4.1 and 7.8.4.4. The potential for, and effects of, seismic soil liquefaction shall also be investigated.

For concrete buried structures with span \((D_h)\) greater than 3 m, 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: Clause 7.8.4.4 is misleading (in title and in text) in that the text addresses only vertical, not horizontal, earthquake loads.
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  8.4.1.2 Concrete strength ......................................................................................................... 3
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<td>8.21 Multi-beam decks ..................................................................................................... 26</td>
</tr>
</tbody>
</table>
8.4 Materials

8.4.1 Concrete

8.4.1.2 Concrete strength

Insert after first sentence:

The specified concrete strength for prestressed members shall not exceed 55 MPa at 28 days or 37.5 MPa at release.

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.

8.4.2.1.3 Yield strength

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

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 CAN/CSA-G30.18.

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

8.8.4 Flexural components

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.

8.8.5 Compression components

8.8.5.6 Reinforcement limitations

For structures located in seismic performance zones 3 and 4, the limitations of Clause 4.7.4.2.2 shall apply.

8.9 Shear and torsion

8.9.1 General

8.9.1.5 Effective shear depth

Commentary: For the seismic design of round reinforced concrete columns or piers, the effective shear area shall be equal to 80% of the gross concrete area, $A_p$.

8.9.3 Sectional design model

Commentary: Design for seismic shear based on S6-06 within ductile substructures does not address shear resistance within a plastic hinge zone. Recent design standards or model standards, such as ATC-32 and ATC-49, as well as the current Caltrans Seismic Design Criteria, attribute higher shear capacities than S6-06 in non-ductile regions of reinforced concrete bridge columns, but lower shear capacities in plastic hinge regions. While designers are encouraged to adopt state-of-the-art seismic design methods for shear, care is required to achieve an appropriate margin of safety against brittle failure modes. The responsibility for achieving an acceptable margin of safety is the responsibility of the designers.

The following approach comprises an acceptable design method for shear resistance within ductile concrete sub-structures, and provides a minimum design shear resistance within plastic hinge zones.

Seismic shear resistance within reinforced concrete columns of ductile substructures may be taken as follows:

$$ \Phi V_n = \Phi_c V_c + \Phi_s V_s + \Phi_p V_p $$
Note that $\phi_s$ is suggested for the $V_p$ term where the column axial load is dominated by gravity loads and flexural hinging in the ductile substructure. The designer should consider using a lower $\phi$ value where conditions warrant.

The various terms of the General Method for shear design are similar to those commonly found in state of the art references and guide standards on seismic design of bridges. Examples of different approaches for each term are provided below. Selected references for shear design of bridge columns include:

- ATC-32 Section 8.16.6 (ATC-32 were provisional recommendations to Caltrans, and may be regarded as superseded by subsequent Caltrans Seismic Design Criteria).
- Caltrans Seismic Design Criteria (latest version, currently Ver. 1.3, 2004), Section 3.6.
- ATC-49 Section 8.8.2.3.1 and 8.8.2.3.2.

8.9.3.4 Determination of $V_c$

Commentary: $\beta$ may be taken as 0.29 for columns of ductile sub-structures of nominal ductility structures, and not less than 0.05 for plastic hinge regions of columns of ductile sub-structures, or where curvature ductilities are not determined. Interpolation between these two values for curvature ductilities between 3 and 15 may be used.
This approach to calculate \( \beta \), the concrete shear contribution in plastic hinge zones, is based on Priestley et al. (2000) and is similar to the approach in ATC-49.

8.9.3.5 Determination of \( V_s \)

**Commentary:** S6-06 currently does not differentiate between rectangular and round columns for the determination of the tie or spiral reinforcing contribution to shear resistance. The formula provided below is adopted from Priestley et al. (2000), and comprises an acceptable method to calculate \( V_s \) as part of the seismic shear resistance of round columns. It may be used within or outside of plastic hinge zones. Designer must satisfy themselves that the column configuration, details and axial load under gravity in combination with seismic loads justify a value of \( \theta = 30^\circ \). Higher angles (cracks crossing fewer spirals, hence lower \( V_c \)) should be used where warranted.

\[
V_s = \phi_s (0.5 \pi A_v f_{yh} D' \cot \theta) / s
\]

Where:

- \( A_v \) = area of one spiral bar
- \( f_{yh} \) = yield stress of spiral reinforcing
- \( D' \) = core diameter of the column, approximately equal to the diameter measured across the centre of the vertical reinforcing steel bar cage
- \( \theta = 30^\circ \) for the purposes of this clause only, or \( 45^\circ \) if axial tensile loads occur under seismic loads
- \( s = \) spacing of spiral reinforcing bars

8.9.3.8 Determination of \( \epsilon_x \)

**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 specification.
8.11 Durability

8.11.2 Protective measures

8.11.2.1 Concrete Quality

8.11.2.1.1 General

For structural elements listed below, concrete mix criteria shall comply with the requirements given in the following table unless otherwise consented to by the Ministry. This information shall also be included in the Special Provisions of the Contract Documents for the Project.

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.)

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

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

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

(3) Application rates shall not exceed 15% by mass of Portland Cement.

(4) Superplasticizer shall not be used.

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

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

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

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

8.11.2.1.3 Concrete placement

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). Bridges with minimum grades of less than 2% may be cast in either direction.

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. Wider cracks shall be effectively sealed to prevent entry of water and chlorides. In specifying the deck pour sequence, the designer shall pay particular attention to the adverse affects 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.
For continuous span bridges, the length of deck casting should be limited to 20 m in order to minimize shrinkage cracking.

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

Placement of parapet concrete shall not proceed until the full deck has been cast and the minimum strength of concrete is 15 MPa, unless otherwise required by the designer.

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**

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

<table>
<thead>
<tr>
<th>Surface</th>
<th>Finish</th>
<th>SS Clause</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surfaces submerged or buried</td>
<td>Class 1</td>
<td>211.17</td>
</tr>
<tr>
<td>Top and inside (exposed) face of parapets, curbs</td>
<td>Class 3</td>
<td>211.17</td>
</tr>
<tr>
<td>Outer face of parapets, curbs; outer edges of deck</td>
<td>Class 2</td>
<td>211.17</td>
</tr>
<tr>
<td>Abutments and retaining walls</td>
<td>Class 2</td>
<td>211.17</td>
</tr>
<tr>
<td>Piers</td>
<td>Class 2</td>
<td>211.17</td>
</tr>
<tr>
<td>Bearing seats</td>
<td>Steel Trowel</td>
<td>211.14</td>
</tr>
<tr>
<td>Top of deck</td>
<td>Tined(^{(2)})</td>
<td>413.31.02.05</td>
</tr>
<tr>
<td>Approach slabs</td>
<td>Float Finish</td>
<td>211.14</td>
</tr>
<tr>
<td>Sidewalks</td>
<td>Transverse Coarse Broom</td>
<td>211.14</td>
</tr>
<tr>
<td>Underside of Deck</td>
<td>Class 1 (or better)</td>
<td>211.17</td>
</tr>
<tr>
<td>Slope Pavement</td>
<td>Transverse Coarse Broom(^{(1)})</td>
<td>211.14</td>
</tr>
</tbody>
</table>

**Notes**

(1) Exposed Aggregate finishes may be considered.

(2) Decks to receive waterproofing membranes shall be finished in accordance with Standard Specification 419.33.
Consideration shall be given to surfaces exposed to public view such as piers and abutments on underpasses where a Class 3 finish may be warranted, and underside of decks where a Class 2 finish may be warranted.

Exposed concrete surfaces of large abutments or retaining walls that are clearly visible to the public may require an architectural finish. The selection of a surface finish shall give consideration for future removal of graffiti. Such consideration may include the application of anti-graffiti paint.

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 S6-06 shall be replaced with “minimum specified cover”.

Table 8.5 in S6-06 shall be amended as follows:

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

<table>
<thead>
<tr>
<th>Environmental exposure</th>
<th>Component</th>
<th>Reinforcement/ steel ducts</th>
<th>Cast-in-place concrete (mm)</th>
<th>Precast concrete (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>All</td>
<td>(3) Top surfaces of Structural components</td>
<td>Reinforcing Steel Pretensioning strands</td>
<td>70 ±6 -0</td>
<td>70 ±6 -0</td>
</tr>
<tr>
<td></td>
<td>Add: Bridge Decks and Approach Slabs</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>All</td>
<td>(10) Precast T, I and box girders</td>
<td>Reinforcing steel</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Add: Ministry Standard Precast Box Girders Pretensioning strands</td>
<td></td>
<td>70 +10 -5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Add: Ministry Standard Precast I-Beams</td>
<td>Reinforcing steel</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Pretensioning strands</td>
<td>200 ±5</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Top surfaces</td>
<td>–</td>
<td>70 +10 -5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Vertical surfaces</td>
<td>–</td>
<td>50 ±5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Soffits</td>
<td>–</td>
<td>40 ±5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Inside surfaces</td>
<td>–</td>
<td>35 ±5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Top surfaces</td>
<td>–</td>
<td>30 +10 -5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Vertical surfaces</td>
<td>–</td>
<td>40 ±5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Soffits</td>
<td>–</td>
<td>30 ±5</td>
</tr>
</tbody>
</table>

Delete Note ‡ under Table 8.5. An additional 10 mm of concrete cover shall not be provided for concrete decks without waterproofing and paving.

**Commentary:** The term “minimum cover” should be avoided as it creates confusion for installers. The term “specified cover” is the preferred term and the appropriate placing tolerances would apply. For top reinforcing in decks,
a “specified cover” of 70 mm with placing tolerances of +6 mm and -0 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

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

Corrosion protection for reinforcing steel shall be achieved by the use of either epoxy coating or galvanizing of the reinforcing steel. If galvanizing 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.

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 Standard Specification Table 412-B
- ensuring Grade W (weldable) reinforcing is used in accordance with Standard Specification 412.11.03
  and
- stress relieving the reinforcing steel after bending and prior to galvanizing. (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.

Stainless steel may be considered as an alternative to epoxy coating or galvanizing if strength requirements are met and its use is found to be comparatively economical. Stainless steel clad reinforcing or reinforcing steel with alloys to increase corrosion resistance (such as low carbon, chromium steel bars for concrete reinforcement) may only be used with consent of the Ministry.

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.
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 reinforcing when detailing the deck reinforcing.

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.

Commentary: The drip groove detail shown above has been used throughout the Province since it was first introduced in 1969 and has functioned well with no adverse feedback from field staff. For this reason the detail has been retained, although it varies from the drip groove detail described in Clause 8.11.2.6 of S6-06.

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 waterproofing membrane and 100 thick asphalt overlay on top of the bridge deck as per SP419. Bridges located in the Southern Interior Region and the Northern Region shall be protected with an application of linseed oil or as directed by the Ministry.
8.12 Control of cracking

8.12.1 General

Control joints shall extend around the perimeter of the barrier, be evenly spaced throughout the length of the barrier with spacing not exceeding 3 m.

Concrete traffic barriers shall have a 6 mm wide joint cut over the supports on continuous spans. The joints may be saw-cut, but the structure shall not be subjected to a single vehicle live load greater than 5 kN prior to the cutting operation.

Figure 8.12.1
Control joint detail

8.13 Deformation

8.13.3 Deflections and rotations

8.13.3.3 Total deflection and rotation

Commentary: The Commentary to S6.06 states that long time deflection and rotation may be calculated by using the empirical multipliers given in Table C8.13.3.3 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 may be used in place of the commentary.
8.14 Details of reinforcement and special detailing provisions

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 extended pile pier caps

![Diagram of transverse reinforcement](image)

**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 Figure 8.20.7.1 shall be used.

**Commentary:** Problems are encountered with stirrup sizes in diaphragms when stirrups are either too long or too short depending on the final depth of
8.15 Development and splices

8.15.6 Combination development length

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.

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 such splices are unavoidable, 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 on the Ministry standard reference details (Ministry Standard Drawings 2978-1 to 2978-24 (latest revision)) for box stringers.

8.18 **Special provisions for deck slabs**

Bridge deck heating systems shall not be incorporated into the design of bridge decks.

*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.

*Commentary: The minimum deck slab thickness is based on providing adequate clear concrete cover between top and bottom layers of deck reinforcement and maintaining top and bottom concrete covers for the deck slab.*

- Concrete cover – top of deck 70 + 6 mm (tolerance)
- Top reinforcing – transverse 18 mm
- Top reinforcing – longitudinal 18 mm
- Minimum clear cover between layers 25 mm
- Bottom reinforcing – longitudinal 18 mm
- Bottom reinforcing – transverse 18 mm
- Concrete cover – soffit of deck 40 + 10 mm (tolerance)
- Total - Minimum slab thickness 223 mm (round up to 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

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 details the requirements for minimum shear key size and reinforcing detail.

**Figure 8.18.4.4**
Full depth precast panel shear key
Add the following additional items:

(h) a minimum 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. 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 applied in accordance with SS and DBSS 415 with a 100 mm thick asphalt wearing surface,

(l) the spacing between shear stud connection pockets shall not exceed 600 mm.

(m) the design of the shear studs must take into account the thickness of the bedding layer under the panels above the supporting beams.

Commentary: Recent research (Kim J.H., Shim C.S., Matsui S., and Chang S.P. “The Effect of Bedding Layer on the Strength of Shear Connection in Full-Depth Precast Deck, Engineering Journal, Third Quarter, 2002, pp 127-135.) has shown that the ultimate strength of the shear connection decreases with an increase in the bedding layer thickness due to increased bending of the connectors and this must be accounted for in the design.

Recent Australian research (Oehlers D.J., Seracino R., and Yeo M.F. “Fatigue Behaviour of Composite Steel and Concrete Beams with Stud shear Connections”, Prog. Struct. Engng Mater., Vol 2, 2000, pp 187-195) has shown that the stud shear connections reduce in strength and stiffness immediately after cyclic loads are applied. This effect shall be accounted for in the design.

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 Figure 8.20.7.3. Steel diaphragms for steel girders shall be fabricated from similar material as the primary members and protected from corrosion with the same system used for the primary members.
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 8.20.8 for specific guidance regarding design of concrete diaphragms for concrete girders.

8.19 Composite construction

8.19.1 General

Prestressed concrete box girders with a concrete overlay shall be designed as non-composite unless mechanical anchorage is incorporated to ensure composite action. For non-composite design, the placement of a concrete overlay 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.

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 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.

Commentary: Refer to Clause 8.11.2.3 regarding use of galvanized reinforcing bars.
Figure 8.19.3 (a)
Additional reinforcement for haunches over 75 mm high

Figure 8.19.3 (b)
Additional reinforcement for haunches over 75 mm high
8.20 Concrete girders

8.20.1 General

Prestressed concrete I-girder and box girder skews over 30° shall be avoided where practical. 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

Ministry Standard Twin Cell Box Stringers shown on Drawings 2978-1 to 2978-24 (latest revision) shall be used as Ministry standards for twin cell boxes.

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 S6-06 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 diaphragms support the slab, bars shall
be added between the longitudinal reinforcing. The bars shall be 15ME and the length shall equal ‘S.’

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

**Figure 8.20.7.1**
**Typical diaphragm tie arrangement**

Abutment and pier diaphragms shall be designed to facilitate future jacking, and to provide access for maintenance inspection, as generally outlined in Figure 8.20.7.2
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. 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.
8.21 Multi-beam decks

The shear key and reinforcement details shown on Ministry Twin Cell Non-composite Concrete Box Girder Standard Drawings 2978-1 to 2978-24 (latest revision) and Ministry Twin Cell Composite Concrete Box Girder Standard Drawings 2310-10 to 2310-17 (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 S6-06.

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 recently completed site investigations 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 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 on the Ministry 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 box beams and a reinforced concrete overlay to achieve composite action.

Commentary: Figures 8.21 (a) and 8.21 (b) are suggested means of achieving composite action between the structural box beam and the reinforced concrete overlay.
Figure 8.21 (a)

Double cell box beam/overlay connection detail

Figure 8.21 (b)

Single cell box beam/overlay connection detail
9.5 General Design

9.5.6 Load-sharing factor

Add to Table 9.3 Values of $D_e$ the following:

<table>
<thead>
<tr>
<th>Structure</th>
<th>$D_e$, m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stringer of Glued-laminated timber stringer bridge</td>
<td>1.75</td>
</tr>
</tbody>
</table>

Commentary: There is no reference to glue-laminated structures.
10.4 Materials

10.4.1 General

Delete the third paragraph and replace with the following:

- Coil steel shall not be used unless specifically 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

Delete the third paragraph and replace with:

Fracture-critical members and primary tension members shall be of type AT Category 3, type WT Category 3, or type QT Category 3 steels as specified in CSA G40.21. Grade 260W shall not be used in bridges.

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 dimension. 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, 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", 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') and 1830 mm (6'). 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. In the future, steel from countries such as Japan, Korea, and China may become more competitively priced and may be available for projects in British Columbia.

6. For reasons of uniformity and simplicity, the design should make use of the same grade of steel throughout the project as much as is practical.

7. 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 bolts: ASTM A307, Grade C (Fy = 250 MPa, 36 ksi).
   • Shear studs: (refer to S6-06 clause 10.4.7)

8. 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.
9. Local fabricator experience indicates that, in reality, angle and channel sections are usually purchased as conforming to ASTM A572, Grade 50 (non-weathering) or ASTM A588 (weathering steel).

10. 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.

11. Local fabricator experience is that structural tee sections are usually purchased as conforming to ASTM A572, Grade 50 (non-weathering) or ASTM A588 (weathering steel).

12. 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.

13. Local fabricator experience is that sections are usually purchased as conforming to ASTM A572, Grade 50 (non-weathering) or ASTM A588 (weathering steel).

14. Higher strength anchor bolts such as ASTM A449 or ASTM F1554 (105 ksi) may be used where required.

15. 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

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 specifically permitted by the Ministry.

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.

4. 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.

5. See the Ministry SS 421.11.03 for coating requirements for bolts.

10.4.10 Galvanizing and metallizing

For steel that is to be hot-dip galvanized, the following restriction is made in addition to the chemical composition (heat analysis) requirements of 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.

10.6 Durability

10.6.3 Corrosion protection

Primary superstructure members shall be corrosion-resistant weathering steel.

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 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.
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 on the Ministry’s Recognized Products List, for the larger of the following two distances from locations of deck joints, such as at expansion joints, fixed joints, and abutments:

- 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 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 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 from the Ministry’s Recognized Products List according to SS 421 or 422.

In marine environments, or where the steel is likely to be sprayed with road salt, the steel shall be coated.

The designer shall make all attempts to avoid situations where water can pool on girder flanges. Where they cannot be avoided, such areas shall be painted with an immersion-grade coating.

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 the environment and rate of corrosion of weathering steel structures located in the area.
10.6.3.2 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 ropes or strands used in bridges.

**Commentary:** Corrosion protection systems for cables are advancing rapidly. As such, discussion with the Ministry is required for the rare instances 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:** Coated piling has not been found to be successful by the Ministry. 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 detail

10.7.1 General

**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.steelbridge.org/AASHTO Docs/GDC-1 AASHTO.pdf](http://www.steelbridge.org/AASHTO Docs/GDC-1 AASHTO.pdf)

NSBA is the US-based National Steel Bridge Alliance.

Add the following Clauses 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 the plate width used for any one flange 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. The designer should take into account that plate comes in 2440 mm (8'-0") and/or 1830 mm (6'-0") widths (depending on availability) when determining flange widths.

### 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

\[
C = (4 \text{ to } 6) \times t_w \text{ BUT NOT LESS THAN 50mm}
\]

\[
X = \begin{cases} 125 \text{mm} & \text{if } b < 400 \text{mm} \\ 150 \text{mm} & \text{if } b > 400 \text{mm} \end{cases}
\]

\[
\phi \geq 30^\circ
\]
Commentary: These 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.

**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.
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.

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 Strength 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)). Two possible solutions are shown. 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

(a) AT PIER

(b) AT ABUTMENT

(c)
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. Of course 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 considered 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 also 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

Camber information shall be provided by the designer. Camber shall be shown at splice points and at intervals not greater than 2 m.

Delete the second paragraph and replace with the following:

A camber diagram shall be included in the Plans and shall include elevations for:

(a) target finished steel girder grades

(b) item (a) cambered for deflections due to the deck, curbs, sidewalks, barriers, railings, wearing surface, creep and shrinkage, and utilities.

(c) Item (b) cambered 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.
10.9  Compression members

10.9.5  Composite columns

10.9.5.4  Compressive resistance

Item (a), delete the formula for $\tau'$ and replace with the following:

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

10.18  Splices and connections

10.18.1  General

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.

10.19  Anchors

10.19.1  General

This clause shall be amended by the addition of the following:

- Anchor bolts, including nuts and washers, shall be galvanized or metallized;
- Anchor bolt 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:** Consideration may be given to the use of anchors in pipe sleeves to provide erection tolerance.

*Based on inspection of existing bridges, it is prudent to galvanize anchor bolts and their components that are not embedded in the concrete and are exposed to damage from corrosion.*
10.24 Construction requirements for structural steel

10.24.1 General

Construction shall be in accordance with SS 421 unless amended by the Supplement or otherwise Approved.

Field splices shall be bolted connections.

10.24.5 Welded construction

10.24.5.1 General

Field welding of girder splices shall not be permitted. Field welding of attachments to girders shall only be permitted with Approval 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.

10.24.6 Bolted construction

10.24.6.3 Installation of bolts

Fully tensioned bolts shall be installed in all bolt holes used for erection.

10.24.8 Quality control

The designer is cautioned that W59 requires the engineer to specify the type and extent of testing for welds. The designer shall specify any testing requirements of the welding that are additional to the testing requirements of SS 421.

10.24.8.2 Non-destructive testing of welds

Delete clause and replace with the following:

The following non-destructive testing of welds shall be performed:

(a) visual inspection of all welds;
(b) 100% radiographic inspection of all flange and web butt welds;
(c) 100% magnetic-particle inspection of web-to-flange fillet welds;
(d) 100% magnetic-particle inspection of flange/stiffener fillet welds;
(e) 25% magnetic-particle inspection of web/stiffener fillet welds;
(f) 25% magnetic particle inspection of bracing/stiffener fillet 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.*

### 10.24.9 Transportation and delivery

After steelwork has been delivered to site it shall be inspected by the Contractor’s QC Inspector. The Contractor shall be responsible for cleaning the steelwork of any dirt and particularly road salts and/or slush that has accumulated during transport.

The cleaning of unpainted steelwork shall be done by power washing, wire wheeling, or light sandblasting. Faying surfaces shall be cleaned by power washing, manually cleaned by steel wire brushing, or by sand blasting. If the design calls for blast-cleaned faying surfaces, they shall be cleaned by sand blasting.

Painted steelwork shall be cleaned by power washing after erection and deck construction is complete.
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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 G164 Table 1 or a coating system which is designated as proven in 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.

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 100mm. 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 third 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 fourth paragraph:

Deck joints with skew angles between 32 deg and 38 deg 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 deg) and this could result in a blade dropping into a deck joint and damaging it.

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 designated as proven in the Ministry’s Recognized Products List.

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

**Commentary:** MoT 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. Approval 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:

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 except for modular joint systems. For modular joint systems the horizontal load shall be developed in consultation with the Ministry with the recommended load consented to by the Ministry.

### 11.5.2 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 the number of deck joints should improve overall lifecycle performance.

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 have resulted in the leakage of surface run-off and de-icing salts onto the substructure and bearings.

### 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. Any other type of open deck joint will not 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.
11.5.3.2 Components

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 approved 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. Countersunk anchor bolts shall not be permitted on any joint connection unless consented to by the Ministry.

11.5.6 Joint seals

Only deck joint seals made of rubber or 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:

The "finger" plate deck expansion joint shall have a drainage trough installed beneath. For the drainage trough, consideration shall be given to the use of a corrosion-resistant plastic such as High Density Poly Ethylene (HDPE). The trough shall be robust (sufficient thickness to prevent deflection when loaded with wet sand). All steelwork supporting the trough shall be galvanized or metallized after fabrication.

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 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

Add to first paragraph the following:

Elastomeric bearings shall be used whenever possible for concrete I-girders and box girders.

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:** 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.

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.
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 Section C11.6.6 at the end of Section 11 for 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 11.6.6.2.2 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. AASHTO refers to 'k values – a compression modulus modifier constant and/or charts to relate stress, shape factor and strain. In addition, there is no relation between temperature ranges and the properties of the elastomer (although AASHTO and S6-88 specify different 'elastomer grades' depending on low temperatures). The designer is responsible for incorporating appropriate properties with the bearing design.

11.6.6.3 Geometric requirements

Contrary to part (a), $h_a$ shall be less than 25 mm and greater than 15 mm. The shape factor must always be checked.

An unreinforced elastomeric pad in the form of a single continuous strip may be used under box girders provided the bearing pressure is in accordance with the requirements of Clause 11.6.6.7.

**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 S6-06.

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 shall be + 3 mm, - 0 mm.

Commentary: It is recommended that a minimum cover of 8 mm be specified. Fabrication tolerances are such that this will likely ensure an actual minimum cover of 5 mm, which is acceptable.

11.6.6.6 Positive attachment

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 plate 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 GIRDER, OPTIONAL FOR INTERIOR GIRDER.

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

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 – “Seismic Base Isolation”.

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

Commentary: Bearings preferably shall be installed level with tapered sole plates to account for girder slopes. If this arrangement will cause geometric problems at deck joint level the bearings may be installed at the same grade as the bridge and the supporting substructure shall be designed for the resulting horizontal force.
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 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 14.7.5.2-1 of the AASHTO LRFD Bridge Design Specifications):

<table>
<thead>
<tr>
<th>Hardness (Shore A)</th>
<th>Shear Modulus @ 23°C (MPa)</th>
<th>Creep deflection @ 25 years divided by instantaneous deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>0.66-0.90</td>
<td>0.25</td>
</tr>
<tr>
<td>60</td>
<td>0.90-1.38</td>
<td>0.35</td>
</tr>
</tbody>
</table>

Notes:

1. Reference Table 14.7.5.2-1, 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:

\[
S_i = \frac{LW}{2h_{ri}(L + W)}
\]

(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:

\[
S_i = \frac{D}{4h_{ri}}
\]

(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:

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

(Equation [3])

For circular bearings:

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

(Equation [4])
Where:

\[ d = \text{the diameter of the hole or holes in the bearing (mm).} \]

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:

\[ \delta = \sum \varepsilon_i h_{ri} \]  

(Equation [5])

Where:

\[ \varepsilon_i = \text{instantaneous compressive strain in } i^{th} \text{ elastomer layer of a laminated bearing;} \]

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

In the absence of material specific data from testing, the following figure (reproduced from Figure C14.7.5.3.3-1 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 C14.7.5.3.3-1 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:

\[ \delta = \sum \delta_i \]  

(Equation [6])

Where:

\( \delta_i \) is the vertical compressive deformation of \( i^{th} \) elastomeric layer and given by

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

(Equation [7])

Where:

\( \sigma_c \) = 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 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:

\[ H_u = GA \frac{\Delta_u}{h_{rt}} \]  

(Equation [8])

Where:
G = shear modulus of the elastomer (MPa);

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

Δui = factored shear deformation (mm); and

hrt = 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 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 Template Special Provisions: Appendix - Supply, Fabrication and Installation of Bearing Assemblies.

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 and the current design practice in the elastomeric bearing industry.

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 S². 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 C14.7.5.3.3-1 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.

CAN/CSA-S6-06 does not include any testing provisions for elastomeric bearings.

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 template Special Provisions, a compression load test is required for every laminated bearing. The compression test specified in the Ministry template Special Provisions 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 the 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 template Special Provisions 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
12.4 Barriers

12.4.2 Barrier joints

12.4.3 Traffic barriers

12.4.3.2 Performance level

12.4.3.2.1 General

12.4.3.2.1.a Performance level PL-1

12.4.3.2.1.b Performance level PL-2

12.4.3.2.1.c Performance level PL-3

12.4.3.2.1.d Performance level for LVR bridges

12.4.3.3 Geometry and end treatment details

12.4.4 Pedestrian barriers

12.4.4.1 General

12.4.4.2 Geometry

12.4.5 Bicycle barriers

12.4.5.2 Geometry

12.4.6 Combination barriers

12.4.6.1.a Use of combination barriers

12.4.6.1.b Pedestrian combination barriers

12.4.6.1.c Bicycle combination barriers

12.4.6.1.d Sidewalks separated from traffic by raised curbs

12.4.6.2 Geometry
12.4 Barriers

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 CSA G164 Table 1.

12.4.3 Traffic barriers

12.4.3.2 Performance level

12.4.3.2.1 General

Bridge traffic barriers as shown in Figures 12.5.2.1.a to 12.5.2.1.i have been accepted by the Ministry for use on highway bridges in B.C. and meet the crash testing requirements of S6-06. Any bridge traffic barriers proposed for use on Ministry bridges in B.C., other than those shown in this document, require proof of meeting crash testing requirements of S6-06 and prior Approval.

12.4.3.2.1.a Performance level PL-1

Figure 12.4.3.2.1.a

Thrie beam bridge railing (box girder side-mounted)
Commentary: The system shown in Figure 12.4.3.2.1.a is based on the crash tested ‘Oregon State Side Mounted Thrie-Beam’ bridge railing. Until such time as the Ministry develops a standard drawing for this system, information regarding the Oregon bridge railing can be found at the following websites:


**Figure 12.4.3.2.1.b**

Thrie beam bridge railing (top-mounted)

Commentary: The system shown in Figure 12.4.3.2.1.b is based on the crash tested ‘Oregon Side Mounted Thrie-Beam’ bridge railing (see Figure 12.4.3.2.1a); however, the system has been modified to a top-mounted anchorage. Use of this system requires that the modified anchorage be designed to resist barrier loads in accordance with Clause 12.4.3.5 of S6-06.

Information regarding the Oregon bridge railing can be found at the following websites:

Alberta Transportation has adopted similar top-mounted thrie beam railing systems as part of its bridge railing standards. Until such time as the Ministry develops a standard drawing for this system, information regarding the Alberta railings can be found at the following websites:

- [http://www.infratrans.gov.ab.ca/INFTRA_Content/docType30/Producti on/S1652-00-rev3.pdf](http://www.infratrans.gov.ab.ca/INFTRA_Content/docType30/Production/S1652-00-rev3.pdf)
- [http://www.infratrans.gov.ab.ca/INFTRA_Content/docType30/Producti on/S1653-00-rev2.pdf](http://www.infratrans.gov.ab.ca/INFTRA_Content/docType30/Production/S1653-00-rev2.pdf)

**Figure 12.4.3.2.1.c**

**Steel two-rail bridge railing (box girder side-mounted)**

**Commentary:** The system shown in Figure 12.4.3.2.1.c is based on the crash tested ‘California Type 115’ bridge railing. Until such time as the Ministry develops a standard drawing for this system, information regarding the Type 115 railing can be found at the following website:

**Commentary:** The system shown in Figure 12.4.3.2.1.d is based on the crash tested ‘California Type 115’ bridge railing (see Figure 12.4.3.2.1.c, however, the system has been modified to a top-mounted anchorage. Use of this system requires that the modified anchorage be designed to resist barrier loads in accordance with Clause 12.4.3.5 of S6-06.

Until such time as the Ministry develops a standard drawing for this system, information regarding the Type 115 railing can be found at the following website:

12.4.3.2.1.b  Performance level PL-2

**Commentary:** The system shown in Figure 12.4.3.2.1.e is the Ministry’s Standard Bridge Parapet – 810 mm High (Standard Drawing No. 2784-1) which is similar to the crash tested ‘32-inch F-Shape’ concrete bridge railing. Use of this system requires that the anchorage be checked to ensure that adequate capacity exists to resist barrier loads in accordance with Clause 12.4.3.5 of S6-06.

Information regarding crash tested F-shape bridge railings can be found at the following website:

- [http://safety.fhwa.dot.gov/roadway_dept/docs/appendixb7g.pdf](http://safety.fhwa.dot.gov/roadway_dept/docs/appendixb7g.pdf)
Commentary: The system shown in Figure 12.4.3.2.1.f is based on a Standard Precast Parapet (Preliminary Standard Drawing No. 2965-4) which the Ministry has used on a number of low volume highway bridges in the past and which is similar to the crash tested ‘L.B. Foster Company’ precast concrete bridge railing. Use of this system requires that the anchorage be designed to resist barrier loads in accordance with Clause 12.4.3.5 of S6-06.

Until such time as the Ministry finalizes a standard drawing for this system, information regarding the L.B. Foster Company precast bridge railing can be found at the following websites:

Commentary: The system shown in Figure 12.4.3.2.1.g is based on the crash tested ‘New York State Two-Rail Steel Bridge Railing’. Until such time as the Ministry develops a standard drawing for this system, information regarding New York State bridge railings can be found at the following websites:

Commentary: The system shown in Figure 12.4.3.2.1.h is based on the crash tested 'New York State Two-Rail Steel Bridge Railing'; however, the system has been modified to include an HSS 127x76 bottom rail. Until such time as the Ministry develops a standard drawing for this system, information regarding New York State bridge railings can be found at the following websites:

12.4.3.2.1.c Performance level PL-3

Figure 12.4.3.2.1.i
Cast-in-place concrete bridge parapet

Commentary: The system shown in Figure 12.4.3.2.1.i is based on the crash tested ‘42-inch F-Shape’ concrete bridge railing. Use of this system requires that the anchorage be checked to ensure that adequate capacity exists to resist barrier loads in accordance with Clause 12.4.3.5 of S6-06.

Until such time as the Ministry develops a standard drawing for this system, parapet reinforcing should, in general, be arranged in a similar pattern to reinforcing shown on Standard Drawing No. 2784-1 ‘Standard Bridge Parapet – 810 mm High’.

Information regarding crash tested F-shape bridge railings can be found at the following website:

- [http://safety.fhwa.dot.gov/roadway_dept/docs/appendixb7g.pdf](http://safety.fhwa.dot.gov/roadway_dept/docs/appendixb7g.pdf)
**12.3.4.2.1.d Performance level for LVR bridges**

The designer is to refer to the Ministry’s Guidelines for Design and Construction of Bridges on Low Volume Roads for guidance with respect to performance levels below PL-1.

**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 PL-2.

*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.

**12.4.4 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.

**12.4.4.2 Geometry**

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

**12.4.5 Bicycle barriers**

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.
12.4.6 Combination barriers

12.4.6.1.a Use of combination barriers

For highway bridges without sidewalks, either a Pedestrian Combination Barrier or a Bicycle Combination Barrier shall be installed on each side of the bridge. The use of Traffic Barriers in lieu of Combination Barriers 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.

On highway bridges with sidewalk(s) intended for pedestrian use only, where the roadway is not separated from the sidewalk(s) by a raised curb, concrete parapet type Traffic Barriers or Pedestrian Combination Barriers shall be used to separate the roadway from the sidewalk(s). The selection of Traffic Barrier or Pedestrian Combination Barrier shall be determined by the Design Engineer, subject to consent by the Ministry, on the basis of the anticipated volume of pedestrian traffic.

On highway bridges with sidewalk(s) intended for both pedestrian and bicycle use, where the roadway is not separated from the sidewalk(s) by a raised curb, concrete parapet type Traffic Barriers, Pedestrian Combination Barriers or Bicycle Combination Barriers shall be used to separate the roadway from the sidewalk(s). The selection of Traffic Barrier, Pedestrian Combination Barrier or Bicycle Combination Barrier shall be determined by the Design Engineer, subject to consent by the Ministry, on the basis of the anticipated volume of pedestrian and bicycle traffic.

On highway bridges with only one sidewalk, either a Pedestrian Combination Barrier or a Bicycle Combination Barrier shall be installed on the side of the bridge with no sidewalk. The use of Traffic Barriers in lieu of combination Barriers 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 details of the crossing.

Use of raised curbs shall only be permitted when the design speed is \( \leq 60 \text{ km/h} \).

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), while it is a requirement that roadway traffic be separated from the 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) such that the sidewalk face of the
barrier has a smooth surface without snag points (i.e. satisfies Clause 12.4.6.2 of S6-06).

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

Pedestrian Combination Barriers as shown in Figures 12.4.6.1.a to 12.4.6.1.d have been accepted by the Ministry for use on highway bridges in B.C. and meet the crash testing requirements of S6-06. Any Pedestrian combination barriers proposed for use on Ministry of Transportation bridges in B.C., other than those shown in these Bridge Standards, require proof of meeting the crash testing requirements of S6-06 and prior Approval.

Performance level PL-2

Figure 12.4.6.1.a
Cast-in-place or precast concrete bridge parapet

810 mm high parapet with steel pedestrian rail
**Commentary:** The system shown in Figure 12.4.6.1.a includes the Ministry’s Standard Bridge Parapet Steel Railing (Standard Drawing No. 2785-2). Until such time as the Ministry updates the Standard Bridge Parapet Steel Railing Drawing, the Design Engineer has the option of either using the current Standard Bridge Parapet Steel Railing or providing an alternate top railing design which meets the requirements of Clause 12.4.6 of S6-06 and which is acceptable to the Ministry.

See Commentary for Figure 12.4.3.2.1e) regarding the cast-in-place concrete bridge parapet (810 mm High).

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**Figure 12.4.6.1.b**

Steel three-rail bridge railing with brush curb

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**Commentary:** The system shown in Figure 12.4.6.1.b is based on the crash tested 'New York State Four-Rail Steel Bridge Railing', however, the system has been modified to replace the bottom rail with a brush curb. Until such time as the Ministry develops a standard drawing for this system, information regarding New York State bridge railings can be found at the following websites:
Figure 12.4.6.1.c
Steel four-rail bridge railing

Commentary: The system shown in Figure 12.4.6.1.c is based on the crash tested 'New York State Four-Rail Steel Bridge Railing'. Until such time as the Ministry develops a standard drawing for this system, information regarding New York State bridge railings can be found at the following websites:
**Performance level PL-3**

**Figure 12.4.6.1.d**

*Cast-in-place concrete bridge parapet (1070 mm High)*

Commentary: See Commentary for Figure 12.4.3..2.1i) regarding the Cast-in-place concrete bridge parapet (1070 mm High).

12.4.6.1.c Bicycle combination barriers

Bicycle combination barriers as shown in Figures 12.4.6.1.e) to 12.4.6.1.g) have been accepted by the Ministry for use on highway bridges in B.C. and meet the crash testing requirements S6-06. Any Bicycle combination barriers proposed for use on Ministry of Transportation bridges in B.C., other than...
those shown in these Bridge Standards, require proof of meeting crash testing requirements of S6-06 and prior Approval.

Performance level PL-2

**Figure 12.4.6.1.e**

**Cast-in-place or precast concrete bridge parapet (810 mm High) with steel bicycle rail**

Commentary: The system shown in Figure 12.4.6.1.e includes the Ministry’s Standard Bridge Parapet Steel Bicycle Railing (Standard Drawing No. 2785-3). Until such time as the Ministry updates the Standard Bridge Parapet Steel Bicycle Railing Drawing, the Design Engineer has the option of either using the current Standard Bridge Parapet Steel Bicycle Railing or providing an alternate top railing design which meets the requirements of Clause 12.4.6 of S6-06 and which is acceptable to the Ministry.

See Commentary for Figure 12.4.3.2.1.e regarding the cast-in-place concrete bridge parapet (810 mm High).
Figure 12.4.6.f
Steel five-rail bridge railing

Commentary: The system shown in Figure 12.4.6.1.f is based on the crash tested ‘New York State Four-Rail Steel Bridge Railing’; however, the system has been modified to increase the overall railing height by the inclusion of an additional top rail. Until such time as the Ministry develops a standard drawing for this system, information regarding New York State bridge railings can be found at the following websites:

Performance level PL-3

**Figure 12.4.6.1.g**
Cast-in-place concrete bridge parapet (1070 mm High) with steel pedestrian rail

**Commentary:** The system shown in Figure 12.4.6.1.g includes the Ministry’s Standard Bridge Parapet Steel Railing (Standard Drawing No. 2785-2). Until such time as the Ministry updates the Standard Bridge Parapet Steel Railing Drawing, the Design Engineer has the option of either using the current Standard Bridge Parapet Steel Railing or providing an alternate top railing design which meets the requirements of Clause 12.4.6 of S6-06 and which is acceptable to the Ministry. (Note that alternate top railing designs must provide an overall minimum barrier height of 1.37 m).

See Commentary for Figure 12.4.3.2.1i) regarding the cast-in-place concrete bridge parapet (1070 mm High).
12.4.6.1.d  Sidewalks separated from traffic by raised curbs

Use of sidewalks separated from traffic by raised curbs requires Approval and is typically only used in urban areas with low traffic volumes where design speeds are not greater than 60 km/h. Where sidewalks separated from traffic by raised curbs are used, only the following Combination Barriers have been accepted for use on highway bridges in B.C. and meet the crash testing requirements of S6-06. Any combination barrier proposed for use on such sidewalks for Ministry bridges in B.C., other than those shown in Figure 12.4.6.1.h below, require proof of meeting crash testing requirements of S6-06 and prior Approval.

**Figure 12.4.6.1.h**

Combination barriers on sidewalks separated from traffic by raised curbs
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.
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13.1 Scope

Commentary: Movable bridges shall not be used unless Approved.

Section 13 Movable bridges of the S6-06 does not address the following items in detail:

• Technical material advances such as UHMW polyethylene bearings and Teflon spherical plain bearings;
• Hydraulic drives;
• PLC control systems.

All these technologies may be acceptable, depending on the particular situation. Any variances from Section 13 requires consent of the Ministry.

13.5 General design requirements

13.5.9 Aligning and Locking

Commentary: CCTV systems are suggested to assist the operator in monitoring mechanisms not visible from the operator’s cabin.

13.5.11 New devices

Delete the second sentence and replace with the following:

If any such devices, materials, or techniques are proposed for use by the designer, they shall be in accordance with good commercial practice, shall have a background of successful application for similar usage, and shall be consented to by the Ministry.

13.5.12 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.

13.5.13 Durability

Commentary: The maintenance and inspection manual shall be prepared by the designer.

13.6 Moveable bridge components

13.6.2 Swing bridge components

13.6.2.3 Main pinions
13.6.2.3.2 Pinion-bearing supports

Delete and replace with the following:

The brackets and connections that support the main pinion bearings are critical to the bridge operation and shall be designed for at least twice the maximum design torque in the pinion.

Commentary: The maximum torque may occur under braking or acceleration.

13.6.3 Bascule bridge components

13.6.3.2 Locking devices

Commentary: The current code requires locking devices on the toe end of each girder. Depending on the design this may contribute to an overly complex mechanical installation. Locking devices on the toe ends of each outside girder is an acceptable alternative.

13.6.5 Vertical lift bridge components

13.6.5.3 Counterweight guides

13.6.5.3.2 Clearances

Commentary: The requirement for shims is to ensure the clearances can be correctly set. In addition the guide shoe mounting design shall facilitate easy adjustment and replacement in the future.

13.7 Structural analysis and design

13.7.3 Wind loads

13.7.3.4 Vertical wind, normal to the floor plan area

Commentary: Note that for unequal arm swing bridges, the surface area shall be the floor plan area of the larger arm.

13.7.6 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.
13.8 Mechanical system design

13.8.6 Wedges

Commentary: Unless separate supports are provided, the end-lift machinery of swing bridges shall also be capable of supporting the span under the specified loading. Systems which might creep under vibration or load shall not be used.

13.8.7 Brakes

13.8.7.1 General

13.8.7.1.3 Holding

Commentary: The braking requirements of this clause are also applicable for hydraulically driven bridges.

13.8.8 Frictional resistance

13.8.8.1 Machinery

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.

13.8.9 Torque

13.8.9.1 Torque at prime mover for main machinery

Commentary: For hydraulic cylinder actuated spans the bridge torque will need to be converted into an equivalent cylinder force.

13.8.9.4 Torque at prime mover for locks and wedges

Commentary: For hydraulic cylinder operated span lock and wedge machinery, the sum of all resistances to be overcome shall be reduced to a single equivalent force in the cylinder.

13.8.13 Bearing pressures (moving surfaces)

13.8.13.2 Determination of bearing pressures

Commentary: Where alternate bearing materials are considered, the maximum bearing pressures shall be in accordance with the supplier’s recommendations.
13.8.17  Machinery fabrication and installation

13.8.17.4  Plain bearings

13.8.17.4.3  Bushings

Delete the first sentence and replace with the following:

Bearings shall have bronze bushings unless otherwise consented to by the Ministry.

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

13.8.19  Power equipment

13.8.19.2  Brakes

13.8.19.2.1  General

*Commentary:* Brakes shall be arranged for hand release regardless of power source.

13.10  Electrical system design

13.10.3  General requirements for electrical installation

*Commentary:* This section includes a number of instructions aimed at the Contractor. The designer shall review the instructions and ensure the relevant instructions to the Contractor are incorporated into the Contract Documents prepared by the designer on behalf of the Ministry.

13.10.4  Working drawings

13.10.4.1  General

*Commentary:* This section includes a number of instructions aimed at the Contractor. The designer shall review the instructions and ensure the relevant instructions to the Contractor are incorporated into the Contract Documents prepared by the designer on behalf of the Ministry.

13.10.8  Motor temperature, insulation, and service factor

*Commentary:* AC motors should have Class F insulation in accordance with CSA or NEMA standards.
Programmable logic controllers

Delete the last paragraph and replace with the following:

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

Circuit breakers and Fuses

Commentary: Electronic Circuit Breakers with programmable trip settings are acceptable types of circuit breakers.

Electrical wires and cables

Commentary: The code prefers wire in conduit. Armoured cables with PVC jacketing may be an acceptable alternative. Therefore external wiring to control panels and consoles shall be wire types as listed in CEC Standard, Table 19, for exposed wiring in wet locations.

Raceways, metal conduits, conduit fittings, and boxes

Wireways and cable trays

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

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. The designer shall detail all wireways such that they do not impose a tripping hazard for the operator.

Commentary: The use of corrosion resistant material and lids is to reduce the problems with birds and their residue.

Spare parts

Commentary: The listing of spare parts specified for the Contractor to 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.

Construction

Commentary: This section includes instructions to the Contractor which need to be reviewed and appropriately transferred to the Contract Documents prepared by the designer on behalf of the Ministry.
13.12 Training and start-up assistance

**Commentary:** This section includes instructions to the Contractor which need to be reviewed and appropriately transferred to the Contract Documents prepared by the designer on behalf of the Ministry.

13.13 Operating and maintenance manual

**Commentary:** The designer should provide the Operation and Maintenance Handbook, not the Contractor. In addition to the drawings specified in this clause and Clause 13.10.4, 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;
- A hardcopy and softcopy of the software program, clearly listing all safety interlocks used in the PLC controls of the movable bridge;
- Calibration and set points of all devices; and
- A copy of the testing and commissioning records.
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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/content/technical/default.aspx

14.9 Transitory loads

14.9.1 Normal traffic

14.9.1.1 General

Delete and replace with:

Unless specified otherwise by the Ministry, evaluation shall be to the Evaluation Level 1 loading (vehicle trains) described in Clause 14.9.1.3. The BCL-625 design loading shall not normally be used for evaluation.

Commentary: Loadings that differ from the CL1-W loadings specified in Section 14.8 may be specified by the MoT on a project-to-project basis.

14.12 Target reliability index

14.12.1 General

If consented to by the Ministry, on low volume road bridges with AADT per lane of less than 500 and ADTT per lane of less than 100, the reliability index, $\beta$, used to determine the evaluation live load factors for Normal Traffic can be reduced by 0.25. However, the reduction in $\beta$ should not be applied if the level of truck weight enforcement at the location is low and it is suspected that the number and size of overloaded vehicles is significantly higher than normal. No reduction is permissible for the reliability index used to determine evaluation dead load factors or permit vehicle live load factors.

Commentary: The evaluation live load factors for Normal Traffic loadings contained in Section 14 are based on Highway Class A traffic volumes, ADT per lane of >4000 and ADTT per lane of >1000. Although the evaluation live load factors are relatively insensitive to variations in the ADTT, very large reductions in the ADTT can slightly reduce the required live load factors. The occurrence of an extremely heavy truck is less likely as the total number of trucks in the population decreases. For Normal Traffic, the reduction in the
required live load factor for a reduction in the ADTT from >1000 to <100 is equivalent to a 0.25 reduction in the reliability index, $\beta$.

Low volume roads may be subject to a lower level of truck weight enforcement which could encourage both a greater percentage of overloaded vehicles and higher levels of overload on the vehicles. Such conditions would counteract the benefits of having a low number of trucks operating on the route.

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.

14.14 Resistance

14.14.1 General

14.14.1.6 Shear in concrete beams

14.14.1.6.1 General

Delete and replace with the following:

Concrete beams shall have their shear resistance calculated in accordance
with Clause 8.9.3 with the exception that the factored sectional shear force
and factored bending moment used to calculate longitudinal strain of the
member, εx in Clause 8.9.3.8 is given by:

\[
\begin{align*}
V_f &= \alpha_D V_{DL} + F (\alpha_L V_{LL}) \\
M_f &= \alpha_D M_{DL} + F (\alpha_L M_{LL})
\end{align*}
\]

where, a value for \( F \) is first assumed, and the calculations repeated, iterating
the value of \( F \), until \( V_f \) 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

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 \( \varepsilon_x \) 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 \( \varepsilon_x \) 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 \( \varepsilon_x \), 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 \( V_r \) and \( M_r \) 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 \( (f_{nu}) \) for glue laminated beams shall be taken from Table 9.15.

### 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 Ministry of Transportation’s Regional Bridge Engineer.

**Commentary:** MoT 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 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 Bridge Standards and Procedures Manual, Volume 4, Seismic Retrofit Design Criteria.

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 Clause 3.8.3.2 shall be used for the rehabilitation design of bridges that are to carry unrestricted normal traffic after rehabilitation.
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16.11.3.1 General........................................................................................................ 3
16.1 Scope

16.1.4 Uses requiring Approval

Delete clause and replace with the following:

The following uses require Approval:

- Any fibre or matrix not listed in 16.12 or 16.13
- FRP as primary for lifeline structures.

16.4 Durability

16.4.3 Fibres in FRC

The use of alternative 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:

The total thickness of the deck slab, t, is at least 175 mm and at least s/15.

Delete Item (e) and replace with:

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. 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.

16.8 Concrete beams and slabs

16.8.1 General

Commentary: Gamal Tadros and John Newhook, under the sponsorship of ISIS Canada, have agreed to share their beam slab design spreadsheet as an aid to the designer. The designer is responsible for the use and results generated by this program. The Ministry does not warrantee the accuracy of this program and does not accept any liability with regards to its use.

Deck_Slab_Bridge_Design.xls

16.11 Rehabilitation of existing concrete structures with FRP

16.11.3 Shear rehabilitation with externally bonded FRP systems

16.11.3.1 General

Seismic retrofit must also conform to the requirements of the Ministry’s Bridge Standards and Procedures Manual - Volume 4, Seismic Retrofit Design Criteria.