




**BRITISH
COLUMBIA**
Ministry of Transportation
BRIDGE ENGINEERING BRANCH

**MANUAL OF
BRIDGE
STANDARDS
AND
PROCEDURES**

Preface

(By MoT)

Table of Contents

1. GENERAL	10
1.2 DEFINITIONS	11
1.2.1 <i>Administrative Definitions</i>	11
1.5 GENERAL PROVISIONS	11
1.5.1 <i>Application</i>	11
1.5.2.1 <i>Design Philosophy</i>	11
1.5.2.6 <i>Economics</i>	11
1.5.2.8 <i>Aesthetics</i>	11
1.5.4 <i>Construction</i>	11
1.5.4.3 <i>Construction Methods</i>	11
1.6 GEOMETRY	12
1.6.2.1 <i>General</i>	12
1.6.2.2 <i>Clearances</i>	12
1.7 BARRIERS	13
1.7.2 <i>Roadside Substructure Barriers</i>	13
1.8 AUXILLARY COMPONENTS	13
1.8.2 <i>Approach Slabs</i>	13
1.8.3 <i>Utilities on Bridges</i>	13
1.9 DURABILITY AND MAINTENANCE	14
1.9.2 <i>Bridge Deck Drainage</i>	14
1.9.2.2.1 <i>Crossfall and Grades</i>	14
1.9.2.2.2 <i>Deck Finish</i>	15
1.9.2.3 <i>Drainage System</i>	15
1.9.2.3.1 <i>General</i>	15
1.9.2.3.3 <i>Downspouts and Downpipes</i>	15
1.9.3 <i>Maintenance Requirements</i>	16
1.9.3.1.2 <i>Removal of Formwork</i>	17
1.9.3.1.5 <i>Access to Primary Component Voids</i>	17
1.9.3.3 <i>Bearing Maintenance and Jacking</i>	17
1.10 HYDRAULICS DESIGN	17
1.10.1.1.1 <i>General</i>	17
1.10.1.2 <i>Normal Design Flood</i>	17
1.10.1.3 <i>Check Flood</i>	18
1.10.1.5 <i>Design Flood Discharge</i>	18
1.10.4.1 <i>Scour Computations</i>	21
1.10.4.2 <i>Soils Data</i>	21
1.10.5.2 <i>Spread Footings</i>	21
1.10.5.2.2 <i>Protection of Spread Footings</i>	22
1.10.5.5 <i>Protective Aprons</i>	22
1.10.6.1 <i>Backwater - General</i>	22

1.10.7.1	<i>Soffit Elevation - Clearance</i>	23
1.10.9.3	<i>Channel Erosion Control – Slope Revetment</i>	23
1.10.11.2	<i>Culvert End Treatment</i>	24
1.10.11.6.6 (a)	<i>Soil-Steel Structures</i>	24
2.	DURABILITY	25
2.4	DESIGN FOR DURABILITY	26
2.4.2.4	<i>Bearing Seats</i>	26
2.4.2.5.1	<i>Expansion and/or Fixed Joints in Decks</i>	26
2.4.2.5.2	<i>Joints in Abutments, Retaining Walls, and Buried Structures</i>	26
2.4.2.6	<i>Drainage</i>	26
2.4.2.7	<i>Utilities</i>	26
3.	LOADS	27
3.8	LIVE LOADS	28
3.8.3	<i>CL-W Loading</i>	28
3.14	VESSEL COLLISION	28
3.14.2	<i>Bridge Classification</i>	28
3.16	CONSTRUCTION LOADS AND LOADS ON TEMPORARY STRUCTURES	28
A3.3	VESSEL COLLISION	28
4.	SEISMIC	29
4.4.1	<i>General</i>	30
4.4.2	<i>Importance categories</i>	30
4.4.3	<i>Zonal Acceleration Ratio</i>	32
4.4.4	<i>Seismic Performance Zones</i>	34
4.4.5	<i>Analysis for Earthquake Loads</i>	34
4.4.5.1	<i>General</i>	34
4.4.6	<i>Site Effects</i>	35
4.4.7.1	<i>Commentary:</i>	35
	<i>Replace the definition of “A” with the following:</i>	35
4.4.8	<i>Response Modification Factors</i>	35
4.4.8.1	<i>General</i>	35
4.4.10	<i>Design Forces and Support Lengths</i>	36
4.4.10.4.2	<i>Modified Seismic Design Forces (SPZ's 3 and 4)</i>	36
4.5	ANALYSIS	36
4.5.1	<i>General</i>	36
4.5.3.4	<i>Time-History Method (Multispan Bridges)</i>	37
4.5.3.5	<i>Static Pushover Analysis</i>	37
4.6.4	<i>Seismic Forces on Abutments and Retaining Walls</i>	38
4.6.5	<i>Soil-Structure Interaction</i>	38
4.6.6	<i>Fill Settlement and Approach Slabs</i>	39
4.7.3	<i>Seismic Performance Zone 3</i>	39
4.7.4.1.1	<i>Longitudinal Reinforcement</i>	39

4.7.4.1.2	Flexural Resistance	40
4.7.4.1.6	<i>Splices</i>	40
4.7.4.3	<i>Column Connections</i>	40
4.7.5.1	Confinement Length	41
4.8.3	<i>Sway Stability Effects</i>	41
4.10.1	<i>General</i>	41
4.10.4	<i>Site Effects And Site Coefficient</i>	41
4.10.6	Analysis Procedures.....	42
4.10.7	Clearance and Design Displacement of Seismic and other loads	42
4.10.11.2 (d)	Prototype Test	42
4.11	SEISMIC EVALUATION OF EXISTING BRIDGES	42
4.12.1	<i>Performance Criteria</i>	43
6.	FOUNDATIONS	44
7.	BURIED STRUCTURES	47
	INTRODUCTION	48
7.5.2	<i>Load Factors</i>	49
7.5.5.3	<i>Seismic Design of Concrete Structures</i>	50
7.6.1.1	<i>Structural Metal Plate</i>	50
7.6.2.1.1	<i>General</i>	51
7.6.2.1.2	<i>Dead Loads</i>	51
7.6.2.1.3	<i>Live Loads</i>	52
7.6.2.4	<i>Connection Strength</i>	52
7.6.3.1	<i>Minimum Depth of Cover</i>	52
7.6.3.3	<i>Durability</i>	52
7.6.4	<i>Construction Requirements</i>	53
7.6.4.5	<i>Structural Backfill</i>	54
7.6.5	<i>Special Features</i>	54
7.7	METAL BOX STRUCTURES.....	54
7.7.2.2	<i>Design Criteria for Connections</i>	55
7.7.3.1	<i>Depth of Cover (and Figure 7.7.4.1.1)</i>	55
7.7.3.2	<i>Durability</i>	55
7.7.4	<i>Construction</i>	55
7.7.4.1.2	<i>Material for Structural Backfill</i>	56
7.8	REINFORCED CONCRETE BURIED STRUCTURES.....	56
7.8.3.2	<i>Minimum Depth of Cover for Structures with Curved Tops</i>	56
7.8.4.4	<i>Earthquake Loads</i>	56
7.6.1.3	<i>Soil Materials</i>	57
	<i>S6-00 Evaluation of Existing Buried Structures for Overload Rating:</i>	58
8.	CONCRETE STRUCTURES	60
8.3	NOTATION	61
8.4	MATERIALS.....	61

8.4.2.1	<i>Reinforcing Bars</i>	61
8.4.2.1.3	<i>Yield Strength</i>	61
8.8.4.5	<i>Maximum Reinforcement</i>	62
8.9.3.5	<i>Nominal Shear Stress</i>	62
8.11	DURABILITY	63
8.11.2.1	<i>Concrete Quality</i>	63
8.11.2.1.2	<i>Concrete Placement</i>	64
8.11.2.1.4	<i>Cold Joints</i>	65
8.11.2.1.5	<i>Slip-Form Construction</i>	65
8.11.2.1.6	<i>Finishing</i>	65
8.11.2.1.6	<i>Finishing</i>	66
8.11.2.2	<i>Concrete Cover and Tolerances</i>	66
8.11.2.3	<i>Corrosion Protection for Reinforcement, Ducts and Metallic Components</i>	67
8.11.2.6	<i>Drip Grooves</i>	68
8.14.3	<i>Transverse Reinforcement for Flexural Components</i>	68
8.15.9	<i>Splicing of Reinforcement</i>	70
8.16.7	<i>Anchorage of Attachments</i>	70
8.18.2	<i>Minimum Slab Thickness</i>	70
8.18.5	<i>Diaphragms</i>	71
8.19	COMPOSITE CONSTRUCTION	71
8.19.1	<i>General</i>	71
8.19.3	<i>Shear</i>	71
8.20	CONCRETE GIRDERS	72
8.20.1	<i>General</i>	72
8.20.3.2	<i>Bottom Flange</i>	72
8.20.4	<i>Web Thickness</i>	72
8.20.7	<i>Post-Tensioning Tendons</i>	73
8.20.8	<i>Diaphragms</i>	73
8.21	MULTIBEAM DECKS	74
10.	STEEL STRUCTURES	76
10.4	MATERIALS	77
10.4.1	<i>General</i>	77
10.4.2	<i>Structural Steel</i>	77
10.4.2	<i>Structural Steel</i>	78
10.4.5	<i>Bolts</i>	80
10.4.10	<i>Galvanizing and Metallizing</i>	80
10.6	DURABILITY	81
10.6.3.1	<i>Structural Steel</i>	81
10.6.3.2	<i>Cables, Ropes, and Strands</i>	82
10.6.4	<i>Other Components</i>	82
10.7	DESIGN DETAIL REQUIREMENTS	82
10.7.1	<i>General</i>	82
10.7.1.a	<i>Flange Widths between Splices</i>	83

10.7.1.b	<i>Transition of Flange Thicknesses at Butt Welds</i>	83
10.7.1.c	<i>Recommended Details</i>	83
10.7.4	<i>Camber</i>	93
10.7.4.1	<i>Design</i>	93
10.7.4.1	<i>Design</i>	93
10.10	BEAMS AND GIRDERS	94
10.10.8	<i>Bearing Stiffeners</i>	94
10.10.8.1	<i>Web Crippling and Yielding</i>	94
10.17	STRUCTURAL FATIGUE	95
10.17.1	<i>General</i>	95
10.17.2	<i>Live Load-Induced Fatigue</i>	95
10.17.2.6	<i>Fatigue Resistance of Stud Shear Connectors</i>	95
10.18	SPLICES AND CONNECTIONS.....	95
10.18.1	<i>General</i>	95
10.19	ANCHORS.....	95
10.19.1	<i>General</i>	95
10.20	PINS, ROLLERS, AND ROCKERS	96
10.20.2	<i>Pins</i>	96
10.23	FRACTURE CONTROL	96
10.23.5	<i>Welding Corrections and Repairs on Fracture-Critical Members</i>	96
10.23.5.6 (g)	<i>Minimum Steps for Repair</i>	96
10.24	CONSTRUCTION REQUIREMENTS FOR STRUCTURAL STEEL	96
10.24.5	<i>Welded Construction</i>	96
10.24.5.1	<i>General</i>	96
10.24.6	<i>Bolted Construction</i>	97
10.24.6.1	<i>General</i>	97
10.24.8	<i>Quality Control</i>	97
10.24.8.2	<i>Non-Destructive Testing of Welds</i>	97
10.24.9	<i>Transportation and Delivery</i>	97
11.	JOINTS AND BEARINGS	98
11.4	COMMON REQUIREMENTS.....	99
11.4.2	<i>Design Requirements</i>	99
11.5	DECK JOINTS.....	100
11.5.1	<i>General Requirements</i>	100
11.5.1.1	<i>Functional Requirements</i>	100
11.5.1.2	<i>Design Loads</i>	102
11.5.3	<i>Design</i>	102
11.5.3.2	<i>Components</i>	102
11.5.3.2.4	<i>Bolts</i>	102
11.6	BRIDGE BEARINGS.....	102
11.6.1	<i>General</i>	102
11.6.1	<i>General</i>	103

11.6.1	<i>General</i>	103
11.6.4	<i>Spherical Bearings</i>	103
11.6.4.1	<i>General</i>	103
11.6.6	<i>Elastomeric Bearings</i>	104
11.6.6.3	<i>Geometric Requirements</i>	104
12.	BARRIERS AND HIGHWAY ACCESSORY SUPPORT	105
12.5	BARRIERS	106
12.5.2	<i>Traffic Barriers</i>	106
12.5.2.1	<i>Performance Level</i>	106
12.5.2.2	<i>Geometry and End Treatment Details</i>	112
12.5.3	<i>Pedestrian Barriers</i>	113
12.5.3.1	<i>Geometry</i>	113
12.5.4	<i>Bicycle Barriers</i>	113
12.5.4.1	<i>Geometry</i>	113
12.5.5	<i>Combination Barriers</i>	113
12.5.5.1	<i>Geometry</i>	121
13.	MOVABLE BRIDGES	122
13.1	SCOPE	123
13.4	MATERIALS	123
13.4.4	<i>Timber</i>	123
13.4.9	<i>Bolts</i>	123
13.5	GENERAL DESIGN REQUIREMENTS	123
13.5.9	<i>Aligning and Locking</i>	123
13.5.12	<i>Access for Routine Maintenance</i>	123
13.5.13	<i>Durability</i>	124
13.6	MOVEABLE BRIDGE COMPONENTS	124
13.6.1.1.7	<i>Concrete</i>	124
13.6.2.1.2	<i>Disc Bearings</i>	124
13.6.1.4	<i>Span aligning and Locking</i>	124
13.6.2.3.2	<i>Pinion Bearing Supports</i>	125
13.6.3.2	<i>Locking Devices</i>	125
13.6.5.3.2	<i>Clearances</i>	125
13.7	STRUCTURAL ANALYSIS AND DESIGN	125
13.7.3.4	<i>Vertical Wind, Normal to the Floor Plane Area</i>	125
13.7.6	<i>Hydraulic Cylinder Connections</i>	125
13.8	MECHANICAL SYSTEM DESIGN	126
13.8.6	<i>Requirements for Wedges</i>	126
13.8.7.1.2	<i>Holding</i>	126
13.8.8.1	<i>Machinery</i>	126
13.8.9.1	<i>Torque at Prime Mover for Main Machinery</i>	126
13.8.9.4	<i>Torque at Prime Mover for Locks and Wedges</i>	126
13.8.9.6	<i>Torque for Lock and Wedge Machinery</i>	127

13.8.11	<i>Machinery Loads</i>	127
13.8.13.2	<i>Determination of Bearing Pressure</i>	127
13.8.17.4.3	<i>Bushings</i>	127
13.8.19.2.2	<i>Electrically Operated Brakes</i>	128
13.10	ELECTRICAL SYSTEM DESIGN	128
13.10.3	<i>General Requirements for Electrical Installation</i>	128
13.10.4.1	<i>General</i>	128
13.10.8	<i>Motor Temperature, insulation, and Service Factor</i>	128
13.10.11	<i>Speed of Motors</i>	128
13.10.15	<i>Electrically Operated Motor Brakes</i>	128
13.10.21	<i>Programmable Logic Controllers</i>	129
13.10.26	<i>Circuit Breakers and Fuses</i>	129
13.10.36.3	<i>Control</i>	129
13.10.39	<i>Electrical Wires and Cables</i>	129
13.10.42.7	<i>Wireways</i>	129
13.10.50	<i>Spare Parts</i>	130
13.11	CONSTRUCTION	130
13.12	OPERATIONAL INSTRUCTIONS	130
13.13	OPERATIONAL AND MAINTENANCE HANDBOOK	130
14.	EVALUATION	131
14.5	CONDITION INSPECTION	132
14.6	MATERIAL STRENGTHS	132
14.6.3	<i>Strengths Based on Date of Construction</i>	132
14.6.3.1	<i>Structural Steel</i>	132
14.6.3.3	<i>Reinforcing Steel</i>	132
14.8	TRANSITORY LOADS	133
14.11	TARGET RELIABILITY INDEX	133
14.11	TARGET RELIABILITY INDEX	134
14.11.2	<i>Element Behaviour</i>	134
14.11.4	<i>Important Structures</i>	135
14.13	RESISTANCE	136
14.13.2	<i>Resistance Adjustment Factors</i>	136
14.13.2	<i>Resistance Adjustment Factors</i>	137
14.13.2	<i>Resistance Adjustment Factors</i>	137
14.17	BRIDGE POSTING	138
14.17.1	<i>General</i>	138
14.18	FATIGUE	138

1.

General

1.2 DEFINITIONS

1.2.1 Administrative Definitions

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

Regulatory Authority: means the Ministry of Transportation of BC (MoT) or its representative.

1.5 GENERAL PROVISIONS

1.5.1 Application

Formulations from other codes can only be used with the written approval of MoT.

1.5.2.1 Design Philosophy

Geotechnical design will be undertaken using Working Stress Design, until further notice.

1.5.2.6 Economics

The first sentence is amended to:

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.

1.5.2.8 Aesthetics

Add:

General guidelines for bridge aesthetics are given in the MoT's Manual of Aesthetic Design Practice.

1.5.4 Construction

The MoT Standard Construction Specifications take precedence over this section of S6-00.

1.5.4.3 Construction Methods

Commentary:

Girders which require transportation by truck on the highway system shall be sized in order that the following limits are not exceeded:

<i>Length</i>	<i>47.5 m out to out including truck</i>
<i>Width</i>	<i>4.4 m</i>
<i>Weight</i>	<i>64 tonnes including truck (GVW)</i>

*(Possible exemptions: Lower Mainland Horseshoe Bay to Langley - 81 tonnes.)
If particular girders are close to the guidelines or if a slightly larger girder is required, the design engineer should inquire with the heavy haul division of Arrow Transport, Davey Cartage and Rocky Mountain, where applicable, to see if the above limiting constraints can be met.*

The approximate limiting constraints for steel girders are the maximum length of 41.5 meters or weight of 43,500 kg.

The approximate limiting constraints for prestressed concrete girders are the maximum length of 39 meters or weight of 43,000 kg. Girder weights shall be calculated using concrete densities of 2650 kg/m³ for I-Girders and 2720 kg/m³ for box girders to provide allowance for spread of formwork and higher reinforcing steel densities.

For the transportation of very short heavy sections, trucking companies should be consulted for girder weights that will meet bridge overload formula and 64 tonne maximum G.V.W.

The design engineer shall determine and verify whether the girder of a particular length and weight can be transported to the bridge site, via, negotiating tight corners and switchbacks and complying with posted load limits on bridges en route.

1.6

GEOMETRY

1.6.2.1

General

This clause is amended such that curb heights shall be at least 200 mm. In addition, the MoT supplement to the TAC Geometric Design Guide for Canadian Roads shall be followed.

1.6.2.2

Clearances

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

The minimum vertical clearance shall be maintained throughout the life of the structure, (i.e., long-term settlement of supports, superstructure deflection must be accounted for).

Horizontal separation between structures shall be maintained at all locations to ensure access for maintenance and to avoid pounding during seismic events...

1.7

BARRIERS

1.7.2

Roadside Substructure Barriers

This clause is amended by the addition of: The standard sidewalk railing shall extend 3 metres beyond the bridge abutments.

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

1.8

AUXILLARY COMPONENTS

1.8.2

Approach Slabs

The inclusion of or provision for approach slabs on paved roads shall be based on site specific conditions as approved by the MoT. The approach slabs shall be 6 metres in length, located at least 100 mm below the finished grade, anchored to the abutments and shall be as wide as the deck. A cover of 70 mm shall be used for the top reinforcing bars.

Approach slabs shall have a 100 mm minimum asphalt overlay but do not require a waterproofing membrane.

The maximum permissible differential settlement between the abutment supported end and the grade supported end of each slab shall be:

- At the end of the Warranty Period-25 mm;
- Long term (25 years)-50 mm

Approach slabs shall be provided for bridges on number routes where settlement of greater than 50 mm is anticipated behind the abutment or in seismic areas 3 and 4. Approach slabs are not required on low volume roads.

1.8.3

Utilities on Bridges

For procedures and guidelines on installation of utilities on or near bridges, see the Ministry's "Utility Policy Manual."

1.9

DURABILITY AND MAINTENANCE

1.9.2

Bridge Deck Drainage

Commentary

In general the following objectives relate to bridge deck drainage:

- *The maximum encroachment into the road way by a flowing swale or gutter shall not be more than 1.2 m rather than the 1.5 m given in the S6-00;*
- *Water must not pond on decks;*
- *Deck drainage inlets should be avoided when possible.*

Deck drainage inlets are often not required for bridges with the following characteristics:

- *Two lanes or less;*
- *Normal crossfall;*
- *More or less symmetrical vertical alignment;*
- *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 the environmental agencies, and the MoT.

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.

Except near the crowns of vertical crest curves, a minimum longitudinal gradient of 1.0% and a minimum of 2% crossfall shall be provided on bridge decks. Sidewalk surfaces shall be provided with 2% crossfall.

1.9.2.2.1

Crossfall and Grades

The last paragraph is amended to read:

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.

1.9.2.2.2 Deck Finish

1.9.2.3 Drainage System

1.9.2.3.1 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.

1.9.2.3.3 Downspouts and Downpipes

This clause is amended by:

“Downspouts shall project a minimum of 500 mm below adjacent members ... “

Commentary

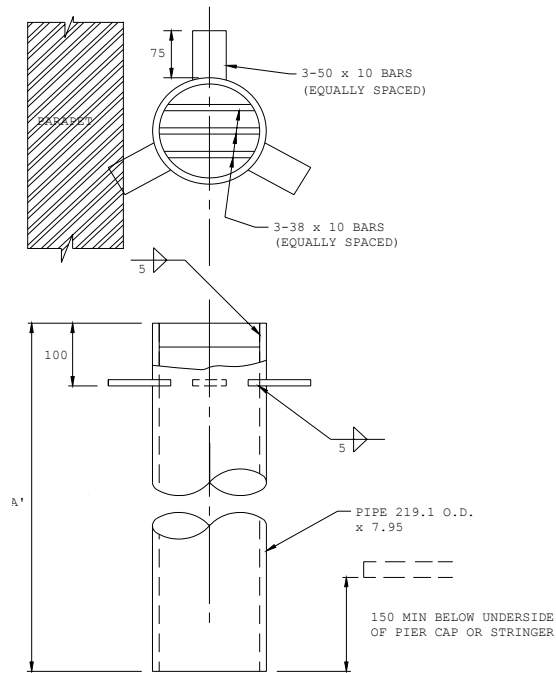
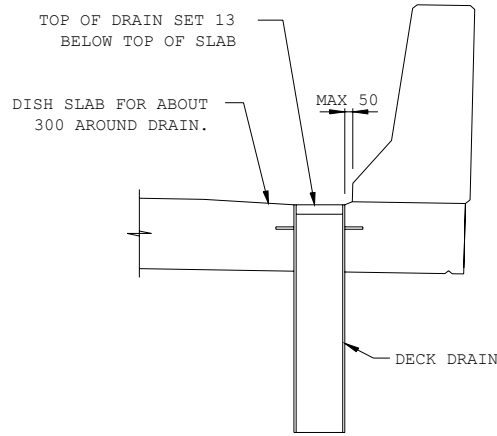
Scuppers for lateral drainage may be more effective and practical on flat grades than having drainpipes installed in the prestressed concrete box stringers. The scuppers are easily installed on a slope with the outlets fully embedded in the deck overlay prior to the construction of the concrete parapets.

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

Catchbasins are normally required just beyond the limits of the structure. A continuous length of curb and gutter should be provided to connect the bridge curb or barrier to the catchbasin to prevent washouts at the ends of the wingwalls.

Drain pipes shall be straight to facilitate cleaning. Drains shall be galvanized.

Support brackets may be required for girders and steel trusses deeper than 2.3 metres.

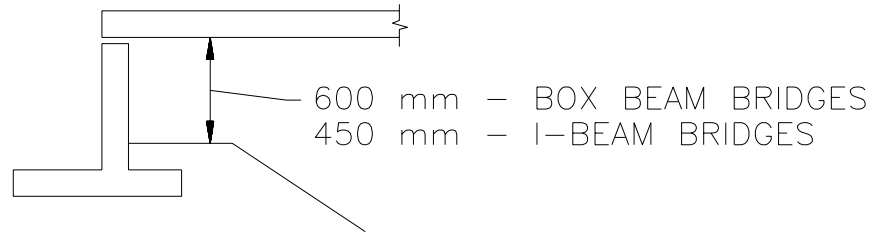


1.9.3

Maintenance Requirements

The following minimum clearances shall be maintained between the top of beam and the underside of the superstructure to facilitate the inspection of bridges:

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



1.9.3.1.2 Removal of Formwork

All formwork shall be removed.

1.9.3.1.5 Access to Primary Component Voids

“Drains shall be screened to prevent birds, animals, insects etc. from entering the voids.”

1.9.3.3 Bearing Maintenance and Jacking

Enough vertical and horizontal space must be provided between the superstructure and the substructure to accommodate the jacks required for bearing replacement. As a rule-of-thumb 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 shoe plates should be bolted and not welded.

1.10 HYDRAULICS DESIGN

1.10.1.1.1 General

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).

1.10.1.2 Normal Design Flood

Replace the first paragraph with the following:

The return period for the design flood is as follows:

Bridges	200-Year
Buried Structures and Culverts (>3m Span)	200-Year
Low-Volume Road - Bridges/Buried Structures	100-Year

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:

<http://wlapwww.gov.bc.ca/wat> and click on "Floodplain Mapping".

Maps BC, Victoria, BC

Low-volume roads shall be considered on 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 highways 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, MoT.

1.10.1.3

Check Flood

Delete the paragraphs since these are not applicable to the Province of British Columbia.

1.10.1.5

Design Flood Discharge

Replace the paragraph with the following:

The design floods shall be estimated by the following methods, unless otherwise approved.

- a) For large drainage areas ($>25 \text{ km}^2$), the recommended design flow calculation methods are:
 - Station Frequency Analysis
 - Regional Frequency Analysis

Commentary:

The most commonly used distributions to describe extreme flows in the Province of British Columbia 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)
<http://www.msc.ec.gc.ca/wsc> and search for "HYDAT".*

b) For drainage areas less than 25 km², design flows can be estimated using the following:

- *SCS Unit Hydrograph Method*

If the drainage areas approach the upper limits, efforts shall be made to 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).*

Hydrologic soil groups and soil/land use curve numbers (CN) can be obtained from the following:

- *Soils Maps from the Ministry of Sustainable Resource Management.*

Textural classifications provided by geotechnical investigations

c) For urban and small drainage areas (<10 km²), the recommended design flow calculation is the Rational Method:

$$Q_p = \frac{CiA}{360} \text{ where}$$

Q_p = peak flow, m³/s

C = runoff coefficient

i = rainfall intensity = P/T_C, mm/hr

P = total precipitation, mm

T_C = time of concentration, hr

A = drainage area in hectares

Commentary:

*For information on the Rational Formula Method, refer to:
TAC Guide to Bridge Hydraulics, Section 3.4.1 (June 2001)*

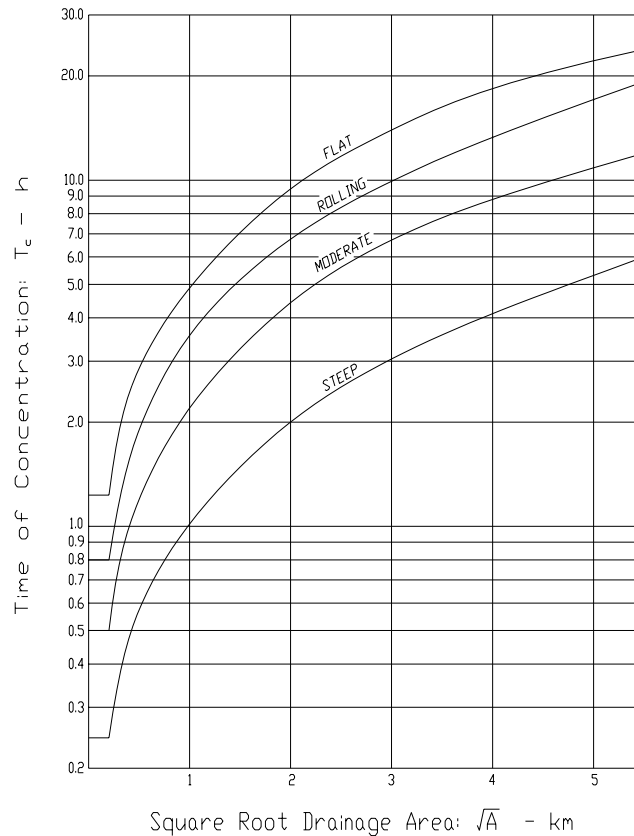
In selecting the runoff coefficient (C), the land is considered as developed to the limit of its zoning. For smaller drainage areas, detailed land use information may be available, resulting in a more precise estimate of the runoff coefficients. With larger drainage basins, only general information is usually available, resulting in the need to use conservative assumptions of the runoff coefficients..

The table below from the Ministry of Sustainable Resource Management, Hydrology Section presents conservative C values for coastal type drainage basins where the maximum runoff occurs as a result of fall and winter rains.

Maximum Runoff Coefficients for Coastal Type Basin

Surface Cover/ Physiography	Impermeable	Forested	Agricultural	Rural	Urban
Mountain (>30%)	1.00	0.90			
Steep Slope (20%-30%)	0.95	0.80			
Moderate Slope (10%-20%)	0.90	0.65	0.50	0.75	0.85
Rolling Terrain (5%-10%)	0.85	0.50	0.40	0.65	0.80
Flat (<5%)	0.80	0.40	0.30	0.55	0.75

Time of Concentration



The Water Management Method was developed by the Ministry of Sustainable Resource Management, Hydrology Section, as shown above, is limited to drainage areas up to 25 km². The time of concentration is dependent on the basin characteristics and the following parameters shall be considered:

<i>flat</i>	<i>approximately 0% slope</i>
<i>rolling</i>	<i>approximately 1% slope</i>
<i>moderate</i>	<i>approximately 2.5% slope</i>
<i>steep</i>	<i>greater than 10% slope.</i>

For agricultural and rural basins, the curves labeled flat and rolling should be used. For forested watersheds, the curves labeled rolling, moderate and steep should be used.

1.10.4.1 Scour Computations

The following information shall be added:

Hec-Ras numerical analysis is approved for the computation of general and local scour based on the D₅₀ and D₉₀ streambed particle sizes.

Commentary:

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

D₅₀ = Bed material particle size in a mixture of which 50% are smaller.

D₉₀ = Bed material particle size in a mixture of which 90% are smaller.

1.10.4.2 Soils Data

The following information shall be added:

If the Hec-Ras numerical analysis is used, the D₅₀ and D₉₀ streambed particle sizes shall be determined.

Commentary:

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

D₅₀ = Bed material particle size in a mixture of which 50% are smaller.

D₉₀ = Bed material particle size in a mixture of which 90% are smaller.

1.10.5.2 Spread Footings

The following information shall be added:

Abutments and piers shall have piled foundations, unless otherwise approved by the Ministry.

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.

1.10.5.2.2 **Protection of Spread Footings**

The following information shall be added:

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 approved by the Ministry.

1.10.5.5 **Protective Aprons**

Replace the second paragraph with the following:

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 these specifications.

The class of riprap used shall be based on the design chart available in the Ministry BC Supplement to TAC Geometric Design Guide, (2001), Section 1030, **Figure 1030A**.

Commentary:

Refer to:

<http://www.gov.bc.ca> and click sequentially on “Ministry and Organizations”, “Transportation”, “Report and Publications”, “Engineering Publications”, “Construction Maintenance Publications”, and “Standard Specifications for Highway Construction”.

<http://www.gov.bc.ca> and click sequentially on “Ministry and Organizations”, “Transportation”, “Report and Publications”, “Engineering Publications”, “Traffic, Electrical, Highway Safety and Geometric Standards Section”, and “BC Supplement to TAC Geometric Design Guide, 2001 Edition”.

1.10.6.1 **Backwater - General**

The following information shall be added:

Hec-Ras numerical analysis is approved for determining the backwater profile.

Commentary:

Numerical analyses for backwater profiles are generally not required for low- volume road bridges/buried structures, unless otherwise specified.

1.10.7.1 Soffit Elevation - Clearance

Amend the first paragraph:

Unless otherwise approved by the Ministry, the clearance between the soffit and the Q_{200} design flood elevation shall not be less than 1.5 metres for bridges; and not less than 0.3 metre on low volume road bridges for the Q_{100} flood elevation.

Commentary:

Clearances shall be increased for crossings subject to ice flows, debris flows and debris torrents.

*For waters the Coast Guard declares 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). 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 meters above the 100-year flood level is considered adequate. For small watercourses less clearance can be considered if cost and road design factors are compromised significantly. The **Coast Guard**, having authority of works over or in **Navigable Waters**, can declare 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 Coast Guard and Harbours' Boards shall be coordinated by the Rail, Navigable Waters Coordinator*

For additional information, refer to

<http://www.gov.bc.ca> and click sequentially on “Ministry and Organizations”, “Transportation”, “Report and Publications”, “Engineering Publications”, “Bridge Engineering Section Publications”, and “Manual of Bridge Standards and Procedures”.

1.10.9.3 Channel Erosion Control – Slope Revetment

The following information shall be added:

Riprap shall be used for protecting the bank slopes and bridge end fills of abutments, and shall conform to the clauses in Section 205, of the Ministry Standard Specifications for Highway Construction. The revetment shall be keyed into the streambed to the estimated total scour depth. 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 Ministry BC Supplement to TAC Geometric Design Guide, Section 1030, **Figure 1030A**.

Commentary: Refer to:

<http://www.gov.bc.ca> and click sequentially on “Ministry and Organizations”, “Transportation”, “Report and Publications”, “Engineering Publications”,

“Construction Maintenance Publications”, and “Standard Specifications for Highway Construction”.
<http://www.gov.bc.ca> and click sequentially on “Ministry and Organizations”, “Transportation”, “Report and Publications”, “Engineering Publications”, “Traffic, Electrical, Highway Safety and Geometric Standards Section”, and “BC Supplement to TAC Geometric Design Guide, 2001 Edition”.

1.10.11.2 Culvert End Treatment

The following information shall be added:

Cut-off walls shall be used at both ends, unless otherwise approved by the Ministry.

Commentary:

This will alleviate failure of culverts from uplift and piping during extreme flood events.

In mid July of 2001, heavy rainfall up to 75mm over a period of 2 days resulted in high flows in the creeks and rivers of Peace Highway District. A number of large diameter culverts, without cut-off walls, on low volume roads were damaged or washed away due to debris, piping and uplift.

1.10.11.6.6 (a) Soil-Steel Structures

The following information shall be added:

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.

2.

Durability

2.4 DESIGN FOR DURABILITY

2.4.2.4 Bearing Seats

Bridges shall be designed with provisions for jacking during future maintenance operations and the proposed locations and procedures for jacking shall be indicated on the contract drawings.

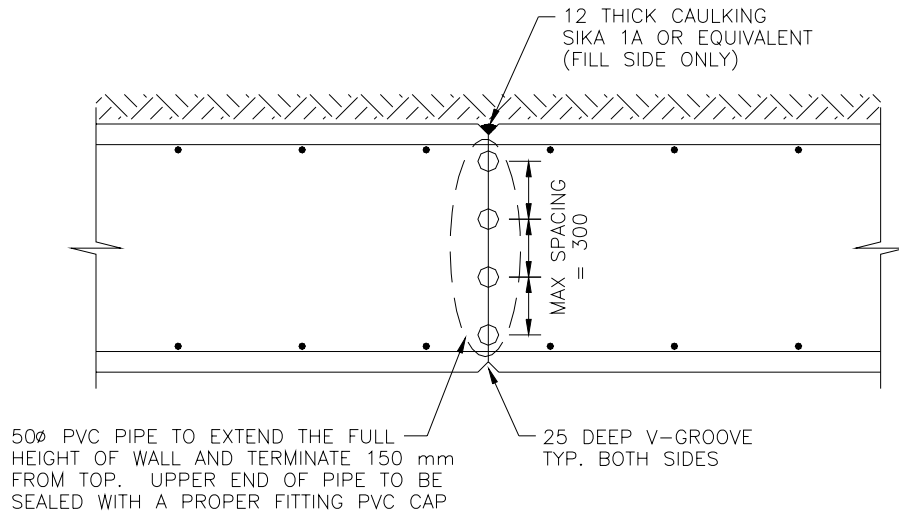
2.4.2.5.1 Expansion and/or Fixed Joints in Decks

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

2.4.2.5.2 Joints in Abutments, Retaining Walls, and Buried Structures

Typical details for control joints are shown *in Figure 2.4.2.5.2* below.

Figure 2.4.2.5.2 Typical Control Joint



NOTES:

- CAULKING IS TO BE PROTECTED WITH 12 THK x 150 WIDE ASPHALT IMPREGNATED FIBREBOARD
- MAXIMUM SPACING OF CONTROL JOINTS = 3.0 m

2.4.2.6 Drainage

This clause is amended to require all downspouts to extend at least 500 mm below adjacent members.

2.4.2.7 Utilities

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

3.

Loads

3.8 LIVE LOADS

3.8.3 CL-W Loading

The use of loads other than CL-625 requires the written approval of the MoT.

3.14 VESSEL COLLISION

3.14.2 Bridge Classification

Class I and II bridges shall correspond to “emergency route” and “other bridges” in Section 4, respectively.

3.16 CONSTRUCTION LOADS AND LOADS ON TEMPORARY STRUCTURES

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 drawings by the designer.

Actual construction live loads shall be approved by the MoT prior to construction.

A3.3 VESSEL COLLISION

Method II analysis is required for “Class II” bridges while either method can be used for “Class I” bridges. This is consistent with the approach taken by AASHTO 1991.

4.

Seismic

This supplement to Section 4, Seismic Design, of the Canadian Highway Bridge Design Code CAN/CSA-S6-00 (S6-00) outlines the specific requirements of the B.C. Ministry of Transportation (MoT), the Regulatory Agency in British Columbia, in the application of S6-00. This document must be read in conjunction with the S6-00 for the seismic design and construction of transportation structures under the jurisdiction of the Ministry.

“Transportation structures” in this document includes permanent structures including:

- Bridges, including their foundations and supporting soils and end fills.
- Retaining walls.
- Sign structures.
- Underground structures (refer also to the Ministry’s supplement to Section 7 of S6-00.)
- Snowsheds.

Commentary:

Transportation facilities not yet described in this Document include:

- *Earth embankments.*
- *Dykes, marine or harbour facilities (including quay-walls).*
- *Other all-soil facilities.*

4.1

SCOPE

Add the following paragraph:

Supplemental requirements for the seismic design, seismic evaluation and retrofit design of bridges contained in this Document shall be adopted as minimum requirements.

4.4

EARTHQUAKE EFFECTS

4.4.1

General

The third paragraph shall have the following sentence added at the end:

For structures in SPZ 3 and 4, earthquake load effects for capacity-protected members shall be determined in accordance with capacity design principles for forces resulting from inelastic action of members with which they connect.

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 both the 475-year and the 1000-year return period events.

Add the following sentences to the end of this clause:

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

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

Commentary:

The Importance Category, along with the zonal acceleration level, lead to a "Seismic performance zone" (SPZ) from Table 4.4.4.1. It is suggested that the SPZ for LVR bridges be limited to a maximum of 2, whereas other highway bridges may be as high as SPZ 4. This parameter, among others, is subsequently used to determine the required minimum level of seismic analysis.

A discussion of the role and characteristics of bridges to assist in classification are contained in S6-00 (code and commentary). The Ministry will designate the Importance Category for each bridge. Additional background on comparable performance objectives and damage levels are contained in ATC-32.

Performance objectives and damage levels contained in the more recent ATC-49 document are based on a 2500-year return period event (probability of of 2% in 50 years), which differs from S6-00.

To relate probabilities of exceedence, return periods, and design life the following relationship is used:

$$R = [1 - (1 - p)^{1/t}]^{-1}$$

Where

R = return period

p = probability of exceedence in period t

t = duration consistent with p (e.g. 1 year for an annual probability of exceedence)

Dividing the period (t) by the annual probability of exceedence provides a useful approximation of the return period.

4.4.3

Zonal Acceleration Ratio

Add sentence to clause (b):

The zonal acceleration ratio obtained from the Pacific Geosciences Centre shall be the value having a probability of exceedence of 10% in 50 years. Requests to the PGC shall specify that the methodology to be used will be consistent with that used to derive seismic design values in Table A.3.1.7 (ie. for a bridge being designed to S6-00).

Commentary:

The zonal acceleration ratio and methodology contained in S6-00 remains appropriate at this time for the seismic design of bridges. It is recognized that the proposed values and methodology contained in the draft NBCC 2005 make several important changes for the design of buildings. Seismicity is based on fourth generation seismic hazard maps developed Canada wide, and incorporate a number of improvements to the definition of seismic hazard. Important changes include:

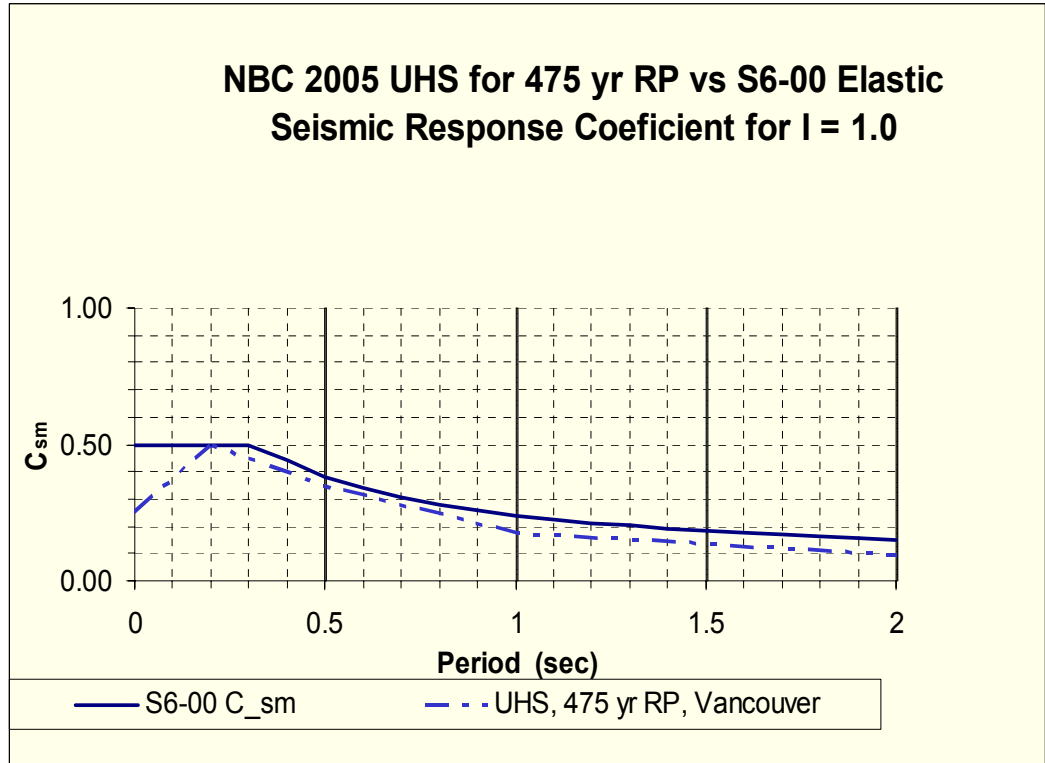
- *Adoption of a probability level of 2% in 50 years (1000 year return period rather than 475-year return period in S6-00)*
- *Definition of spectral accelerations defining a complete firm ground uniform hazard spectrum.*
- *Adoption of a median level of hazard (50th percentile shaking) rather than a mean level (65th to 75th percentile range) as implicitly used previously.*

It is important to specify to the PGC that the methodology to be used is consistent with that used to derive values for the S6-00 code. The methodology currently used by default assumes zonal acceleration values will be consistent with the NBCC 2005 methodology. Zonal accelerations and velocities derived for the same return period (in this case 475 years) will therefore not be the same using the old and new methodology. PGC scientists are aware of this issue, and are able to provide values suitable for use with either the NBC2005 or the S6-00. Harmonization of the two codes and methodologies may take a number of years to complete

Figure 4-1 below illustrates the design spectra corresponding to S6-00 and the firm ground uniform hazard spectrum for Vancouver, scaled down to a 475-year return period. NBCC 2005 will provide the VHS for a 2500-year return period. This plot illustrates that the spectral shapes are similar to this example, but ordinates vary significantly at periods exceeding 0.5 seconds. Applying an importance factor of 1.5 to the S6-00 plot would yield a design spectrum significantly higher than the 2500-year firm ground VHS. As noted above, harmonization and appropriate calibration to earthquake experience of S6-00

with the proposed NBCC 2005 is necessary, and is expected to be performed for a future release of S6-00.

Figure 4-1



In addition, other parameters on both sides of the “demand” and “resistance” side within NBC 2005 of the equation have been adjusted and calibrated for consistency. These include importance factors, structural ductility factors, etc. In particular, ductility factors have been broken into two components (Ro (over strength) and Rd (ductility), which are multiplied together for a net reduction for modified seismic design forces. During the calibration of forces undertaken for past building codes in Canada, one objective was to avoid significant unjustifiable changes in design force levels between codes. For the 2005 NBCC, significant changes in seismic forces will be made, which vary with location and structural period.

The necessary calibration and integration of the NBCC 2005 methodology into the S6-00 framework has not been performed at this time. One aspect of the 4th generation seismic hazard models that may be integrated into the seismic design of bridges would be the spectral shapes for the firm-ground uniform hazard spectra (UHS). These would need to be calibrated and scaled to correspond to a 10% in 50 year hazard. A reasonable scale factor to convert the 1000-year return period 5% damped firm ground acceleration spectrum to a 475-year return period is 0.8.

While period-dependent, this single factor is believed to be applicable in British Columbia in the period range of 0 to 2 seconds. It should be recognized that any such linear scaling of the UHS would alter the probability of exceedence at some periods; however, the error is likely to be less than currently contained in the spectra implicit in S6-00.

Additional information is available in the Canadian Journal of Civil Engineering (Volume 30, 2003) and through the PGC web site. Two selected references are:

- *Overview of seismic provisions of the proposed 2005 edition of the National Building Code of Canada, by Art Heidebrecht, pg 241-254*
- *Development of seismic hazard maps for the proposed 2005 edition of the National Building Code of Canada, by John Adams and Gail Atkinson, pg. 255-271.*

Refer also to Clause 4.4.7 of this supplement.

The Pacific Geosciences Centre in Sidney, B.C. can be contacted at:

(www.pgc.nrcan.gc.ca/index_e.html,

Phone: (250) 363-6500 Fax :(250) 363-6565).

4.4.4 Seismic Performance Zones

Commentary:

S6-00 prescribes a higher level of seismic design importance to structures in SPZ 3 and SPZ 4. The SPZ is defined by the peak horizontal ground acceleration (at 10% in 50 year probability) and the importance classification of the bridge. SPZ affects analysis, performance, and detailing requirements. At this time the implications on seismic design of various sites and bridges around the Province has not been determined. Future editions of this Supplement may propose modifications or refinements to the design process for selected bridges and sites.

4.4.5 Analysis for Earthquake Loads

4.4.5.1 General

Replace second sentence with: "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 S6-00, and Clause 4.4.6.6 allows for engineering judgement. 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.

4.4.7.1

Commentary:

Replace the definition of “A” with the following:

A = zonal acceleration ratio specified in clause 4.4.3 or the PHA obtained from the PGC or Appendix A4.

Importance factors “I” contained in this clause were based on a calibration of S6-00 with the AASHTO LRFD guide seismic specifications in place at the time of S6-00 writing. For “Lifeline” bridges an Importance Factor of 3 is specified. The use of I=3 may lead to conservative design forces for some bridges and components (e.g. abutment and connection forces, design plastic hinge forces in single-column piers) as well as a disincentive for a ‘capacity design’ approach. For “design-build” delivery, unless otherwise specified in project-specific design criteria, the use of I = 3 shall apply.

4.4.8 Response Modification Factors

4.4.8.1 General

Add the following paragraph after the first paragraph:

The axial loads shall be taken as those consistent with the plastic mechanism adopted for design.

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.

*It further **requires** that substructure elements that resist lateral seismic loads be ...”designed and detailed to be ductile...” This requirement addresses piers rather than most typical abutments.*

The intent of this clause is to ensure that good details are provided. For piers in SPZ 1, only Clause 4.4.10.2 is applicable, and therefore the R=2 requirement of Clause 4.4.8.1 applies only to minimum sizing.

This document requires that lateral load-resisting substructures in SPZ 3 and SPZ be designed using capacity design principles, and therefore will meet the intent of this clause.

*For lateral load resisting substructures (piers) in SPZ 1 and SPZ 2 **elastic** design forces may be used. Clause 4.4.8.1 in conjunction with Clause 4.4.10.3 requires that piers in SPZ 2 be sized as a minimum based on an $R=2$ using nominal section capacities. Detailing requirements must, in addition, satisfy Clause 4.7 (and as modified in this document).*

4.4.10 Design Forces and Support Lengths

4.4.10.4.2 Modified Seismic Design Forces (SPZ's 3 and 4)

The second paragraph shall be deleted and replaced with the following paragraph:

Capacity-protected elements, such as superstructures, cap-beams, beam column joints and foundations (including footings, pile caps, and piles but not including pile bents and retaining walls) shall be designed to have factored resistances equal to or greater than the maximum force effects that can be developed by the ductile substructure element(s) attaining their probable resistance.

Commentary:

For structures in SPZ 3 and 4, a seismic design approach based on 'capacity design' principles is preferred over designs based on elastic force levels, or over designs that do not consider demands arising from plastic mechanisms.

A design concept based on a clearly defined inelastic mechanism (whether ductile sub-structures, base isolation, or other mechanisms) for which capacity-protected elements are appropriately designed, and in which brittle failure modes are prevented is preferred.

4.5 ANALYSIS

4.5.1 General

Remove third paragraph and replace with:

In modeling reinforced concrete sections of ductile substructures the effects of cracking in appropriate members shall be taken into account in calculating periods, force effects, force distributions, and deflections.

In other substructures, uncracked section properties shall be used in calculating periods and force effects

Add additional paragraph to the end of 4.5.1:

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.

4.5.3.4 Time-History Method (Multispan Bridges)

Commentary:

For the design of most new highway bridges in B.C. time history methods is not required. Where time history methods are proposed, the objectives (design benefits) should be clearly outlined, and the number of 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 to be developed by the Designer.

Not less than three 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 analyses.

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.

4.6.4 Seismic Forces on Abutments and Retaining Walls

Add the following after the first sentence:

Active or passive pressures found from the Mononobe-Okabe method on abutment walls shall be considered as appropriate.

A seismic coefficient (generally designated k_h in the literature) of not less than 50% of the ground acceleration should be used, and k_v of zero may be used. Where the design PGA (1:475, or 10% probability of exceedence in 50 years) in excess of 0.25g is applicable, or where the wall system is rigid, then higher values of k_h shall be adopted, or calculations demonstrating the anticipated deformations of the wall shall be made. For wall or abutment designs where displacement-based methods are proposed the approach and consequences of analyses should be discussed and agreed with the Ministry and documented in a project-specific Design Criteria Report.

Commentary:

The seismic coefficient is generally designated k_h in geotechnical references. Note also that the nomenclature and equations contained in AASHTO differ from those typically found in the literature on the Mononobe-Okabe method, requiring care to ensure a consistent design approach results.

The most recent version of AASHTO allows 50% of the ground acceleration for checking global stability, and 100% for internal stability of MSE walls. Note that the implication of using 50% ground acceleration is normally that some permanent deformation occurs, reportedly (AASHTO) under 50 mm in the majority of walls. Tall walls (greater than eight meters) can be investigated in this regard as part of the design.

4.6.5 Soil-Structure Interaction

Replace sentence with:

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 warrant soil-structure interaction, geotechnical input shall be obtained.

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.

A discrete spring approach for soil-pile behavior represents one relatively straight-forward yet reasonable approach. In the absence of specific input from a geotechnical engineer (e.g. for minor bridges) representative spring values from either recent NAVFAC publications or other generally accepted technical references should be used. For bridge designs that warrant soil-structure interaction, geotechnical input should be obtained.

4.6.6 Fill Settlement and Approach Slabs

Replace first sentence with:

Approach slabs shall be provided in accordance with clause 1.8.2 of the B.C. MoT Supplement.

Commentary:

Project specific design criteria will typically specify settlement slabs (6 m long, measured normal to the abutment) as part of the structural design criteria. In general approach slabs have benefits for post-seismic performance and vehicle access. Settlement slabs shall be used for all bridges in SPZ 3 and 4 unless specified otherwise.

4.7 CONCRETE STRUCTURES

4.7.3 Seismic Performance Zone 3

Replace second sentence with:

The transverse reinforcement at potential plastic hinge zones of beams or columns shall be as specified in Clauses 4.7.4.1.4 and 4.7.4.1.5.

Commentary:

The reference to column tops is not applicable to all columns. A distinction between hinging beams and columns is not appropriate in this context. Normal design practice and Ministry requirements adopt hinging columns. Plastic hinges in beams may occur, for example when link beams are used in tall piers.

4.7.4.1.1 Longitudinal Reinforcement

Replace entire clause with:

The area of longitudinal reinforcement shall not be less than 0.008 (0.8%) nor more than 0.04 (4%) times the gross cross sectional area, A_g , of the column. The centre to centre spacing of longitudinal bars shall not exceed 200 mm.

4.7.4.1.2 Flexural Resistance

Delete the second paragraph of this clause.

4.7.4.1.6 Splices

Replaces the second paragraph with:

Lap splices in longitudinal reinforcement shall be permitted only in regions where plastic hinges are not expected to occur, and the splice length shall not be less than the greater of 60 bar diameters or 400 mm. These regions of potential plastic hinges shall comprise the centre half of column heights or for tall piers where rational analysis of a ductile substructure considering potential plastic mechanisms shows moments to be less than 70% of the maximum moments. 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 100mm,

Commentary:

Splices should be limited to the centre half of columns where standard bar lengths allow without adding un-necessary extra splice cost.

4.7.4.3 Column Connections

Delete the second paragraph of this Clause and replace with:

Lifeline and emergency route

For 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
- Seismic Design and Retrofit of Bridges, Priestley and Calvi (1996).
- 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 4.7.4.1.4, 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.

4.7.5.1 **Confinement Length**

Add sentence to the end of this clause:

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.

Commentary:

Hinging in piles may be expected or may be explicitly designed in some bridges, for example in extended pile bents or single-column caisson piers. In such instances, or others where plastic hinges may occur below grade, the concrete piles should be designed and detailed to ensure ductile behaviour.

4.8.3 **Sway Stability Effects**

Commentary:

This clause is included for steel structures, but not concrete structures. Guidance on incorporating P- Δ effects can be found in ATC – 32 Clause 3.21.15.

4.10 **SEISMIC BASE ISOLATION**

4.10.1 **General**

Add to end of clause:

For designs using base isolation, the Designer shall submit to the Ministry a Seismic Design Criteria document outlining 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, basic firm ground and soft soil time history records, and how displacements are accommodated at joints shall also be provided.

4.10.4 **Site Effects And Site Coefficient**

Replace asterisk sentence with:

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.

4.10.6 Analysis Procedures

Add sentence to the end of this clause:

Foundation flexibility and other relevant soil-structure interaction effects on structural response 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

Add sentence to the end of this clause:

Allowance shall be made for thermal deformation demands in combination with seismic isolation deformation demands on joint, bearing and railing details unless otherwise approved by the Ministry. 40% of the thermal deformation demands shall be combined with deformation demands from the base isolation system.

4.10.11.2 (d) Prototype Test

Change cross reference from Clause 4.10.11.3 to 4.10.10.3.

4.11 SEISMIC EVALUATION OF EXISTING BRIDGES

4.11.1 Bridge Classification

Second paragraph should be edited to begin with:

“The lifeline and emergency route bridges,…”

Third paragraph should be replaced with:

For other bridges the provisions of clause 4.11 shall apply.

Commentary:

The Ministry’s stand-alone document “Seismic Retrofit Design Criteria in (2004) document for the assessment portion of the retrofit criteria of existing bridges is more comprehensive than this section of S6-00, and shall be adopted for existing bridges.

4.12

SEISMIC REHABILITATION

4.12.1

Performance Criteria

Add to the end of this clause:

Seismic rehabilitation (retrofit) design shall be in accordance with the Ministry's document "Seismic Retrofit Design Criteria (2004)."

6.

Foundations

Introduction

The Ministry has concerns that the use of Limit States Design for foundation design and geotechnical work, as mandated in CSA S6-00, may result in inappropriate designs. Currently in the United States, AASHTO has provisions for bridge foundation design in both Load Factored (limit states) and Service Load (working stress). In Canada however, CSA-S6-00 provides only Limit States Design provisions, and the Ministry would like designers to continue to design bridge foundations using “working stress” practices.

Current bridge foundation design practice in BC varies. For the most part, structures are designed in Limit States design (using factored loads according to S6-00). However, geotechnical engineers often still provide “working stress” allowable loads for foundation design. Structural designers then usually compare these allowable with specified loads (unfactored) in the proportioning of the foundations.

There are no valid Canadian Bridge Code provisions which provide recommendations for unfactored load combinations. Also working stress codes usually recommend increasing allowable loads by 25%, 33% or 50% when designing for extreme loads. There does not appear to be consistent application of these increases in BC practice. Designers likely vary in the load combinations and factors they use for foundation design. In addition there is likely inconsistency in how bearing stresses and pile loads are distributed.

Live Load design trucks, and load factors within AASHTO differ from those in CSA S6-00, and it is potentially dangerous to apply AASHTO Service Load Design provisions in Canada.

The Ministry is therefore providing the following guidance to designers in order to obtain more consistency in design practice.

S6-00 Section 6 – Foundations

Foundation designs for bridges shall meet the following requirements.

- Working stress design shall be used in the proportioning of foundations in lieu of the Ultimate Limit States requirements of CSA S6-00 for geotechnical design. Loads shall be developed in accordance with CSA S6-00, and shall be combined according to the following table. Geotechnical working stress allowable stresses and loads may be increased for exceptional loads as indicated.

Loads	Permanent Loads		Transitory Loads		Exceptional Loads	% of geotechnical working capacities
	D	E	L	W	EQ	%
1	1	1	1			100
2	1	1		1		125
3	1	1	.85	.5		125
4	1	1			1.0	200

D *dead load*

E *loads due to earth pressure and hydrostatic pressure including surcharges other than dead load*

L *live load, including dynamic load allowance when applicable, based on CL-625 Truck or Lane*

EQ *earthquake load*

- The serviceability requirements of CSA S6-00 shall be met.
- The distribution of soils pressure shall be consistent with properties of the soil and the structure, and with established principles of soil mechanics.

7.

Buried Structures

INTRODUCTION

Previously, neither the MoT standards nor CAN/CSA-S6-88 Design of Highway Bridges included any specific reference to Buried Structures. Section 7 of CAN/CSA-S6-00 (CHBDC) now addresses Buried Structures.

CHBDC Section 7 – Buried Structures

7.1

Scope

Add the following paragraph:

Buried structures with span smaller than, or equal to, 3m may also be designed to CHBDC 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 S6-00 Commentary (C7.1 Scope, and C7.6 Soil-Metal Structures) indicates that the provisions of Section 7 apply only to buried structures with span (D_H) greater than 3m, but the S6-00 does not provide design guidance for smaller structures.

Add the following paragraph:

In addition to the design of new buried structures, the provisions of Section 7 shall also apply for evaluation or overload rating of existing buried structures. However, the provisions of Section 14 (Evaluation) shall be used to derive the appropriate live load factor for evaluation or overload rating of buried structures.

Add the following requirements:

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

- Type of Buried Structure;
- Highway Design Loading;
- Unit Weight of Backfill;
- Depth of Cover, H ;
- Depth of Cover, H_C , at intermediate stages of construction;
- Construction Live Loading assumed in the design (corresponding to H_C);
- Geometric Layout and Key Dimensions;
- Foundation and Bed Treatment;
- Foundation Allowable Bearing Capacity;
- Extent of Structural Backfill;
- Conduit End Treatment;

- Hydraulic Engineering Requirements, as appropriate;
- Roadway Clearance Envelope, as appropriate; and,
- Concrete Strength, as appropriate.

Add the following requirements:

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

- Design Life;
- Plate(s) 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_H and Span, D_V ;
- Radius at Crown, R_C ;
- Radius at Spring-line, R_S ; and,
- Radius at Base, R_b .

Add the following note:

Specifications for materials, fabrication and construction of buried structures shall be in accordance with MoT Standard Specifications for Highway Construction (2004), Section 303 Culverts and Section 320 Corrugated Steel Pipe. In the event of any inconsistency or conflict between the MoT Standard Specifications and S6-00, then the MoT Standard Specifications will take precedence and govern.

7.3

Notation

Add the following note:

Two separate notations for “ A_L ” appear in S6-00 Section 7, one on page 254, and one on page 255. Both descriptions for “ A_L ” are valid.

7.5.2

Load Factors

Add the following requirement:

When checking buried structures for buoyancy (Clause 3.11.3 also refers), 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.

7.5.5.3

Seismic Design of Concrete Structures

The contents of this Clause are deleted and replaced with the following sentence:

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:

It is inappropriate to rely solely on the requirements of Clause 7.8.4.4, especially for large span buried structures, since that clause addresses only vertical, not horizontal, earthquake loads.

7.6.1.1

Structural Metal Plate

Add the following limitation:

Aluminum and aluminized steel products shall not be used for buried structures designed to the CHBDC, unless otherwise approved by MoT based on site-specific test data.

Commentary:

Clause 7.6.3.3 and Clause 2.5.2 “Durability of Aluminum Structures - Protective Measures” precludes use of aluminum in contact with soil. Clause C2.5.1 states that chemical corrosion occurs when aluminum is in contact with concrete (e.g. headwalls). MoT’s 2004 Highway Construction Specification, Section 303 “Culverts”, refers to Section 320 “Corrugated Steel Pipe” for CSP culverts, which stipulates steel as base material. Section 320 allows a choice of zinc galvanizing or asphalt coating (AC) on steel CSP / SPCSP buried structures, but not aluminized coating.

7.6.2.1.1 General

Replace the contents of this Clause with:

The thrust, T_f , in the conduit wall due to factored live loads and dead loads shall be calculated for ULS load combination 1 of Table 3.5.1 (a), according to the following equation:

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

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 span less than or equal to 10m, T_D and T_L shall be calculated in accordance with Clauses 7.6.2.1.2 and 7.6.2.1.3, respectively;
- b) For soil-metal structures with span more than 10m, 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 7.6.2.1.2 and 7.6.2.1.3, respectively, should be used as an additional check to ratify the results of the finite difference, or finite element, method;
- c) For deeply buried soil-metal structures, the S6-00 expressions for T_D and T_L may be too conservative. The S6-00 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 S6-00 methodology or, if Approved, they may use an alternate finite difference or finite element soil-structure interaction analysis method to determine the dead and live load thrusts.

Commentary:

S6-00 does not place any limitations on the applicability of Section 7 for soil-metal structures with large spans, or for those deeply buried. Research indicates that S6-00 design formulae may not be appropriate for all large span, or deeply buried, soil-metal structures.

7.6.2.1.2 Dead Loads

Add the following note:

"H" is measured vertically from crown of structure to finished grade, as shown on Commentary Figure C7.6.2.1.2.

Commentary:

The depth of cover or height of overfill, "H", is missing on Figure 7.6.2.1.2.

7.6.2.1.3 **Live Loads**

Replace the second expression for “ T_L ” with the following:

“ $T_L = 0.5 L_T \sigma_L m_f$ ” (not “ $T_L = 0.5 D_T \sigma_L m_f$ ”).

Commentary:

L_T is defined as “A length of dispersed live load at crown level measured transversely”, and $L_T = (1.45 + 2H)$. The expression for “ L_T ” is specific to normal highway vehicle/axle loading, and may not be applicable for special vehicle/axle layouts.

7.6.2.4 **Connection Strength**

Add the following note:

Designers are advised that values of unfactored seam strength, S_s , for standard corrugation profile with bolted connections are shown in Commentary Figure C7.6.2.4.

7.6.3.1 **Minimum Depth of Cover**

Add the following note:

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

7.6.3.3 **Durability**

Add the following requirement:

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

Commentary:

S6-00 Section 7 Commentary suggests that an expected design life of up to 100 years is achievable, and presents sample values for corrosion loss.

Add the following requirements:

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/m^2 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:

MoT's 2004 Highway Construction Specification, Section 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).

Add the following requirements:

For non-saturated soil conditions, the "AASHTO corrosion loss model", as presented in S6-00 Commentary Table C7.6.3.3(a), 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 S6-00 Section 7 Commentary presents two sets of values for Non-Saturated Loss Rates (i.e. UBC 1995 & AASHTO 1993) in Table C7.6.3.3(a), and a single set of values for Saturated Loss Rates (i.e. UBC 1995) in Table C7.6.3.3(b). Practical experience suggests that some of these corrosion loss results are too conservative in typical applications.

7.6.4

Construction Requirements

Add the following requirement:

MoT Standard Specifications for Highway Construction (2004), Section 320 Corrugated Steel Pipe, stipulates identification markings on galvanized steel sheet used in buried structures. In addition to those requirements, all sheets shall be marked with radius of plate curvature. All identification features shall be stamped in a prominent location on all plates, to facilitate field assembly and future inspection.

7.6.4.5 **Structural Backfill**

Add the following requirement:

Refer to MoT Standard Specifications for Highway Construction (2004), Section 303 Culverts, for backfill materials and compaction requirements.

7.6.5 **Special Features**

Add the following requirement:

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. The CHBDC 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**

Add the following two paragraphs:

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 Approved by MoT, soil-structure interaction shall not be considered for metal box structures larger than 8.0m span, or 3.2m rise.

Commentary:

The 8.0m span limit, and the 3.2m rise limit, for metal box structures are based on limitations in the original research. The S6-00 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.

7.7.2.2 Design Criteria for Connections

Add the following paragraph:

Designers are advised that values of unfactored seam strength, S_s , for standard corrugation profile with bolted connections are shown in Commentary Figure C7.6.2.4.

Commentary:

Values of unfactored seam flexural strength are not presented in the S6-00 or in the AASHTO Standard Specifications for Highway Bridges (2002) Clauses 12.4.2 and 12.6.2.

7.7.3.1 Depth of Cover (and Figure 7.7.4.1.1)

Add the following paragraph:

The Designer shall determine a practical minimum cover depth (>0.3m) for each specific application.

Commentary:

Although the prescribed minimum cover depth of 0.3m may be achievable in certain situations, it is likely that the roadway structure thickness and construction practices will require a larger depth of cover for most metal box applications.

7.7.3.2 Durability

Add the following requirements:

The design life and durability requirements for Metal Box Structures shall be the same as stipulated for Soil-Metal Structures in clause 7.6.3.3 above.

7.7.4 Construction

Add the following requirement:

MoT Standard Specifications for Highway Construction (2004), Section 320 Corrugated Steel Pipe, stipulates identification markings on galvanized steel sheet used in buried structures. In addition to those requirements, all sheets shall be marked with radius of plate curvature. All identification features shall be stamped in a prominent location on all plates, to facilitate field assembly and future inspection.

7.7.4.1.2 **Material for Structural Backfill**

Add the following note:

Refer to MoT Standard Specifications for Highway Construction (2004), Section 303 Culverts, for backfill materials and compaction requirements.

7.8

REINFORCED CONCRETE BURIED STRUCTURES

Add the following paragraph:

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.

Commentary:

The analysis and design provisions of Section 7.8 appear to focus on medium size 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 7.8.5.3.1-7.8.5.3.3 may not be appropriate for large structures.

7.8.3.2 **Minimum Depth of Cover for Structures with Curved Tops**

Add the following paragraph:

The Designer should determine a practical minimum cover depth (>0.3m) for each specific application.

Commentary:

Although the prescribed minimum cover depth of 0.3m (below unpaved or flexible pavements) may be achievable in certain situations, it is likely that the roadway structure thickness and construction practices will require a larger depth of cover for most concrete box/pipe applications.

7.8.4.4 **Earthquake Loads**

Add the following two paragraphs to Clause 7.8.4.4:

For concrete buried structures with span (D_H) less than or equal to 3m, 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 be investigated.

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

Section 7 – Buried Structures – Commentary

7.6.1.3

Soil Materials

Replace the expression for secant soil stiffness or modulus, E_s (on page 280 of the Commentary), with the following;

$$"E_s = E_i [1 - (\sigma_d / (\sigma_d)_f) * R_F]"$$

Commentary:

This expression is consistent with the result shown on page 281. of the commentary.

S6-00 Evaluation of Existing Buried Structures for Overload Rating:

Existing buried structures shall be evaluated as follows:

- a) Use Section 14, Evaluation, to determine the appropriate live load factor to account for traffic type, system/element behaviour, inspection level, etc; and then,
- b) Use Section 7, Buried Structures, to analyze the soil-structure behaviour and check the capacity of each buried structure.

Commentary:

S6-00 includes a specific process (Section 14, Evaluation) for evaluation, and overload rating, of existing bridges. However, Section 14 specifically excludes buried structures from its scope. The relevant section of S6-00 for buried structures (Section 7) gives no guidance for evaluation of existing buried structures loaded by specific overload vehicles.

A two-stage evaluation process shall be used for overload rating of buried structures:

- 1) Stage 1 is an initial “screening” process to eliminate any buried structures that satisfy either of the following configuration conditions:
 - a) Buried structures with depth of cover greater than 0.7m, and with a “depth of cover to span ratio” greater than $\frac{2}{3}$; or,
 - b) Buried structures with a span of less than 1.0m.

Commentary:

Any structure found to satisfy either of these two conditions is screened out, as MoT deems such structures adequate for all overload conditions.

- 2) Stage 2 is required for any buried structures that do not satisfy the Stage 1 “screening” process. Such structures shall be inspected in the field and receive a more detailed evaluation as follows;
 - a) Buried structures with span less than or equal to 10m shall be evaluated in accordance with the analysis/design provisions of Section 7, while using Section 14 to determine the appropriate live load factor. However, only the ultimate limit states from Table 7.5.1 that apply to the as-built condition of the structure need to be investigated (i.e. limit states that may have occurred during original construction do not need to be investigated);
 - b) Buried structures with span more than 10m shall be evaluated using a finite difference, or finite element, soil-structure interaction analysis method. The analysis/design provisions of Section 7, and Section 14 for live load factor, should be used as an additional check on the results of the finite difference, or finite element, method;

Commentary:

S6-00 does not place an upper limit on the applicability of the Section 7 methodology for large-span buried structures.

- c) In Section 7, Live Loads Clause 7.6.2.1.3, the expression $L_T = (1.45 + 2H)$ (where L_T is the length of dispersed live load at crown level measured transversely), is not necessarily applicable for overload vehicles which have axle spacings different from the standard CHBDC truck. Engineering judgment shall be used to determine an appropriate alternate expression for the overload vehicle in question.
- d) In Section 7, Live Loads Clause 7.7.2.1.3, the expression $L_L = A_L / k_4$ (where L_L is a line load equivalent to the live load acting on a metal structure), is not necessarily applicable for overload vehicles which have axle spacings different from the standard CHBDC truck. Engineering judgment shall be used to determine an appropriate alternate expression for the overload vehicle in question.
- e) Seismic requirements do not need to be evaluated for overload vehicle rating, regardless of structure span; and,
- f) Dynamic load allowance shall be as specified in Clause 3.8.4.5. The restricted speed reduction factors expressed in Clause 14.8.3 (a, b, c, d) shall not be applied for overload evaluation of buried structures.

Commentary:

In general, for buried structures there is no particular advantage to be gained by requiring an overload vehicle to cross at a slow speed or at a specific location. In contrast to bridge structures, a slower transit over a buried structure may in fact allow larger soil strains to develop which could be detrimental. At a buried structure with variation in cover depth along its length, advantage may be gained by requiring the overload vehicle to cross at a specific location.

8.

Concrete Structures

8.3 NOTATION

Add sentences to definition of D as follows:
For round columns, the outer diameter of the column..

Add sentences to definition of V_p as follows:
For non-prestressed columns in ductile sub-structures, the V_p shall be taken as the contribution to column shear resistance arising from the horizontal component of the column axial load, assumed to act as a strut between compression regions of plastic hinge zones or points of application of axial load (refer to Clause 8.9.3.5)

8.4 MATERIALS

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

Where Grade 400R reinforcing bars are specified for structural components which resist seismic forces, an upper limit for yield strength of 525 MPa shall be specified for flexural reinforcement in plastic hinge regions. The specified upper limit for yield strength shall be verified from test samples prior to placement of reinforcement.

Commentary:
Specification/verification of an upper limit for yield strength is intended to ensure Grade 400R reinforcing bars used in plastic hinge regions possess expected ductility characteristics.

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

8.8.4.5 Maximum Reinforcement

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

8.9.2.5 Effective Shear Depth

Add sentence to end of clause:

For the design of reinforced concrete columns or piers against seismic demands in ductile sub-structures, V_p shall be taken as:

$$V_p = k_1 P (D-a) / H$$

Where

k_1 = 1 for columns in double curvature, and 0.5 for columns in single curvature.

P = axial load including dead load and seismic effects (the latter determined from a plastic mechanism analysis using nominal material properties).

a = effective depth of compression block

8.9.3.4.1 General Method

Add sentence to end of clause:

(c) β 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.

8.9.3.5 Nominal Shear Stress

Add sentence to end of clause:

For the design of round reinforced concrete columns or piers against seismic demands in ductile sub-structures the effective shear depth shall be that producing an effective shear area equal to 0.80 of the gross concrete area, A_g .

8.9.3.8 Determination of V_s

Add sentence to end of clause:

(c)For the design of round columns in ductile sub-structures, V_s shall be taken as:

$$V_s = 0.5 \pi A_v f_{yh} D' \cot \Theta / s$$

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 longitudinal reinforcing steel bar cage.

Θ = 30° for the purposes of this clause only, or 45° if axial tensile axial loads occur under seismic loads.

s = spacing of spiral reinforcing bars

8.11 DURABILITY

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

Commentary:

Heating of bridge decks has been determined to be inoperable based on research and practice. Its use has therefore been discontinued.

8.11.2.1 Concrete Quality

For the following structural elements, concrete mix criteria shall be specified in the Special Provisions and shall comply with the following requirements:

ELEMENT	MIN. 28- DAY COMPRESSIVE STRENGTH (MPa)	MAX. NOMINAL AGGREGATE SIZE (mm)
Deck Slab, Parapets, Approach Slabs	35	28
Deck Overlays	35	12
Piers & Abutments	30	28
Footings	30	28
Working Floors	20	28
Fill for Pipe Piles	30	28
Keyways between Box Stringers	35	12

Commentary:

The above criteria do not apply to high-performance concrete applications such as deck and overlay mix designs utilizing silica fume and super-plasticizers. The criteria for these specialty mix designs shall be established by the mix design engineer.

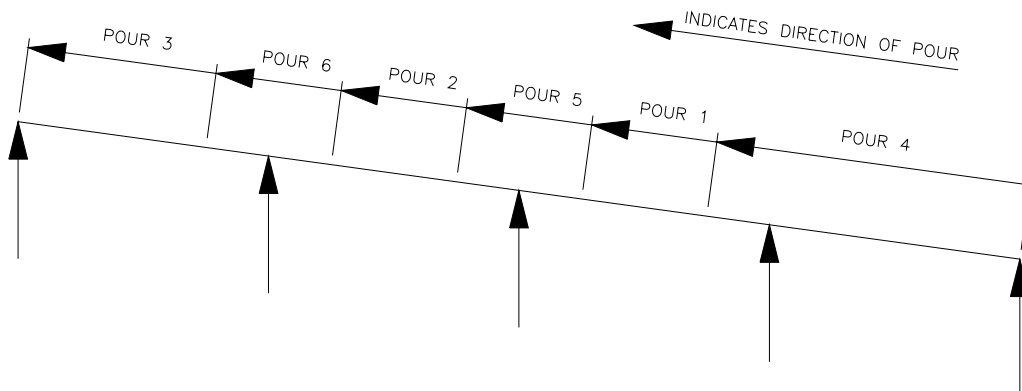
8.11.2.1.2 Concrete Placement

The deck casting sequence and the detail for construction joints shall be shown on the drawings. Typically, deck slabs shall be cast in the direction of increasing grade (uphill). Bridges with minimum grades may be cast in either direction. Consideration shall be made in specifying the casting sequence to minimize the number of construction joints (cold joints) in the deck.

For continuous structures, concrete shall typically be cast as follows:

<p><u>Concrete in positive-moment zones:</u></p>	<p>All concrete in these zones shall typically be cast prior to concrete in negative-moment zones.</p>
<p><u>Concrete in negative-moment zones:</u></p>	<p>Concrete in these zones shall typically not be cast until adjacent concrete in positive- moment zones has cured, unless cast monolithically with the positive-moment concrete as shown below.</p>

EXAMPLE SCHEMATIC OF DECK POUR SEQUENCE (to be shown on the drawings)



Concrete placing sequence for integral abutments shall be given special consideration to reduce stresses induced by deflection of the girders.

Commentary:

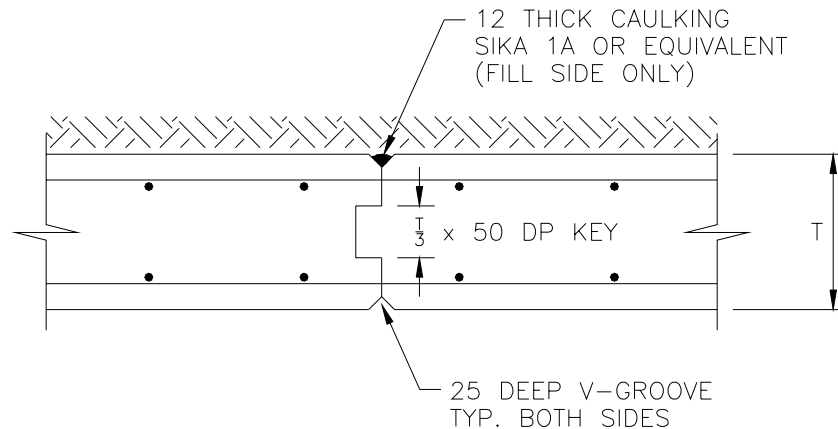
A deck casting sequence is required in order to minimize the potential for deck cracking due to improper concrete placement sequencing.

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.

8.11.2.1.4 Cold Joints

Typical details for cold joints are shown in **Figure 8.11.2.1.4** below.

Figure 8.11.2.1.4 - Typical Cold Joint



NOTES:

- MAXIMUM SPACING OF COLD JOINTS = 10.0 m
- FOR WALLS LESS THAN 10 m IN LENGTH, CONCRETE MAY BE PLACED IN ONE CONTINUOUS OPERATION

8.11.2.1.5 Slip-Form Construction

Extruded concrete barriers shall not be used.

Commentary:

It has been observed that extruded concrete barriers do not result in a watertight joint at the interface with the deck which allows seepage of water to cause staining of the deck and superstructure.

8.11.2.1.6 Finishing

Surface finishes shall be specified in the Special Provisions.

Exposed concrete surfaces of large abutments or retaining walls that are clearly visible to the public may receive an architectural finish. In the selection of this surface finish, consideration shall be made regarding methods required for the removal of graffiti. Such consideration may include the application of anti-graffiti paint.

8.11.2.1.6 Finishing

Surface	Finish	Standard Specification Clause
Surfaces submerged or buried.....	Class 1	211.16
Top and inside (exposed) face of parapets...	Class 3	211.16
Outer face of parapets.....	Class 2	211.16
Abutments and retaining walls.....	Class 2	211.16
Piers.....	Class 2	211.16
Bearing seats.....	Steel Trowel	211.13
Top of deck	Tined **	413.31.04
Approach slabs.....	Float Finish	211.13
Sidewalks.....	Transverse Coarse Broom	211.13
Underside of Deck.....	Class 1	211.16
Slope Pavement.....	Transverse Coarse Broom *	211.13

* Exposed Aggregate finishes may be considered.

** Decks to receive waterproofing membranes shall be finished in accordance with Standard Specification 419.33.

Consideration shall be given to surfaces exposed to 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.

8.11.2.2 Concrete Cover and Tolerances

Concrete Cover

All references to “minimum cover” in CAN/CSA-S6-00 shall be replaced with “specified cover”. Table 8.11.2.2 in CAN/CSA-S6-00 shall be amended as follows:

8.11.2.2	Specified Concrete Covers and Tolerances		Concrete Covers and Tolerances	
			Environmental Exposure	Component
All	(3) Top surfaces of Structural components Add: Bridge Decks and Approach Slabs	Reinforcing Steel	70 +6 -0 *	as per Table 8.11.2.2 of CAN/CSA-S6-00

* For concrete decks without waterproofing and paving, increase concrete covers by 10 mm to allow for wearing of the surface concrete.

Commentary:

The term “minimum cover” should not be used 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

Reinforcing Steel

As a minimum, the top mat of deck reinforcing steel, all reinforcing steel in cast-in-place parapets and reinforcing steel in approach slabs shall be protected against corrosion.

Epoxy coating or galvanizing may be used for corrosion protection of reinforcing steel. If galvanizing is used, all reinforcing steel in the component shall be galvanized. Galvanized bars and uncoated bars shall not be used in combination in any one structural component.

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.

Ends of Prestressing Strands

Ends of prestressing strands shall be painted with a Ministry-approved galvanizing agent where the ends of stringers are incorporated into concrete diaphragms or are otherwise embedded in concrete.

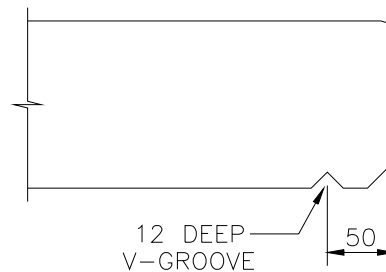
Ends of prestressing strands shall be given a minimum 3 mm coat of thixotropic epoxy in 100 mm wide strips applied in accordance with the manufacturer's requirements where ends of stringers are not embedded in concrete.

Commentary:

Galvanized reinforcing steel and uncoated steel should not be used in combination because of 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).

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:



Commentary:

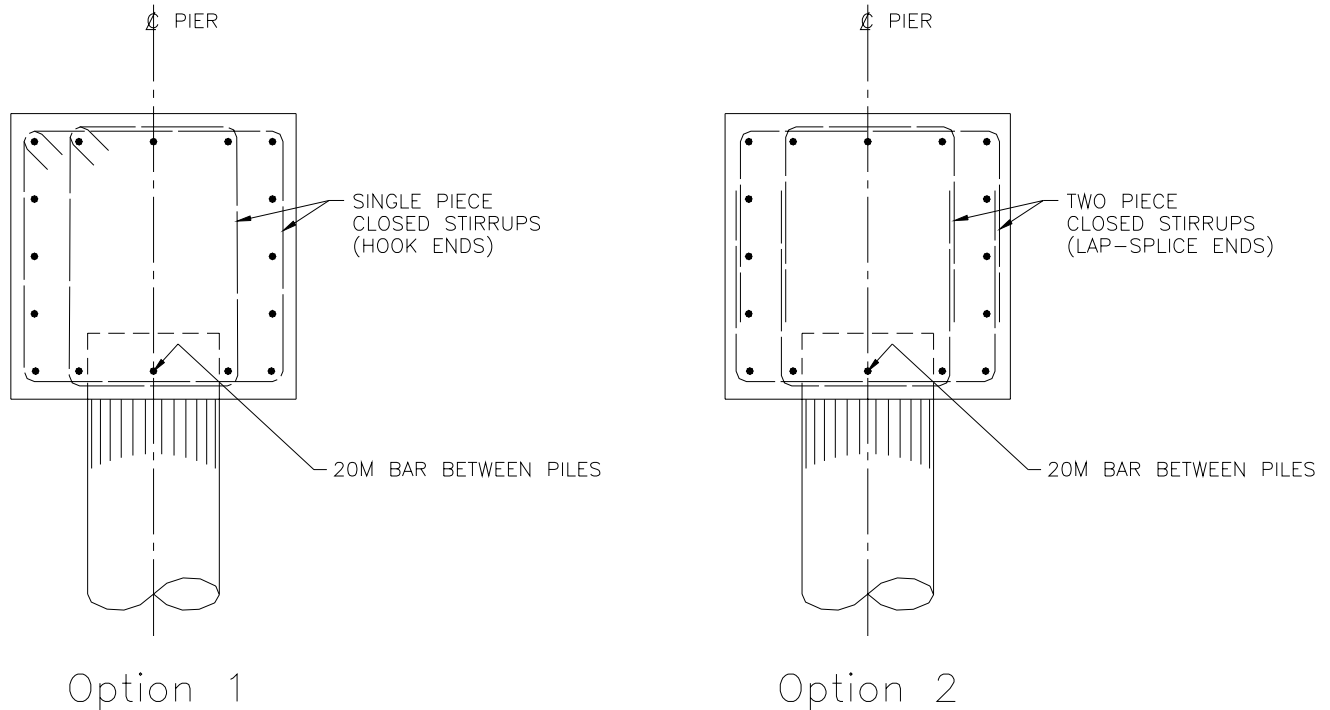
The drip groove detail shown above has been used throughout the Province since it was first introduced in September 1989 and has functioned well with no adverse feedback from field staff. For this reason the detail has been retained, although it varies from the drip groove detail shown in clause 8.11.2.6 of CAN/CSA-S6-00.

8.14.3 Transverse Reinforcement for Flexural Components

Pier Caps

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

Figure 8.14.3 – Typical Transverse Reinforcement of Pier Caps



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.

Diaphragms and Other Varying Depth Members

Closed stirrups formed from two piece lap-spliced U-stirrups are preferred for diaphragms and other varying depth members.

Commentary

Problems are encountered with stirrup sizes in diaphragms when stirrups are either too long or too short depending on the final depth of the haunches. The method of using two piece U-stirrups of suitable depth alleviates problems in accommodating variations in depth of diaphragms.

8.15.9 Splicing of Reinforcement

All splices that are critical to the structure shall be indicated by the Design Engineer.

Splicing of transverse reinforcing bars in bridge decks shall be avoided. If such splices are unavoidable, their location shall be indicated by the Design Engineer.

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 for box stringers and further described as follows:

a) Fixed Joint

Dowel holes shall be filled with non-metallic, non-shrink grout with minimum compressive strength of 25 MPa at 7 days.

b) Expansion Joint

Dowel holes and dowels shall be sized to ensure there is adequate space for vertical movement.

Dowel holes shall be protected from moisture and debris prior to placing Styrofoam and grout.

8.18.2 Minimum Slab Thickness

A minimum deck slab thickness of 225 mm shall replace the minimum slab thickness of 175 mm required in Clause 8.18.2 of CAN/CSA-S6-00.

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	15 mm
Top reinforcing – longitudinal	15 mm
Minimum clear cover between layers	25 mm
Bottom reinforcing – longitudinal	15 mm
Bottom reinforcing – transverse	15 mm
Concrete cover – soffit of deck	50 + 10 mm (tolerance)
Total - Minimum slab thickness	221 mm (round up to 225 mm)

8.18.5 Diaphragms

Additional Deck Reinforcing at End Diaphragms and Intermediate Diaphragms

Consideration shall be given to deck reinforcing requirements for negative moment over monolithic cast-in-place concrete end and intermediate diaphragms.

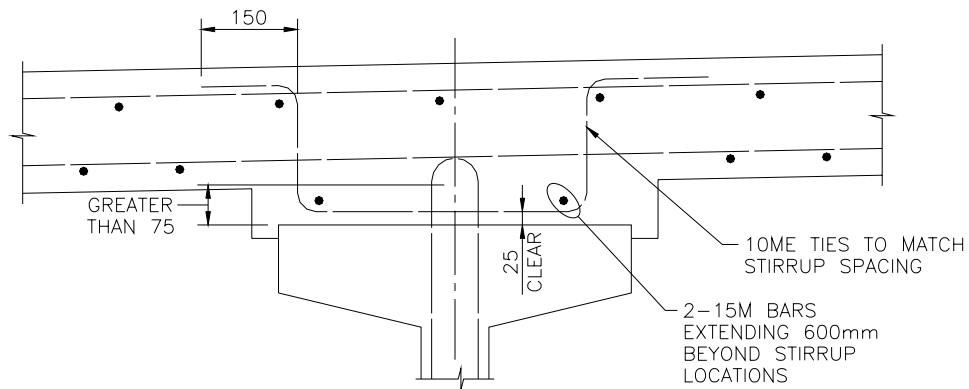
8.19 COMPOSITE CONSTRUCTION

8.19.1 General

Prestressed concrete box girders shall be designed as non-composite. 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 stirrups in prestressed I-beams shall extend 125 mm above the top of the beam. When the haunch height exceeds 75 mm, additional 10ME ties matching the spacing of the shear stirrups and additional 15M corner reinforcing at the haunch level shall be provided as shown in the following sketch.



Additional 10ME ties with 15M corner reinforcing at the haunch level shall also be provided in haunches above steel girders where haunch heights exceeds 75 mm.

8.20

CONCRETE GIRDERS

8.20.1

General

Skew Limitations - Prestressed Concrete I-Girders and Box Girders
Skews over 30° shall be avoided. 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°.

8.20.3.2

Bottom Flange

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

Commentary

The bottom flange thickness of MOT 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 requirements of CAN/CSA-S6-00 therefore do not seem to be warranted.

8.20.4

Web Thickness

Prestressed concrete I-beam sections summarized on Drawing D202-A shall be used as Ministry standards for pretensioned concrete I-beams. For post-tensioned applications, the minimum web thickness shall conform to the requirements of Clause 8.20.4 of CAN/CSA-S6-00.

Commentary:

Although the Ministry standard prestressed I-beam sections with 127 mm thick web do not meet the code minimum requirement of 160 mm, they have functioned well since they were introduced in 1987. The 127 mm thick web has not resulted in any problems to fabricators in placing of prestressed and non-prestressed reinforcement. Increase in web thickness in order to conform to code requirements will result in transportation limitations for Type 5 and higher girder types and increased cost of production.

For post-tensioned construction, the web thickness shall be increased to satisfy code requirements for cover and tolerances.

8.20.7 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.

8.20.8 Diaphragms

Orientation of Diaphragms

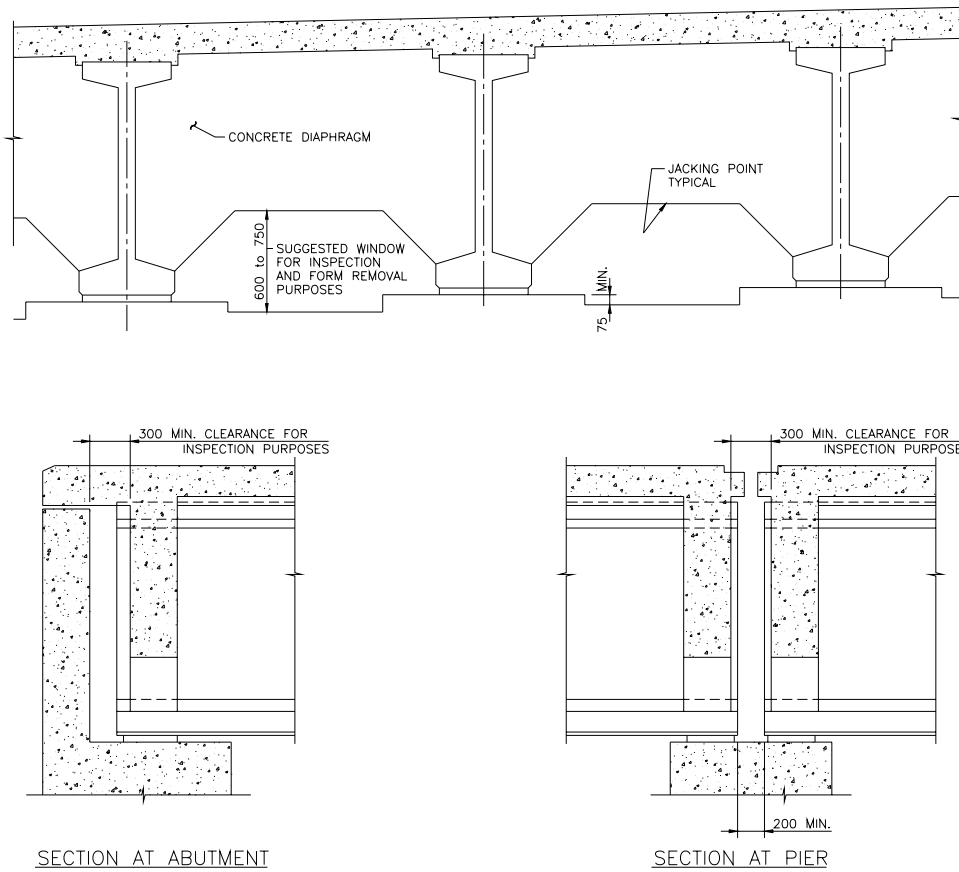
Abutment and pier diaphragms shall be oriented parallel to the bridge skew.

Intermediate diaphragms shall be oriented at right angles to the deck.

Maintenance and Inspection Provisions

Abutment and pier diaphragms shall be designed to facilitate future jacking, and to provide access for maintenance inspection, as generally outlined in **Figure 8.20.8**.

Figure 8.20.8 – Typical Concrete Diaphragm



Hole Sizes through Ends of Prestressed Girders

The hole size for abutment and pier diaphragm reinforcing which passes through the ends of prestressed girders shall be 50 mm diameter for 20M bars.

8.21

MULTIBEAM DECKS

MOT standard twin cell concrete box stringers shown on Drawings 2978-1B to 2978-24B which include shear keys filled with 35 MPa concrete and reinforced with 15ME longitudinal bars may be used at the discretion of the design engineer without lateral post-tensioning in accordance with Clause 8.21(c) of CAN/CSA-S6-00.

Commentary:

MOT standard box stringers less than 20 m in length without lateral post-tensioning have performed extremely 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 was less than 20 m spans.

8.11.2.1.6 Finishing

SURFACE	FINISH	STANDARD SPECIFICATION CLAUSE
Surfaces submerged or buried	Class 1	211.16
Top and inside (exposed) face of parapets	Class 3	211.16
Outer face of parapets.	Class 2	211.16
Abutments and retaining walls	Class 2	211.16
Piers	Class 2	211.16
Bearing seats	Steel Trowel	211.13
Top of deck	Tined **	413.31.04
Approach slabs	Float Finish	211.13
Sidewalks	Transverse Coarse Broom	211.13
Underside of Deck	Class 1	211.16
Slope Pavement	Transverse Coarse Broom *	211.13

* Exposed Aggregate finishes may be considered.

** Decks to receive waterproofing membranes shall be finished in accordance with Standard Specification 419.33.

Consideration shall be given to surfaces exposed to 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.

10.

Steel Structures

10.4

MATERIALS

10.4.1

General

The third paragraph shall be removed in its entirety and shall be replaced with the following:

- Coil steel shall not be used unless specifically approved by the Ministry.

Commentary

Coil steel undergoes stressing during the rolling and unrolling process that may result in undesirable properties, given the application. It may also be difficult to straighten. As a general rule, coil steel shall not be allowed. In special circumstances, for non-critical members, the Ministry may approve its use.

10.4.2

Structural Steel

The following paragraph shall be added:

Supplementary information regarding material sizes and availability shall be:

1. Available plates (thickness x width x length) and costs should be obtained prior to starting design.
2. Maximum thickness of plate should not exceed 102 mm (4") but preferably should not be more than 76 mm (3").
3. Plate widths of 2440 mm (8'-0") and, in some instances, 1830 mm (6'-0") should be used whenever possible.
4. Availability, cost, and delivery lead time should be considered prior to choosing welded wide flange (WWF) and welded reduced flange (WRF) sections for design.
5. Availability and costs of rolled shapes should be obtained prior to starting design.
6. A trim allowance of 10 mm around the perimeter of the available plate sizes, plus an allowance for sweep and for camber, should be made when sizing the member plates.
7. When notch tough steels (WT and AT) are used, category 3 shall be specified for all locations within British Columbia.

Commentary

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*

Most structural steel comes from the US and therefore common dimensions are Imperial. If a large order is placed, mills will produce plates in Metric sizes.

2. *Standard Metric plate thicknesses are: 6mm, 9mm, 13mm, 16mm, 19mm, 22mm, 25mm, 32 mm, 38 mm, 44 mm, 51 mm, 57mm, 64mm, 70mm, and 76mm. (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'-0") and 1830 mm (6'-0"). Wider plates may be obtained as a special mill order but long supply times can be expected. Girders more than 8' deep will generally require a longitudinal web splice and, therefore, designers should take into account the added cost associated with the splice when determining the optimum girder depth.*
4. *Provided sufficient quantities are specified (≥100 tonnes) plates and welded wide flanged shapes (WWF) are available in both Imperial and Metric sizes.*
5. *Rolled shapes are no longer available from Canadian mills. Rolled shapes from US mills are currently available only in Imperial sizes. Common Metric angle sizes and their Imperial equivalents currently available are: L90x90x8 (L3-1/2"x3-1/2"x 5/16"), L100x100x6 (L4"x4"x1/4"), L100x100x10 (L4"x4"x3/8"), and L125x125x8 (L5"x5"x5/16"). Metric sizes included in steel handbooks are soft conversions of the Imperial equivalents. In the future, steel from countries such as Japan, Korea, and China may become more competitively priced and may be considered for projects in British Columbia*

10.4.2 Structural Steel

The following information shall be added:

Where appropriate, all superstructure members shall be corrosion-resistant steel.

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.

Bracing members shall be as noted below. Bracing members of 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 metalizing. If galvanizing or metalizing are inappropriate (e.g. for aesthetic reasons), bracing shall be coated with a paint system from the Ministry Recognized Products List.

Grade 260W shall not be used in bridges.

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:

1. Angles and channels, non-weathering: 350W (equivalent to ASTM A572, Grade 50); weathering: ASTM A588, Grade 50.
2. Hollow structural sections: 350W or ASTM A500, Type B
3. HP Sections: 350W (equivalent to ASTM A572, Grade 50)
4. Plate: 300W, 350W, 350WT, 350A, 350AT
5. Structural tees: 350W (equivalent to ASTM A572, Grade 50)
6. Welded reduced wide flange shapes: 350AT, 350W
7. Welded wide flange shapes: 350AT, 350W
8. Wide flange shapes, non-weathering: 350W (equivalent to ASTM A572, Grade 50), weathering ASTM A588, Grade 50.
9. Anchor bolts: ASTM A307, Grade C ($F_y = 250$ MPa, 36 ksi).
10. Shear studs: (refer to S6-00 clause 10.4.7)

Commentary:

Due to the cost of painting, it is recommended that corrosion-resistant steel be used where appropriate.

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.

1. *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).*
2. *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.*
5. *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)*
- 6.,7. *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.*
8. *Local fabricator experience is that sections are usually purchased as conforming to ASTM A572, Grade 50 (non-weathering) or ASTM A588 (weathering steel).*
9. *Higher strength anchor bolts equivalent to ASTM A325 or ASTM A193, Type B7 ($F_y = 720$ MPa, 105 ksi) may be used with prior Ministry approval in special circumstances (e.g. seismic loading).*
10. *It is recommended that designers not specify one particular grade as manufacturers will not guarantee studs to meet one grade.*

10.4.5

Bolts

The following sentences shall be added:

1. Bolts shall be 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. See the Ministry SS 421.11.03 for coating requirements for bolts.
4. ASTM Standard A490 bolts, nuts, and washers shall not be used unless specifically permitted by the Ministry.
5. All bolts shall be installed with a hardened washer on the side that is being turned, generally under the nut.
6. Bolt tension shall conform with the requirements of CAN/CSA-S6-00.
7. Fully torqued bolts shall be installed in all bolt holes used for erection.

Commentary:

1. *Bolts are not available in Metric sizes without ordering an entire lot. Therefore, the designer should specify Imperial bolt sizes.*
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 will be considered by the Ministry.*

10.4.10

Galvanizing and Metallizing

The following sentence shall be added:

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:

- Mn maximum of 1.35%
- Si maximum of 0.03% or within range of 0.15% to 0.25%.

Commentary:

When the silicon content of the steel is between 0.04% and 0.15% there can be a negative effect on the zinc thickness when galvanizing.

10.6

DURABILITY

10.6.3.1

Structural Steel

The first paragraph shall be deleted and replaced with the following:

For weathering steel structures, all structural steel, including contact surfaces of bolted joints, diaphragms and bracing but excluding surfaces in contact with concrete, shall be coated with an Approved coating system for the larger of the following two distances from 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.

Consideration by the Design Consultant may be given to metallizing the zone as described above. If the metallized zones will be visible from the outside of the bridge, they should also be topcoated with paint to match to colour of the adjacent steel elements.

The following paragraph shall also be added:

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 Representative.

For structures not using weathering steel, the steel shall be coated with a coating system from the Ministry Recognized Products list according to the provisions SS 421.

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

The designer should make all attempts to avoid situations where water can pool on girder flanges. Where they cannot be avoided, such areas should 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 the good design practice would require both.

10.6.3.2 Cables, Ropes, and Strands

The first paragraph is deleted and replaced with the following:

A method of corrosion protection as approved 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.

10.6.4 Other Components

The second paragraph is deleted.

Commentary:

Table 10.6.4

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

- Piling shall be sized for a corrosion allowance of at least 3 mm (1/8") over the life of the structure.
- Coated piling shall not be allowed.

Commentary:

Coating 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.

10.7 DESIGN DETAIL REQUIREMENTS

10.7.1 General

This clause is amended by the addition of the following sentence:

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.

The document may be referenced at:

www.steelbridge.org/AASHTO%20Docs/GDC-1%20AASHTO.pdf

NSBA is the National Steel Bridge Alliance, an US-based organization.

10.7.1.a Flange Widths between Splices

The following information shall be added:

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.b Transition of Flange Thicknesses at Butt Welds

The following information shall be added:

Transition of flange thickness at butt welds should 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. 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.c Recommended Details

The following items shall be added as preferred details:

Coping of Stiffeners and Gusset Plates

As shown in **Figure 10.1**, for I-girders with vertical webs, copes on details such as stiffeners shall be 50 mm horizontally x the greater of 50 mm and 6 times the girder web thickness vertically. Copes on details such as gusset plates shall be the greater of:

- 50 mm x 50 mm; and
- times the girder web thickness x 6 times the girder web thickness.

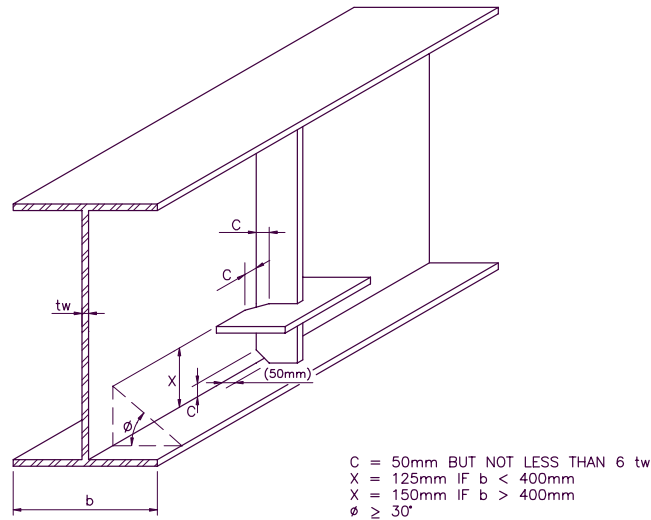


Figure 10.1 - Coping of Stiffeners and Location of Gusset Plates

Commentary

These copes as dimensioned above are desirable for the following reasons:

- they prevent the possibility of intersecting welds;
- they reduce the high weld shrinkage strains associated with smaller copes;
and
- they allow drainage.

At end diaphragms, copes are not permitted. 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. This reference has a comprehensive set of suggested steel design details.

Gusset Plates for Lateral Bracing.

All gusset plates for lateral bracing should be fillet welded. As shown in **Figure 10.1**, 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. The latest edition of “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.

Frames for Lateral Bracing, Cross-Frames and Diaphragms

Frames 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 designed for shipping and erection as a single unit. A sample arrangement is shown in **Figure 10.2**.

All frames should be designed for fabrication from one side, eliminating the need for “turning over” during fabrication. A “K” brace angle system is preferred, using rectangular gusset plates. Oversized holes in the gusset plates are permitted.

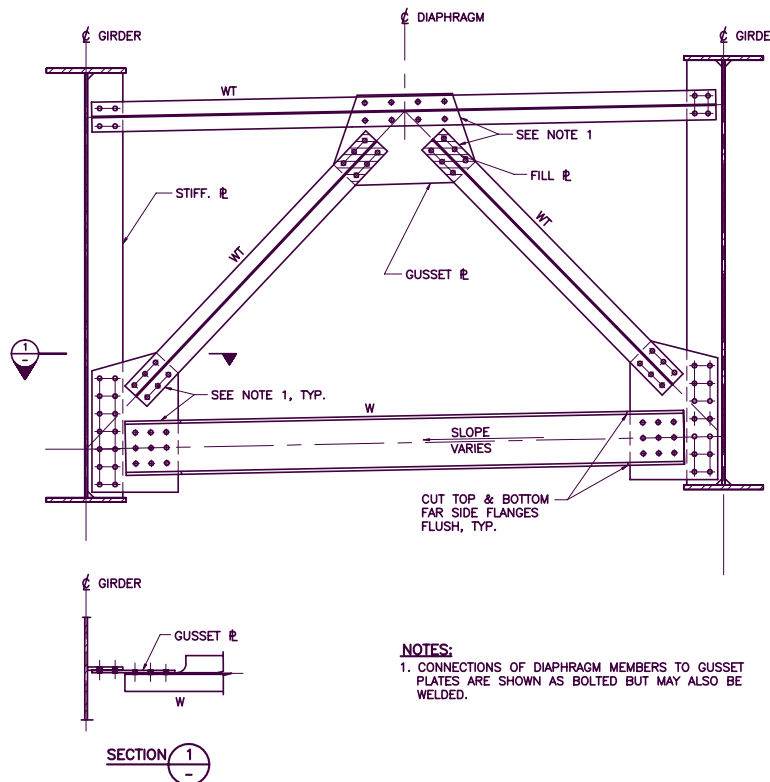


Figure 10.2 Typical Diaphragm

Commentary:

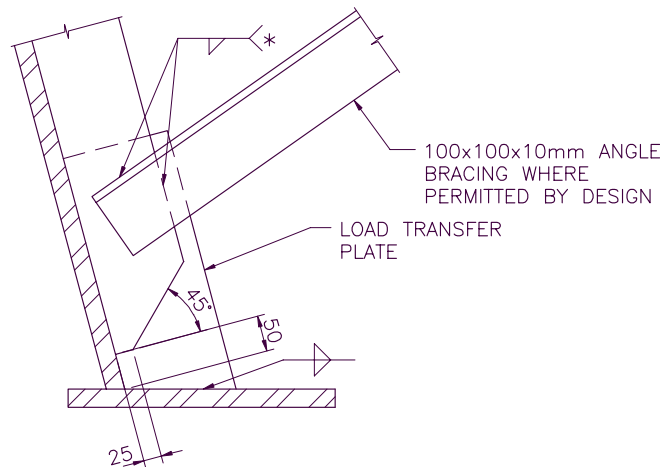
The preferred “K” brace system for use between girders should consist of angles shop welded to one side of gusset plates which would be field bolted to the girder stiffeners. It results in more economical fabrication and erection procedures when all frames are produced in one jig and when fewer pieces are handled in the field.

The designer shall account for eccentric force effects for both strength and fatigue arising from the arrangement described above.

For curved girder design, the arrangement described above may result in heavy members because of additional stresses from eccentric connections.

Box Girder Bracing.

Unless design requirements dictate otherwise, 100 x 100 x 10 mm angles should be used 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 show this on the shop drawings which shall be subject to approval by the Designer. The designer should ensure that the interior bracing can be welded to the web stiffeners (see **Figure 10.3**); and if design permits in the case of X-bracing, the intersection of the two bracing elements need not be connected.



*: WHERE POSSIBLE, WELD ALONG TOP AND BOTTOM LANDINGS OF BRACING, OTHERWISE ALONG VERTICAL FACE OF STIFFENER AND TOP OR BOTTOM LANDING OF BRACING.

Figure 10.3 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 size chosen is believed to be adequate for the normal range of bridge spans. Additional economies can be achieved when connection details are such that fabrication can be performed more easily.

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 fabricated from channel or beam sections would be less expensive in shallow bridges.

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 10.4a**). 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 10.4b**). Diaphragms between box girders at piers and abutments should be of constant depth, and bolted to exterior box girder web stiffeners (see **Figure 10.4c**). Oversized holes in diaphragms or stiffeners are permitted.

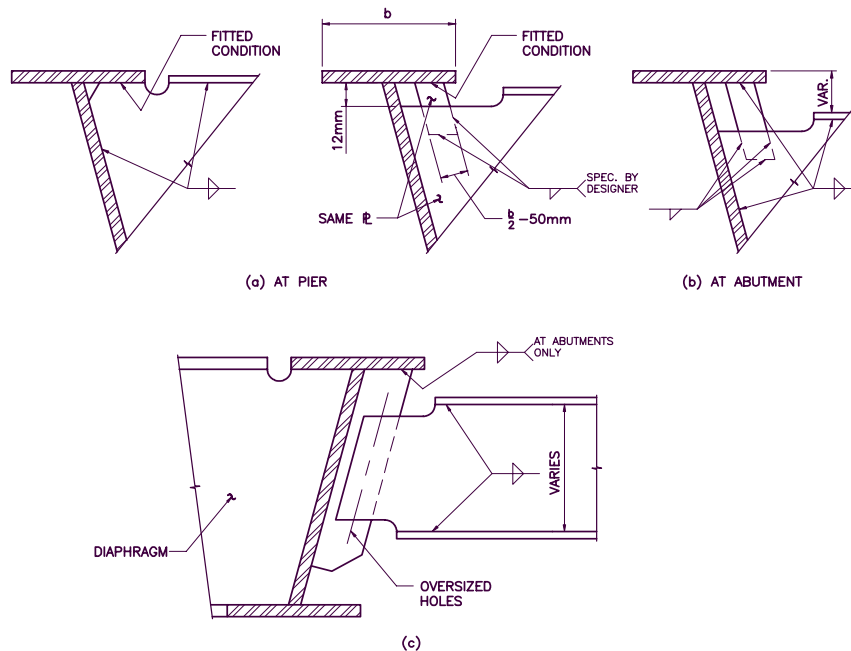


Figure 10.4 – Box Girder Diaphragms

Commentary:

*The details as shown in **Figure 10.3**, were developed to meet design and fabrication needs.*

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 outer 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 outer face is maintained facilitate web splice details. In the event a horizontal web stiffener is required by design, a flush outer face makes fabrication easier. 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 constant and the plate thickness transitions should be made into the web.

Grinding of Butt Welds.

Grinding of butt welds shall be in accordance with:

Butt welds in webs of girders designed for tension in Category B shall be “flush” for a distances 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”.

Where: “Flush” is defined as: the condition in which there is 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 as: the condition in which the surface finish of weld reinforcement has 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:

These recommendations are consistent with the latest fatigue rules. For welds specified to be ground “flush”, weld reinforcement of 1 mm is allowed (except in the case of faying surfaces) reducing the possibility of overgrinding and repair. In webs of girders, butt welds more than 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.

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 are not met.

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.

Bearing Stiffener to Flange Connection

As shown in **Figure 10.5**, 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.

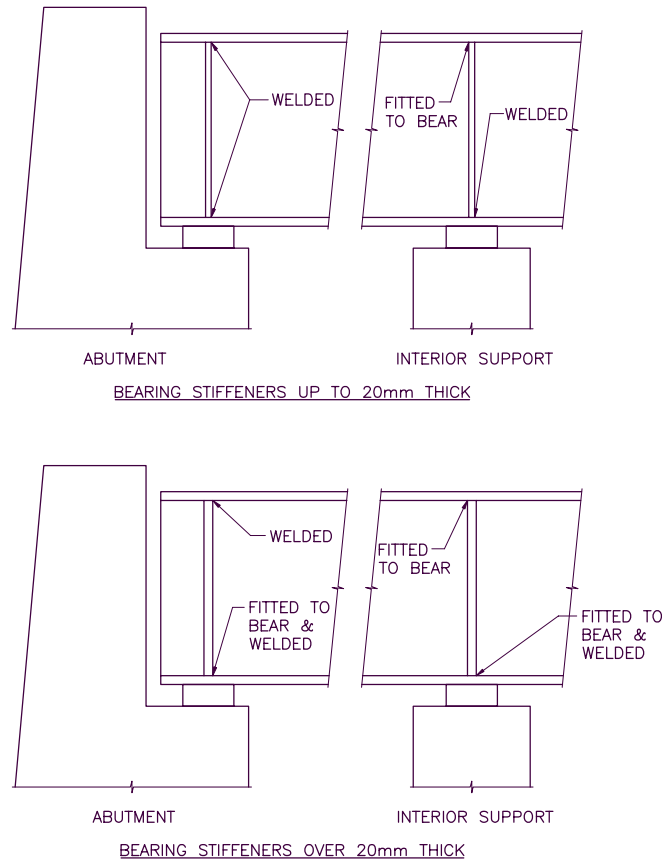


Figure 10.5 – Bearing Stiffener to Flange Connections

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.

Intermediate Stiffener to Flange Connection

In plate girders up to a depth of 2000 mm, in the positive moment regions, the intermediate stiffeners between bottom lateral bracing, cross-frames, and diaphragm connection points should be cut short of the tension flange. At the bracing connection points, intermediate stiffeners may be either fitted or welded transversely to the tension flange, depending on the fatigue requirements. In negative moment regions, all intermediate stiffeners should be fitted to 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 either welded, bolted or fitted to the tension flange, depending on the allowable fatigue stress 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 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.

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.

Intersecting Longitudinal and Transverse Stiffeners

Where possible, longitudinal stiffeners shall be located on the opposite side of the girder web to intermediate transverse stiffeners. 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.

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.

Box Girder Web Stiffeners.

Web stiffeners on the inner and outer faces of box girders should be cut short of the bottom flange (see **Figures 10.6a** and **10.6c**). If a fitted condition is required due to design, an additional plate may be provided (see **Figure 10.6b**).

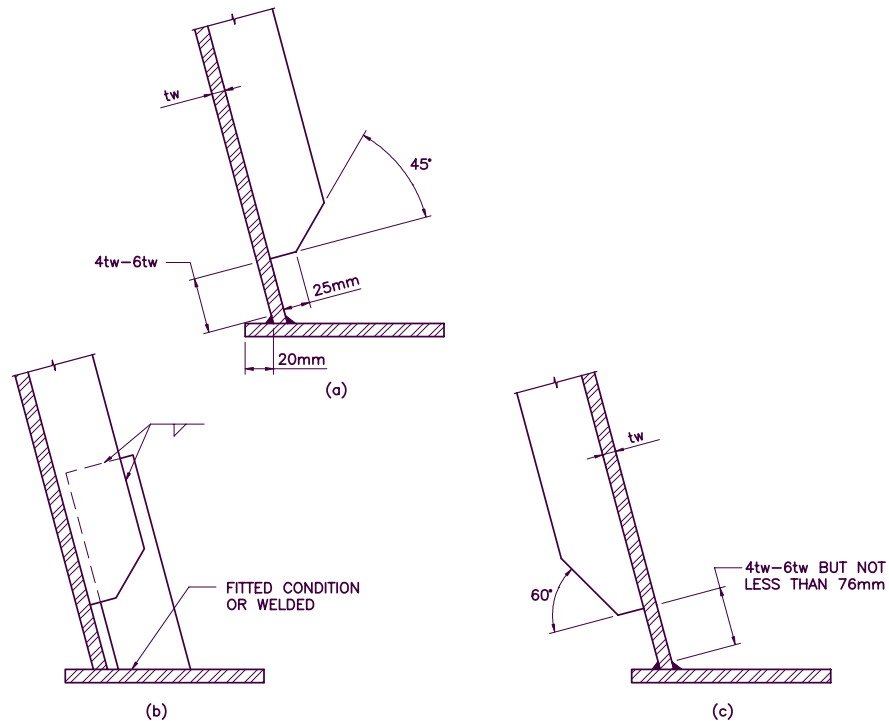


Figure 10.6 – Box Girder Intermediate Web Stiffeners

Commentary:

*In order to allow the use of automatic welding of the web-to-flange joint, the details as shown in **Figures 10.6a** and **10.6c** 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. If a fitted condition is required by design, for example at intermediate bracing locations, horizontally curved girders or skewed girders, the detail shown in **Figure 10.6b** is a possible solution.*

Box Girder Bottom Flange Stiffener Details

For box girders, wide flange “I” or “T” section longitudinal bottom flange stiffeners are preferred in lieu of plate sections as shown in **Figure 10.7**. The sections should be spaced a minimum of 305 mm between flanges to allow the use of automatic welding equipment. Channel sections, welded to the top of the wide flange longitudinal stiffeners, and to the inner web stiffeners, are preferred as transverse bottom flange stiffeners.

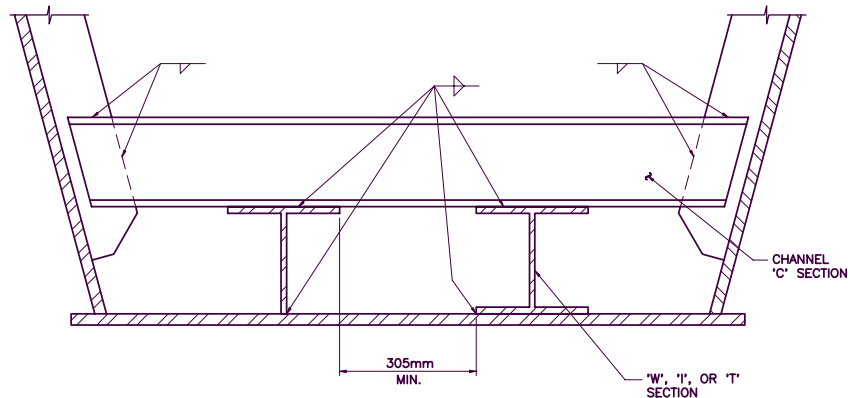


Figure 10.7 – Box Girder Bottom Flange Stiffener Details

Commentary:

10.7.4 Camber

10.7.4.1 Design

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

Camber shall be provided by the Design Consultant. Camber shall be shown on splice points and on intervals not greater than 2m.

A camber diagram shall be included and shall include elevations for:

- Girders laid flat (no gravity);
- Erected girders (girder self-weight and bracing only);
- Deck complete (excluding asphalt or high density concrete overlays, parapets, and long term effects); and
- Final stage (including asphalt or high density concrete overlays, parapets, and long term effects).

Commentary:

The camber for the “girders laid flat” is required for the fabricator, the “erected girders” is required for the steel erector, and the “deck complete,” together with the theoretical computed camber for the “erected girders” compared to the actual elevation profile of the girders after they are erected, is required to determine screed elevations.

10.7.4.1 Design

This clause shall be amended by the addition of the following regarding moment and shear diagrams and bearing reactions:

Moment and Shear Diagrams

The following combinations of the moment and shear diagrams for the Ultimate Limit States design of the continuous or semi-continuous (where applicable) structures shall be shown as follows:

- a) A
- b) B
- c) (B+C)
- d) (B+C+D)
- e) (B+C+D+E)
- f) (B+C+D+E+F)

where:

- A = Factored Resistance
- B = Factored non-composite dead load moment and shear (including effect of deck pour sequence, if any)
- C = Factored composite dead load moment and shear (including effect of deck pour sequence, if any).
- D = Factored live load moment and shear (consideration shall be given to displaying maximum moment and maximum shear, or maximum moment with corresponding concurrent shear and maximum shear with corresponding concurrent moment)
- E = Factored dynamic load allowance for moment and shear
- F = Factored moments and shears from shrinkage, creep, temperature, prestress, etc. (where applicable)

Tabulations of the load factors and distribution factors shall also be included. The method of obtaining the distribution factors shall be indicated.

Service Bearing Reactions

Service level bearing reactions for each of combinations a) through f) shall also be shown on the moment and shear diagram drawing.

Commentary:

Service bearing reactions provide valuable information to bearing manufacturers.

10.10

BEAMS AND GIRDERS

10.10.8

Bearing Stiffeners

10.10.8.1

Web Crippling and Yielding

The second sentence of the first paragraph shall be replaced with:

“The factored compressive resistance of webs shall be computed as:...”

Commentary:

Bearing stiffeners are incorrectly referred to in the Code.

10.17

STRUCTURAL FATIGUE

10.17.1

General

This clause shall be amended by the addition of the following sentence:
Where diaphragms, lateral bracing, and floor beams are assumed to be effective in distributing live load effects, they shall be investigated for fatigue.

Commentary:

If members are utilized to distribute live load, they are susceptible to fatigue issues. These fatigue issues must be investigated.

10.17.2

Live Load-Induced Fatigue

10.17.2.6

Fatigue Resistance of Stud Shear Connectors

Correct the existing equation to read:

$$Z_{sr} = (238 - 29.5 \log N_C) d^2 \geq 38d^2/2$$

Commentary:

Equation in original issue of S6-00 is incorrect.

10.18

SPLICES AND CONNECTIONS

10.18.1

General

Dead end connections for cables (hangers, suspension cables, cable stays, etc.) shall be designed so that the ultimate breaking strength of the connection exceeds the minimum 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 sentences:

- Exposed anchor bolts shall be galvanized or metallized.
- Anchor bolt nuts shall be secured with lock-nuts or locking screws;
- Proprietary anchorage systems may be used only with Ministry approval.
- Mechanical anchorage systems shall not be used.

- Consideration may be given to the use of anchors in pipe sleeves to provide erection tolerance.

Commentary:

10.20

PINS, ROLLERS, AND ROCKERS

10.20.2

Pins

Add the following sentence:

Pin hole diameter shall not be more than 1 mm greater than the pin diameter.

Commentary:

10.23

FRACTURE CONTROL

10.23.5

Welding Corrections and Repairs on Fracture-Critical Members

10.23.5.6 (g)

Minimum Steps for Repair

This sentence on preheat and interpass temperatures contradicts information provided in CSA Standard W59. The Ministry requires the use of W59.

Commentary:

Temperatures listed in the CAN/CSA S6-00 are different from CSA Standard W59. Since W59 is the applicable welding code, until this issue is resolved, the Ministry requires that designers default to it.

10.24

CONSTRUCTION REQUIREMENTS FOR STRUCTURAL STEEL

10.24.5

Welded Construction

10.24.5.1

General

Add the following sentence:

Field welding shall not be permitted without prior approval of the Ministry.

Commentary:

Quality Assurance of field welding is problematic. General field welding and field welding of splices are strongly discouraged but permission for these operations may be granted in unique circumstances.

10.24.6 Bolted Construction

10.24.6.1 General

Add the following sentence:
Field splices generally shall be bolted connections.

Commentary

10.24.8 Quality Control

This clause shall be amended by the addition of the following sentence:
The requirements of this clause shall be coordinated with SS 421 “Structural Steelwork.”

Commentary:

The designer is cautioned that W59 requires the engineer to specify the type and extent of testing for welds. Even when QA/QC is both the responsibility of the Contractor, the designer must specify the testing requirements of the welding.

10.24.8.2 Non-Destructive Testing of Welds

This clause shall be amended by the addition of the following sentence:
Ultrasonic Testing (UT) may replace Radiographic Testing (RT) at the direction of the Ministry.

Commentary:

This requires further investigation.

10.24.9 Transportation and Delivery

Add the following statement regarding cleaning steelwork after transport:
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. Cleaning the steelwork after erection has been completed shall be required for all areas that will be accessible.

11.

Joints and Bearings

11.4

COMMON REQUIREMENTS

The following sentence shall be added:

The structural steel/concrete interface shall be detailed such that no rust staining of the concrete occurs.

Commentary:

11.4.2

Design Requirements

The following requirements for elastomeric bearings shall be added:

Steel reinforced elastomeric bearings shall have at least two steel reinforcing plates and the minimum cover of elastomer for the top and bottom steel reinforcing plates shall be 5 mm. Allowable tolerances shall be + 3 mm, - 0 mm.

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

The design of un-reinforced and steel reinforced elastomeric bearings for compressive deformation shall account for the different deformation responses (when subjected to the same compressive force) of the following:

- the internal layers of elastomer in steel reinforced bearings
- the cover layer of elastomer in steel reinforced bearings
- plain unreinforced bearings

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.

The following requirements for the tabulation of design loads and bearing pressures shall be added:

The tabulation of permanent vertical load, total vertical load, and bearing pressures at serviceability limit states design shall be shown on the drawing for each bearing.

Commentary:

11.5 DECK JOINTS

11.5.1 General Requirements

11.5.1.1 Functional Requirements

This clause is amended by the addition of the following:

Unless otherwise approved 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.

The "finger" plate deck expansion joint shall have a drainage trough 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 hose connection shall be provided to allow easy attachment for flushing and cleaning of the drainage trough for future maintenance.

Commentary:

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

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.

The 10% slope is to assist in self-cleaning of the trough.

This clause is also amended by the addition of the following:

- Only deck joint seals made of rubber or neoprene will be allowed.

Commentary:

Deck joint seals made of tyfoprene, santoprene, and silicon have been observed to perform poorly and will not be allowed.

This clause is also amended by the addition of the following:

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.

This clause is also amended by the addition of the following:

- The number of deck joints within a structure shall be kept to a minimum.

Commentary:

There has been a growing tendency towards the design of joint-free continuous bridges in recent years. The main weakness in the various forms of deck joints has been the lack of durability and associated maintenance problems.

Continuity in bridge decks with pretensioned precast concrete girders has been researched and designed since 1960.

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 to the substructure and bearings. Reinforced concrete crossheads and piers often corrode following attack by chloride penetration.

A single line of bearings in lieu of a double row of bearings over the piers may result in a reduction in construction costs.

This clause is also amended by the addition of the following:

- Proprietary joint products must be approved prior to use by the Ministry.

This clause is also amended by the addition of the following:

Water ingress into or onto the substructure or abutment wall backfill from the superstructure above shall be prevented. Joints between the superstructure/end diaphragm and the substructure shall be waterproofed.

This clause is also amended by the addition of the following:

Modular deck joints should be avoided and may be used only with the approval of the Ministry.

Commentary:

Modular joints are expensive and have maintenance problems and should only be used if no other option exists.

This clause is also amended by the addition of the following:

- The joint shall be accessible for inspection and maintenance and shall be replaceable except for elements permanently attached to the structure.

Commentary:

11.5.1.2 Design Loads

The third paragraph is deleted and replaced 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 as approved by the Ministry.

Commentary:

11.5.3 Design

11.5.3.2 Components

11.5.3.2.4 Bolts

The contents of this clause are deleted and replaced with the following:

All anchor bolts for bridging plates, joint seals, and joint anchors shall be high-strength bolts fully torqued/tensioned as specified. Cast-in-place anchors shall be used only in new concrete. Expansion anchors shall not be permitted on any joint connection. Countersunk anchor bolts shall not be permitted on any joint connection unless approved by the Ministry.

Commentary:

11.6 BRIDGE BEARINGS

11.6.1 General

The following requirements for bearing accessibility shall be added:

Bearings shall be designed for easy maintenance, inspection, and replacement. Bearing replacement procedure shall be shown on the bridge drawings, including jacking locations and jacking loads. Proprietary products must be approved by the Ministry prior to use.

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, e.g. between bearings and shoe plates, must be bolted or make use of accessible screws and must not be welded.

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 shelf and the soffit of the diaphragm, and as many sides of the bearing should be accessible as possible.

11.6.1

General

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.

Commentary:

The designer shall ensure compatibility between the various structural elements (shear keys and their allowable gaps, joints, and bearings).

11.6.1

General

The sixth paragraph is deleted and replaced with the following:

- The design-bearing rotation, θ_u , shall be taken as the sum of the rotations due to the ULS loads and tolerances in fabrication and installation, plus one degree, except for elastomeric bearings.

Commentary:

Elastomeric bearings are excluded because the design process is not dependent on the variable θ_u .

11.6.4

Spherical Bearings

11.6.4.1

General

This following sentence shall be added to this clause:

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

Commentary:

Installing these types of bearings with the concave part facing upwards creates a "bowl" which can collect water and dirt and affect bearing performance and durability.

11.6.6 Elastomeric Bearings

11.6.6.3 Geometric Requirements

Contrary to part (a) of this clause, the thickness specified on the contract drawings for plain bearings should be between 15 mm and 25 mm. The shape factor must always be checked.

Commentary:

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

12.

Barriers and Highway Accessory Support

12.5

BARRIERS

12.5.2

Traffic Barriers

12.5.2.1

Performance Level

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 CAN/CSA-S6-00. Any bridge traffic 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 CAN/CSA-S6-00 and prior approval from the Ministry.

Performance Level PL-1

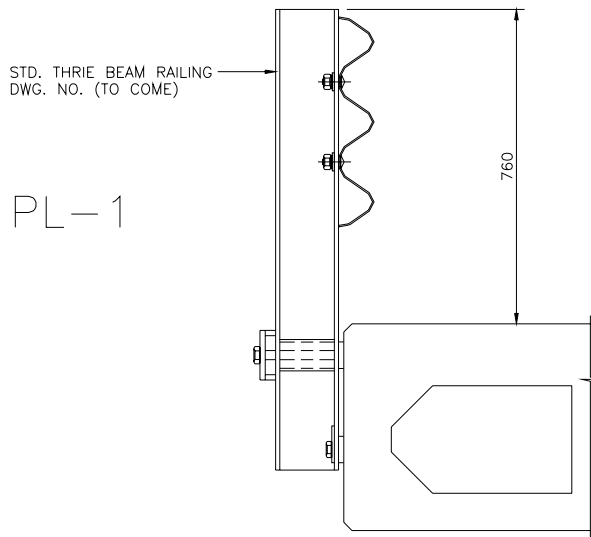


Figure 12.5.2.1 a) Thrie Beam Bridge Railing (Box Girder Side-Mounted)

Commentary:

The system shown in **Figure 12.5.2.1a** 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 which may be of assistance to designers can be found at the following websites:

- <http://safety.fhwa.dot.gov/fourthlevel/hardware/pdf/appendixb7b.pdf> (page 4 of 9)
- http://www.odot.state.or.us/tsspecs/pub/pdf/dwgs/met/y02_br233.pdf (metric units)
- <http://www.odot.state.or.us/tsspecs/pub/pdf/dwgs/eng/ebr233.pdf> (imperial units)

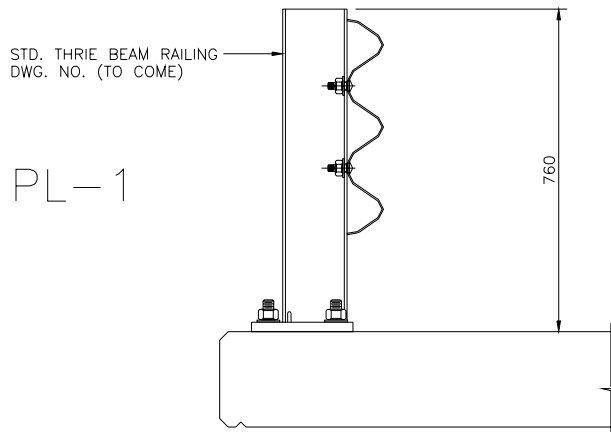


Figure 12.5.2.1 b) Thrie Beam Bridge Railing (Top-Mounted)

Commentary:

The system shown in **Figure 12.5.2.1b** is based on the crash tested 'Oregon Side Mounted Thrie-Beam' bridge railing (see **Figure 12.5.2.1a**), 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 12.5.2.4 of CAN/CSA-S6-00.

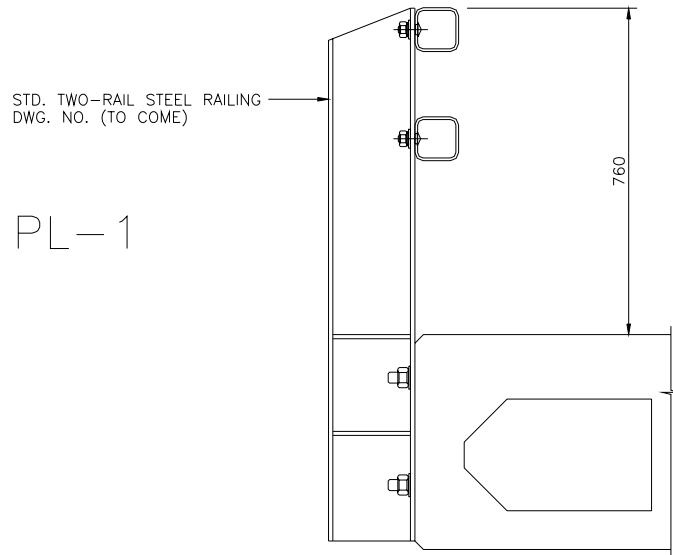
Alberta Transportation has adopted similar top-mounted thrie beam railing systems as part of its bridge railing standards. Until such time as the Ministry develops a standard drawing for this system, information regarding the Alberta railings which may be of assistance to designers can be found at the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

- <http://www.trans.gov.ab.ca/Content/doctype30/production/S1652-00-rev2.pdf>
- <http://www.trans.gov.ab.ca/Content/doctype30/production/S1653-00-rev1.pdf>

Information regarding the Oregon bridge railing which may be of assistance to designers can be found at the following websites:

- <http://safety.fhwa.dot.gov/fourthlevel/hardware/pdf/appendixb7b.pdf> (page 4 of 9)
- http://www.odot.state.or.us/tsspecs/pub/pdf/dwgs/met/y02_br233.pdf (metric units)
- <http://www.odot.state.or.us/tsspecs/pub/pdf/dwgs/eng/ebr233.pdf> (imperial units)

Figure 12.5.2.1 c) Steel Two-Rail Bridge Railing (Box Girder Side-Mounted)



Commentary:

The system shown in **Figure 12.5.2.1c** 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 which may be of assistance to designers can be found at the following website:

- <http://safety.fhwa.dot.gov/fourthlevel/hardware/pdf/appendixb7c.pdf> (page 7 of 25)

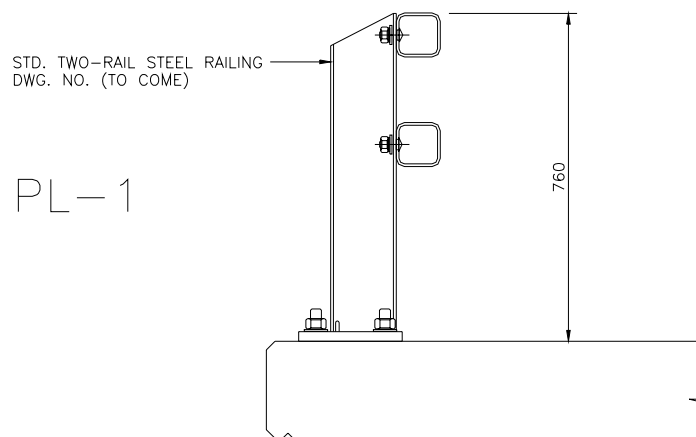


Figure 12.5.2.1 d) Steel Two-Rail Bridge Railing (Top-Mounted)

Commentary:

The system shown in **Figure 12.5.2.1d** is based on the crash tested 'California Type 115' bridge railing (see **Figure 12.5.2.1c**), 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 12.5.2.4 of CAN/CSA-S6-00.

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

- <http://safety.fhwa.dot.gov/fourthlevel/hardware/pdf/appendixb7c.pdf> (page 7 of 25)

Performance Level PL-2

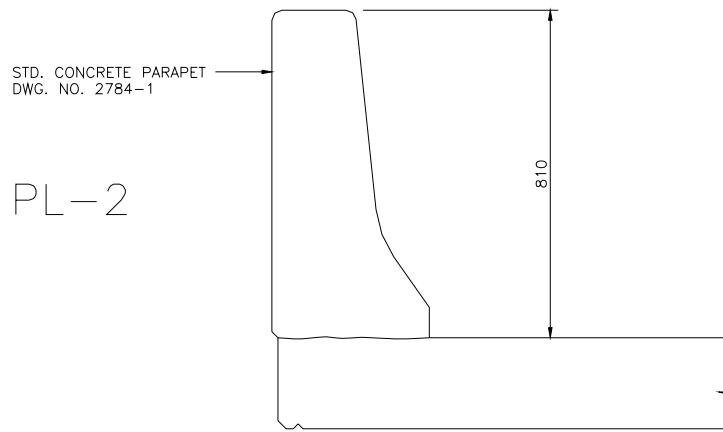


Figure 12.5.2.1 e) Cast-in-Place Concrete Bridge Parapet (810 mm High)

Commentary:

The system shown in **Figure 12.5.2.1e** 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 12.5.2.4 of CAN/CSA-S6-00.

Information regarding crash tested F-shape bridge railings which may be of assistance to designers can be found at the following website:

- <http://safety.fhwa.dot.gov/fourthlevel/hardware/pdf/appendixb7g.pdf>

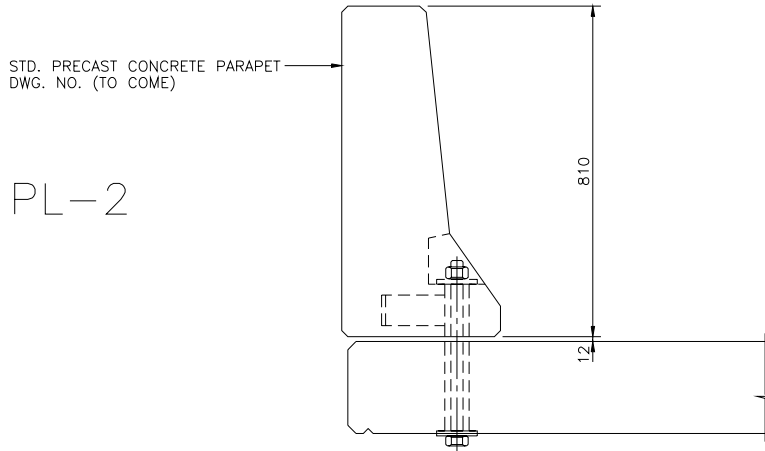


Figure 12.5.2.1 f) Precast Concrete Bridge Parapet

Commentary:

The system shown in Figure 12.5.2.1f is based on a Standard Precast Parapet (Preliminary Standard Drawing No. 2965-4) which the Ministry has used on a number of 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 12.5.2.4 of CAN/CSA-S6-00.

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

- <http://safety.fhwa.dot.gov/fourthlevel/hardware/barriers/pdf/b-5.pdf>
- <http://safety.fhwa.dot.gov/fourthlevel/hardware/barriers/pdf/b-5a.pdf>

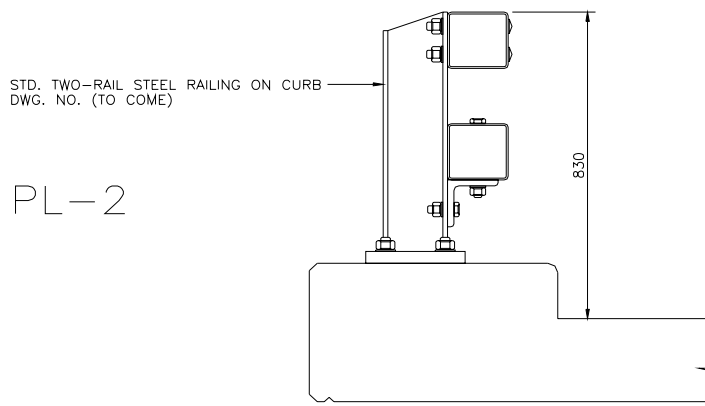


Figure 12.5.2.1 g) Steel Two-Rail Bridge Railing with Brush Curb

Commentary:

The system shown in **Figure 12.5.2.1g** 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 which may be of assistance to designers can be found at the following websites:

- <http://www.dot.state.ny.us/caddinfo/structures/files/bdrs1.pdf>
- <http://safety.fhwa.dot.gov/fourthlevel/hardware/barriers/pdf/b-72.pdf>

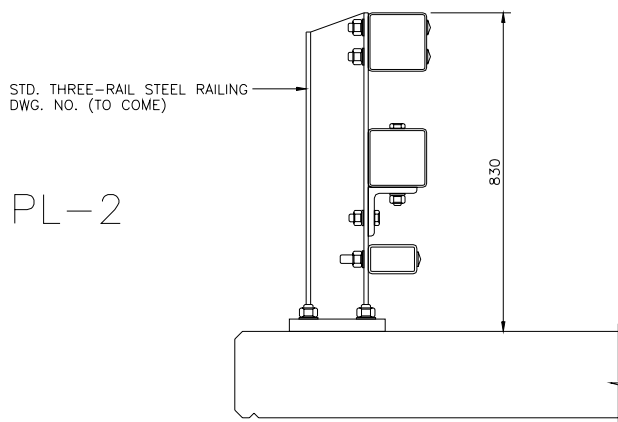


Figure 12.5.2.1 h) Steel Three-Rail Bridge Railing

Commentary:

The system shown in **Figure 12.5.2.1h** 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 which may be of assistance to designers can be found at the following websites:

- <http://www.dot.state.ny.us/caddinfo/structures/files/bdrs1.pdf>
<http://safety.fhwa.dot.gov/fourthlevel/hardware/barriers/pdf/b-72.pdf>

Performance Level PL-3

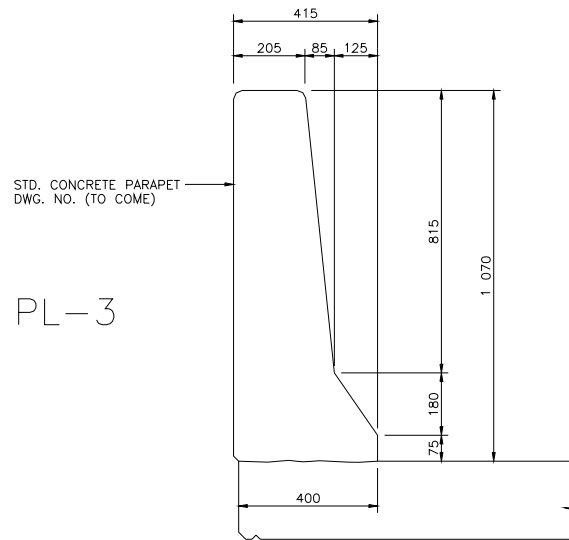


Figure 12.5.2.1 i) Cast-in-Place Concrete Bridge Parapet (1070 mm High)

Commentary:

The system shown in **Figure 12.5.2.1i** 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 12.5.2.4 of CAN/CSA-S6-00.

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 which may be of assistance to designers can be found at the following website:

- <http://safety.fhwa.dot.gov/fourthlevel/hardware/pdf/appendixb7g.pdf>

12.5.2.2 Geometry and End Treatment Details

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

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 or super elevation.

12.5.3 Pedestrian Barriers

Where pedestrian barriers are required in accordance with Clause 12.5.3 of CAN/CSA-S6-00, the Ministry Standard steel sidewalk fence shall be used (Standard Drawing 2891-1K). The standard sidewalk fence shall extend three (3) metres beyond the bridge abutments.

Commentary:

The Ministry Standard steel sidewalk fence shall be used on an interim basis for the safety of pedestrian traffic on bridge sidewalks until such time as the design has been reviewed in accordance with CAN/CSA-S6-00 and any revisions to the design have been finalized.

12.5.3.1 Geometry

Pedestrian barriers shall be constructed such they are oriented vertically.

12.5.4 Bicycle Barriers

Where bicycle barriers are required in accordance with Clause 12.5.4 of CAN/CSA-S6-00, the Ministry Standard steel bicycle fence shall be used (refer to Standard Drawing 2891-2C).

Commentary:

The Ministry Standard steel bicycle fence shall be used on an interim basis for the safety of pedestrian and cyclist traffic on bridge sidewalks until such time as the design has been reviewed in accordance with CAN/CSA-S6-00 and any revisions to the design have been finalized.

12.5.4.1 Geometry

Bicycle barriers shall be constructed such they are oriented vertically.

12.5.5 Combination Barriers

Use of Combination Barriers

For highway bridges without sidewalks, either a Pedestrian Combination Barrier or a Bicycle Combination Barrier shall be installed on both sides of the bridge. The use of Traffic Barriers in lieu of Combination Barriers may be acceptable in remote areas, as determined by the Design Engineer in consultation with the Ministry, on the basis of the anticipated volume of pedestrian and/or bicycle traffic and 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, in consultation with 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, in consultation with 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.

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 12.5.2.2 of CAN/CSA-S6-00).

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.

Pedestrian Combination Barriers

Pedestrian Combination Barriers as shown in **Figures 12.5.5 a) to 12.5.5 d)** have been accepted by the Ministry for use on highway bridges in B.C. and meet the crash testing requirements of CAN/CSA-S6-00. 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 crash testing requirements of CAN/CSA-S6-00 and prior approval from the Ministry.

Performance Level PL-2

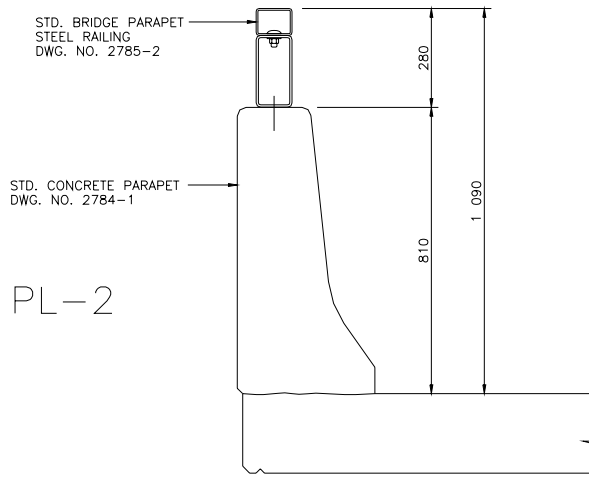


Figure 12.5.5 a) Cast-in-Place or Precast Concrete Bridge Parapet (810 mm High) with Steel Pedestrian Rail

Commentary:

The system shown in **Figure 12.5.5a** 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 to address rail setback concerns, 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 12.5.5 of CAN/CSA-S6-00.

See Commentary for **Figure 12.5.2.1e** regarding the Cast-in-Place Concrete Bridge Parapet (810 mm High).

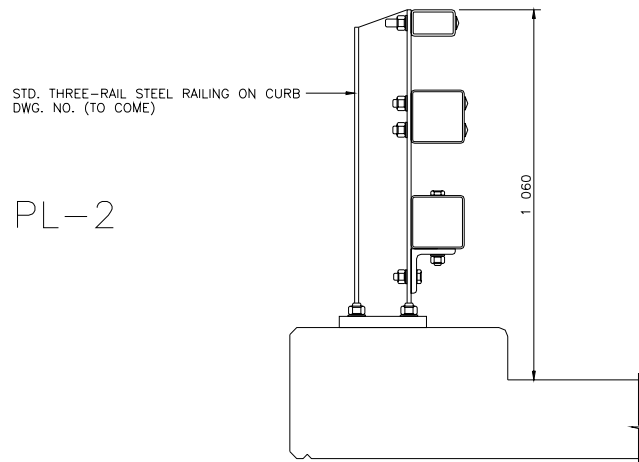


Figure 12.5.5 b) Steel Three-Rail Bridge Railing with Brush Curb

Commentary:

The system shown in **Figure 12.5.5b** 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 which may be of assistance to designers can be found at the following websites:

- <http://www.dot.state.ny.us/caddinfo/structures/files/bdrs1.pdf>
- <http://www.dot.state.ny.us/caddinfo/structures/files/bdrs2.pdf>
- <http://safety.fhwa.dot.gov/fourthlevel/hardware/barriers/pdf/b-72.pdf>

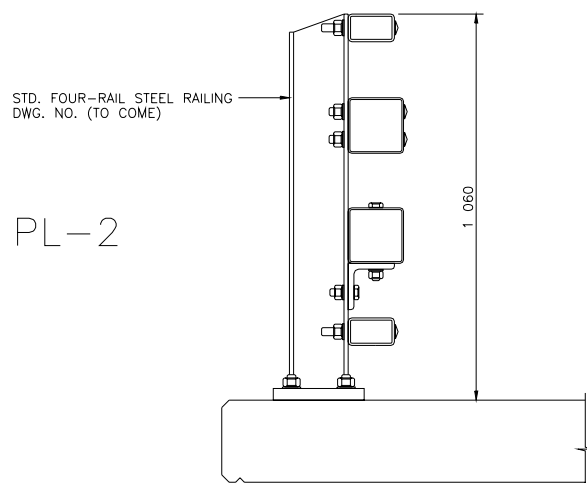


Figure 12.5.5 c) Steel Four-Rail Bridge Railing

Commentary:

The system shown in **Figure 12.5.5c** 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 which may be of assistance to designers can be found at the following websites:

- <http://www.dot.state.ny.us/caddinfo/structures/files/bdrs2.pdf>
- <http://safety.fhwa.dot.gov/fourthlevel/hardware/barriers/pdf/b-72.pdf>

Performance Level PL-3

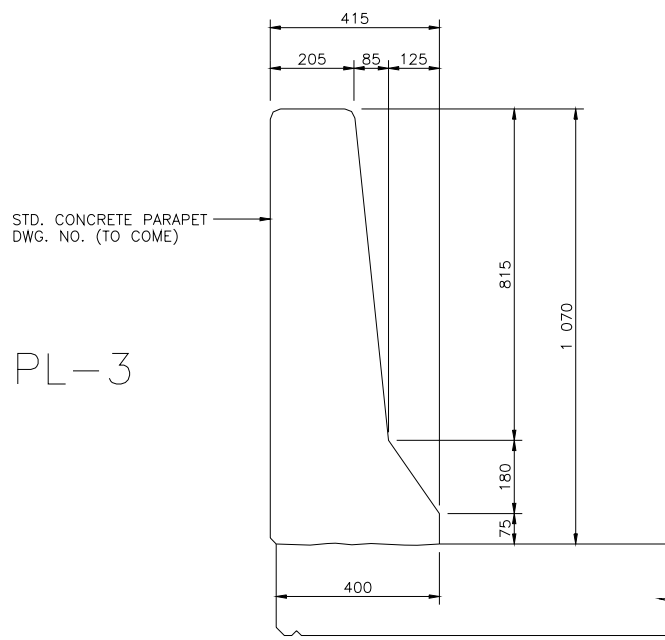


Figure 12.5.5 d) Cast-in-Place Concrete Bridge Parapet (1070 mm High)

Commentary:

See Commentary for **Figure 12.5.2.1i** regarding the Cast-in-Place Concrete Bridge Parapet (1070 mm High).

Bicycle Combination Barriers

Bicycle Combination Barriers as shown in **Figures 12.5.5 e) to 12.5.5 g)** have been accepted by the Ministry for use on highway bridges in B.C. and meet the crash testing requirements of CAN/CSA-S6-00. 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 CAN/CSA-S6-00 and prior approval from the Ministry.

Performance Level PL-2

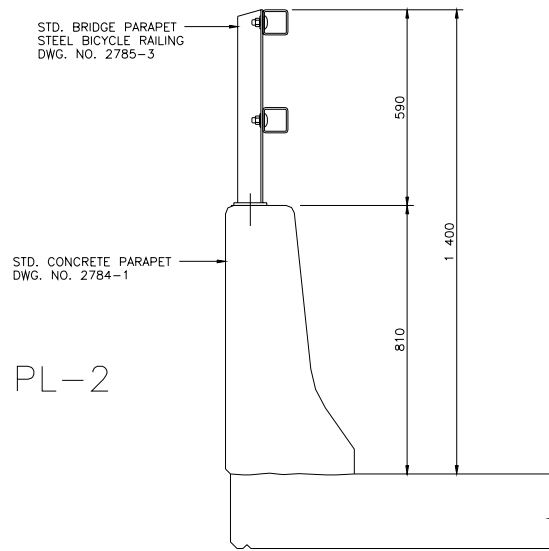


Figure 12.5.5 e) Cast-in-Place or Precast Concrete Bridge Parapet (810 mm High) with Steel Bicycle Rail

Commentary:

The system shown in **Figure 12.5.5e** 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 to address rail setback concerns, 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 12.5.5 of CAN/CSA-S6-00.

See Commentary for **Figure 12.5.2.1e** regarding the Cast-in-Place Concrete Bridge Parapet (810 mm High).

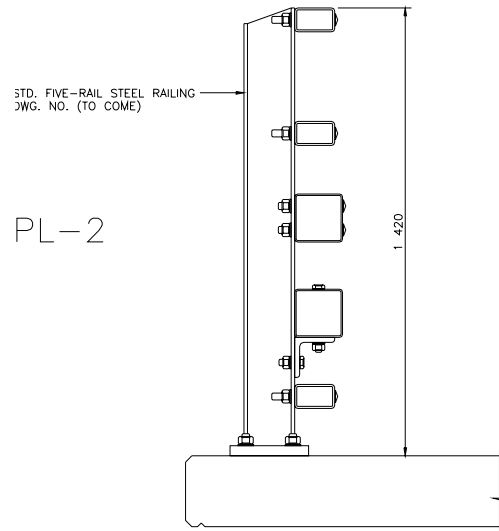


Figure 12.5.5 f) Steel Five-Rail Bridge Railing

Commentary:

The system shown in **Figure 12.5.5f** 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 which may be of assistance to designers can be found at the following websites:

- <http://www.dot.state.ny.us/caddinfo/structures/files/bdrs3.pdf>
- <http://safety.fhwa.dot.gov/fourthlevel/hardware/barriers/pdf/b-72.pdf>

Performance Level PL-3

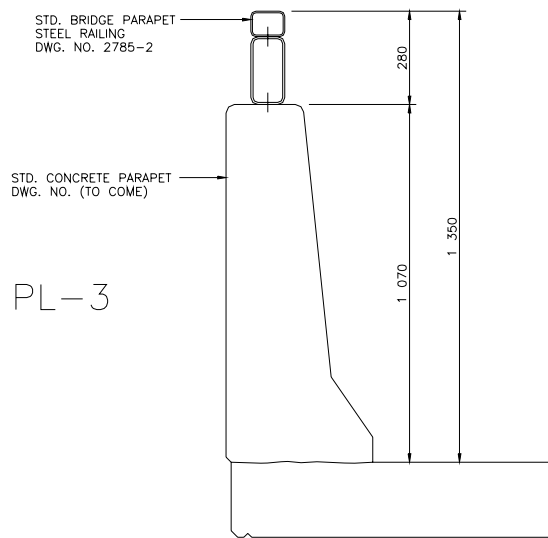


Figure 12.5.5 g) Cast-in-Place Concrete Bridge Parapet (1070 mm High) with Steel Pedestrian Rail

Commentary:

The system shown in **Figure 12.5.5g** 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 to address rail setback concerns, 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 12.5.5 of CAN/CSA-S6-00. (Note that alternate top railing designs must provide an overall minimum barrier height of 1.37 m).

See Commentary for **Figure 12.5.2.1i** regarding the Cast-in-Place Concrete Bridge Parapet (1070 mm High).

Sidewalks Separated from Traffic by Raised Curbs

Use of sidewalks separated from traffic by raised curbs requires Ministry approval and is typically only used in urban areas with low traffic volumes where design speeds are not greater than 60 km/hr. 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 CAN/CSA-S6-00. Any Combination Barrier proposed for use on such sidewalks for Ministry of Transportation bridges in B.C., other than those shown in **Figure 12.5.5 h**) below, require proof of meeting crash testing requirements of CAN/CSA-S6-00 and prior approval from the Ministry.

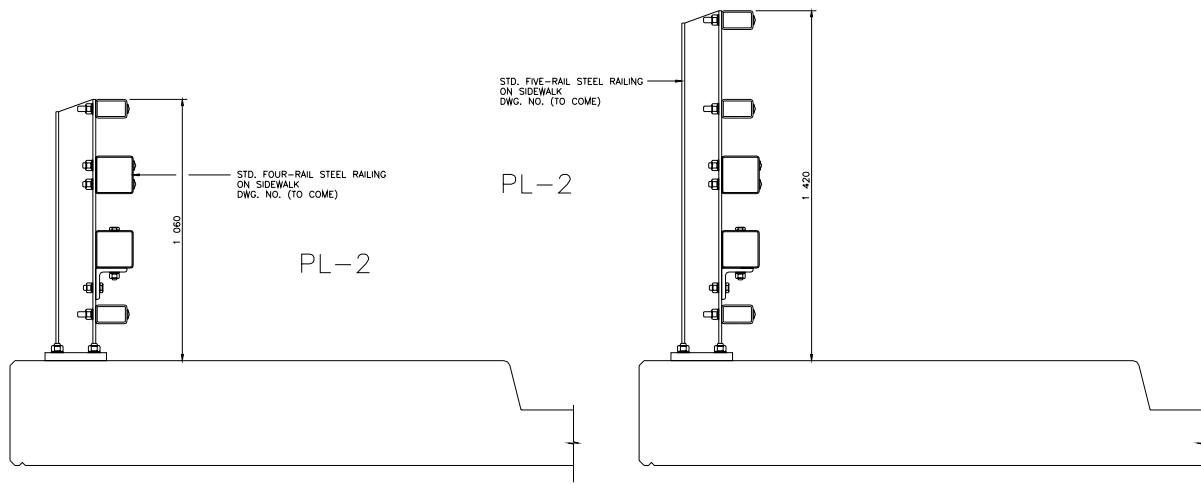


Figure 12.5.5 h) Combination Barriers on Sidewalks Separated from Traffic by Raised Curbs

Commentary:

See Commentary for **Figure 12.5.5c** regarding the Steel Four-Rail Bridge Railing.

See Commentary for **Figure 12.5.5f** regarding the Steel Five-Rail Bridge Railing.

12.5.5.1

Geometry

Where Combination Barriers are installed on sidewalks separated from traffic by raised curbs, the barriers shall be constructed such they are oriented vertically. 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 remain correctly oriented to withstand vehicle impacts which may be inclined due to deck crossfall or super elevation.

13.

Movable Bridges

13.1

SCOPE

Commentary:

Movable bridges shall not be used unless approved in writing by MoT.

Section 13 Movable Spans of the S6-00 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. The variances from Section 13 will require approval.

13.4

MATERIALS

13.4.4

Timber

Revise to read:

Timber materials and fasteners shall be in accordance with Section 9.

13.4.9

Bolts

Commentary:

High strength structural bolts shall conform to ASTM A325M unless otherwise approved.

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.

13.5.12

Access for Routine Maintenance

Commentary:

The requirement for elevators in tower-drive vertical lift bridges shall apply 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

13.6.1.1.7 Concrete

Revise the second to last sentence to read:

The design of the counterweight shall be modified if the experimental density differs from the design density.

Commentary:

The counterweight density may not be designed at 2355 kg/m³.

13.6.2.1.2

Disc Bearings

Revise the first sentence to read:

Centre-bearing swing bridges shall rotate on spherical thrust bearings.

Commentary:

Traditional bronze disc thrust bearings are being replaced by self lubricated spherical thrust bearings and the change is intended to allow such technology.

13.6.1.4

Span aligning and Locking

Revise the last sentence to read:

Where the ends of bascule bridge decks are located behind the centre of rotation and where calculations indicate the toe may be lifted from toe rests under the passage of live load, tail locks shall be provided in order to resist the maximum reactions from live load.

Commentary:

To minimize bridge complexity and future maintenance, tail locks can be eliminated if the toe stays in position under all live load conditions.

13.6.2.3.2 Pinion Bearing Supports

Commentary:

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 torque in the pinion. Note that the maximum torque may occur under braking or acceleration.

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. Therefore a more efficient requirement is to have locking devices on the toe ends of each outside girder.

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

13.7.3.4 Vertical Wind, Normal to the Floor Plane Area

Commentary:

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

13.7.6 Hydraulic Cylinder Connections

Revise to read:

The loads on the structural connections to the cylinders shall be based on the greater of:

- wind, ice, inertia or other structural loads assuming the cylinder as a rigid link; and,
- driving and braking mechanical loads assuming a cylinder force developed by 150% of the setting of the pressure-relief valve that controls the maximum pressure available at the cylinder.

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 Requirements for 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.

13.8.7.1.2 Holding

Commentary:

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

13.8.8.1 Machinery

Commentary:

Self-lube bearing materials may be appropriate for some applications. For proprietary bearing materials the coefficients of friction shall be as advised by the suppliers.

13.8.9.1 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.

13.8.9.6 Torque for Lock and Wedge Machinery

Add the following sentence:

Where span locks and wedges are operated by hydraulic systems, the hydraulic systems shall be capable of providing 150% of the maximum span lock torque or equivalent force at the normal operating pressure.

Commentary:

Cylinder operated locks and wedges are common and therefore basic design guidelines are required.

13.8.11 Machinery Loads

Add the following:

(e) machinery operated by hydraulic systems shall be designed for 100% of the maximum hydraulic system relief valve pressure.

Commentary:

The relief valve pressure is above the normal operating pressure and therefore the system could potentially develop the relief valve pressure with all the resultant forces in the machinery. Therefore the relief valve pressure must be one of the design cases.

13.8.13.2 Determination of Bearing Pressure

Commentary:

Where alternate bearing materials are considered, the maximum bearing pressures shall be in accordance with the supplier's recommendations.

13.8.17.4.3 Bushings

Revise the first sentence to read:

Bearings shall have bronze bushings unless otherwise approved.

Commentary:

Self-lube non bronze bushings may be appropriate for some applications, however their use is subject to approval by MoT.

13.8.19.2.2 Electrically Operated Brakes

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

Revise the second paragraph to read”

“.....(NEMA), and Canadian Standards Association (CSA) as applicable.”

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 MoT.

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 MoT.

13.10.8 Motor Temperature, insulation, and Service Factor

Commentary:

AC motors should have class F insulation in accordance with CSA or NEMA standards.

13.10.11 Speed of Motors

Commentary:

Motors for hydraulic pumps shall not exceed 1800 rpm.

13.10.15 Electrically Operated Motor Brakes

Commentary:

Interlocks is a more common word instead of escapements, thus brakes for the span operation shall be provided with hydraulic, mechanical, or electrical interlocks, such that all the brakes will not be applied at the same time.

13.10.21 Programmable Logic Controllers

Commentary:

The specified applicable standards should also include CSA as well as NEMA and IEEE.

Instead of regulating/ isolation transformers for the PLC and the input-output (I/O) power supply should come from a UPS. The AC source shall feed a UPS inverter, complete with batteries and battery charger, which provides power to the PLC and I/O systems. The UPS must be able to provide power for a minimum of 1 hour.

The PLC shall be provided with a communication card installed to allow remote communication monitoring by MoT.

13.10.26 Circuit Breakers and Fuses

Commentary:

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

13.10.36.3 Control

Commentary:

Industrial type touch screens or menu driven graphical interfaces may be provided for the normal control functions as an alternative to the pushbuttons specified.

Fibre optic wires may also be used, either multi-mode or single mode fibres pending equipment manufacture recommendations.

13.10.39 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.

13.10.42.7 Wireways

Add 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.

Commentary:

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

13.10.50 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 MoT. The list should be reviewed to include spare parts for PLC's

13.11 CONSTRUCTION

Commentary:

This section has instructions to the contractor which need to be reviewed and appropriately transferred to the Contract Documents prepared by the Designer on behalf of MoT.

13.12 OPERATIONAL INSTRUCTIONS

Commentary:

This section has instructions to the contractor which need to be reviewed and appropriately transferred to the Contract Documents prepared by the Designer on behalf of MoT.

13.13 OPERATIONAL AND MAINTENANCE HANDBOOK

Commentary: The Designer shall provide the O & M 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 shall be supplied.

14.

Evaluation

14.5

CONDITION INSPECTION

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

- Condition inspections shall be considered valid for a bridge evaluation if:
- the inspection has been conducted within the MoT-specified inspection period for that bridge,
- accurate as-constructed drawings of the bridge are available, and
- to the satisfaction of the evaluator, the condition inspection report contains sufficient information to assess the effects of deterioration on the behaviour and capacity of the structural components.

Commentary:

Bridge evaluations may often be conducted based on condition inspection reports produced by qualified inspectors who were not under the direct supervision of the evaluator at the time of the inspections. These provisions are intended to help ensure that the bridge evaluation is based on accurate and up to date information.

14.6

MATERIAL STRENGTHS

14.6.3

Strengths Based on Date of Construction

14.6.3.1

Structural Steel

Further information on steel grades may be found on the CISC website, specifically at the following URL:

http://www.cisc-icca.ca/historical_steels.html

Commentary:

Further information provided for use of designers.

14.6.3.3

Reinforcing Steel

In Table 14.6.3.3, revise the Date of Bridge Construction as follows:

- 1914 – 1955 to 1914 – 1972
- 1956 – 1978 to 1973 – 1978

All other dates remain the same.

Commentary:

The Dates of Bridge Construction and the corresponding minimum yield strengths of reinforcing steel, f_y , specified in S6-00 were erroneously based on the values given in 1983 Edition of OHBDC instead of those from the OHBDC 3rd Edition. The lower minimum yield strengths specified by OHBDC 3rd Edition are considered to be more representative of actual minimum yield strengths for reinforcing steel in wide scale use from 1956 to 1972.

14.8

TRANSITORY LOADS

This clause shall be amended by the addition of the following:
Evaluation loading shall be defined by the Ministry of Transportation on a project-specific basis.

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.

14.11

TARGET RELIABILITY INDEX

The following is provided as background information:

Commentary:

The following provision contained in the MTO Exceptions to the S6-00, dated 2002 June, shall not be adopted into the MoT manual.

“If the bridge is to be re-evaluated within 5 years for Normal Traffic, the Reliability Index, β , specified in Table 14.11(a), shall be reduced by 0.25. This value shall not be less than 2.5.”

Although this provision has been implemented for use in Ontario by the MTO, the rationale on which it is based is inconsistent with the philosophy used to develop the evaluation provisions of Section 14 of S6-00. The provisions of Section 14 were developed based on maximum annual traffic loadings and not 75 year traffic loadings. Re-evaluation of the bridge is required if there is a significant change in the bridge traffic, legal weights or truck traffic volume, or if there is significant deterioration of the bridge. Re-evaluating the bridge within 5 years does not improve on either the knowledge of traffic loadings or the condition of the bridge. Therefore, such a provision provides no benefit to the evaluation philosophy contained in Section 14. The current sub-committee for Section 14 of S6-00 2nd Edition is reviewing this issue but a rationale has yet to be developed for adopting a provision similar to MTO's into S6-00.

14.11

TARGET RELIABILITY INDEX

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

If approved 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, β , used to determine the evaluation live load factors for Normal Traffic can be reduced by 0.25. However, the reduction in β 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, β .

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.

14.11.2

Element Behaviour

This clause shall be amended by the following:

Steel in tension at net section shall remain in Category E1 but, for evaluations, the new resistance adjustment factor specified under Clause 14.13.2 of this Supplement shall be applied to the axial tensile resistances determined in accordance with Clauses 10.8.2(b) and 10.8.2(c).

Commentary:

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 14.11.2 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 14.13.2 of this document.

14.11.4 Important Structures

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

A bridge shall not be evaluated as an “Important Structure” except at the direction of the Ministry of Transportation.

Commentary:

Typically, unless a higher than normal level of reliability was a requirement during the original design of a bridge, the bridge should not be evaluated as an “Important Structure”. However, a bridge could be classified as being an “Important Structure” if failure of the structure represents an unusually high level of risk to life, isolation of a community for an extended period of time or severe economic consequences.

Risk to life is the basis for the levels of reliability specified for the evaluation of bridges and typically the reliability index, β , should only be increased when a very large number of people are at risk, such as on long span bridges. However, in S6-00, the β for longer span bridges has been increased in a “hidden” way, since the lane loading given in Section 3 is considered to be somewhat conservative for longer spans.

Economic impacts resulting from the failure of a bridge are usually secondary to the risk to life issues. Therefore, a bridge should only be classified as an “Important Structure” for economic reasons if the type of bridge component failure being considered results in total closure of the bridge to traffic and no reasonable detour routes are available and it is impractical to restore at least temporary access in a reasonable timeframe (less than two weeks as a guideline).

When assessing whether or not a bridge should be classified as “Important”, consideration should be given to the expected type and extent of damage to a bridge component resulting from an overload. If the damage can be quickly repaired or only results in a partial closure of the bridge to traffic, the bridge should not be considered as “Important”.

Note that permit controlled (PC) loads are permitted to operate at a reduced reliability index, β , compared to other types of traffic, due to the reduced risk to life (driver of PC truck typically the only one at risk). However, on an economic basis, this corresponds to a higher probability of loss of use of the structure for PC vehicles than posed by other types of traffic. Therefore, MoT should give greater consideration to classifying a bridge as "Important" for PC vehicles than for other types of traffic.

14.13

RESISTANCE

14.13.2

Resistance Adjustment Factors

The following is provided as background information:

Commentary:

The following provision contained in the MTO Exceptions to the S6-00, dated 2002 June, shall not be adopted into the MoT manual.

"For all components, which have no visible sign of defect or deterioration, the factored resistance, as calculated in accordance with Clause 14.13.1., shall be multiplied by a resistance adjustment factor, U , where $U = 1.0$ for all cases. Table 14.13.2 shall not be used."

MTO has indicated that their reason for not using the resistance adjustment factors in Table 14.13.2 is that some of the values are significantly less than 1.0. This creates a situation where a newly designed bridge could be evaluated as being deficient. Although some of the resistance adjustment factors provided in Table 14.13.2 require updating, the use of resistance adjustment factors is considered to be appropriate for the evaluation of bridges. Revisions to some of the resistance adjustment factors are recommended in this document. Further revisions to the resistance adjustment factors are expected to be included in the scheduled 2005 amendment to S6-00.

Section 14 is explicit in relating load factors to expected potential failure mechanisms. In Sections 1 to 13, there are also compensations, but they are "hidden." However, these are not as detailed. The U factor removes the hidden compensations, so that the explicit ones in Chapter 14 can be used. U should not be ignored, or set always = 1.0 as this will reduce accuracy of the evaluation.

14.13.2 Resistance Adjustment Factors

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

Add to Table 14.13.2 the following resistance adjustment factor to be applied to the axial tensile resistance of a steel component determined in accordance with Clause 10.8.2(b) and 10.8.2(c).

Steel in tension (fracture on the net section) 1.17

Commentary:

This resistance adjustment factor removes the double penalty previously being applied to the axial tensile resistance of steel components when governed by fracture on the net section.

14.13.2 Resistance Adjustment Factors

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

The resistance adjustment factors provided in Table 14.13.2 for both the shear resistance of precast and cast-in-place concrete in shear shall be modified as follows:

Change:	Shear (> min. stirrups)	0.94
To:	Shear (>minimum transverse reinforcement)	1.02

Commentary:

Updated statistical information on the resistance of concrete components in shear was provided to the calibration committee subsequent to the calibration of these resistance adjustment factors. These statistics, $\delta=1.15$ and $V=0.14$, are included in Table CA.3.2 of the Calibration Report contained in Appendix A of the S6-00 Commentary. These statistics are considered to be appropriate for use in the derivation of resistance adjustment factors for both shear in cast-in-place and precast concrete when minimum transverse reinforcement requirements of the code are achieved.

It is likely that a similar increase in the resistance adjustment factor could be applied when the minimum transverse reinforcement requirements are not met ($U=0.82$ would increase to $U=0.89$). However, it needs to be established that the statistical information included conditions with less than minimum transverse reinforcement prior to recommending an increase.

14.17

BRIDGE POSTING

14.17.1

General

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

Posting requirements for a bridge evaluated as being deficient shall be determined by the Ministry of Transportation.

Commentary:

MoT posting requirements and standards may vary from those specified in Clause 14.17 of S6-00.

14.18

FATIGUE

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

For fatigue in riveted connections, the stress Category "D" shall be used in determining the allowable range of stress in tension or reversal for base metal at net section of riveted connections.

Commentary:

This category will be useful during the evaluation and rehabilitation of existing riveted bridge structures.