

STRUCTURAL MANUAL



Ministry of Transportation Ontario
Transportation Infrastructure Management
Division, Standards and Contracts Branch
Structures Office
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STRUCTURAL MANUAL ENQUIRIES

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RECORD OF REVISIONS

RECORD OF REVISIONS

This Structural Manual dated January 2024 includes Revision #60 and supersedes the previous edition. Numerous changes were made from the previous edition by re-ordering and renumbering of the clauses to group similar concepts together, along with other minor clarification corrections. Major changes are summarised as follows:

DIVISION 1 - EXCEPTIONS TO THE CHBDC

Section 4:

- CHBDC reference clarified to use NBCC 2020 seismic hazard values. Slight adjustment to calculation of Seismic Performance Category (SPC).

Section 8:

- Concrete crack width (allowable reinforcing steel stress) equation modified.

Section 10:

- Allowance for crash tested barriers for use on sidewalks.

Low Volume:

- Minor change in TL1 barrier warrants.

DIVISION 2 - PROCEDURES

Section 2:

- Description of 500 MPa reinforcing steel.
- Changes to standard drawing format.
- Addition of section on structure naming and clarification of site numbers.

Section 3:

- Renaming section as “Deep Foundations” and having separate sections for driven and drilled foundations.

Section 4:

- Renaming of section as “Shallow Foundations”.

Section 5:

- Moved approach slabs from section 16 and allowing approach slabs >6 m length to improve drainage.
- Moved slopes and slope protection segment from section 16.
- Updated section about Wingwall Design Aid to describe changes to DA-5-1 through 6.

Section 8:

- Added information about use and detailing of atmospheric corrosion resistant (ACR) steel.
- Permit additional types and grades of steel allowed in the CHBDC to be used. Steels which are not ACR shall be coated.

RECORD OF REVISIONS

- Addition of 1" bolts for bridges with spans greater than 50m.
- Clarified protection of steel clauses.
- Updated details and figures.

Section 9:

- Added section on partial depth precast deck panels.
- Added details for curb forming (still to come).

Section 10:

- Added Section on inspector Guards.
- Moved Noise walls/barriers section from Section 16 to Section 10.

Section 12:

- Added descriptions for 500 MPa reinforcing steel.
- Added information on reinforcement couplers.
- Moved segment on dowels from section 16 to Section 12.

Section 15:

- The entire section has been revised to include up to date information on sawn timber, prefabricated mass timber and treatment with preservatives.

Section 16:

- Moved approach slabs and slopes and slope protection to section 5.
- Moved dowels in concrete to section 12.
- Removed most of the pedestrian bridge requirements and referred to CSA S7.
- Moved segment on Noise walls / barriers to section 10.

DIVISION 3 - DESIGN AIDS

- Revised DA5-1 through DA5-6 on cantilever wingwall design.
- ~~Added design aid DA 10-1 & DA 10-2 containing dead load of permanent and temporary barriers respectively.~~
- ~~Revised Deleted the design aid DA12-1.1 on reinforcement bends to update for 500W reinforcing steel.~~
-

DIVISION 4 - STRUCTURAL STANDARD DRAWINGS

Division 4 has been removed. All the Structural Standard Drawings have been included in the Structural Standard Drawings Manual which is available on MTO's Technical Publications Website. They are available either individually, or collectively under the title "SSD Complete Manual". Changes and updates of Structural Standard drawings information about new and archived drawings are listed in "SSD Revision Sheet All-Rev" and available on MTO's Technical Publication Website.

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STRUCTURAL MANUAL

PREFACE

PREFACE

The Structural Manual was first developed by the Structural Office in the 1970's to provide a convenient reference for procedures, policies, design provisions, design aids, and Structural Standard Drawings that were to be used in the preparation of Ministry structural contract documents. Subsequently the manual has been continuously updated to reflect changes due to metrication, new bridge codes and current practices. Since its inception the Structural Manual has been revised to be compatible with the AASHTO code, OHBDC, and now with this edition the CHBDC. To accommodate the growth in bridge engineering knowledge the Structural Manual will continue to be revised by the Structures Office when appropriate.

The current manual is divided into the following three divisions:

Division 1 - Exceptions to the Canadian Highway Bridge Design Code

Division 2 - Procedures

Division 3 - Design Aids

A Table of Contents or a List precedes each division as appropriate.

Although the Ministry of Transportation (MTO) has developed this manual as a standard for their use, other bridge owners may use it as a resource document from which they can develop their own standards and policies. However, any variation should not be a substantial departure from the Structural Manual or cause an adverse effect on the safety and movement of people and goods. Any deviation from this manual should be based on operational experience and objective analysis. The MTO does not accept responsibility in any connection with the variations. For MTO structures, any deviation from the requirements of this manual shall be approved by the MTO Structural Section (of the Design and Engineering Branch) in consultation with Structures Office. In MTO design-build projects, all uses of the word 'should' shall be interpreted as mandatory requirements unless a higher standard is applied and approved by MTO Structural Section.

Suggestions for the improvement of this manual, or for the addition of new material, should be forwarded to:

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NOTATIONS

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AADT	Average Annual Daily Traffic
AASHTO	American Association of State Highways and Transportation Officials
ACR	Atmospheric Corrosion Resistant
AREMA	American Railway Engineering & Maintenance of Way Association
ASTM	American Society for Testing and Materials
AWPA	American Wood Preservers Association
CDED	Contract Design, Estimating and Documentation
CHBDC	Canadian Highway Bridge Design Code, CSA S6
CNWA	Canadian Navigable Waters Act
CPCI	Canadian Prestressed Concrete Institute
CPS	Contract Preparation System
CSA	Canadian Standards Association International
CSP	Corrugated Steel Pipe
DA	Design Aids
DD	Design Details
DOT	Department of Transportation
DSM	Designated Sources of Materials
ERT	Effective Rubber Thickness
FHWA	Federal Highway Administration
FIDR	Foundation Investigation and Design Report
FLS	Fatigue Limit State
GFRP	Glass Fibre Reinforced Polymer
HPC	High Performance Concrete
HSS	Hollow Structural Section
HWL	High Water Level
kN	kilo Newton
m	metre
MASH	Manual for Assessing Safety Hardware
mm	millimetre
MPa	Mega Pascals
MSE	Mechanically Stabilized Earth system (also known as RSS, see Section 5.7.1)
MTOD	Ministry of Transportation of Ontario Drawing

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NOTATIONS

MUP	Multi-Use Path
NCHRP	National Cooperative Highway Research Program
NSSP	Non-Standard Special Provision
NTS	Not to Scale
OHBDC	Ontario Highway Bridge Design Code
OPSD	Ontario Provincial Standard Drawing
OPSS	Ontario Provincial Standard Specifications
PI	Point of Intersection
PTFE	Polytetrafluoroethylene Polymer
RSS	Retained Soil System (also known as MSE, see Section 5.7.1)
SC	Spiral to Curve
SLS	Serviceability Limit States
SPCSP	Structural Plate Corrugated Steel Pipe
SS	Structural Standard
SSD	Structural Standard Drawing
SSP	Standard Special Provision
TAC	Transportation Association of Canada
TC	Tangent to Curve
TL	Test Level
TMB	Temporary Modular Bridge
TTI	Texas Transportation Institute
ULS	Ultimate Limit States
WP	Work Project/Working Point
WWR	Welded Wire Reinforcement

DIVISION 1 - EXCEPTIONS TO THE CANADIAN HIGHWAY BRIDGE DESIGN CODE

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**SECTION 1 - EXCEPTIONS TO THE CANADIAN HIGHWAY BRIDGE
DESIGN CODE (CHBDC), CSA S6-19**

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**SECTION 1 - EXCEPTIONS TO THE CANADIAN HIGHWAY BRIDGE
DESIGN CODE (CHBDC), CSA S6-19**

1 EXCEPTIONS TO THE CHBDC, CSA S6-19**1.1 Scope**

These exceptions implement provisions that CSA S6-19, Canadian Highway Bridge Design Code (CHBDC) and Commentary delegates to the Regulatory Authority for highway structures in Ontario. They also set forth design criteria that are exceptions to those included in the CHBDC. These exceptions may be in the form of deletions from, additions to, or modifications of the CHBDC. All owners must always use these two documents jointly to prepare contract plans and specifications for structural elements and/or systems on Ontario highways. Such elements and/or systems include, but are not limited to, bridges, overhead sign structures, earth retaining structures, buried structures and miscellaneous roadway appurtenances.

Exceptions for low volume roads are provided in APPENDIX A - GUIDELINES FOR THE DESIGN OF BRIDGES ON LOW VOLUME ROADS. In the event of any inconsistency or conflict in its contents and the exceptions given in Section 1.4 below, the appendix will take precedence and govern.

1.2 Authority

Ontario Regulation 104/97, and its amendments, made under the *Public Transportation and Highway Improvement Act (PTHIA)*.

1.3 Implementation

Immediately for all designs according to CHBDC.

1.4 Exceptions

In the following changes, the relevant CHBDC clause numbers are given next to each provision and the latest version of any standard referred to shall apply.

CHBDC SECTION 1 GENERAL**1.3 Definitions****1.3.2 General Administrative Definitions**

The definition for Regulatory Authority is amended by the addition of the following at the end of the definition:

The nominee for the Minister of Transportation shall be the Chief Engineer of the Ministry of Transportation of Ontario.

1.3.3 General Technical Definitions

The definition for Deck Width is deleted and replaced with the following:

**SECTION 1 - EXCEPTIONS TO THE CANADIAN HIGHWAY BRIDGE
DESIGN CODE (CHBDC), CSA S6-19**

Deck width — the horizontal distance, measured at deck level perpendicular to the direction of travel, from face to face of sidewalks or curbs (or barrier walls if there are no curbs), but not taken as less than the Bridge Width minus 1 m unless approved by the Owner.

1.4 General Requirements**1.4.2 Design****1.4.2.5 Single-Load-Path Structures**

This clause is amended by the addition of the following:

For bridges, a single load path structure shall not be used unless approved by the Owner.

1.4.4 Construction**1.4.4.5 Plans**

The third paragraph is deleted and replaced with the following:

In order to validate and certify the design shown on the plans, the engineer and checker shall each affix his or her Professional Engineers Ontario seal to each and every drawing, and each shall sign and date the appropriate seal.

The following shall be added to the provisions of this clause:

Specifications for construction and rehabilitation shall be in accordance with the *Ontario Provincial Standards for Roads and Public Works*, or another standard approved by the Owner. In the event of any inconsistency or conflict in the contents of these standards and the CHBDC, the *Ontario Provincial Standards for Roads and Public Works*, or other standard approved by the Owner, will take precedence and govern.

1.5 Geometry

In this subsection all references to "...the Regulatory Authority, or in their absence, with the TAC Geometric Design Guide for Canadian Roads." are deleted and replaced by "...Transportation Association Canada (TAC) Geometric Design Guide for Canadian Roads in conjunction with the MTO Design Supplement or other standard approved by the Owner."

1.6 Barriers

In this subsection all references to "...the Regulatory Authority, or in their absence, with the TAC *Geometric Design Guide for Canadian Roads*." are deleted and replaced by "...the *MTO Roadside Design Manual* or other standard approved by the Owner."

**SECTION 1 - EXCEPTIONS TO THE CANADIAN HIGHWAY BRIDGE
DESIGN CODE (CHBDC), CSA S6-19**

1.9 Hydraulic Design**1.9.10.1 Stream stabilization works**

The last paragraph is deleted and replaced with the following:

Stabilization works shall be designed to suit the requirements of the site and comply with the requirements of the regulatory authority or with the Transportation Association of Canada's Guide to Bridge Hydraulics.

1.9.1 Design Criteria**1.9.1.2 Normal Design Flood**

The first paragraph is deleted and replaced with the following:

The normal design flood in Ontario shall have a return period in accordance with the MTO Highway Drainage Design Standard WC-1, Design Flows (Bridges and Culverts).

1.9.1.3 Check Flood

The first paragraph is deleted and replaced with the following:

The check flood in Ontario shall have a return period in accordance with the MTO Highway Drainage Design Standard WC-1, Design Flows (Bridges and Culverts).

CHBDC SECTION 3 LOADS**3.8 Live Loads****3.8.3 Traffic Loads****3.8.3.1 Normal Traffic****3.8.3.1.1 CL-W loading**

The last three paragraphs are deleted and replaced with the following:

A loading exceeding CL-625 may be specified by the Owner.

3.8.4 Application**3.8.4.3 Local Components**

Item (b) is deleted and replaced by the following:

For modular expansion joints, the axle load considered shall be axle no. 4 of the CL-625-ONT truck for ultimate and serviceability limit states, and axles 2 and 3 of the CL-625-ONT truck for the fatigue limit state.

**SECTION 1 - EXCEPTIONS TO THE CANADIAN HIGHWAY BRIDGE
DESIGN CODE (CHBDC), CSA S6-19**

For structural components other than joint armouring, the horizontal load shall be 20% of the vertical load applied at the roadway surface and considered individually or in combination with vertical loads, whichever produces a greater load effect.

For the FLS, the truck shall be positioned anywhere along the length of the expansion joint but need not be placed closer than 600 mm to a barrier wall.

The maximum (positive) stress at a given location in the joint shall be calculated from the truck positioned at any location along the length of the expansion joint. The minimum (negative) stress at that same location shall be calculated from the truck positioned at any other location along the length of the expansion joint. The calculated fatigue stress range at each location along the joint shall be the algebraic difference between the maximum and minimum stress above.

The required length of the support bar shall be determined based on an installation temperature of 15°C. Force effects shall be calculated based on factored movements in relation to the joint's articulation relative to its positions at 15°C.

CHBDC SECTION 4 SEISMIC DESIGN
4.4.3 Seismic Hazards
4.4.3.1 General

The first paragraph is amended with the addition of the following statement at the end.

The 2020 National Building Code of Canada Seismic Hazard Tool shall be used.

4.4.4 Seismic Performance Category
~~**4.4.3.1 General**~~

Table 4.10 shall be deleted and replaced with the following:

Table 4.10
Seismic performance category based on 5% damped 2% in 50-year
exceedance spectral values
(See Clauses 4.2, 4.4.4, 4.10.3, and 6.14.2.3)

Spectral accelerations	All Bridges (Lifeline, Major route and other bridges).
$S_a(1.0) < 0.05$	0
$0.05 \leq S_a(1.0) < 0.10$	1
$0.10 \leq S_a(1.0) \leq 0.30$	2
$S_a(1.0) \geq 0.30$	3

Note: For lifeline bridges in seismic performance category 1, detailing of structural elements shall adopt requirements for seismic performance category 2 as a minimum.

**SECTION 1 - EXCEPTIONS TO THE CANADIAN HIGHWAY BRIDGE
DESIGN CODE (CHBDC), CSA S6-19**

4.11 Seismic evaluation of existing bridges**4.11.1 General**

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

The evaluation shall assess the bridge for seismic performance objectives, as defined by the owner, using one or more of the seismic hazard levels considered in Table 4.15, or as defined by the owner in accordance with Clause 4.11.3. The evaluation and performance criteria, including the selection of performance objectives and seismic hazard levels, may adopt a risk-based approach that considers the remaining functional/intended life of the bridge, the economic and societal consequences of damage to or failure of the bridge, its age and condition, deficient details or arrangements, and any condition, strength or functional upgrades that are envisaged.

4.11.3 Seismic hazard and evaluation

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

The evaluation shall assess the bridge for acceptable performance objectives targeted for one or more seismic hazard levels considered in Table 4.15. For bridges in SPC 2 and 3, the evaluation and seismic rehabilitation shall seek to prevent collapse of all or part of the bridge at a hazard level not less than a 5% in 50-year probability of exceedance, unless a higher probability of exceedance is specified by the owner.

CHBDC SECTION 5 METHODS OF ANALYSIS**5.5 Requirements for Specific Bridge Types****5.5.3 Voided slab**

The contents of Clause (b) are deleted and replaced with the following:

- b) for rectangular voids, the thickness of the web defined by adjacent voids shall be not less than 20% of the total depth of the section. The depth of the void shall not exceed 80% of the total depth of the section and the transverse width of the void shall not exceed 1.5 times its depth, although a ratio of up to 2.5 times the depth is permitted with approval of the Owner.

CHBDC SECTION 6 FOUNDATIONS AND GEOTECHNICAL SYSTEMS**6.9 Geotechnical Resistance**

Table 6.2 is amended with the addition of the following note:

The value of geotechnical resistance factor for Deep foundations and embankment fills may be increased by up to 0.05, with approval of the Owner. The value of geotechnical resistance factor for settlement of embankment fills may be increased to 1.0 with approval of the Owner.

**SECTION 1 - EXCEPTIONS TO THE CANADIAN HIGHWAY BRIDGE
DESIGN CODE (CHBDC), CSA S6-19**

6.11 Deep Foundations**6.11.4 System Design and Construction Considerations****6.11.4.5 Degradation of Pile Foundations**

This clause is amended by the addition of the following:

Exposed steel H and steel tube piles shall have an Approved protective coating applied from an elevation 600 mm below the low water level or 1000 mm below finished ground surface up to at least 1000mm above high-water level.

6.11.4.9 Splices

The last sentence is deleted and replaced with the following:

Wood piles shall not be spliced.

CHBDC SECTION 7 BURIED STRUCTURES**7.1 Scope**

This subsection is amended by the addition of the following:

The provisions of this Section are mandatory only for structures that are greater than 3 m in span.

Specifications for construction, rehabilitation, and design provisions, where applicable, shall be in accordance with the *Ontario Provincial Standards for Roads and Public Works*, or another standard approved by the Owner.

7.8 Reinforced Concrete Pipe, Boxes, and Three-Sided Buried Structures**7.8.9 Strength Design****7.8.9.2 Design for Shear****7.8.9.2.2 Box Structures without Stirrups or Ties**

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

The shear strength shall be determined in accordance with Section 8, unless approved by the Owner.

CHBDC SECTION 8 CONCRETE STRUCTURES**8.4 Materials****8.4.2 Reinforcing Bars and Deformed Wire**

**SECTION 1 - EXCEPTIONS TO THE CANADIAN HIGHWAY BRIDGE
DESIGN CODE (CHBDC), CSA S6-19**

8.4.2.1 Reinforcing Bars**8.4.2.1.1 Specification**

This clause is amended by the addition of the following:

Other grades of reinforcing bar not covered by CSA G30.18 are only permitted when approved for use by the Ministry of Transportation Ontario (MTO).

8.5 Limit States**8.5.3 Fatigue Limit State****8.5.3.1 Reinforcing Bars**

The clause is modified with the addition of the following paragraph, immediately following the bulleted list.

Unless approved by the Owner, tack welding of reinforcing bars shall not be permitted. If approved, tack welding shall follow the requirements of CSA W186.

8.8 Flexure and Axial Loads**8.8.4 Flexural Components****8.8.4.6 Prestressed Concrete Stress Limitations**

The contents of Clause (a)(ii) are deleted and replaced with the following:

- ii) For all prestressed concrete elements, the limiting concrete tensile stress at transfer shall be $0.6f_{cr}$.

The contents of Clause (b)(iii) are deleted and replaced with the following:

- iii) The maximum concrete tensile stresses in service shall not exceed $0.75f_{cr}$. Tension shall not be permitted across the joints of segmental components unless bonded reinforcing bars pass through the joints in the tensile zone.

8.11 Durability**8.11.2 Protective Measures****8.11.2.2 Concrete Quality**

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

The durability properties for structural concrete shall be as specified in the *Ontario Provincial Standards for Roads and Public Works* or other standard, approved by the Owner, for the appropriate combination of deterioration mechanisms and environmental conditions.

**SECTION 1 - EXCEPTIONS TO THE CANADIAN HIGHWAY BRIDGE
DESIGN CODE (CHBDC), CSA S6-19**

8.11.2.4 Concrete Cover

The title of this clause is deleted and replaced with the following:

8.11.2.4 Concrete Cover and Tolerances

The second paragraph is deleted and replaced with the following:

Tolerances for concrete cover shall be shown along with concrete covers on the plans or in the specifications. Unless approved by the Owner, the tolerances shall be as follows:

- a) Reinforcement in Cast in Place Concrete: ± 10 mm against formed surfaces, ± 20 mm on finished surfaces and ± 25 mm against earth.
- b) Reinforcement in Precast Concrete: ± 10 mm and $+10/5$ for precast, prestressed girders.
- c) Prestressing steel in Precast Concrete: ± 5 mm.
- d) Post-Tensioning Duct in Cast in Place Concrete: ± 10 mm against formed surfaces, and ± 15 mm on finished surfaces.
- e) Posttensioning Duct in Precast Concrete: ± 10 mm.

Where approved by the Owner, the cover and tolerance for Stainless Steel bars shall be the values for reinforcing steel found in CHBDC Table 8.5, reduced by 10 mm, however, the minimum clear cover after consideration of tolerance shall not be less than 30 mm.

8.12 Control of Cracking**8.12.3 Crack Control Reinforcement in Zones of Computed Tensile Stress****8.12.3.2 Concrete Surface Categories**

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

Concrete surfaces shall be classified as Category A, AA, or Category B in accordance with the following criteria:

- a) Category A surfaces are surfaces for which a high standard of appearance is required and/or surfaces that will be exposed to one or more of the following agents: de-icing chemicals, spray or surface runoff containing de-icing chemicals, marine spray, swamp or marsh water, salt water, or aggressive backfill.
- b) Category AA surfaces are Category A deck surfaces of slab on girder bridges built with un-shored construction that are predominantly subject to live loads.
- c) Category B surfaces are all surfaces that are not classified as Category A or AA.

Waterproofing membranes shall be disregarded in the classification of concrete surfaces.

**SECTION 1 - EXCEPTIONS TO THE CANADIAN HIGHWAY BRIDGE
DESIGN CODE (CHBDC), CSA S6-19**

8.12.3.4 Tensile Stress Limits for Reinforcing Steel

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

Tensile stress, f_s , in reinforcing steel produced by the governing cases defined in Clause 8.12.3.3 shall not exceed either the following limits:

$$f_s \leq \sqrt{\frac{W_{cat} f_{cr} E_s}{\alpha_b d_b} \left(1 + \frac{A_b E_s}{2 c_b s E_c}\right)}, \text{ and } f_s \leq 0.75 f_y$$

Where:

W_{cat} is the category factor, taken as for 2.8 for Category A exposure, 1.6 for Category AA exposure, 3.9 for Category B exposure.

α_b is the bar coating factor, taken to be equal to 1.0 for uncoated bars and 1.2 for epoxy-coated bars.

d_b is the bar diameter, taken as the diameter of bar corresponding to the average area of steel on the tensile face, in mm².

A_b is the average area of steel bar in the layer nearest the tensile face, in mm².

c_b is the clear cover from the bar controlling the crack to the tensile face, but not greater than 70 for more than 2 layers of reinforcement, in mm.

s is the average spacing of reinforcement nearest the tensile face, in mm.

and the steel/concrete ratio term $\left(1 + \frac{A_s E_s}{2 c_b s E_c}\right)$ may be taken as unity in lieu of calculation.

CHBDC SECTION 10 - STEEL STRUCTURES**A10.1 Construction Requirements for Structural Steel****A10.1.1 General**

Clause A10.1.1.2 is deleted.

CHBDC SECTION 11 JOINTS AND BEARINGS**11.6 Bridge bearings****11.6.3 Sliding surfaces****11.6.3.3 Mating surface**

This clause is amended as follows:

The words "stainless steel or anodized aluminum alloy" shall be replaced by "stainless

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steel, anodized aluminum or other material approved by the owner”.

CHBDC SECTION 12 – BARRIERS AND HIGHWAY ACCESSORY SUPPORTS

12.4.3.4 Crash test requirements

12.4.3.4.1 General

This clause is amended with the addition of the following:

The above requirements may be waived where otherwise specified by the MTO standards.

CHBDC SECTION 14 EVALUATION

14.7 Material Strengths

14.7.4 Strengths Based on Date of Construction

14.7.4.4 Reinforcing Steel

The contents of Table 14.2 in this clause are deleted and replaced with the following:

Table 14.2
Minimum yield strengths of reinforcing steel, MPa
(See Clause 14.7.4.4)

Date of Bridge Construction	Structural Grade	Medium or Intermediate Grade	Hard Grade	Unknown Grade
Before 1914	-	-	-	210
1914 - 1955	230	275	345	230
1956 - 1972	275	275	345	275
1973 - 1978	300	345	415	300
After 1978 - stirrups and ties	300	350	400	300
After 1978 - remainder	300	350	400	350

14.12 Target Reliability Index

14.12.1 General

This clause is amended by the addition of the following:

If the bridge is to be re-evaluated within 5 years for Normal Traffic, the Reliability Index, β , specified in Table 14.5, shall be reduced by 0.25. This value shall not be less than 2.5.

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14.14 Resistance**14.14.2 Resistance Adjustment Factor**

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

For all components, which have no visible sign of defect or deterioration, the factored resistance, as calculated in accordance with Clause 14.14.1, shall be multiplied by a resistance adjustment factor, U . The value of U shall be as specified in Table 14.10 but shall not be taken as less than $U = 1.0$. Where no value for U is specified in Table 14.10, and in lieu of better information, a value of $U = 1.0$ may be used.

14.17 Bridge Posting**14.17.2 Calculation of Posting Loads****14.17.2.2 Based on Axle Weight**

This clause is deleted.

14.17.3.4 Posting Sign for Axle Weight

This clause is deleted.

CHBDC SECTION 16 FIBRE-REINFORCED STRUCTURES**16.3 Abbreviations and Symbols****16.3.2 Symbols**

The following definition of ψ_s is added to the clause:

ψ_s = curvature at a section when the moment is M_s , mm⁻¹ (see Clause 16.8.2.1).

16.4 Durability**16.4.5 Cover to Reinforcement**

The first sentence of this clause is deleted and replaced with the following:

The cover and tolerance for FRP bars and grids shall be the same as the values for reinforcing steel found in Table 8.5, reduced by 10 mm, however, the minimum clear cover after consideration of tolerance shall not be less than 30 mm.

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16.5 Fibre-Reinforced Polymers**16.5.7 Minimum Bend Radius to Bar Diameter Ratio of Bent FRP Bars**

The clause is deleted and replaced with the following:

The bend radius of an FRP bar, r , shall not be less than $4d_b$ unless approved by the Owner.

16.8 Concrete Beams, Slabs, and Columns**16.8.2.1 Design for deformability**

The equation in this clause is replaced with the following:

$$J = \frac{M_{ult} \Psi_{ult}}{M_s \Psi_s}$$

16.8.4 Development Length for FRP Bars and Tendons**16.8.4.1 General**

The clause is amended with the addition of the following at the end of the clause:

c) When the transverse reinforcement is steel, in the expression for shall be taken as 1.0.

16.8.4.3 Development length of FRP bent bar

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

The development length, λ_d , for FRP bent bars in tension shall be calculated using the equation given in Clause 16.8.4.1.

However, the development length, λ_d , may be reduced by accounting for the tensile stress taken by the bent portion of the bar, σ_v , for bars that are bent 90 degrees or more from straight. The development length may be reduced by an amount equal to, $l_d \left(\frac{\sigma_v}{f_{FRPbend}} \right)$, but λ_d shall not be less than $12d_b$ or 230 mm.

Where:

$$\sigma_v = \frac{(0.05r/d + 0.3)f_{FRPbend}}{1.5}$$

The tail length of a bent bar, λ_t , should not be less than $12d_b$.

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16.8.7 Design for Shear and Torsion**16.8.7.3 Interface Shear Transfer**

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

16.8.7.3.1 General

The interface shear resistance shall be designed so as to be provided by only one type of shear-friction reinforcement material, either FRP or steel; the effects of FRP and steel shear-friction reinforcement providing resistance simultaneously shall not be combined. Where the shear-friction reinforcement is composed entirely of steel, the interface shall be designed as per Clause 8.9.5.

Otherwise, a crack shall be assumed to occur along the shear plane and the relative displacement shall be considered to be resisted by cohesion and friction maintained by the shear-friction reinforcement crossing the crack. In lieu of more detailed calculations, the factored shear resistance of the plane, v_r , shall be calculated as follows:

$$v_r = \phi_c(c + \sigma)$$

Where v_r shall not exceed the lesser of:

- a) $0.25\phi_c f'_c$, or
- b) 6.5 Mpa.

16.8.7.3.2 Values of c, μ , and λ_1

The title of this clause is deleted and replaced with the following:

16.8.7.3.2 Values of Compressive Normal Stress

The value of the compressive stress across the shear-friction plane shall be calculated as follows:

$$\sigma = \alpha_v \rho_v f_d + \frac{\mu N}{A_{cv}}$$

Where α_v is the dowel term reduction factor, ρ_v is the shear reinforcement ratio, f_d is the dowel shear resistance contribution (MPa), μ is the coefficient of friction, N is the unfactored permanent load perpendicular to the shear plane (positive for compression and negative for tension) (N), and A_{cv} is the area of concrete resisting shear transfer (mm^2).

$$\rho_v = \frac{A_{vf}}{A_{cv}}$$

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Where A_{vf} is the area of shear-friction dowel reinforcement (mm^2). A minimum amount of reinforcement must be provided such that $\rho_v \geq 0.44\%$.

16.8.7.3.3 Values of σ

The title of this clause is deleted and replaced with the following:

16.8.7.3.3 Values of c , α_v , f_d , and μ

The following values shall be taken for c , α_v , f_d , and μ in Clauses 16.8.7.3.1 and 16.8.7.3.2:

- a) For concrete placed against hardened concrete, with the surface clean and free of laitance but not intentionally roughened:

$$c = 0.25 \text{ MPa}$$

$$\mu = 0.60\lambda_1$$

$$\alpha_v = 0.80$$

$$f_d = \text{Transverse shear strength as per CSA S807-19, which may be taken as 180 MPa for straight or bent Grade III GFRP bars.}$$

- b) For concrete placed against hardened concrete, with the surface clean and free of laitance and intentionally roughened to a full amplitude of about 5 mm and spacing of about 15 mm:

$$c = 0.50 \text{ MPa}$$

$$\mu = 1.00\lambda_1$$

$$\alpha_v = \mu$$

$$f_d = 0.0025E_{FRP}$$

Where E_{FRP} is the longitudinal modulus of elasticity as per CSA S807-19 and may be taken as 60 GPa for straight Grade III GFRP bars and 50 GPa for bent Grade III GFRP bars.

- c) For concrete placed monolithically:

$$c = 1.00 \text{ MPa}$$

$$\mu = 1.40\lambda_1$$

$$\alpha_v = \mu$$

and where f_d is as per (b) above.

Where the values of λ_1 shall be as follows:

- Normal density concrete: 1.00
- Semi-low density concrete: 0.85
- Low-density concrete: 0.75

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16.8.7.3.4 Anchorage of Shear-Friction Reinforcement

The title of this clause is deleted and replaced with the following:

16.8.7.3.4 Orientation and Anchorage of Shear-Friction Reinforcement

FRP reinforcement for shear-friction shall be oriented perpendicular to the shear plane.

FRP reinforcement for shear-friction shall be capable of developing a strength of $2f_d$ on both sides of the shear-friction plane.

16.8.9 Compression Components**16.8.9.2 Maximum factored axial resistance**

Clause 16.8.9.2 is deleted and replaced with the following:

The maximum factored axial load resistance, P_r , of compression members shall be calculated in accordance with Clauses 8.8.5.6 and 16.8.9.1, where A_s shall be taken as zero where longitudinal reinforcement is not steel.

16.8.9.4 Transverse Reinforcement**16.8.9.4.3 FRP ties**

The requirements for items (b) and (d) of the list in this clause are deleted and replaced with the following:

- b) 24 times the minimum cross-sectional dimension (or diameter) of the GFRP tie or grid;
- d) 300 mm in compression members.

16.8.11 Strut-and-Tie Model for Deep Beams, Corbels, and Short Walls**16.8.11.1 General**

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

Strut-and-tie models shall be used only where approved by the Owner.

When approved, strut-and-tie models may be used to determine internal force effects near supports and the points of application of concentrated loads, or for the design of deep footings and pile caps or other situations in which the distance between the centres of applied load and the supporting reaction is less than twice the component thickness.

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2 APPENDIX A - GUIDELINES FOR THE DESIGN OF BRIDGES ON LOW VOLUME ROADS**2.1 Scope**

These design guidelines apply for bridges on roads with an average annual daily traffic (AADT), in both directions, of 400 or less. These guidelines may also be used, with Approval, for existing roadways where operational issues such as collisions, traffic delays, etc. are minimal.

2.2 Introduction

Present design codes, standards and policies have typically been developed for bridges with high traffic volumes. It has become apparent, however, that for bridges with low traffic volumes these requirements have become too stringent. In order to achieve economies by the relaxing of requirements, and without compromising safety, a task force with MTO Regional and Provincial Office participation was set up to develop guidelines for this purpose. As a result, this section has been formulated. Where applicable, the CHBDC clause number that is being modified is provided, along with the reference to justify the provision. A commentary is also provided.

Application of these recommendations to bridges on low volume roads will provide an opportunity for savings on structures in these situations. Examples of changes in design criteria that will be effective in achieving this aim are:

- Reduction in minimum soffit clearance over waterways it was considered that the temporary consequences of possible flooding, including disruption to traffic, were tolerable.
- Reduction in minimum lane and shoulder width with the lower traffic volumes it is believed that the probability of vehicles encountering or stopping on bridges was not a high risk.

2.3 Definitions

Recreational Road means a road used for the access of parks, scenic and historic sites, or seasonal cottages.

Resource Access Road means a road used for mining, forestry, and energy development.

Local Road means a road primarily used for land access.

Collector Road means a road providing for land access and traffic circulation.

Arterial Road means a road primarily used for high volumes of through traffic.

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AADT means Average annual daily traffic.

TL-0 (formerly LVPL1) means a railing able to successfully redirect a ¾ ton pick-up truck with a speed of 25 km/h and an impact angle of 15°. The level of performance is less than that for TL-1 and is intended for very low volume and low speed traffic.

TL-1 means a Test Level 1 (TL-1) as described in CHBDC and satisfies NCHRP Report 350, successfully redirecting a ¾ ton pick-up truck with a speed of 50 km/h and an impact angle of 25°. The level of performance is intended for low volume traffic at moderate speeds.

CHBDC means CSA-S6-19, Canadian Highway Bridge Design Code.

2.4 Provisions, Guidelines, and Commentaries

The CHBDC should be used for all bridges with the following exemptions allowed for bridges on low volume roads. The relevant CHBDC clause numbers are given next to each provision.

CHBDC SECTION 1 - GENERAL

1.4 General Requirements

1.4.2 Design

1.4.2.3 Design Life

- a) The design life of the bridge shall be 75 years unless reduced at the request of the Owner (Cl. 1.4.2.3).

Commentary: Certain cases may require a lower life. Consideration should be given to a lower life cycle where alignments are substandard, but improvements are cost prohibitive. In those situations, the owner may not be able to correct the alignments but may also not wish to be committed to the substandard alignment for 75 years.

1.4.2.5 Single-Load-Path Structures

- a) Some single load path structures such as trusses and 2 girder bridges are acceptable. Alternate load paths may be desirable due to the lack of maintenance that is common on low volume roads (Cl. 1.4.2.5).

Commentary: This overrides the CHBDC Exceptions and allows their use.

1.5 Geometry

1.5.1 Planning

- a) The 10-year growth need not be considered for AADT unless a significant change

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in the road use is foreseen (Cl. 1.5.1).

Commentary: For these roads, the growth would generally be small.

b) The horizontal and vertical alignment should be determined using roadway design criteria and not the bridge code. Where existing roadway alignments have proven to perform well, then the existing alignment can be used (Cl. 1.5.1).

Commentary: This allows structure replacement at same location if old structure was adequate.

1.5.2 Structure Geometry

a) The use of single lane bridges is acceptable for bridges on some low volume roads. (Cl. 1.5.2) The minimum bridge, lane, and shoulder widths shall be as given in Table 2.4-1 (Ref. 2,3). Adequate warning should be given to traffic in the case of narrow or single-lane bridges.

Commentary: Reduced widths result in lower costs. With AADT less than 200, the probability of 2 vehicles meeting on the bridge is low.

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TABLE 2.4-1 - MINIMUM LANE, SHOULDER, AND BRIDGE WIDTHS

AADT	Design Speed (km/h)	# Lanes	Min. Lane Width (m)	Min. Shoulder Width (m)	Min. Bridge Width (m) ¹
≤ 200	≤ 60	1	3.0	0.5	4.0 ^{2,3,4} (4.9 m max) ⁵
		2 ⁶	2.75	0.25	6.0
	≥ 70	2	3.0	0.5	7.0
> 200 and ≤ 400 ⁷	≤ 40	1	3.0	0.5	4.0 ^{2,3,4} (4.9 m max) ⁵
		2 ⁶	2.75	0.25	6.0
	50 to 60	2	3.0	0.5	7.0
	≥ 70	2	3.3 ⁸	0.6 ⁸	7.5 ⁸

Table 2.4-1 Notes:

- 1 - Width measured between the inside face of the barriers, guiderails, or curbs.
- 2 - Horizontal and vertical sight distances, or adequate turn-outs, shall be provided to allow approaching motorists to observe an opposing vehicle on a single lane bridge or its far approach. If there are sight distance issues, a single lane bridge should not be used.
- 3 - Farm or other special vehicles may require larger widths. Consultation shall be made with local officials. It may also be acceptable to have a lower barrier to accommodate these farm or other special vehicles.
- 4 - For AADT > 400, or for locations where the Seasonal Average Daily Traffic is significantly > 400, consideration should be given to using additional traffic control measures at the bridge for single lane structures (i.e., traffic signals, yield sign for one direction, stop sign, etc.).
- 5 - New single lane bridges wider than 4.9 m should be avoided as they may give the appearance of a full two-lane bridge.
- 6 - Two lane bridge is also an acceptable alternative if a 4.9 m single lane bridge cannot accommodate the traffic (i.e., farm or special vehicles, sight distances, etc.).
- 7 - Range also applies to existing roadways with AADT > 400, where operational issues have been minimal and Approval for use of these guidelines has been obtained from the Owner.
- 8 - For AADT > 400, or for locations where the Seasonal Average Daily Traffic is significantly > 400, lane width of 3.25 m, shoulder width of 1.0 m and minimum bridge width of 8.5 m shall apply.

1.7 Auxiliary Components

1.7.2 Approach Slabs

- a) The approach slabs may be omitted. Approach slabs may be beneficial for high abutments to reduce the effects of live load surcharge (Cl. 1.7.2).

Commentary: These roads are generally lower service roads and settlement would not be a great inconvenience compared to other hazards on the road.

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1.8 Durability and Maintenance

1.8.2 Bridge Deck Drainage

1.8.2.3 Drainage Systems

1.8.2.3.1 General

- a) Deck drains are only required as given by Clause 1.8.2.3.1. Shorter bridges and bridges with catch basins at the ends of the structure often do not require deck drains.

Commentary: Traditionally, bridges generally had more drains than required.

1.9 Hydraulic Design

- a) Hydrologic and Hydraulic design standards shall be as specified in the Highway Drainage Design Standards, January 2008.

1.9.9 Channel Erosion Control

1.9.9.1 Slope Protection

- a) Slope protection, where required, needs only to be extended 150 mm above HWL (Cl. 1.9.9.1).
- b) Scour and erosion protection is required only for susceptible structures. The stone sizes for scour and erosion protection may be taken from Table 2.4-2 instead of RTAC recommendations (Cl. 1.9.9.1) (Ref. 5).

Commentary: Slightly smaller stones are acceptable.

TABLE 2.4-2 – REQUIRED STONE SIZE FOR SCOUR AND EROSION PROTECTION

Velocity (m/s)	< 2.0	< 2.6	< 3.0	< 3.5	< 4.0	< 4.7	< 5.2
Nominal Stone Size* (mm)	100	200	300	400	500	800	1000

* Maximum stone size to be 1½ times the nominal stone size. 80% of stones (by mass) must have diameter of at least 60% of nominal stone size.

1.9.11 Culverts

1.9.11.6 Closed-Invert Culverts

1.9.11.6.5 Concrete Box Structures

- a) For closed culverts in scour resistant soils, a concrete cut-off wall is not required (Cl. 1.9.11.6.5).

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CHBDC SECTION 3 - LOADS

3.4 Limit States Criteria

3.4.4 Serviceability Limit States

- a) The deflection limit may be reduced to $L/360$ and need not be checked for temporary modular bridges (Cl. 3.4.4).

Commentary: A reduced passenger comfort is allowed.

3.5 Load Factors and Load Combinations

- a) The full DLA and load factors should be used (Cl. 3.5 & 3.8.4.5.3).

3.8 Live Loads

3.8.3 Traffic Loads

3.8.3.1 Normal Traffic

3.8.3.1.1 CL-W Loading

- a) The bridge can be designed to a load lower than the CHBDC CL-625 truck (Cl. 3.8.3.1.1) provided that the bridge will be properly posted. The live load surcharge can be proportionately decreased with the decrease in truck load (Cl. 6.12.5). Emergency and maintenance vehicle weights should be considered when determining the appropriate design loading.

Commentary: If properly posted and for specific uses, a lower load limit bridge may be adequate.

3.8.3.1.3 CL-W Lane Load

- a) Lane load need not be considered (Cl. 3.8.3.1.3).

Commentary: Only 1 truck will be present in the lane.

3.8.4 Application

3.8.4.5 Dynamic Load Allowance

3.8.4.5.3 Components Other than Buried Structures

- a) See 3.5 above.

CHBDC SECTION 6 - FOUNDATIONS

6.12 Ground Pressures

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6.12.5 Surcharge

- a) Bridges designed to a lower live load than the CHBDC CL-625 truck may have the 800 mm live load surcharge reduced proportionately (Cl. 6.12.5).

Commentary: 800 mm surcharge load is calibrated for the full design truck.

- b) See 3.8.3.1.1 above.

CHBDC SECTION 8 - CONCRETE STRUCTURES

8.18 Special Provisions for Deck Slabs

8.18.2 General

8.18.2.4 Allowance for Wear

- a) The 10 mm allowance for wear on exposed concrete decks may be omitted (Cl. 8.18.2.4).

CHBDC SECTION 9 - WOOD STRUCTURES

9.11 Sawn Wood

- a) Hardwoods may be used, and strengths and other properties shall be taken from CSA O86-14 (Cl. 9.11) (Ref. 10).

9.17 Durability

9.17.12 Stress-Laminated Timber Decking

- a) Water borne preservatives may be used for stress laminated timber decks with a minimum dimension of less than 50 mm (Cl. 9.17.12).

Commentary: Stressing closes the gap between laminates, minimizing ingress of water.

9.21 Nail-Laminated Wood Decks

9.21.2 Transversely Laminated Wood Decks

9.21.2.2 Assembly

9.21.2.2.1 Nailing

- a) Transverse nail laminated decks shall have a nail length that pass through two laminates and 10 mm into the third laminate (Cl. 9.21.2.2.1).

Commentary: This slightly shorter length permits the use of 3½" nails.

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9.21.3 Longitudinal Nail-Laminated Wood Decks

- a) Longitudinally nail laminated decks can be used and the truck load shall be assumed to be distributed over a width of 1.7 m. (Cl. 9.21.3).

Commentary: Taken from AASHTO and applies to low volume roads where limited traffic means less chance of nails loosening.

CHBDC SECTION 11 - JOINTS AND BEARINGS

Commentary: Joint systems that are used for high volume bridges need not be used for low volume bridges upon Approval by Owner.

CHBDC SECTION 12 - BARRIERS AND HIGHWAY ACCESSORY SUPPORTS

12.4 Barriers

12.4.3 Traffic Barriers

12.4.3.2 Test Level

12.4.3.2.5 Test Level for Barriers on Low Volume Roads

- a) A lower performance level is acceptable for some low volume roads (Cl. 12.4.3.2.5) (Ref. 8).

Commentary: The lower performance barriers can be used on lower speed, low height bridges.

b) A TL-0 and TL-1 railing can be used for bridges meeting the criteria of Table 2.4-3. If the criteria of Table 2.4-3 are not met, a TL-1 barrier may be used where the barrier exposure index, B_e , is less than 20% of the TL-2 limit shown in CHBDC Table 12.5 through 12.7. Otherwise CHBDC TL-2 through TL-5 must be used. (Cl. 12.4.3.2.5).

c) If the bridge has significant pedestrian and bicycle traffic, the height of the barrier shall be increased according to Table 12.8 as long as the vehicle barrier interaction is not adversely affected.

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TABLE 2.4-3 – BARRIER SELECTION CRITERIA

	AADT	Height above water	Design Speed	Bridge Width
TL-0	≤ 100	≤ 2.5 m	≤ 30 km/hr	no limit
			≤ 40 km/hr	≤ 5.0 m
TL-1	≤ 400*	≤ 5.0 m	≤ 50 km/hr	no limit
			≤ 80 km/hr	≤ 5.0 m

* Range can be higher for existing roadways where operational issues have been minimal and Approval for use of these guidelines has been obtained.

12.4.3.4 Crash Test Requirements

12.4.3.4.2 Crash Test Requirements for Traffic Barriers

- a) Railings that have been successfully crash tested to the above performance limits are given in Figure 2.4.1 and Figure 2.4.2. Other railings that satisfy the crash test requirements may be used (Cl. 12.4.3.4.2).

Commentary: These have been used for low volume roads in the United States.

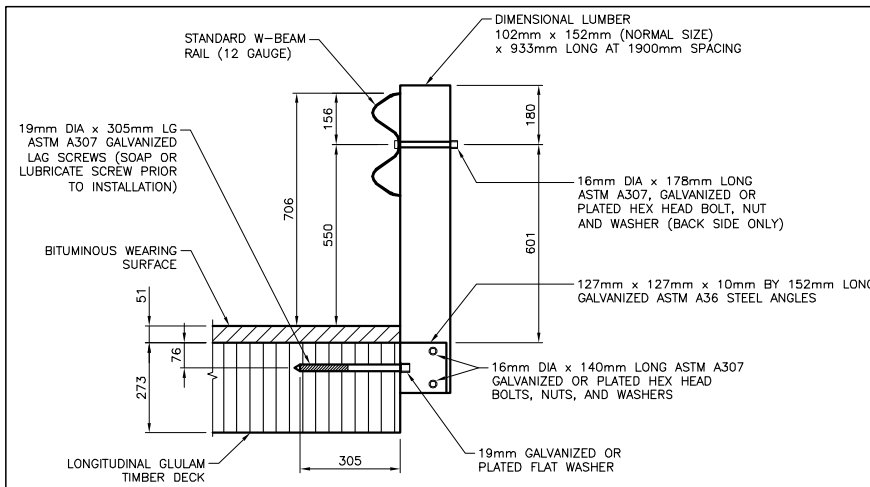


FIGURE 2.4.1 – APPROVED TL-1 BARRIER

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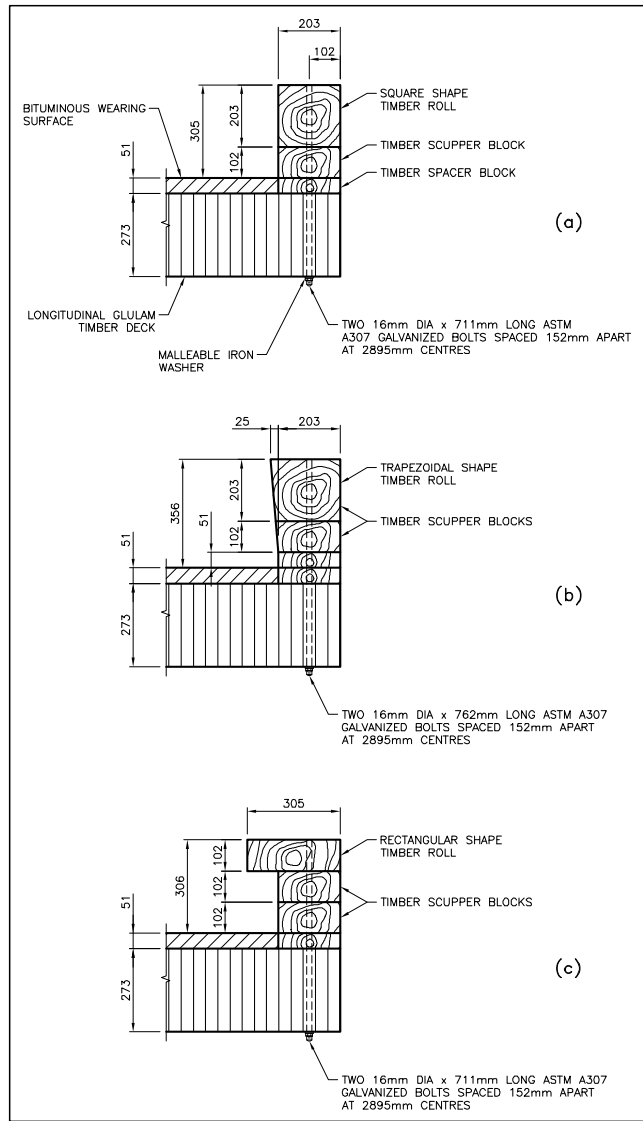


FIGURE 2.4.2 – APPROVED TL-0 BARRIER SYSTEMS

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CHBDC SECTION 15 - REHABILITATION AND REPAIR

15.8 Structural Steel

15.8.1 Member and Connection Repair and Strengthening**15.8.1.1 General**

- a) Reuse of existing bridge materials may be considered at the Owner's discretion. Material condition and physical properties should be determined prior to their use (Cl. 15.8.1.1).

Commentary: Cost savings from used material may be significant.

2.5 References

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SECTION 1 - INTRODUCTION

1 INTRODUCTION**1.1 General**

The design and drafting procedures in this division are to be followed when preparing structural contract documents for the Ministry. The preparation of contract documents estimation of quantities, and the referencing of Ontario Provincial Standard Specifications (OPSS) are not covered in this manual. Technical revisions and additions to the Procedures Division are indicated by the date shown in the left portion of the footer block.

When additions or revisions are necessary, they will be made available through online website, as detailed in Section 1.3.

1.2 Definitions

For the purpose of this manual the following definitions apply:

Average Annual Daily Traffic (AADT) means the total yearly traffic volume on a given road/highway, in all lanes and both directions including over twin bridges of divided highways, divided by the number of days in the year.

Engineer means an engineer who is licensed by Professional Engineers Ontario (PEO).

Ministry means Ministry of Transportation, Ontario.

MTO means Ministry of Transportation, Ontario.

1.3 Distribution of Structural Manual's Copies

Digital copies of the Manual and revisions may be obtained from the [MTO Technical Publications](#) website.

1.4 Numbering System of Structural Manual

Design aids bear the prefix DA and Structural Standard Drawings bear the prefix SS. Procedures have no prefix. All Structural Standard Drawings are assigned with a three-digit number with a drawing number after the prefix (1XX-Y). The first digit (1) is common for all groups. The next two digits (XX) show the Section Number of the procedure's division. Y represents the drawing number within the Section, which contains 1 to 3 digits. All divisions use the same subject-oriented section numbering system. E.g., SS113-3 would represent the Section 13 of the procedure's division, 'Bearing Assemblies and Expansion Joints' - Drawing 3. The same principle applies to design aids.

The Structural Standard Drawing numbers in the system are not sequential. This is to permit the addition of future sheets in the appropriate place without disrupting the numbering system.

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1.5 Structural Standard Drawings**1.5.1 Format of Structural Standards Drawings**

The following two formats are used for Structural Standard (SS) drawings:

- a) Structural Standard drawings for attachment on or insertion into rest of the contract drawings in the manner specified in Section 2.6.13. These drawings do not require the designer to fill in any additional information from what is shown:
- b) Structural Standard drawings requiring project specific information for use as contract drawings. These drawings serve as templates for a standard structure or element e.g., NU prestressed concrete girder, Sign Support Structure, Traffic Barrier, etc. In this category of Structural Standard Drawings, the designer shall complete all the required information specific to the design of the structure or element under consideration.

Some Structural Standard Drawings being issued in category (a) are in 216 x 279 mm (letter size) format. Standards in this category are referred to as "small size" Structural Standard Drawings.

Structural Standard drawings in category (b) are issued in the structural contract drawing format of size 914 x 610 mm. They contain two title blocks that must be completed by the user. Standards in this category are referred to as "full size" Structural Standard Drawings.

Copies of all the full size Structural Standard Drawings are available in Structural Standard Drawings Manual which can be downloaded from the [MTO Technical Publications](#) website.

1.5.2 Use of Structural Standards Drawings

Most Structural Standard Drawings bearing an "SS" prefix require project specific information to be added by the user. The project specific information should be added at the time the standards are included in the contract drawings package.

AutoCAD file names are established by removing one "S" from the standard drawing name prefix and adding the extension "DWG".

The AutoCAD layer named "UPDATE" in the digital file, contains a revision information sheet for the current revision of the drawing.

AutoCAD files of Structural Standard Drawings shall have the drawing and drawing check initial blocks filled in.

1.5.3 Completion of Structural Standards Drawings

Structural Standard drawings shall be reviewed to determine what information, if any, needs to be added to them or to be modified to suit the project under consideration.

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Where information in tables and dimensions are transferred to a Structural Standard Drawing from an MTO manual for its completion, the drawing shall bear the seal, date, and signature of an Engineer. This Engineer accepts full responsibility for the accuracy of the added information only.

Where details and notes shown on a Structural Standard Drawing are not applicable, they shall be deleted, the drawing shall be identified as "Modified", and shall bear the seal, date, and signature of an Engineer. This Engineer accepts full responsibility for the modifications to the drawing only.

Where project-specific information is added, or changes are made on Structural Standard Drawing that affect or alter the original details, the drawing shall be identified as "Modified" and shall bear the seals, dates and signatures of design Engineer and a design-checking Engineer. These Engineers accept full responsibility for the design that results from the changes.

Those drawings without a Stamping Block are intended to be inserted into the contract package and are complementary to another Structural Standard Drawing. They do not require any engineering seal. If they are modified for any reason, they need to be labelled as such and bear the seals, dates and signatures of design Engineer and a design-checking Engineer.

1.5.4 Availability of Structural Standards Drawings

Electronic CAD files containing standard drawings in AutoCAD may be obtained from the CPS.

1.6 Ontario Provincial Standard Drawings (OPSDs)

OPSD's are standards for provincial roads and municipal services which have been developed in consultation with provincial and municipal owners, designers, and contractors. They are suitable for use by all owners and reflect a consensus of opinion about acceptable practices to ensure owners' standard requirements for quality and the contractor's need for efficiency are satisfied.

OPSD's are provincial publications and are not automatically used by the Ministry. Those that the Ministry has implemented may be found in chapter F of the Ministry's Contract Design, Estimating and Documentation (CDED) Manual, and the CPS. Sometimes it happens that the Ministry implements or deletes standards prior to publishing in the CDED Manual. In this case, confirmation of implementation dates prior to CDED manual revisions will be shown on the CPS News File of the CPS Main Menu.

OPSD standards are to be referenced by drawing number in the contract documents and their copies are not to be included with the contract drawings.

OPSD drawings are not to be modified. If a designer, however, determines that an OPSD drawing, as published, does not meet their project's needs, then it may be modified provided that:

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- the modified drawing is marked with the date of modification and the contract number and name of the project to which the drawing applies.
- all references to Ontario Provincial Standards, the logo and the OPSD title, are removed from the original drawing.
- the drawing shall bear the seal, date and signature of an Engineer and a design-checking Engineer who have done the modifications. The copies of modified OPSDs are required to be inserted in the contract drawings package.

1.7 Ministry of Transportation Ontario Drawings (MTODs)

MTODs are developed for use on provincial highway projects. MTODs typically introduce pilot products and/or designs for highway construction not currently covered by OPSDs. MTODs successfully used on ministry contracts will be forwarded to the appropriate Ontario Provincial Standard (OPS) committee for consideration of conversion into OPSDs at a later date.

MTODs are published and implemented in the Ministry's CPS and also available at [MTO Technical Publications](#) website.

Reference to the MTODs selected (by MTOD number only) shall also be included as appropriate in the contract drawings/documents. Copies of MTODs are not to be inserted into contract drawings package unless modified for a specific project.

Details on the use and modification of MTODs shall follow the policy and procedures described in CDED Manual.

1.8 Structures Office Policy Memos

Whenever an immediate change to manuals, codes, procedures, policies, or an innovation is required, the Structures Office issues Policy Memos. These memos enable their content to be implemented on date of issue, thereby avoiding delay due to waiting for revision of the corresponding publication. Where conflict occurs with the content of two documents, the publication/memo with the latest date takes precedence.

Current policy memos may be obtained from the [MTO Technical Publications](#) website.

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SECTION 2 - GENERAL DESIGN AND DRAFTING

2 DESIGN AND GENERAL DRAFTING**2.1 Design**

This section discusses general structural design aspects of structural elements.

2.1.1 Design Specifications

All structures (bridges, culverts, and tunnels with spans equal or greater than 3 m and retaining walls/systems and sign support structures) shall be designed in accordance with the CHBDC and exceptions to CHBDC provided in Division I of this manual. CHBDC clauses identified in this revision refer to the 2019 edition of the CHBDC CSA S6. Structures supporting railway loading shall be designed to criteria provided by the railway authority.

~~• **Live Load Design Forces in Slab-on-Girder Bridges**~~

~~At ULS and SLS, live load effects used for the design of girders shall not be less than the factors determined in accordance with the simplified methods of analysis in Clause 5.6 of the CHBDC, unless approved by the Head of Structural Section. If a bridge does not satisfy the conditions for use of the simplified methods of analysis, the traffic load fraction used for girder design shall not be less than those determined if the bridge had met those criteria.~~

2.1.2 Design and Design-Checking Calculations

The scope of the design check should be sufficient for the engineer to confirm the contract package is complete and accurate for construction, and the structural design is compliant with the requirements of CSA S6 and MTO Exceptions.

For all Ministry bridges designed in-house, the design and the design-checking calculations, shall be filed with the Bridge Design Section, Structures Office or Structural Section as applicable. This shall be done by the Design Engineer immediately after the design and the design-checking are completed and the design drawings are sealed, dated and signed.

For consultant's designs, a copy of calculations of the final new and rehabilitation bridge designs including any changes as a result of design checking shall be submitted to the Ministry project manager, within six (6) weeks of the executive review. For Design Build Projects, the design and design checking Calculations along with all the changes shall be submitted minimum six (6) weeks before the start of construction. The design calculations shall be submitted on letter or legal-size paper. The use of any commercial computer software, its name and version shall be clearly identified, and a hard copy of the input and output data and graphs of computer model and output shall also be included with the design calculations.

The submission shall identify the designer and design checker and shall be dated. The MTO Structural Engineer shall be responsible for adding the submission to the Bridge Management System for future reference.

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2.1.3 Structural Continuity

All bridges shall be designed as continuous over intermediate supports unless approved by the Structural Section. Continuity may be achieved with either a continuous superstructure, or through continuity of the deck slab by means of a flexible link slab.

All bridges with a skew of up to 30 degrees and 100 m or less shall be designed as rigid frames or integral abutment bridges, where foundation conditions allow. For skew up to 20 degrees, steel and concrete girder bridges of up to 150 m shall be designed as integral abutment bridges where foundation conditions allow.

2.1.4 Critical Design Details

During the new and rehabilitation design process of structures, the designer shall identify all critical details and ensure that they can be easily inspected in the future after construction.

2.1.5 Shored Construction for Bridges

For all slab on girder bridges, the girders shall be designed for un-shored construction, unless approved by the Structural Section.

2.1.6 Live Load Design Forces in Slab-on-Girder Bridges

To protect for future staging and rehabilitation and the possible future change in functional use of a bridge, at ULS and SLS, live load effects used for the design of girders shall not be less than the factors determined in accordance with the simplified methods of analysis in Clause 5.6 of the CHBDC, unless approved by the Head of Structural Section. If a bridge does not satisfy the conditions for use of the simplified methods of analysis, the traffic load fraction used for girder design shall not be less than those determined if the bridge had met these criteria.

2.2 Bridge Geometry**2.2.1 Bridge Deck Cross-Sections**

The CHBDC, CHBDC Commentary and the TAC Geometric Design Guide for Canadian Roads (2017) in conjunction with the MTO Design Supplement shall be used to determine deck cross-sections and side clearances.

In deciding the bridge deck cross-section, and determining the dimensions, location, and design of the structure as a whole, the designer shall aim to provide a bridge on which driver reaction and vehicle placement will be essentially the same as elsewhere on the highway considering planned future needs.

2.2.2 Horizontal Clear Zone Width Requirements at Bridge Substructures

This subsection gives direction in setting the ministry's clear zone width requirements to highway bridge abutments and piers. The railway owner shall be contacted for clearance

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envelopes on railway overhead bridges.

For the purpose of this subsection the following definitions apply:

Auxiliary Lane means a lane in addition to, and placed adjacent to, a through lane intended for a specific manoeuvre such as turning, merging, diverging, weaving, and for slow vehicles, but not for parking.

Clear Zone means the unobstructed, traversable area provided beyond the edge of the through travelled way available for use by errant vehicles. The clear zone includes shoulders, bike lanes, and auxiliary lanes, except those auxiliary lanes that function like through lanes. The clear zone also includes recoverable slopes, and non-recoverable slopes with a clear run-out area. The selected clear zone width is dependent upon traffic volumes and design speed, and roadside geometry.

Clear Zone Width means the distance from the edge of the travelled way to the face of an unprotected hazard.

Freeway means a fully controlled access road limited to through traffic, with access through interchanges.

Gore Area means the area between edge of highway, edge of ramp and bullnose.

Travelled Way means that part of a roadway intended for vehicular travel. This includes through lanes, turn lanes, and other auxiliary lanes. This does not include shoulders or ancillary space.

Through Lane means a lane intended for through traffic movement.

Through Travelled Way means that part of a roadway intended for vehicular use, excluding shoulders and auxiliary lanes.

For further information, refer to the Roadside Design Manual.

Minimum horizontal clear zone on Freeways

The minimum horizontal clear zone width from the outside edge of the ultimate through travelled way to the face of abutments or bridge piers for all bridges over freeways shall be the greater of:

- i) 10 m;
- ii) 10 m multiplied by the appropriate horizontal curve adjustment factors given in the Roadside Design Manual for the design speed and radius of the through travelled way;
- iii) Width of gore area plus width of auxiliary lane or ramp plus 7 m from the outside edge of the auxiliary lane or ramp.

For exceptions see below.

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Minimum horizontal clear zone on Non-Freeways

The minimum horizontal clear zone width from the outside edge of the ultimate through travelled way to the face of abutments or bridge piers for all bridges over non-freeways shall be the greater of:

- i) The clear zone width as given in the Roadside Design Manual for the appropriate design speed for $AADT \geq 6000$ and radius of the through travelled way;
- ii) Width of gore area plus width of auxiliary lane or ramp plus the clear zone width given in (i) from the outside edge of the auxiliary lane or ramp.

For exceptions see below.

Exceptions

Where the requirements above cannot be met due to constraints including those resulting from alignment, physical, environmental, and property concerns, or would lead to a cost prohibitive structure, a reduction in the minimum horizontal clear zone width may be considered with mitigation, and with approval of the Structural Section. When this occurs, reasons for this deviation from the policy above shall be demonstrated and documented in the structural design report. Mitigation measures can include an approved barrier system as specified in the Roadside Design Manual. The barrier system shall be positioned a distance in front of the abutment or pier to allow for a deflection characteristic of the barrier system. The offset from the traffic face of the barrier system to the edge of the closest travelled lane shall be the greater of the following requirements:

- i) The minimum clearance to barriers as required by the Roadside Design Manual;
- ii) The shoulder widths on the approaches as required by the TAC Geometric Design Guide for Canadian Roads (2017) in conjunction with the MTO Design Supplement.

Consideration shall also be given to provide greater clearances that would not increase the cost of the structure where the bridge configuration would allow it. For example, when the abutment height exceeds 8 to 8.5 metres, it is sometimes less expensive to increase the span and reduce the abutment height especially when the bridge is narrow. An open abutment arrangement is more pleasing and is preferred wherever possible without an impact on cost.

Bridge abutments and slope treatments shall also be offset adequately to provide the necessary stopping sight distances.

2.2.3 Treatment of Slopes in Front of Abutments

At overpass and underpass locations where the back slope parallel to the roadway in front of the abutment is within the clear zone width and at a slope not steeper than 2H:1V, protection is not warranted provided the slope is free of obstacles and of a smoothness that allows it to be traversable. However, if the abutment is on an intersecting fill slope as

SECTION 2 - GENERAL DESIGN AND DRAFTING

shown in Figure 2.2.1 this slope and the exposed side of the abutment may be hazardous to errant vehicles. In this case the approach grading shall be flattened and contoured to redirect errant vehicles away from the exposed side of the abutment. An example of such grading is shown in Figure 2.2.1.

Flattening and contouring the grading on the approaches is preferred and is generally a safer and more economical practice than placing a traffic barrier in front of the abutment. A traffic barrier is also a hazard, and it is closer to traffic and significantly longer than an abutment, which is not desirable.

In situations where it is not practical to contour the approaches to the abutment fill, for example at closed abutments, appropriate mitigation measures such as traffic barriers shall be considered.

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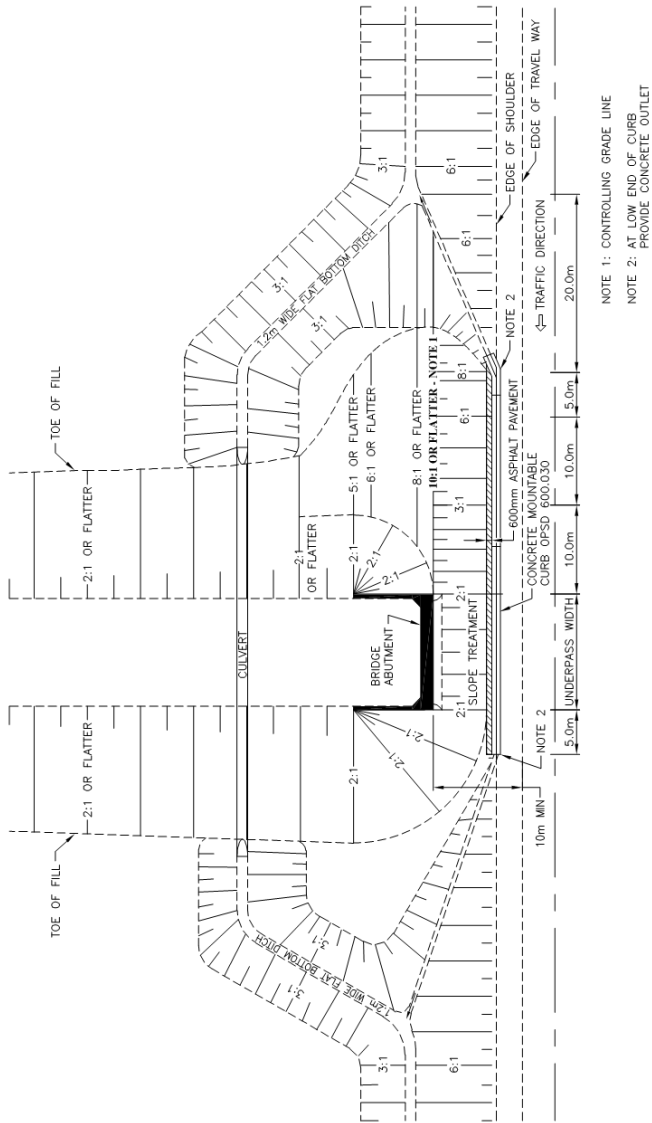


FIGURE 2.2.1 – TREATMENT OF SLOPES IN FRONT OF ABUTMENTS

SECTION 2 - GENERAL DESIGN AND DRAFTING

2.2.4 Clearances for CNWA Structures

For structures over navigable waterways, the minimum clearance provided above the appropriate water level and as specified in the structural design report shall be shown on the preliminary and final versions of the general arrangement.

This minimum clearance shall be assumed to apply during as well as after construction unless the drawings indicate otherwise. If a lesser clearance during construction has been approved, it must be shown as the "Construction" clearance. This construction clearance should be given in the structural planning report or be covered by subsequent correspondence.

2.2.5 Vertical Clearance

Vertical clearances for structures are prescribed in the publications "TAC Geometric Design Guide for Canadian Roads (2017) in conjunction with the MTO Design Supplement". Possible reduction of vertical clearance, due to settlement of an overpass structure, shall be investigated. If the expected settlement exceeds 25 mm, it shall be added to the specified clearance.

For the purposes of vertical clearance, voided concrete slab bridges would qualify for reduced clearance shall have exterior web no less than 500mm thick.

The determination of a new structure's profile shall also take into account required falsework clearances and falsework depth. Table 2.2.1 lists typical falsework dimensions for various lane arrangements.

TABLE 2.2.1 – TYPICAL FALSEWORK CLEARANCES

Required Opening	2 lanes		3 lanes	
Steel beam depth	24 in. / 610 mm		36 in. / 915 mm	
Joist depth (mm)	90	140	90	140
Plywood (mm)	17	17	17	17
Crossfall + camber (mm)	150	150	150	150
TOTAL (mm)	867	917	1172	1222

Existing structures with a clearance of less than 4.5 m must be posted.

Resurfacing or reconstruction that reduces clearance under a structure to less than 4.5 m is not permitted. In addition, any reduction of clearance below that specified for new bridges requires Ministry approval and shall be avoided if this can be achieved at reasonable cost.

SECTION 2 - GENERAL DESIGN AND DRAFTING

2.3 Geotechnical Design**2.3.1 Protection Systems**

Excavations required for construction of foundations, culverts, and other below-grade components or structures are generally made with sloping sides or with vertical or near vertical sides. These conditions depend on a variety of factors such as available space, type of soil, water table, depth of cut, duration of work etc. In all cases, the conditions shall provide for the stability and protection of the new construction, adjacent existing structures, and the safety of the public and construction workers. Where the actual site, soil and neighbouring conditions will not permit construction with allowable slopes, then supplementary measures shall be provided such as a "Protection system."

Protection systems are structures designed to protect and preserve existing structures, materials, utilities, or other works while facilitating the safe construction of new work. Roadways, adjacent buildings, property, railways, existing bridge work (new and old), gas and water mains, are examples of items that may require protection. They are, in general, temporary structures required, essentially, to keep the earth out of excavations.

The type of retaining structure or protection system is selected and designed by the contractor except as modified below. Contractors may design their schemes according to their proposed staging, available materials, expertise, and enterprise. The result shall be a scheme that is structurally adequate and economic to both the owner and contractor. The design shall not compromise the safety of the public, construction workers and the protection requirements.

The following policy applies to Ministry projects:

1. The complete design of protection systems shall not be detailed on the contract drawings, except as follows:
 - a) Where the protection system will be incorporated into the final design of the structure.
 - b) If the owner whose property is being protected, such as a railway company, requests a detailed design to be part of the contract documents.
 - c) Special cases where the Ministry perceives that, for significant safety concerns, control of the protection system's design should remain with the permanent structure designer.

For these situations the structural design report shall make the appropriate recommendations for the protection system's design.

In order for the Ministry to benefit from a contractor's design, construction ingenuity, experience, and ability to work through or around restrictions, the contractor may still submit alternate schemes. These, however, will only be allowed under certain conditions. Some important elements that should form part of these conditions are:

SECTION 2 - GENERAL DESIGN AND DRAFTING

- There should be a cost saving which the ministry shares with the contractor.
- The change proposal shall not compromise any essential design criteria, safety or preliminary engineering commitments.
- The change proposal cannot be the basis for a contract claim.

Where applicable, the criteria above shall be included in the contract documents.

2. When an anticipated protection system is not detailed on contract documents, adequate information shall be given on the contract drawings to enable the contractor to design and construct the protection system. Such information shall include all or some of the following:
 - a) Approximate location;
 - b) Allowable slope geometry;
 - c) Earth pressure parameters;
 - d) Design bond stress for anchors where applicable;
 - e) Performance level;
 - f) Operational constraints;
 - g) Known utilities;
 - h) Live load surcharge.
3. For all protection systems, the design and construction shall meet the requirements of the Ontario Provincial Standard Specification OPSS 539 "Construction specification for temporary protection systems".
4. It is possible that a protection system may not be anticipated but required because of the contractor's method of operation. In this situation, drawings of the contractor's proposal shall be submitted to the Contract Administrator (CA) as required by OPSS 539. Review of these cases will be at the discretion of the regional office.
5. The protection systems proposals shall be submitted to CA for information only. For a complex system included in the contract, Structural Section may be involved in the review with the CA before construction, the Structures Office may also be consulted at the discretion of the Structural Section.

For assistance in the design or review of protection systems the designer may consult the "Guide Design Specifications for Bridge Temporary Works, 2nd edition" 2020 INTERIM REVISIONS, AASHTO and the "Construction Handbook for Bridge Temporary Works, 2nd edition", published by AASHTO in 2017 shall be referenced. Where conflict occurs in these publications with OPSS 539 or the recommendations of the Foundation Section, the requirements of the latter shall take precedence.

2.3.2 Sheet Piling Around Footings

When sheet piling is to be left permanently in place around a footing, typically as scour or tremie concrete protection, it shall be stated on the contract drawings. The cut-off and tip elevations shall be specified, together with the minimum thickness and minimum section modulus per unit width. In this case, payment for such work shall be included in the tender item for concrete in footings.

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Shoring and bracing required to do the work are usually not shown on the contract drawings but shall remain the responsibility of the contractor.

2.3.3 Fill Stages at Structures

Where fill is to be brought up to footing level, piles (if any) driven, etc., the first fill stage or interim grade line shall be shown to the bottom of footing elevation on the general arrangement drawing.

When piles are to be driven through the fill, a note is required limiting the maximum particle size in the fill to 75 mm when steel H-piles are specified, and 50 mm when tube piles are specified. If either type of piles may be used, the maximum particle size for tube piles shall be specified.

2.3.4 Sub-Excavation and Fill at Structures

When the foundation investigation report recommends sub-excavation of unacceptable material in the vicinity of structures and replacement with granular material, the type of granular material shall be established in consultation with the Foundation Section and the Structural Section and shall be specified on the general arrangement drawing. When the extent of sub-excavation is not shown on structural drawings, a note saying "FOR LIMITS OF SUB-EXCAVATION, SEE GRADING DRAWING" is required, close to where the sub-excavation is indicated.

2.4 Design - Material**2.4.1 Specified 28-Day Concrete Compressive Strengths**

The 28-day concrete compressive strength shall be specified based on Table 2.4.1 below. The concrete compressive strength is the concrete ~~strength~~strength, which is determined by means of cylinders or cores, tested at 28 days of age, as specified by the governing OPSS. The 28-Day compressive strengths listed below are to be specified for normal concrete. MTO performs in-situ testing for Rapid Concrete Permeability (RCP), and salt scaling as means of improving the durability of in-place concrete. -Specifications currently require concrete RCP values of between 1000 and 2500 Coulombs, depending on the use and type of concrete. As such, concrete strength should be specified only for structural requirements ~~only~~ since the durability properties can be achieved by other various means allowed in the concrete specifications.

Higher strengths of concrete come with increased cost ~~and~~, higher carbon footprint, generate more heat, and have higher shrinkage which may result in more cracking. In typical bridge designs, it is difficult to make use of the higher strength in the structural design since shear and tensile cracking are a function of the square root of the concrete strength. -Acceptable 28-day compressive strengths of concrete for various applications are as follows:

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TABLE 2.4.1 – 28-DAY CONCRETE COMPRESSIVE STRENGTHS

28-day Compressive Strength	Applications
<30 MPa	Not used for structural purposes.
30 MPa	For bridge decks, curbs, sidewalks, approach slabs, barrier walls, cast-in-place concrete culverts and substructures.
35 MPa	For precast culverts with span 3m and less, prestressed post-tensioned concrete, columns where strength requirements dictate, and cast in place segmental structures.
40 MPa	For precast prestressed panels, and precast segmental units. (See also Section 7.2)
45 MPa	For precast concrete culverts with spans greater than 3m
50 MPa	For precast prestressed box beams. (See also Section 7.2)
60 MPa	For precast prestressed NU girders. (See also Section 7.2)

Higher strengths are permissible only with justification and if written approval at no later than its 30% design is obtained by the designer from the Head of Structural Section.

The required concrete strength shall be shown on the preliminary version of the general arrangement drawing.

2.4.2 Mass Concrete

According to OPSS 904, components with the least dimension thicker than 1.5 m are classified as mass concrete. For mass concrete components, a temperature control plan is required by OPSS 904 and is to be developed by the Contractor. To mitigate against excess heat development during curing of the concrete, components should be no thicker than necessary, should be detailed with the lowest practical strength of concrete, and should use the largest practical aggregated size. Nevertheless, there are situations where the designer may need to include additional provisions in design to ensure the temperature limits are controlled. In design, this could include planned and designed construction joints through a component to limit the size of each placement to allow for natural cooling between staged concrete placements which usually requires additional reinforcement to be designed.

The designer shall review and determine if large concrete components potentially require thermal intervention during curing. If the temperature of the components is likely to exceed 70° C and cannot be mitigated by routine construction measures (e.g. lowering the initial concrete temperature, adding supplementary cementing materials to the limits allowed in OPSS 904), an NSSP should be added to the contract where cooling systems are anticipated. For components which are not exposed to de-icing salts, alternate concrete mix designs should be considered. Notes should be added to the GA to account for any optional construction joints or cooling system(s).

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2.4.22.4.3 Availability of Concrete

There are areas in the province where cast-in-place concrete is difficult to obtain. The designer shall confirm the availability of cast in place concrete, from a certified ready-mix plant, before starting the design.

2.4.32.4.4 Reinforcing Steel (Grade 500W)

In 2021, the Ministry announced the policy to move towards a higher grade of 500W MPa strength steel reinforcement. Since 2010, Grade 400W reinforcing steel has been specified according to CSA G30.18-09 (R2014): Carbon Steel Bars for Concrete Reinforcement, as the standard grade of reinforcing steel on MTO contracts. The same CSA specification and its current version includes grade 500W which is 25% stronger than 400W. The higher strength of 500W steel helps to reduce the total area of steel in reinforced concrete and precast elements. This specially becomes advantageous in reducing the rebar congestions and offers additional advantages such as:

- Reduced time and labour costs to place reinforcing steel;
- Easier concrete placement and result in improved quality;
- Lower greenhouse gas emissions due to less tonnage of reinforcing steel; and,
- Design yield strength of 500 MPa aligns with current design yield strength of 500 MPa for stainless reinforcing steel.

As per MTO's policy the switch to 500W reinforcing steel began in 2022 with new designs started to use the new grade. By April 2024, it is expected that all projects currently in design phase I will have been completed, and all projects after that time will only use 500W grade in design.

-The following policy shall be applied:

1. The designer shall utilize only the 500W grade, mixing of 400W and 500W on the same structure ~~is not permitted~~**being allowed**;
2. The designer shall consider and take advantage of the higher yield strength, where permitted by CHBDC;
3. For SSD's, any reference from 400W shall be changed to 500W. Structures Office is in the process of updating the drawings to 500W and has determined that there are no negative implications in using the currently specified reinforcement quantities with the higher strength;
4. For OPSD's, and MTOD's, which are not allowed to be modified, the drawings shall be replicated, but with the steel grade changed to grade 500W, and inserted into the drawing package for the bridge. The drawings shall be signed and sealed by the design engineer as per Section 1.5.3.

2.4.42.4.5 Durability

Durability requirements for new structures, rehabilitation and replacement work shall meet

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the provisions of Section 2 of the CHBDC except, where modified by the contents of this manual.

2.4.52.4.6 Cover to Reinforcing Steel, P.S. Strands, and P.S. Tendon Ducts

The nominal cover and tolerance specified on drawings are based on CHBDC Table 8.5, and the CHBDC Exceptions. Contrary to CHBDC Commentary Clause C8.11.2.4 and based on satisfactory past performance, MTO treats the overhead deck slab soffit as a moderate (and not severe) environmental exposure and does not increase the cover by the 10mm suggested in the notes to CHBDC Table 8.5.

The concrete cover, and placement tolerances for several typical conditions are provided in Section 2.6.6 which shall be added to the Contract Drawings as General Notes. For member types, surfaces and exposure conditions not listed, the designer shall refer to Clause 8.11.2.4 of the CHBDC. Some nominal alterations to the cover are specified in MTO Structural Standard Drawings, with the specific values for concrete cover and tolerance shown on the individual drawings. Some reductions in cover due to the use of Premium Reinforcement have already been incorporated into values provided in Section 2.6.6 and in the Structural Standard Drawings which are applicable in conjunction with the details and practices used throughout this Manual.

In detailing and scheduling reinforcing steel it is important to ensure that nothing will prevent obtaining the specified cover. Particular attention is drawn to the following:

- 1) Stirrups which extend from the bottom to the top of the deck shall be sized to fit without causing an encroachment on the cover in any location. Generally, this results in the stirrups being 120 mm min. less than the deck depth at the shallowest location where the stirrups are to be placed;
- 2) In slabs and walls, Stirrups-stirrups shall-need not be hooked over the ~~topouter~~-most reinforcing bars. They ~~shall-may~~ be hooked over bars in the lower layer of the top mat with 135 degree hooks or greater;
- 3) The appropriate standard drawing showing the method of supporting the reinforcing steel shall be referenced as follows:
 - OPSD 3329.101 for thick slabs;
 - OPSD 3329.100 for thin slabs (i.e., less than 300 mm thick);
- 4) If fanned bars are used, a location shall be found for them such that there is no encroachment on cover and that location shall be clearly shown in plan and section.

2.4.62.4.7 Requirements for Sulphate Resistance in Bridge Elements

Concrete used in buried concrete structural elements including but not limited to piles, caissons, footings, retaining walls, pile caps, culverts, pipes, utility chambers, box maintenance holes and retained soil systems can be exposed to sulphate attack. Concrete is considered subject to sulphate attack when:

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- Water-soluble sulphate (SO₄) content of the adjacent soil is equal to or greater than 0.10%; or,
- Sulphate (SO₄) in groundwater is equal to or greater than 150 mg/L.

When concrete is identified as subject to sulphate attack, the concrete shall be resistant to sulphate attack as required by Special Provision CONC0006.

2.4.72.4.8 Corrosion Protection Policy for Concrete Bridge Components**Definitions**

Black Steel means carbon steel as per CSA G30.18.

Corrosion Resistant Materials means Premium Reinforcement and HPC.

Premium Reinforcement or Premium Reinforcing means Stainless Steel or GFRP.

Conventional concrete means concrete containing no silica fume, having a rapid chloride permeability at 28 days of 2500 coulombs or less, and having a minimum specified 28-day compressive strength of 30 to 60 MPa, in accordance with OPSS 1350.

High Performance Concrete (HPC) High Performance Concrete (HPC) means concrete with a minimum specified 28-Day compressive strength of at least 50 MPa that includes silica fume and may include other supplementary cementing materials and having specified rapid chloride permeability at 28 to 32 Days of 1,000 coulombs or less.

Stainless Reinforcing Steel means stainless steel reinforcing bars conforming to ASTM A276 and ASTM A955M, minimum grade 500, Type 316 LN or 2205 Duplex or 2304 Duplex. All structural designs which specify stainless reinforcing steel in accordance with OPSS 1440 and this manual shall take advantage of the higher yield strength of 500 MPa in proportioning the reinforcement size and spacing.

GFRP Reinforcing means Grade III Glass Fibre Reinforced Polymer reinforcing bars meeting the high durability as D1 category, with nominal areas according to CSA S807, and material properties according to OPSS 1640.

Background

Current corrosion protection strategies include use of black (carbon) steel in low susceptible bridge components, the use of deck waterproofing membrane and the use of Premium Reinforcement for bridge components within salt susceptible areas, including the splash zones as defined in Table 12.2.1 of the Structural Manual. These strategies were developed based on life-cycle financial analysis. Given the higher unit price of Premium Reinforcement, the Ministry uses it judiciously and only in those components for which a life-cycle benefit can be demonstrated.

The Ministry conducted studies on a number of decks built with epoxy-coated rebars in the early 1980's. The studies showed that the waterproofing membrane and the concrete cover were effective in minimizing the ingress of moisture and chlorides, making a service

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life of 75 years possible provided the waterproofing is replaced every 30 years. Studies have also shown that epoxy coated steel was not effective in providing additional corrosion protection and it is therefore no longer used by the Ministry as a corrosion protection strategy.

The Structures Office, in conjunction with the Engineering Materials Office, continues to explore new types of corrosion resistant reinforcement and corrosion protection strategies will be reviewed from time to time as new cost-effective materials are identified.

Corrosion Protection Requirements

1. Premium Reinforcing shall be used in proximity to all bridge surfaces within the splash zone or otherwise exposed to chlorides. Details of reinforcement and locations are provided in Section 12.
2. Normal concrete is typically used. However, when considered appropriate by the Structural Section, HPC may be specified for bridge components except the following:
 - Footings;
 - Post-tensioned bridges;
 - Precast pre-stressed concrete girders with 28-day design strength less than or equal to 50 MPa.
3. All concrete decks shall be protected by a hot poured rubberized asphalt waterproofing system with protection board, followed by paving with asphalt. In rare cases where the Structural Section decides to use exposed concrete decks (e.g., to reduce dead loads), Premium Reinforcement shall be required in the deck beyond what is specified in Section 12 of this manual.

Additional Guidelines for Rehabilitation

In addition to the requirements described above, the following policies shall be followed with respect to the application of premium corrosion protection systems in rehabilitated concrete bridges:

- Concrete patch repairs:
Conventional concrete or proprietary patching materials shall be used in patch repairs, depending on the size and dimensions of the repair area. HPC shall not be used in patch repairs, regardless of the remaining life of the bridge or its exposure condition.
- Overlays:
Two low permeability concrete overlay options are available, silica fume concrete and latex-modified concrete, as outlined in the Structure Rehabilitation Manual. There is no "HPC" overlay option available in current MTO specifications and this shall not be specified. The current strength specification for silica fume concrete overlay is 40 MPa compressive strength at 28 days. Conventional concrete remains an option for overlays.

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Premium Reinforcement requirements for Rehabilitations shall be as per Section 12 of this manual.

2.5 Miscellaneous Design Guidance**2.5.1 Design for Protection of MSE Systems Abutments and Walls**

The use of Mechanically Stabilized Earth (MSE) Systems (RSS) for false and true abutments allows them to be constructed close to travelled lanes and in salt splash zones which exposes them to deterioration, requiring the use of premium reinforcement. They are also susceptible to loss of integrity in the event they are hit by errant vehicles. The MSE false abutment may also be designed as an integral or semi-integral abutment that may be vulnerable to an impact from vehicles unless it is designed to absorb the traffic load, or it is adequately protected. Integral abutment bridges supported on a single row of piles are somewhat more vulnerable than semi-integral abutment bridges, which are supported on columns on rigid foundations. Consideration therefore shall be given to their protection as indicated below:

- i) Where MSE concrete facing panels are located within the clear recovery zone, they shall be protected by an approved traffic barrier system or a slope treatment.
 - a. The approved traffic barrier system shall be installed at least the TL-5 cab Zone of Intrusion distance in front face of the abutment wall, which is 1.3 m for OPSD 911.132 concrete tall wall barrier or 1.65 m for MTOD 912.600 steel railing:
 - b. The slope treatment should have a 2H:1V to 2.5H:1V upward slope for a distance of at least 4.0 m measured horizontally from the edge of shoulder to the face of panels and the approach grading should be flattened and contoured as described in the previous section;
- ii) Where integral or semi integral abutment design is used, the deflection and 4.0 m distance shown in i) shall be increased by 0.8m.

2.5.2 Design for Fire Protection of Tunnel Like Structures

Structures that span over a roadway with vehicular access limited only to the two ends of the enclosed area can be considered to be tunnels irrespective of the presence of fill on top of the structure. These structures present specific safety challenges in the case of a fire breaking out within the structure; the degree of risk to life safety depends on the length of the roadway between access points.

In the absence of a policy dealing with the fire protection of tunnel-like highway structures, designers were left to their own professional judgement on when to implement fire protection. Although internationally recognised standards like National Fire Protection Association (NFPA) 502 provide guidelines for fire resilience design of road tunnels, many practising bridge engineers may not be aware of its requirements and there is no direct reference to it in the CHBDC. Furthermore, the CHBDC does not have load combination factors involving fire events nor provisions for material properties at elevated temperatures. The guidelines described below are to ensure a consistent design

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approach for fire protection of tunnel like structures and to ensure life safety related to fire events is duly considered where there is a higher risk of occurrence of a large hydrocarbon fire.

A highway structure will be treated as a tunnel for the purposes of fire resilient design when:

- a) The structure spans over a motor vehicle roadway; and,
- b) The vehicular access is limited to the two ends of the fully or partially enclosed area; and,
- c) The distance between any open-air access points for emergency equipment to the roadway below is greater than 90m.

A structure, through an engineering analysis according to NFPA 502 Chapter 4, may also be categorized as a fire resilient design tunnel by the ministry.

Unless otherwise approved by the ministry, new highway structures categorised as tunnels and with AADT for the enclosed roadway exceeding 1500 shall be designed for fire resilience according to the following requirements:

1. Design for fire resilience shall be according to Chapter 7 of the latest edition of NFPA 502;
2. Road tunnels shall be designed to withstand the required fire exposure without structural collapse for a minimum duration of 2 hours under all dead loads. Load factors for all permanent loads including snow load, fill and landscaping, as applicable, shall be 1.0. Material properties at elevated temperatures shall be according to EN-1991-1-2 Eurocode 2: Design of Concrete Structures Part 1-2, General Rules – Structural Fire Design, 2004 edition, unless otherwise specified by the Ministry. Steel structures shall not be used unless approved by the Ministry.
3. In addition to criteria (2) above, road tunnels that carry vehicular traffic on top shall be designed to withstand the required fire exposure without structural collapse for a minimum duration of 1 hour with full traffic live load and all dead loads. Load factors for live loads and all permanent loads including snow load, fill and landscaping, as applicable, shall be 1.0 and with no application of dynamic load allowance. Material properties at elevated temperatures shall be as stated in criteria (2) above;
4. Any fire protection materials used to retard the temperature increase of structural components shall be durable and able to withstand moisture, chloride, freeze thaw cycles and the dynamic effect due to in-service live load for a service life of 50 years.

New tunnels with AADT less than 1500 and rehabilitation of existing tunnels, including extending the width or length of a structure to meet the above criteria a) through c) for a tunnel, shall be required to follow the above design requirements 1) through 4) only when specified by the ministry.

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2.5.3 Design Guidance for Seismic Importance Categories

The seismic importance category shall be shown in the structural planning report and shall be determined as follows:

- i) Lifeline bridges shall include all bridges:
 - a) With AADT exceeding 20,000 and overall length exceeding 1000 m, or deck area exceeding 10,000 m²; or,
 - b) Identified in an economic planning exercise as structures that are vital to the integrity of the regional transportation network, the ongoing economy, and/or the security of the region and/or represents significant investment and would be time-consuming to repair, with approval of the Structures Office;
- ii) Major-route bridges shall include all bridges:
 - a) Carrying Core and Feeder National Highway System bridges (see Figure 2.5.1 and Table 2.5.1); or,
 - b) Carrying 400 series highways; or,
 - c) Exceeding 300 m in length; or,
 - d) Exceeding 5000 m² of deck area; or,
 - e) That provide sole access to emergency vehicles and those with exceptionally long detour lengths, with the approval of the Structural Section; or,
 - f) Identified in an emergency planning exercise as being critical to an emergency response of a community, with approval of the Structural Section;
- iii) Other bridges are all remaining bridges.

Note: AADT and deck area, in the case of twin bridges and core/collector freeways, shall be for all lanes of traffic.

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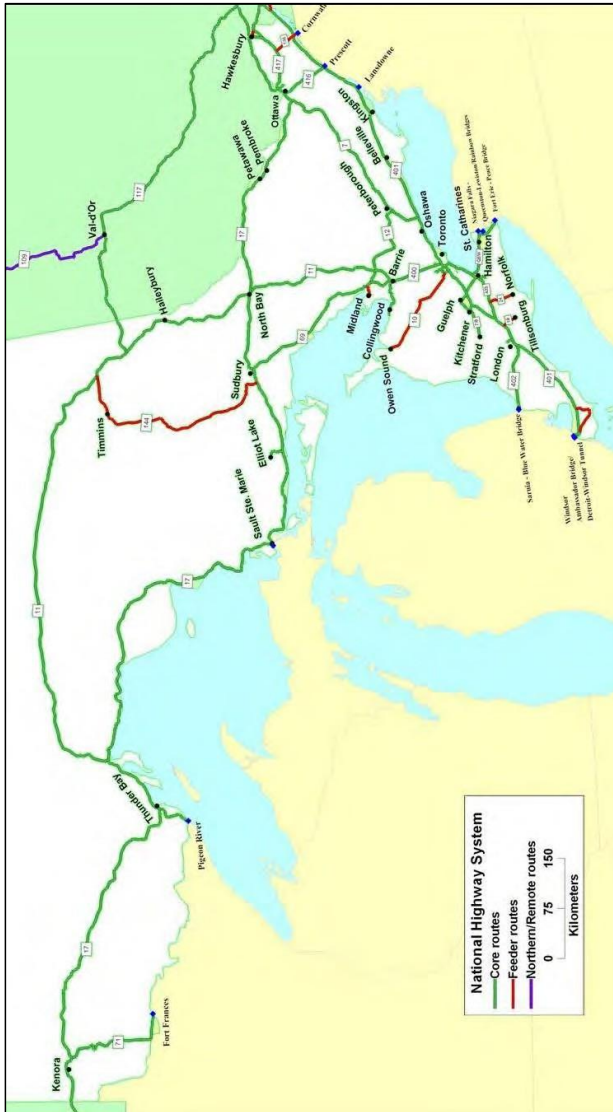


FIGURE 2.5.1 – CANADA’S NATIONAL HIGHWAY SYSTEM – ONTARIO

Source: Annual Report 2017 (<https://www.comt.ca/Reports/NHS%20Annual%202017.pdf>)

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TABLE 2.5.1 – CANADA'S NATIONAL HIGHWAY SYSTEM – ONTARIO HIGHWAYS AND LOCATIONS

Source: Annual Report 2017 (<https://www.comt.ca/Reports/NHS%20Annual%202017.pdf>)

Ontario				Core	Feeder	Northern and Remote
Route	From	To	Length (km)	km	km	km
QEW	Fort Erie	Toronto	139.0	139.0		
401	Que. Border	Windsor	827.0	827.0		
402.0	London	U.S. Border (Sarnia)	102.6	102.6		
405	QEW	U.S. Border (Queenston - Lewiston Bridge)	8.8	8.8		
427	Hwy 401	QEW	7.7	7.7		
137	Hwy 401	U.S. Border (Lansdowne)	4.3	4.3		
416	Ottawa (Jct. 417)	Hwy 401	75.9	75.9		
16	Hwy 416	U.S. Border (Prescott)	3.8	3.8		
417	Quebec Border	Hwy 17 Sheel Drive (Arnprior)	194.0	194.0		
400	Toronto (Jct. Hwy 401)	Hwy 69 (IC-241)	225.0	225.0		
69	Hwy 400 (IC-241)	Sudbury (Jct. Southwest Bypass)	139.0	139.0		
17	Hwy 417 Sheel Drive (Arnprior)	Manitoba Border	1,948.0	1,948.0		
66	Quebec Border	Kirkland Lake	58.0	58.0		
11	North Bay	Nipigon	991.0	991.0		
71	U.S. Border (Fort Frances)	Hwy 17	194.0	194.0		
61	U.S. Border (Pigeon River)	Thunder Bay (Jct. 17)	61.0	61.0		
403	QEW (Burlington)	Hwy 401 (Woodstock)	82.0	82.0		
11 / 400A	Barrie	North Bay	239.0	239.0		
35 / 115	Hwy 401	Peterborough (S Jct. Hwy 7 / 115)	44.8	44.8		
7 / 115	Peterborough (S Jct. Hwy 7 / 115)	Ottawa (Jct. Hwy 417)	250.0	250.0		
7 / 12	Peterborough (S Jct. Hwy 7 / 115)	Hwy 11	130.0	130.0		
12	N Jct. Hwy 11	Hwy 400	21.0	21.0		
26	Hwy 400 (Barrie)	Collingwood (County Road 19)	63.0	63.0		
6.0	Hwy 403 (Hamilton)	Hwy 401 (Guelph)	25.9	25.9		
6.0	Hwy 401 (Guelph)	Guelph (Woodlawn Rd.)	15.4	15.4		
7.0	Guelph (Woodlawn Rd.)	Kitchener (Conestoga Parkway)	21.0	21.0		
8.0	Kitchener (Conestoga Parkway)	Stratford (Erie)	45.0	45.0		
8.0	Hwy 401	Kitchener (Conestoga Parkway)	7.7	7.7		
108	Hwy 17	Elliot Lake (Hillside Dr.)	27.2	27.2		
34	Hwy 417	Hawkesbury (Quebec Border)	19.2	19.2		
17B	Hwy 17	U.S. Border (Sault Ste. Marie)	10.6	10.6		

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Ontario				Core	Feeder	Northern and Remote
Route	From	To	Length (km)	km	km	km
3.0	Hwy 401	U.S. Border (Ambassador Bridge)	11.0	11.0		
3B	Hwy 401	U.S. Border (Detroit-Windsor Tunnel)	11.0	11.0		
420	QEW	U.S. Border (Rainbow Bridge)	5.0	5.0		
Nicholas / Rideau / King Edward	Hwy 417	Quebec Border (Gatineau)	4.0	4.0		
403	QEW	Hwy 401	21.0	21.0		
410	Hwy 401	Hwy 10	24.0	24.0		
427	Hwy 401	York Regional Road 7	12.3	12.3		
409	Hwy 401	Hwy 427	6.0	6.0		
6	Hwy 403	Hamilton Airport (Airport Rd.)	10.0	10.0		
Bronson / Airport Parkway	Hwy 417	Ottawa Airport	9.8	9.8		
Airport Rd. / Oxford St. E	Hwy 401	London Airport	10.0	10.0		
RR7 / RR50 / Rutherford	Hwy 427	CP Intermodal Terminal (Vaughan)	6.0	6.0		
Steeles / Airport Rd / Intermodal Dr.	Hwy 410 (Bovaird Dr.)	CN Intermodal Terminal (Brampton)	7.1	7.1		
Dery Rd / Airport Rd / Intermodal Dr.	Hwy 427	Steeles Ave. - CN Intermodal Terminal (Brampton)	5.6	5.6		
Gardiner Expy / Kipling / Queen	Hwy 427	CP Obico Intermodal Terminal	3.5	3.5		
Trafalgar	Hwy 401	Dery Rd. - CP Expressway Intermodal Terminal	1.7	1.7		
RR7 / Keele / Administration	Hwy 400	CN RoadRailer Intermodal Terminal (Vaughan)	4.3	4.3		
McCowan Road	Hwy 401	CP Expressway Intermodal Terminal (Scarborough)	1.6	1.6		
138	Hwy 401	Hwy 417	35.0		35	
138	U.S. Border (Cornwall)	Hwy 401 IC	8.0		8	
CR17	Hawkesbury E	Hwy 417	10.0		10.0	
144 / 101	Hwy 17 (Sudbury)	Timmins (Mountjoy St.)	292.0		292	
101	Timmins (Mountjoy St.)	Hwy 11	62.0		62	
12	Hwy 400	Midland (Hwy 93)	18.0		18.0	
10	Hwy 410	Owen Sound (Hwy 26)	136.0		136	
77	Hwy 401	Leamington (Hwy 3)	23.0		23	
3.0	Leamington (Hwy 77)	Hwy 401	39.0		39	
19	Hwy 401	Tillsonburg (Vienna Rd.)	23.0		23	
24	Hwy 403	Simcoe (Hwy 3 / Queensway Dr.)	36.0		36	
Total			6,816.8	6,134.8	682.0	-

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2.6 Drawings

This section provides criteria and guidance on the preparation on design drawings and contract packages.

2.6.1 Drawing Numbers

Final drawings for all new structures are numbered consecutively with numbers only. [The drawing number is added to the lower title block, as explained in 2.6.7.](#)

Preliminary drawings are numbered with the letter P1 for the first scheme. If preliminary drawings are revised and re-issued, the number is incremented, e.g., from P1 to P2. If more than one preliminary scheme is prepared, the drawings are numbered PA1, PB1 etc. For cases where there is more than one drawing per scheme then it shall be numbered PA1, PA2, etc.

Rehabilitations or modifications shall be numbered with the prefix RX-1, 2, etc., where X represents the number of times it has been rehabilitated.

In the following examples, the simplest and the most complex results are shown, however improbable the latter.

50	A bridge final, fiftieth drawing.
P1	Bridge preliminary, one scheme, one drawing.
PA2	Bridge preliminary, first scheme, second sheet.
R4-P1	Bridge rehabilitation preliminary, fourth rehab, one proposal.
R4-12	The rehabilitation above, final drawing, twelfth sheet.

2.6.2 Sequence and Titles of Structure Drawings

The structure drawings are generally placed in the order of construction, with some standard detail and electrical drawings located at the end. Drawings shall be included as appropriate for specific bridges in the order shown below.

For all types of bridges:	General Arrangement Borehole Locations and Soil Strata Construction Staging Roadway (Track) Protection Foundation Layout Footing Reinforcement North (West) Abutment North (West) Wingwall South (East) Abutment South (East) Wingwall Retaining Walls Mechanically Stabilized Earth (MSE) System (RSS) Piers Bearings
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For bridges with post-tensioned decks:	Deck Details Longitudinal Tendons I, II, etc. Transverse Tendons I, II, etc.
For bridges with precast, prestressed concrete girders:	Prestressed Girders Prestressed Girders and Bearings Deck Details
For bridges with steel girders:	Structural Steel I, II, etc. Deck Details
For all types of bridges:	Deck Reinforcement Joint Anchorage and Armouring Barrier Wall with Railing Barrier Wall w/o Railing Railing for Barrier Wall 6000mm Approach Slab Details of Concrete Slope Paving As Constructed Elevations and Dimensions. Pile Driving Control Standard Details I, II, etc. Electrical Embedded Work

2.6.3 Existing Bridges Access to Contract Drawings

Requests for contract drawings and shop drawings of existing structures shall be submitted to the Structural Section. The Structural Section may contact the Structures Office if the drawings are not available in their files.

2.6.4 Water Level Designations

The following procedure shall apply to the designation of water levels on drawings:

- a) A water level shall be shown;
- b) The water level shown shall be a factual one, i.e., an elevation taken at some specific time, such as when the profile was run, and E plan contours obtained. This elevation shall be shown on the drawings with the date, e.g., "429.00 (day/month/year)";
- c) If more than one elevation is known, the Head of Structural Section shall be consulted as to the benefits of showing water levels measured at different dates;
- d) An estimated High Water Level (HWL), required for the assessment of waterway opening, shall be shown on the preliminary version of the general arrangement drawing only. This water level shall be removed for the final version;
- e) The Structural Section shall be responsible for supplying all pertinent water levels in the structural planning report.
- f) When the Structural Section deems it prudent, an additional water level, to be used for

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the design of protection schemes, shall be included on the general arrangement drawing with the note:

PROTECTION SCHEME DESIGN WATER LEVEL _____

2.6.5 Notes and Labels Forming Part of Views and Details

Notes are used to complement the information described graphically and should be placed directly at the end of leaders. Use of item references, in the form of numbers or letters at the end leaders referring to a legend or list, shall be avoided. Notes placed on views or details, describing components or requirements shall be precise, unambiguous, contain no unnecessary words, and be used only when necessary.

Some common examples are as follow:

- a) "Barrier wall and rail". There is no need for the word "Standard". The note shall state whether or not there is a rail. This note is required on the preliminary and final versions of the general arrangement drawing on the elevation. It shall not be repeated on the cross section;
- b) "Slope paving (typ.)". There is no need for other description and the note shall appear only once on the elevation;
- c) "Top of concrete end dams to suit pavement profile". This note shall always appear on the abutment drawing;
- d) "Top of cleat to be cast 35 mm below approach slab ledge". Reference to the to closed-cell extruded polystyrene or neoprene is unnecessary since this is on the approach slab drawing.
- e) Notes calling for the removal of formwork or expanded polystyrene shall never be used. The specifications require all formwork be removed and if expanded polystyrene is not to be left in place, it shall not be shown at all;
- f) Notes concerning construction joint grooves and sealing shall not duplicate or conflict with OPSD-3950.100 if this standard is included in the drawings.

2.6.6 General Notes

The following are standard notes to be shown below the title block on the general arrangement drawing. The notes shall be worded to cover the requirements specific to the particular project and should only be used if applicable. Other notes may be required in special circumstances.

In specifying the classes of concrete, different components may be itemised together if the classes are the same.

For clarity the wording of the notes is shown in upper case (capital) lettering. Explanations shown in brackets in lower case lettering are not part of the notes.

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a. General Arrangement Drawings Slab-On-Girder Bridges

1) SPECIFIED 28-DAY COMPRESSIVE STRENGTH

30 MPa

XX MPa (CLOSURE POURS) (if applicable)

SPECIFIED 28-DAY COMPRESSIVE STRENGTH FOR PRECAST GIRDERS ARE GIVEN ON PRESTRESSED GIRDER DRAWINGS

2) Clear Cover to Reinforcing Steel

FOOTINGS		100 ± 25
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DECK	TOP	70 ± 20
	BOTTOM	40 ± 10

PIER COLUMNS, SHAFTS AND CAPS 70 ± 10

REMAINDER 70 ± 20 UNLESS OTHERWISE NOTED

3) Reinforcing Steel

REINFORCING STEEL SHALL BE GRADE 500W.

UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES FOR REINFORCING STEEL BARS SHALL BE CLASS B.

STAINLESS REINFORCING STEEL SHALL BE TYPE 316LN OR DUPLEX 2205 AND HAVE A MINIMUM YIELD STRENGTH OF 500 MPa, UNLESS OTHERWISE SPECIFIED.

BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS.

GLASS FIBRE REINFORCED POLYMER REINFORCING BARS SHALL BE GRADE III, AND THE NOMINAL DIAMETER, TENSILE MODULUS OF ELASTICITY AND GUARANTEED MINIMUM TENSILE STRENGTH SHALL BE AS SPECIFIED IN THE CONTRACT DOCUMENTS.

BAR MARKS WITH THE PREFIX 'G' DENOTE GRADE III GLASS FIBRE REINFORCED POLYMER BARS.

BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWING SS112-1, UNLESS INDICATED OTHERWISE. HOOKS AND BENDS FOR GFRP BARS SHALL HAVE A MINIMUM BEND-RADIUS-TO-BAR-DIAMETER RATIO (r/d) OF 4.0.

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(The designer shall include SS112-1 on the contract drawings).

- 4) Mechanically Stabilized Earth (MSE) System (RSS) (where applicable)

APPLICATION: (To be completed according to MTO RSS guidelines)

PERFORMANCE: (To be completed according to MTO RSS guidelines)

APPEARANCE: (To be completed according to MTO RSS guidelines)

- 5) Construction Notes

(Notes as applicable, see list under Section 2.6.7(d) below)

b. General Arrangement Drawings Concrete Rigid Frames and Integral Abutments

- 1) SPECIFIED 28-DAY COMPRESSIVE STRENGTH
30 MPa

- 2) CLEAR COVER TO REINFORCING STEEL

FOOTINGS		100 ± 25
DECK	TOP	70 ± 20
	BOTTOM	50 ± 10 (if deck slab <300, then 40mm +/- 10mm)

PIER COLUMNS, SHAFTS AND CAPS 70 ± 10

REMAINDER 70 ± 20 UNLESS OTHERWISE NOTED

- 3) REINFORCING STEEL

Same as Notes for Slab-on-Girder Bridge (Section 2.6.7(a).3).

- 4) MECHANICALLY STABILIZED EARTH (MSE) SYSTEM (RSS) (where applicable)

Same as Notes for Slab-on-Girder Bridge (Section 2.6.7(a).4).

- 5) CONSTRUCTION NOTES

BACKFILL SHALL NOT BE PLACED BEHIND THE ABUTMENTS UNTIL THE DECK SLAB IS IN PLACE AND HAS REACHED 70% OF ITS DESIGN STRENGTH.

(See Section 11.1.1 of this Manual for Exception)

BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH ABUTMENTS KEEPING THE HEIGHT OF THE BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 500 mm.

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(See Section 11.1.1 of this Manual for note to be used for special site access conditions)

CONSTRUCT ABUTMENTS AND WINGWALLS TO THE BEARING SEAT ELEVATIONS. THE CONTRACTOR SHALL SUPPLY TEMPORARY LATERAL BRACING FOR THE ABUTMENTS. FORMWORK AND LATERAL BRACING SHALL NOT BE REMOVED UNTIL CONCRETE HAS REACHED 70% OF ITS SPECIFIED 28-DAY STRENGTH.

(Other notes as applicable, see list under Section 2.6.7(d) below).

c. General Arrangement Drawings Cast-In-Place Post Tensioned Concrete Bridges

1) SPECIFIED 28-DAY COMPRESSIVE STRENGTH

DECK & SIDEWALKS	35 MPa
REMAINDER	30 MPa

2) CLEAR COVER TO REINFORCING STEEL

FOOTINGS	100 ± 25
PIER COLUMNS, SHAFTS AND CAPS	70 ± 10

ROUND VOIDED DECKS

DECK	TOP	70 ± 20
	BOTTOM, SIDES, WEBS	50 ± 10
REMAINDER		70 ± 20 UNLESS OTHERWISE NOTED

RECTANGULAR VOIDED DECKS

DECK	TOP SLAB, TOP	70 ± 20
	TOP SLAB, BOT	40 ± 10
	BOT SLAB, TOP	40 ± 10
	BOT SLAB, BOT	50 ± 10
	SIDES, WEBS	60 ± 10
REMAINDER		70 ± 20 UNLESS OTHERWISE NOTED

3) REINFORCING STEEL

Same as Notes for Slab-on-Girder Bridge (Section 2.6.7(a).3).

4) MECHANICALLY STABILIZED EARTH (MSE) SYSTEM (RSS) (where applicable)

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Same as Notes for Slab-on-Girder Bridge (Section 2.6.7(a).4).

5) CONSTRUCTION NOTES

Notes as applicable, see list under Section 2.6.7(d) below.

d. CONSTRUCTION NOTES (To be used where applicable.)

THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.

(In addition to the preceding note, the following shall be shown on the drawing where top of bearing elevations are shown: Top of bearing elevations shall be denoted with an asterisk (*), and add note,) " * SEE CONSTRUCTION NOTES ON DRAWING 1 ".

THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS, DETAILS AND ELEVATIONS OF THE EXISTING STRUCTURE THAT ARE RELEVANT TO THE WORK SHOWN ON THE DRAWINGS PRIOR TO COMMENCEMENT OF THE WORK. ANY DISCREPANCIES SHALL BE REPORTED TO THE CONTRACT ADMINISTRATOR AND THE PROPOSED ADJUSTMENT OF THE WORK REQUIRED TO MATCH THE EXISTING STRUCTURE SHALL BE SUBMITTED FOR APPROVAL.

(use for the rehabilitation of structures).

SIDES OF FOOTINGS TO BE CAST AGAINST UNDISTURBED SOIL.

(use when factored horizontal forces exceed factored sliding resistance at the bottom of footing).

SHEET PILING INDICATED ON THE CONTRACT IS REQUIRED AS A PERMANENT PART OF THE STRUCTURE. IT IS NOT INTENDED TO REPRESENT A COMPLETE SHORING SCHEME.

(use when sheet piling is indicated on the contract.)

CONCRETE BARRIER WALLS ON RETAINING WALLS SHALL NOT BE CAST UNTIL THE RETAINING WALL BACKFILL HAS BEEN COMPLETED.

COMPACTED FILL, MAXIMUM GRAIN SIZE 50 mm SHALL BE PLACED UP TO THE BOTTOM OF FOOTING ELEVATION PRIOR TO DRIVING PILES (see Section 2.3.3, 50 mm is for tube piles).

EXISTING ROADBED IN AREAS THROUGH WHICH PILES MUST PENETRATE TO BE REMOVED BEFORE PLACING FILL.

IF THE DEPTH OF BLOCKOUT FOR THE SELECTED MODULAR JOINT IS

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DIFFERENT FROM THAT GIVEN ON THE MODULAR EXPANSION JOINT ASSEMBLY DRAWINGS, THE CONTRACTOR SHALL ADJUST THE DEPTH OF BLOCKOUT AND THE REINFORCING STEEL TO SUIT THE SELECTED MODULAR JOINT.

- e. **(For structural plate corrugated steel structures only. Only standard metric material may be used).**

STRUCTURAL PLATE CORRUGATED STEEL PIPE SHALL CONFORM TO CSA G401, EXCEPT THAT THE ZINC COATING MASS (TOTAL ON BOTH SIDES) SHALL BE NOT LESS THAN 915 g/m² WHEN TESTED BY THE TRIPLE SPOT TEST (TST). NOMINAL BASE METAL THICKNESS (give) mm. LONGITUDINAL SEAM BOLT SPACING (give number) PER CORRUGATION.

PRIOR TO ALLOWING HEAVY CONSTRUCTION EQUIPMENT OVER THE STRUCTURE, THE DEPTH OF COMPACTED COVER MATERIAL SHALL BE NOT LESS THAN (give depth).

- f. **STRUCTURAL STEEL NOTES**

See **Section 8.7.**

- g. **PRE-TENSIONED GIRDER NOTES**

See **Section 7.2.9.**

- h. **POST TENSIONED DECK NOTES**

See **Section 7.3.8.**

- i. **PRESERVATIVE TREATED WOOD STRUCTURE NOTES**

See **Section 15.4.**

- j. **MISCELLANEOUS**

1. No instruction or note on the drawings shall imply that there is more than one contractor. For example, reference must be to "THE CONTRACTOR" only and not to "THE STRUCTURAL STEEL CONTRACTOR".
2. For temperatures, only the Celsius scale shall be used. Temperatures shall be shown thus: 23°C, 0°C, 10°C, etc.

2.6.7 Contract Drawing Border

All structural drawings shall use the master drawing border for which a reduced scale hardcopy is shown in Figure 2.6.1. Line thicknesses and text sizes given are specified to ensure legibility after drawing reduction. A "**thin**" line shall be as thin as can be depended upon to print clearly after 1/2 size reduction. This width is between 0.2 and 0.3 mm. A "**medium**" line width is between 0.35 and 0.5 mm and a "**thick**" line shall have a width

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between 0.6 and 0.7 mm. Text need be used in only three sizes: **large** 4 to 4.5 mm high, medium 3 to 3.5 mm, and **small** 2.5 mm. Vertical, uppercase, Roman Simplex text font is preferred. Text within the upper title block shall be 3.5 mm high. All drawings shall be identified as Metric.

Upper Title Block

The Ontario Ministry of Transportation logo shall be present at the top of the title block. If desired when a design is completed by a consultant, the design engineering firm's name and logo may be added at the bottom of the upper title block, not larger than 20 mm tall.

The name of the structure (see Section 2.6.10) and the W.P. number shall be as given in the structural design report. For clarification in the use of some terms, such as, underpass, overpass, subway and overhead, see Figure 2.6.2 and Figure 2.6.3.

The Contract number and the sheet numbers are required to be shown when they become known, which may be close to the time of tendering.

The standard north point, pointing true north, shall be shown on the general arrangement drawing upper title block and on other drawings which contain a plan view.

The drawing title shall be as given in Section 2.6.2.

The highway number is not required in the title block but shall appear on the plan.

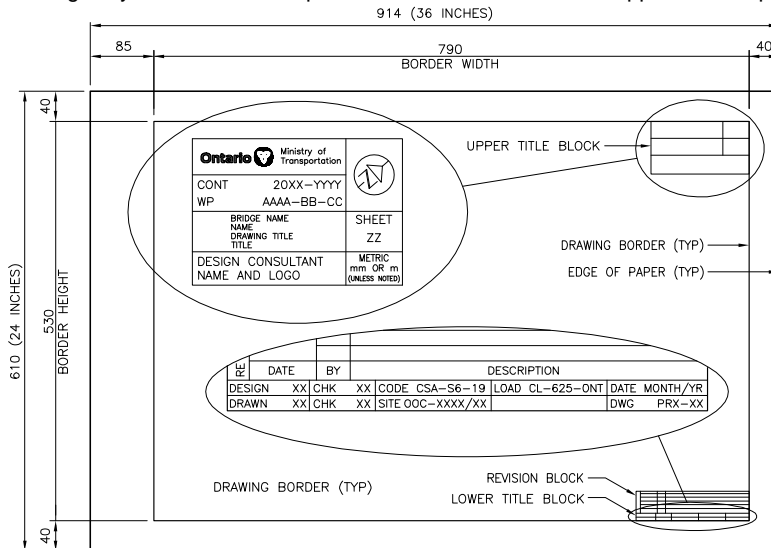


FIGURE 2.6.1 – MASTER DRAWING BORDER

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Revision Block

All revisions, after tender prints have been prepared, must be identified by a number placed in a hexagon. In the "Revision" column, the same symbol shall be used. The date, initials of the drafter making the changes and a short description of the nature of the revision shall also be given. See Section 2.6.15 for more details.

Lower Title Block

All information shall be filled as indicated below and illustrated on Figure 2.6.1.

DESIGN	Initials of <u>actual</u> designer for the work depicted on the drawing. For the general arrangement drawing the initials shall indicate general responsibility for the preliminary design, and not that complete design computations have been carried out. The designation of professional responsibility is made by the application of an engineer's seal performing or overseeing the work.
CHK (DESIGN)	Initials of <u>actual</u> design checker who checked the design, for the work depicted on the drawing.
CODE	Name of design code used in design. For current MTO projects this is the Canadian Highway Bridge Design Code (CHBDC) and shall be designated as CSA-S6-19 . For railway structures this may be American Railway Engineering & Maintenance of Way Association Manual for Railway Engineering (AREMA).
LOAD	Loading used in design specified in the applicable code above. For new MTO highway structures this will be CL-625-ONT. Railway loading may be according to AREMA and rehabilitations as per Sections 14 and 15 of the Canadian Highway Bridge Design Code.
DATE	The month and year when final drawings were completed and submitted to the Structural Section, or to Planning and Design.
DRAWN	Initials of drafter or technician who produced the drawing.
CHK (DRAWN)	Initials of checker with responsibility for checking accuracy, dimensions, geometry, etc. of the drawing.
SITE	Add site number as per Section 2.6.11.
DWG	Add drawing number as per Section 2.6.1.

2.6.8 General Arrangement Drawings Content and Format

The following instructions are not exhaustive and are intended only to provide guidance in the preparation and checking of general arrangement drawings. A general arrangement drawing shall appear as the first drawing for work on any structure.

Key Plan

A key plan is not required on the general arrangement if it is to appear on the soils drawing

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or elsewhere on a drawing that will remain on the structural file and appear in the contract documents.

If a key plan is required, it shall normally be to a scale of 1:50 000, not smaller than 100 mm x 100 mm and not larger than 150 mm x 150 mm and appear just to the left of or below the title block (top right-hand corner). The structure site shall be clearly indicated, and the plan shall be oriented with north to the top and a north point shown. If a place shown on the Ontario Road map does not occur within the key plan, notes shall be added indicating the direction and distance of two such places along routes appearing on the key plan. If this is not practicable, a smaller scale plan shall be considered.

The key plan shall show only major roads, railroads, rivers, lakes, and towns. The roads shall be shown in **thick** single lines, railroads thus ++++++, lakes and rivers in **thin** "shaky" lines. Double parallel lines shall be avoided unless, for example, a river is of great width. The title of "Key Plan" shall appear below the plan in **large** text with a scale in **small** text that is also given as a graduated line. A **medium** line border shall surround the plan itself with a **thick** line border around plan, title, and scale.

Plan

The plan, located at the top and left portion of the drawing, shall be to a scale of 1:200 unless the size of the structure precludes this, in which case scales of 1:250, 1:500, 1:750, 1:1000, 1:1250 shall be considered to obtain the largest plan possible within the space limits of the master drawing border.

The plan shall be oriented with the upper roadway direction as nearly horizontal as possible and with the chainage increasing from left to right, if this can be arranged. A construction north arrow, rotated up to 45° from true north, may also be shown on the plan and marked "North for Construction", but this shall be done only if the various parts of the structure could not otherwise be unambiguously named.

North arrows that point downwards are very undesirable and shall not occur normally because chainages throughout the province are supposed to increase from west to east and from south to north. If it is found that the north point points downwards, the problem shall be discussed with Project Delivery Section to find out if the chainage direction was correctly determined and to ensure consistency between the road and bridge drawings.

Existing contours shall always be shown, dashed (or dotted) if and where new work will change them and solid if not. If final contours are available or are specifically requested, these shall be shown, using short-dashed lines in this case for all existing contours. Contours normally are required at 1 m intervals, but larger intervals may be used for steep surfaces. Lines used shall be thin, except that 5 m interval contours may be of medium thickness.

On the plan, the following are required:

- a) Creek water edges in thick lines, consistent with the given water level and ground contours; direction of flow; name of creek in large text written along the creek and to be read from the right-hand side of the drawing.

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- b) All centre lines and control lines, pier centre lines and abutment bearing centre lines in thin centre lines.
- c) Edges of pavement, except on a bridge deck where no physical demarcation other than painted lines will exist; curbs and copings; approach slabs; deck-ballast wall joints; top and bottom of slopes; shoulder edges, retaining wall, pier and wingwall stems, when not hidden, and drainage openings shall be shown in lines of medium thickness. Railings shall not be shown.

Hidden work such as pier or column shafts, abutment faces and grading details under the structure shall be shown in thin dashed lines. Footing outlines shall be shown to indicate proximity to pavements, tracks, creeks, etc., but shall not be shown for abutments that are remote from such features. Services likely to affect or be affected by construction shall be indicated with centre lines and a description that includes the words "existing" or "proposed".

Unless a careful study of the structure by the project Engineer has indicated that none is required, the position and length of roadway or track protection shall be shown in medium double lines with the words "Track Protection" or "Roadway Protection" in small text. If it is necessary to show Traffic Protection, centre lines of medium thickness shall be used.

Dimensions for roadway widths, spans, final horizontal clearances, wingwall lengths, extent of rock protection, etc., shall be shown using thin lines and small text. If the overall length of the bridge cannot be clearly shown on the Elevation, it shall be shown on the Plan.

A plan title shall be shown in large text and the scale in small text. The direction and location of the cross-section plane and the direction of the elevation projection shall be shown by triangular symbols on the plan.

Working points, at the intersection of all centre or control lines and pier or abutment bearing centre lines shall be shown and labelled WP #1, WP #2, etc; names shall be written along all centre or control lines and the skew angle should be given, all in small text. In small text also, give stations, slopes, at least one top of pavement elevation at some well-defined central point, and control line geometry, e.g., degree of curvature, azimuth, etc. If coordinates are available or obtainable, they shall be shown for TC's, SC's, PI's, etc., on the general arrangement drawing, and for WP's, on the foundation layout. In any case, they shall be shown together with stations. These coordinates may be tabulated separately to avoid congestion on the Plan.

On the approach slab the following note should appear "6000 mm approach slab with 90 mm asphalt and waterproofing". This note shall be shown with a dimension line showing the 6000 mm dimension. The "6000 mm" shall appear above the dimension line and the remainder of the note below it.

Elevation

The elevation shall be immediately below the plan, located as if projected from it, and to the same scale. Normally the view is obtained as if by projecting the elevation parallel to

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pier centre lines or abutment faces onto a plane through the face of the bridge that is at the bottom of the plan, and then projecting vertically downwards to the location of the elevation. The ground lines shown therefore, shall be at the near bridge face and skew effects do not appear.

The outline appearing on the elevation, in lines of medium thickness, shall include barriers, an indication of railings, coping, soffit, and parts of piers, abutment faces and wingwalls that are above ground, final ground line, and preliminary grading lines (see Section 2.3.3). Hidden detail lines, thin and dashed, shall show footings, piles, and other parts of piers and abutments that are buried.

A thin long-dashed line shall show the existing ground line. Do not show centre lines of piers and abutment bearings unless essential for a particular purpose. Dimension the overall length of the bridge, generally the total barrier wall length, the thickness of any pier shafts, and both the required and the actual minimum vertical clearances under the structure, with the location of the critical point indicated. Sometimes it is not possible to indicate this location clearly on the elevation and an indication also has to be given on the plan. Use a leader pointing to a circle with 45° cross. Print in small text footing top elevations, water levels, (see Section 2.6.4) an approximate lower pavement level, bearing types ("exp" or "fixed"), existing ground line designation, minimum soffit elevation for water crossings, railing type, railing anchorage required, rock protection or slope paving notes, and such other notes as are necessary. Note that elevations shall always be given to the top of footings and never to the bottom. If it is necessary to define the bottom of footing level, give the top elevation and dimension the footing depth.

A triangular numbered symbol (optional elevation title) followed by the scale shall show, by reference to the corresponding symbol on the plan, where and in what direction the view is taken.

Cross-Section

The cross-section shall be located to the right of the Plan but may be placed elsewhere if the Plan extends across the width of the sheet. The scale shall be 1:100 if possible. One cross-section is generally sufficient. If the cross-section varies in some way, this may be shown by giving more than one value to a dimension or taking half of the section at one point and half at another. Such a split shall be clearly shown on the plan in association with the triangular symbol and a plane of section line. If dimensions are given showing "varies," limits shall be given.

The cross-section should show the outline of the deck together with barrier walls, railing, wearing surface, exterior beams (others may be shown by beam centre lines), pier cap, pier shaft or columns, typical drainage outlet if any, and utilities.

Centre lines and control lines shall be thin lines and shall be labelled with small text.

Dimensions shall be given showing overall cross-section width, distances between centre lines, curb or median widths, barrier wall face to outer deck edge width, slab thickness, pier shaft width or column diameter and the lane and shoulder widths even if there is no physical demarcation between them. Beam spacing shall be covered by a note stating

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the number, type and spacing of the beams.

The following note shall be given, where applicable, with an arrow pointing to the wearing surface:

"ASPHALT AND WATERPROOFING SYSTEM, 90 mm TOTAL".

No title shall be used, but a triangular symbol, numbered to agree with the one showing the section plane on the plan. This shall appear beneath the Cross-Section, followed by the scale in small text. The direction of the cross-section shall be indicated also by labelling the sides of the bridge "East", "North", etc., in medium text. This is unnecessary if the section is symmetrical. Notes in small text shall be added to show crossfall, wearing surface, number, size and type of ducts, etc. The section shall be taken looking in the direction of increasing chainage.

Crossfall and super-elevation are normally stated as a percentage, for example, 2%. Slopes for embankment slopes, curb side, backslopes and concrete slope paving shall be given by a numerical ratio X:Y, where X is the horizontal dimension and Y is the vertical dimension of a right-angle triangle, the hypotenuse of which parallels the slope, for example 2H:1V. The use of a symbol as shown in the following example is preferable:



Bench Mark

In the lower left corner of the drawing, give in medium text, the elevation of the benchmark to be used, e.g., BM 217.565 and below it in small text, the datum and description of the bench mark given on the E Plan (Bridge Site Plan).

General Notes

In a column about 150 mm wide below the title block and under the title "General Notes" in large text, give the notes in small text as specified elsewhere (See Section 2.6.7). These include Construction Notes.

List of Drawings

Below, or if necessary, to the left of the general notes, list vertically the number and title of all drawings forming part of the structural contract documents (see Section 2.6.2). This may include embedded work drawings. For reference to SSD, OPSD and MTOD, see Section 2.6.13.

Profiles

To the left of the general notes, show diagrammatically the profiles of all roadways, railroads and if available, new channel beds. Roadway profiles should be given only to top of pavement. The profile line should be a medium line. In thin lines, show tangent extensions and PI's. The location and length of the structure should be shown on the profile where appropriate. A title in large text is required below each profile giving the name of the profile, under which "N.T.S." should appear in small text. At each end of each

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profile, print in medium text, "North", "East", etc. The location of the profile (e.g., "COUNTY RD. 10") should be shown in medium text.

In small text, give PI elevations and stations, curve visibility and length and grades. Directions (e.g., "To Beaverton") at each end of the profiles are helpful and shall be given especially if the compass directions could be misinterpreted.

Miscellaneous

When road, rail or water traffic is permitted under the structure during construction, construction clearances shall be shown and unless these can be clearly specified by a single dimension on the elevation, a separate diagram is necessary. If the clearances are standard, OPSD 3390.150 can be referenced. A separate final clearance diagram is always required for a railway opening and this shall show construction clearances if these are less than final clearances.

Non-standard backfill requirements shall be shown as specified in Section 5.2.1.2.

Functions of skew angles shall not be given.

Hatching shall be used where required to show a specific material (asphalt, etc.) or differences in materials (concrete, steel, etc.). In rehabilitation projects, hatching is used to indicate areas to be removed.

The signed P.Eng. stamps of the designer and checker shall appear to the left of the revision block (bottom right-side corner). P.Eng. stamps shall always be signed and dated.

Preliminary Version of the General Arrangement

The preliminary version shall be the same in content and format as the general arrangement. On the preliminary version however, it is not necessary to show a drawing list, and the drawing number shall be "P1" rather than 1. The title shall be "General Arrangement".

The "Design/Check" initial block need not be completed at this stage, but the "Drawn/Check" block must be. A P.Eng. stamp is not required.

The profile shown on the preliminary version of the general arrangement drawing may in some cases be a proposal by the designer, necessary because of insufficient or excess clearance. If so, this shall be clearly noted, and some assurance that it is feasible shall be obtained by telephone before issuing the preliminary version. The note shall not appear on the final drawing.

2.6.9 Use of Word "MINIMUM" on Drawings

There is a possibility of a contractor taking advantage when the word "minimum" is used to specify the depth of footing or the depth to which a footing is to be recessed into rock. When the word "minimum" is used in such a manner, the Ministry may be obliged to pay

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for unnecessary over excavation and additional concrete.

The word "minimum" shall not be used in such cases. The required depth of footing and the required recess into rock should be shown (see Section 4.2.2). The word "minimum" shall not be used to specify the strength of concrete, as it results in a conflict with the strength requirements in OPSS 1350. Concrete shall be specified by metric Specified 28-day Compressive Strength (see Section 2.4.1).

2.6.10 Construction Staging and Sequence

When the bridge is constructed in two or more stages, a construction staging drawing shall be included in the contract drawings which includes a typical section in each stage, along with a list of the work to be done in each stage.

When tasks within a construction stage are provided, the words "suggested" or "proposed" shall not be used in notes on drawings.

For new bridges, a construction sequence shall not be provided unless required by Clause 1.4.4.3 of CHBDC.

2.6.11 Site Numbers

Every structure owned by the Ministry, or located within the Ministry's right-of-way, is identified by an alphanumeric structure identifier which is commonly known as the "site number".

Based on the descriptions below, the following highway structures shall receive a site number:

- 1) Bridge, with span 3m or larger.
- 2) Culverts (buried structures), defined as follows:
 - a) all culverts with spans 3m and greater, or
 - b) all multi-cell and adjacent culverts, separated by not more than the span of the smallest culvert, with individual span greater than 2 m, or
 - c) all other culverts of importance that are managed by the Regional Structural Sections.
- 3) Tunnels, with a minimum length of 90 m (<90m may be categorized as a bridge or culvert). Tunnels are typically constructed below water level or through soil, used to convey pedestrian, highway or railway traffic, and is usually incorporated with some other design considerations (e.g., special lighting, ventilation, fire protection systems or emergency egress measures).
- 4) Retaining Walls:
 - a) with exposed wall height greater than 2 metres, or
 - b) within 2 times the wall height from a travelled lane, or
 - c) whose failure would cause significant adverse effects (e.g., a wall near utilities, or wall near limit of MTO property, or wall retaining superimposed surcharge

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loads).

Retaining walls are deemed to be Standalone by:

- i) being built away from (not immediately adjacent to) a structure (bridge, culvert, etc.), or
- ii) being built immediately adjacent to a structure, but exceedingly longer than typical – in which case a construction joint, different year of construction, different material, or other convenient point – is used to differentiate the portion of wall associated with the bridge from the standalone wall.

By extension, they are not deemed Standalone if they are reasonably short and can be inspected and/or rehabbed easily with the structure, as a result simply become an element of the bridge, culvert, etc. (see also, Figure 2.6.2).

- 5) Ferry Docks, used to convey passengers and motor vehicles encompassing docks, piers, wharfs, ramp and other accessories.
- 6) Sign Support Structures that extend over the roadway, including overhead span, cantilever, bridge mounted and pole mounted types as well as tolling gantries.

Once a site number is assigned to a structure, the site should retain this number even after the structure has been modified or replaced with the exception of its 3rd segment (i.e., it's suffix). For example, in situations where a bridge changes to a culvert, or vice versa, the 3rd segment would change to suit the type of structure and similarly, in situations where a bridge becomes twinned or part of a core/collector system by the addition of structure(s), the 3rd segment shall be modified to follow Figure 2.6.3. Where twin structures are merged, they should change their previous "/1" and "/2" numbering and become "/0", unless there are unusual circumstances that benefit the management of the structure.

In preparing contract documents for new bridges, a bridge replacement or bridge rehabilitation, such that the existing site number was removed, MTOD 3941.210 should be used to have the site number cast back on the bridge. The MTOD also includes a year of construction – which shall be the year of superstructure construction, not the year a particular less extensive rehabilitation is being done (i.e., barrier wall and patch, waterproof and pave).

The following precise format for site numbering was established in May 2018 and is required for managing all assets digitally. The site number is composed of 9 alphanumeric characters with no spaces in-between. The site number is divided into three segments by the symbols "-" and "/":



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1st Segment:

The first two boxes are used to identify the county, based on the counties in existence in the 1960's when the county numbering system was first developed. For a single digit number, a leading zero is required. The last box is used for the county code. For counties 38, 39, 41 or 48 in the large northern regions, county code can be equal to N, S, E, W or C. For all other counties, county code is X.

03X Carleton	37X York	39E Cochrane	48C Thunder Bay
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2nd Segment:

These four boxes are used to give a unique identifier for each structure within a given county, using a number from 0000 to 9999. If the site number is less than 4 digits, leading zeroes are required to fill all the boxes.

Note: Do not duplicate the 2nd segment number for different structure types (i.e., a bridge with a Stand-Alone retaining wall etc.) with the exception of bridge mounted signs.

03X-0225	37X-1206	39E-0201	48C-0087
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3rd Segment:

The first character is used to identify the type of structure.

B	- Bridge (except ferry dock bridges)	39E-0201/B0
C	- Culvert	21X-0495/C0
T	- Tunnel	06X-0709/T0
W	- Retaining wall	24X-0792/W0
F	- Ferry dock structure	17X-0223/F0
S	- Overhead sign support structure	33X-0406/S0

The second character is used:

- a) For bridges culverts and tunnels, as the traffic stream (e.g., direction NB) and facility (e.g., Core, Collector, etc.), identifier is as follows:

0	When there is only one structure at the site – Note: Multi-cell culverts, as described above, are considered one structure.
---	---

Note: Typically, a single structure always receives a single number (e.g., "/B0", "/C0").

However, CR has situations where they have treated a single freeway bridge as separate structures (i.e., B1, B2, B3, etc.) as shown below based on travel streams on the bridge. This typically occurs on an overpass or river crossing structure which is sufficiently wide (e.g., a rigid frame or culvert) and carries several travels streams (e.g., core and collector) or directions.

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This unusual practice is intended to be limited to the Greater Toronto Area where core and collectors exist. All other typical overpass and underpass structures that do not carry 'core + collector' should generally not be divided into multiple site numbers, since these structures can be inspected and rehabbed as one structure.

When there is more than one structure at the same site location:

1	For: - bridge carrying - culvert under - tunnel carrying	NB or EB lanes in twin structures or NB or EB collector lanes of core/collector system
2		SB or WB lanes in twin structures or SB or WB collector lanes of core/collector system
3		NB or EB core lanes of core/collector system
4		SB or WB core lanes of core/collector system
5, 6 etc.	For other structures (e.g., ramps) at the same site. A sequential number is assigned to each structure. See example below in Figure 2.6.3.	

Note: For sites on highways that change directions at times the site numbers should be created according to the predominant direction of the highway (Ex. 401 is East/West, QEW is North/South).

b) For Overhead Sign support structures, as the sign number at the site, which is as follows:

0	For all sign support structures except bridge mounted supports and tolling gantries
1,2 etc.	For bridge mounted sign supports on the same bridge site, sequential numbers are assigned. The same 1st and 2nd segments as the bridge should be used. See example.
	For tolling gantries, which are always found in pairs, assign 1 to the first structure and 2 to the second structure based on the direction of traffic. See Figure 2.6.3.

c) For standalone Retaining walls, as the wall number, which is as follows:

0	For a standalone retaining wall.
1,2 etc.	In virtually all cases, "0" is used, but other sequential numbers may be used for unique situations.

Note: Retaining walls, either RSS (MSE) or concrete walls, that are associated with the structure should be entered as an Element within the structure, and a separate Site Number is not required.

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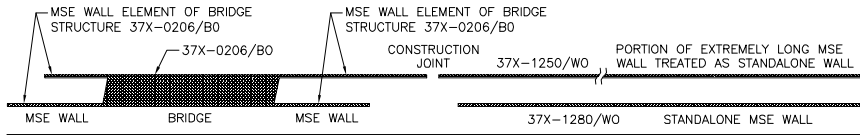


FIGURE 2.6.2 – STRUCTURE IDENTIFICATION OF MSE WALLS

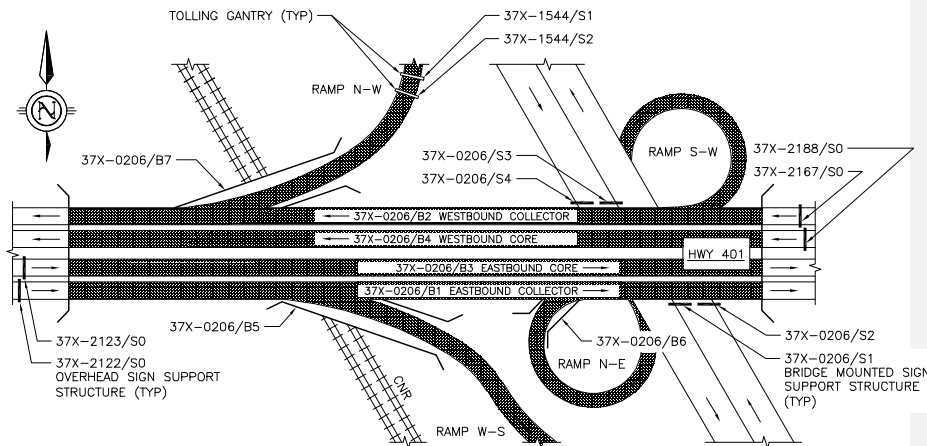


FIGURE 2.6.3 – SITE NUMBERS FOR BRIDGES AND SIGN SUPPORTS

2.6.12 Structure Name

The structure name appears in the Structural Design Report and is typically assigned based on the following:

1. The bridge shall be identified in relation to the highway it is on, and where highways are concurrent, the more major highway taken using the following order, QEW, then the 400 series highways, numerically starting from 400, then other highways numerically starting with highway 2.
2. The major obstacle being crossed appears in the bridge name,
 - a. when multiple obstacles are being spanned, the most significant obstacle name is provided; with the most significant obstacles being bodies of water, then railways, then roads.

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- b. Where the body of water is small, and/or the roads are major, the order may be altered. An additional obstacle being crossed may be added to the bridge name when multiple significant obstacles are crossed.
 - c. Where twin bridges exist, the highway direction (and Express and Collector Lanes) shall be included in the name - See Figure 2.6.3.
 - d. Where the same obstacle is crossed multiple times, a geographic location may be added for clarity (i.e., south crossing, at Hearst, etc.).
 - e. A complex interchange shall contain the bridge number.
3. The next part of the name shall be whether the major crossing is passing over or under the obstacle, using the terminology from Figure 2.6.4.
- a. When crossing water, "Bridge" shall be added and "Overpass" is not required, while for roadway crossing, "Bridge" is not required, and "Overpass" or "Underpass" is required.
 - b. Abbreviations U/P, O/P, O/H maybe be used.
4. Where required for clarity, the highway number may be added at the end of the bridge name.
5. Interchange should be added to the name when applicable, which may be abbreviated as I/C.
6. When approved by the Assistant Deputy Minister (ADM) of Transportation Infrastructure Division (TIMD), or when pre-existing, the bridge may be named after a person.
- a. The name description in #2 shall become the secondary name, unless short enough to fit in the primary name.
 - b. For bridges named for fallen officers, this shall appear as a secondary name.
 - c. The secondary name may be added to the title block, when space permits.
7. The bridge name is not legally binding and may be changed if the highway or waterway changes names. The former name may be retained as the secondary name.

Some examples of bridge names are shown in below:

Primary Name	Secondary Name
Keele Street U/P (I/C on Hwy. 401)	
Glendale Ave. NBL O/P (I/C) (on Hwy. 406)	
Etobicoke Creek Bridge WBL Express (Hwy. 401)	
Hwy. 40 O/P (on Hwy. 401)	Sgt. Margaret Eve Bridge

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CNR O/H @ East of Rosspoint (on Hwy. 17)	
Sgt. Aubrey Cosens V.C. Memorial Bridge	Montreal River Bridge @ Latchford (on Hwy. 11)
Trout River Bridge (on Hwy. 11/17)	
Jones Creek Culvert East Crossing (on Hwy. 15)	
McLean Young Bridge over Penetangore River (on Hwy. 21)	
Hwy. 400/401 I/C Bridge #5	401 W.B. RAMP TO 400S.B.
CNR Subway @ Pickering (on Hwy. 401)	
Welland River/CPR Bridge (on QEW)	
Magnetawan River/Hwy. 520 Bridge (on Hwy. 11).	
Still River/Hwy. 526/CPR Bridge (on Hwy. 69)	

Sign supports shall be named as per the Ontario Sign Support Inspection Manual. Retaining walls shall be named after the highway they are associated with, the nearest road or feature, and the ordinal or quadrant (e.g., N, S, NE, etc.) they are located in.

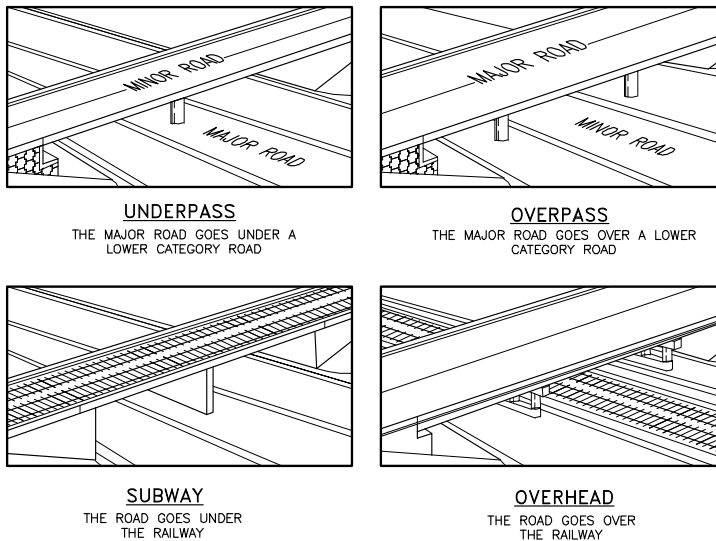


FIGURE 2.6.4 – UNDERPASS, OVERPASS, SUBWAY AND OVERHEAD BRIDGES

2.6.13 Small Size Standard Drawings

For inclusion with contract drawings, small size drawings bearing an SS number shall be added to a full-size standard drawing sheet as per Section 1.5.1 of this manual.

The title block of the full-size drawing shall be completed. The title should be "Standard

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Details I" or II, (etc.). Initials are required as per Section 1.5.2 of this manual.

2.6.14 Referencing Standards on Structural Drawings

OPSD and/or MTOD only, shall be listed on the particular drawing showing the details to which they apply. The number and title of the standard shall be listed just above the revision block under the heading "Applicable Standard Drawings".

If there is any ambiguity as to a standard's application, reference to the standard drawing shall also be made in close proximity to the affected detail.

2.6.15 Drafting Scales

For Structural drawings, the following scales are acceptable:

- 1:20, 1:25, 1:50, 1:75, 1:100, and 1:125.

The scales above are widely used in other highway design work and are preferred. The recommended range of scales may be extended, provided that the required scale is derived from a recommended scale by multiplying by whole number powers of 10 (e.g., 1:2, 1:20, 1:200). Scales with decimal number denominators are not acceptable such as 1:12.5.

In addition, the following intermediate scales may be used to avoid detail views on excessively large or small drawings. They may only be used in such cases.

- 1:30, 1:40, 1:60, 1:150, and 1:250.

2.6.16 Revisions to Drawings

The following procedure shall be followed in making revisions to drawings for all structures, including culverts and sign supports.

a. For all Structures not Requiring Railway or CNWA Approval

- 1) Revisions Prior to Submitting Contract Package for Tendering
 - Revisions generally may be made freely
 - Revisions shall be conveyed to Structural Section and Project Manager to ensure implications to the broader project are understood.
- 2) Revisions During Tendering Period, but Prior to Award
 - Sheet number shall have a letter to designate the revision number (i.e., Sheet 22A, 22B, etc.).
 - General description of revisions added to revision block.
 - Actual revision on drawings shall be issued with an addendum and may be highlighted with revision cloud as directed by Project Manager or Contract Management Office.
 - Must allow for at least 10 business days for most contracts before tender close, or tender close must be extended. Details shall be obtained from Project Manager or

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Contract Management Office as some larger contracts require a longer time.

- Drawings shall be sealed in the same manner as drawings prepared for tender.

3) Revisions to Contract Package after Award

- Sheet number shall have a letter to designate the revision number (i.e., Sheet 22A, 22B, etc.).

- General description of revisions added to revision block.

- Actual revision on drawings shall be highlighted with revision cloud.

- As-built drawings should include all changes, revisions from the tendered package after completion of the contract.

- Drawings shall be sealed in the same manner as drawings prepared for tender.

b. For all Railway Overheads and Subways, and Structures Subject to the CNWA

1) Before the First Submission to the Appropriate Authority for Approval

The procedures for the revisions outlined in section (a), paragraph 1 applies.

2) After the First Submission to the Appropriate Authority for Approval/Record

The procedures for revisions outlined in section (a), paragraph 3 applies, when changes affect the design requirements such as clearances, etc. affecting the authority.

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SECTION 3 - DEEP FOUNDATIONS

3 DEEP FOUNDATIONS**3.1 General**

Deep foundations achieve the required foundation capacity by transferring the superstructure loads to deeper layers of soil, which are either more competent than the top layers or soil or less susceptible to erosion. They can generally be divided into two installation methods; driven piles and drilled shafts.

3.1.1 Design Criteria

Prior to the initiation of the foundation engineering investigation, the structural Engineer shall provide the geotechnical Engineer with a conceptual layout of the proposed structure foundations.

The subsequent Foundation Investigation and Design Report (FIDR) produced by the geotechnical Engineer shall contain information and recommendations with respect to deep foundations. When deep foundations are selected, the final design shall be developed through consultation between the structural Engineer and the geotechnical Engineer.

In the Foundation Investigation and Design Report, the geotechnical Engineer shall provide recommendations for:

- i) Type of foundations, e.g., spread footing, driven piles, drilled shafts, etc.;
- ii) Axial and horizontal resistances for a single pile with the provision that these values may require modifications to account for the pile group effect once a proposed pile layout is known. They shall be given as follows:
 - Factored axial and horizontal resistances at ULS where the resistances are based on the geotechnical resistance of the piles.

(N.B. Factored geotechnical resistance at ULS = Ultimate geotechnical resistance x resistance factor);
 - Axial and horizontal resistances at SLS for identified settlements or displacements and corresponding subgrade reactions. Where deformations of piles are larger than 15 mm, including integral abutments or foundations elements that provide lateral resistances under any loading conditions, p-y curves shall be provided. Pile resistances at SLS should be determined from the consideration of stress/strain characteristics of the pile and unfactored geotechnical parameters of the soil appropriate to the conditions of the site.

(N.B. Geotechnical reaction at SLS = Values calculated for specific settlement or displacement based on stress/strain performance);
- iii) Depth to which the pile should be driven;

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- iv) Requirements for pile tip reinforcement to be used;
- v) Any pile driving constraints licence (e.g., noise, driving hours) and obstacles (e.g., boulders).

Pile resistances given in the Foundation Investigation and Design Report are based on the assumption that the piles will be driven into soil that provides full lateral support against buckling. The structural Engineer may need to modify the resistances, taking into account the un-supported length of the pile, if piles are driven and left partially exposed or immersed in water.

Minimum pile length should be kept to 5.0 m. When piles less than 5 m in length are required for pile foundations, the elastic shortening of the piles is not sufficient to distribute the loads according to the inertia method and an advanced method of analysis shall be used. Piled foundations with piles less than 3 m long shall not be used. In such cases, alternate foundation systems such as spread footings on compacted granular pad (engineered fill) or on mass concrete should be investigated.

Where pile lateral displacement exceeds 15 mm, p-y curves shall be used in the final structural design.

3.2 Driven Piles

Driven piles are appropriate for a wide range of subsurface conditions encountered in Ontario and the resistance may be achieved by end bearing or shaft friction, or a combination of both. For steel piles which rely on shaft friction to develop their resistance, the section with the highest surface area to cross-sectional area shall be used unless driving conditions dictate a larger size of pile.

3.2.1 Design

The designer shall specify the pile tip treatment and embedment of the piles into the pile cap and shall account for deterioration of the pile over the design service life of the structure. Steel piles shall be embedded a minimum of 300 mm into the pile cap when the foundation relies only of the axial resistance of the pile (e.g., a single column supported on a pile cap with grid of piles, battered to resist horizontal loads), and a minimum of 600 mm where the foundation relies on transfer of bending into the pile (e.g., integral abutments).

3.2.1.1 Pile Tip Treatment

The following recommendations on pile tip treatment are guidelines only and may be superseded by the geotechnical Engineer on a project-specific basis:

- 1) For piles driven to a specified elevation in soft or moderate driving conditions, no pile tip treatment is required;
- 2) For piles driven to end bearing on a hard material, through materials presenting only soft to moderate driving conditions and no obstructions or only minor obstructions,

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reinforced flanges per OPSD 3000.100 or OPSD 3001.100 respectively for H-piles or tube piles are required;

- 3) For piles driven to end bearing on hard material, through material presenting hard driving or obstructions such as boulders, bearing points such as the Titus 'H' bearing pile point or APF hard bite, standard model shall be specified;
- 4) For piles driven to and seated into bedrock (usually where the angle of intersection of pile and bedrock is less than 60°), Oslo Point rock points, as per OPSD 3000.201 shall be specified and set into bedrock as detailed in the contract.

3.2.1.2 Corrosion of Steel Piles

The corrosion rate of steel piles embedded in soil is influenced by a number of factors such as oxygen availability, pH, chloride content, sulphate content, sulphide ion content and soil moisture content. Measurement of these parameters can give an indication of the corrosivity of the soil; however, because of the number of factors involved and the complex nature of their interaction, actual corrosion rates are challenging to estimate.

In general, the corrosion behaviour of steel piles embedded in soil can be divided into two categories, corrosion in disturbed soil and corrosion in undisturbed soil. A disturbed or freshly placed soil is defined as a soil in which digging, backfilling or other soil upheaval has taken place allowing the creation of an oxygen-rich environment. Driven steel piles generally have the majority of their length in undisturbed soil; however, excavation and backfilling for footing and pile caps create a region of disturbed soil near the top of the piles, increasing the availability of oxygen and the opportunity for corrosion.

Although this may not be an issue with piles under axial compression alone, as is the case with pile groups under piers or typical abutments in bridges with joints, in integral abutments the piles are under coincident axial compression and bending and the consequences of section loss due to corrosion become more serious.

In typical integral abutment situations, the upper three metres of the pile is encased in a corrugated steel pipe which is subsequently filled with uniformly graded loose sand. This upper part of the pile is in an oxygen rich environment and although the backfill is usually non-aggressive. After a number of movement cycles, there is a strong probability that there may be a void between the bottom of the concrete abutment wall and the top of the sand leaving that part of the pile exposed to oxygen. In addition, the pile at this location is usually under the maximum coincident axial compression and bending making any section loss critical.

Consequently, design Engineers shall account for a loss of 1.0 mm steel thickness per exposed face of steel piles in integral abutments instead of the allowance required by the CHBDC Clause 2.3.6.11 Steel pile corrosion protection.

Steel piles shall not be used in corrosive ground water. Precast concrete piles may be used under these conditions, if specified with sulphate resisting cement.

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The designer shall not use steel piles that are exposed above ground or above low water level without approval from the Structural Section. When approved, exposed piles shall comply with the following criteria:

- 1) Exposed piles shall have an additional sacrificial steel thickness added based on the chemical properties of the soil or water in which they are located and be coated, and/or protected from corrosion by other means, over a length of at least 1000 mm above the high water level to 600 mm below the low water level or 1000 mm below ground elevation;
- 2) When concrete-filled circular steel piles are exposed, they shall have a diameter of not less than 450 mm.

3.2.1.3 Other Considerations

- 1) Refer to Section 6 of the CHBDC for minimum pile edge distance, embedment and spacing.
- 2) Maximum batter for all piles is 1:3. The geotechnical Engineer must approve exceptions to this.
- 3) Pile driving tip reinforcement and rock points are to be used for steel piles only when specified in the Foundation Investigation and Design Report (see Section 3.2.3.1).
- 4) Accessibility for pile driving equipment, allowing for batter, and any infringement on required construction clearances during driving should be considered.
- 5) The possibility of causing damage to buried utilities by driving piles must always be considered. Generally, if there are utilities within 3 m of piles measured at the elevation of the utility, the piles should be pre-augured to an elevation below the elevation of the utility.

3.2.2 Pile Materials

Steel piles are typically used with CSA G40.20/G21 350W and ASTM A572 grade. The ministry has recently used for higher grade H-piles in various projects and ASTM A913 450 grade is readily available for some common sections. When considering the use of high grade of H-piles, the designer should check the availability of sections in Ontario.

3.2.2.1 Steel H-Piles

Steel H piles are adequately described by giving the standard steel section designation only, e.g., HP 310x110. The most common sizes specified in Ontario are HP 310x79, HP 310x110, HP 310x132, and HP 360x108.

For these H-piles, the width-thickness ratio requirement given in Section 10.9.2.1 of the CHBDC, $\left(\frac{b}{t} < \frac{200}{\sqrt{F_y}}\right)$, need not be satisfied for them to be treated as Class 3 sections.

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The type and grade of steel shall be specified by adding the following note to the foundation drawing:

STEEL H-PILES SHALL CONFORM TO CSA G40.20/G40.21 GRADE (Note: The designer shall specify Grade 350W or Grade 450W depending on the structural capacity required).

MTO allows substitution of G40.21/G40.21 piles with ASTM A572/A572M or ASTM A913/A913M. The design shall be based on the yield strength of the grade of product per G40.20/G40.21, and the imperial equivalents are deemed acceptable substitutions. Splices for steel H-piles shall be according to OPSD 3000.150.

3.2.2.2 Steel Tube Piles

When steel tube piles are used in the design, the pile cross section must be specified by giving the outside diameter and the wall thickness as part of the pile data. The piles may or may not be filled with concrete, depending on soil conditions. ASTM A252 is the most available tube for piles. Grade 3 has a yield strength of 310 MPa, and the design should use the nominal wall thickness.

Steel tube piles with longitudinal or helical butt splices shall be used.

The most common sizes are O.D. Diameters of 324, 356, 406, 508, 610 mm with wall thickness of 9.5, 11, and 13 mm.

The type and grade of steel shall be specified by adding the following note to the foundation drawing:

STEEL TUBE PILES SHALL CONFORM TO ASTM A252 GRADE 3.

Splices for steel tube piles shall be according to OPSD 3001.150.

3.2.2.3 Wood Piles

When wood piles are used in the design the pile size must be specified as part of the pile data table by giving the following:

- i) Species;
- ii) Minimum diameter at extreme butt or large end (mm); and,
- iii) Minimum diameter at tip or small end (mm).

The sizes of wood piles which are normally available in Canada are given in the publication Wood Piles published by the Canadian Wood Council or in CSA-056 Round Wood Piles.

Sometimes wood piles are designated as "Size 30." Size designation in this case refers to the minimum diameter at the extreme butt or large end in centimetres i.e., Size 30

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means the minimum diameter of the pile at the large end is 30 cm or 300 mm. The most common sizes used by MTO are Size 30 and Size 36.

The document Wood Piles referred to above is a useful aid for the design, protection, construction and specification of wood piles in structures.

Splices in wood piles shall not be permitted.

Refer to Section 9 of the CHBDC for restrictions concerning the use of untreated wood piles, which are less expensive than treated wood piles. Refer to Section 15.2.3 for treatment requirements. Where required the type of preservative treatment must be given.

The maximum length of timber piles is typically 15 m or less.

3.2.2.4 Precast Concrete Piles

Proprietary precast concrete piles may be used with permission of the Structures Office Manager.

3.2.3 Drawings**3.2.3.1 Pile Data Table**

A table or a statement entitled "Pile Data" is required on the foundation layout drawing giving the number, lengths, batter, cross section, and type of piles. The length should be the length measured along the pile between cut off and tip elevations given or estimated by the design Engineer, rounded up to the nearest 0.5 m.

Pile notes are required close to the pile data table as appropriate. See also Section 3.2.3.3.

The following are typical pile notes for various conditions:

- 1) PILE SPACING IS MEASURED AT THE UNDERSIDE OF FOOTINGS.
- 2) PILE LENGTHS SHOWN ARE THE THEORETICAL LENGTHS BELOW CUT-OFF.
- 3) THE PILE DRIVING EQUIPMENT SHALL BE APPROPRIATE TO THE DRIVING CONDITIONS AND CAPABLE OF DELIVERING A MINIMUM SPECIFIED HAMMER ENERGY OF ... kJ.
- 4) PILES SHALL HAVE REINFORCED TIPS AS PER OPSD 3000.100 or OPSD 3001.100 TYPE ... OR AS APPROVED. *(Specify if pile reinforcement is required, see also Section 3.2.1.1.)
- 5) PILES SHALL HAVE DRIVING SHOES AS APPROVED.
- 6) PILES SHALL BE FITTED WITH ROCK POINTS AS PER OPSD 3000.201. (Specify if rock points are required, and if so, the type. See also Section 3.2.1.1.)

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7) PILE SPLICES SHALL BE AS PER OPSD 3000.150 or OPSD 3001.150 AND IN ACCORDANCE WITH OPSS 903.

* Titus "H" Bearing Pile Points or APF Hard Bite are alternate products that are acceptable.

3.2.3.2 Pile Driving Control

Pile driving control has been done using the Hiley formula for driven piles with uncertain end bearing or friction. Recently, the ministry has been testing a newer method, Pile Driving Analyzer (PDA), which appears to be more accurate than Hiley, and with less scatter of results. For the time being, use of PDA requires approval of the Foundation Section and is not applicable for general use.

3.2.3.2.1 Hiley Formula

Pile driving control is a construction technique that is used in the field to control pile installations and thereby provide some assurance about the validity of design assumptions.

MTO's pile driving control tool is the Hiley formula. This is an empirical formula that models pile behaviour by relating the energy of the hammer blow to the penetration of the pile and rebound of the hammer. That is, it is a monitoring tool, not a design tool. It provides a reasonable approximation of actual pile resistance (for piles that are essentially friction-type rather than end-bearing) in non-cohesive soils, but not in cohesive soils unless the excess pore water pressures are accounted for or allowed to dissipate. Note that the hammer has to rebound enough to maintain its energy per blow and hence the soil must provide sufficient rebound for the Hiley formula to be effective.

In order to minimise misinterpretations of the Hiley formula that have occurred in practice, Structural Standard Drawing SS103-11 has been revised, and renamed "Pile Driving Control." Instead of the graphical method that was used in the past, SS103-11 now gives only a calculation method to determine R, the ultimate pile capacity. The method of applying the Hiley formula, as well as some explanatory notes are given on the Structural Standard Drawing and are further elaborated here below.

When applying the Hiley formula, hammers should be operating at 100% of their available capacity. That is, for example, controls for diesel hammers should be turned to full capacity.

The formula for use with drop hammers and single-acting steam hammers is:

$$R = \frac{n e_f W g H}{S + \frac{C}{2}}$$

$$e_f = 0.75 \text{ for drop hammers}$$

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The formula for use with double-acting, differential-acting steam and diesel hammers is:

$$R = \frac{n e_f E}{S + \frac{C}{2}}$$

$e_f = 0.6$ to 0.8 for steam hammers

$e_f = 1.0$ for diesel hammers

Diesel hammers are currently the most commonly used type.

Where:

$R =$ Ultimate pile resistance (pile capacity) by Hiley formula [kN]

$$n = \text{Efficiency of blow} = \frac{W + P e^2}{W + P}$$

$e_f =$ Efficiency based on gross manufacturer's rated energy
(typically 0.6 to 0.8)

$W =$ Mass of components (ram) delivering blow (from tables) [kg]

$g = 9.8$ [m/s²]

$H =$ Height of free fall of mass [m]

$S =$ Measured penetration of pile per hammer blow [mm]

$C =$ Measured rebound of pile per hammer blow [mm]

$E =$ Maximum rated energy of hammer (from contractor, or tables)
[Joules/blow]

$P =$ Mass of components receiving blow (pile + cushion + anvil) [kg]

$e =$ Coefficient of restitution

$e = 0.25$ for timber pile using cushion

$e = 0.32$ for steel pile using cushion

$e = 0.55$ for steel pile without cushion

A) Design Stage

The requirement for pile design is as follows:

Factored geotechnical resistance at ULS > Design load at ULS

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Factored geotechnical resistance at ULS = Ultimate geotechnical resistance x resistance factor

Factored geotechnical resistance at ULS: Given in the Geotechnical Report.

Ultimate geotechnical resistance: Established by the geotechnical Engineer, based on formulae, field testing, or assessment.

Resistance factor: The factor by which the ultimate geotechnical resistance is multiplied to establish the factored geotechnical resistance at ULS, typically 0.5.

B) Construction Stage

The requirement for pile resistance is as follows:

Ultimate Pile Resistance R > Ultimate Geotechnical Resistance

- The **Ultimate pile resistance R** is calculated in the field by use of the Hiley formula, based on measured observations of pile penetration (S) and rebound (C), and the pile driving characteristics of energy of hammer and efficiency of blow.
- The **Ultimate geotechnical resistance = 2 x design load at ULS** and must be given by the designer in the pile driving notes on the contract drawings.

In the equation above:

- **R** must be greater than **2 x design load at ULS** (rather than **2 x factored geotechnical resistance at ULS**).

The **design load at ULS**, calculated by the structural Engineer is always less than or equal to the **factored geotechnical resistance at ULS** established by the geotechnical Engineer. The factor of 2 needs to provide safety for the actual ULS design load only, in order that the pile not be driven to an unnecessarily high capacity, risking damage during driving.

- The **design load at ULS** is the maximum factored design load per pile at the ultimate limit states, calculated by the designer.

During the process of pile driving and its monitoring, if the required ultimate pile resistance (as calculated by the Hiley formula) is not reached when expected at a prescribed elevation or in a depth of stratum bounded by prescribed elevations, the advice and recommendations of the geotechnical Engineer should be sought and followed.

It should be noted that the Hiley formula incorporates a factor to account for hammer efficiency so that the maximum rated energy should be used in the equation.

MTO's principle alternate pile driving control tool is the Pile Driving Analyser (PDA) which measures force imparted to the pile by measuring acceleration and/or strain of the pile in

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response to blows and through wave equation analysis has the potential to provide a more accurate model of pile resistance. It is used in critical applications that warrant the higher monitoring costs involved.

3.2.3.3 Pile Driving Notes

When piles are specified, the pile driving notes shall be in the form of one of the seven listed below.

When reference is made to SS103-11 (Hiley formula for steam and diesel hammers), it shall be included in the drawings, which is usually the case unless the pile is driven to the bedrock.

Foundation Investigation and Design Reports are to indicate which note is applicable.

1. PILES TO BE DRIVEN IN ACCORDANCE WITH STRUCTURAL STANDARD DRAWING SS103-11 USING AN ULTIMATE GEOTECHNICAL RESISTANCE OF ... KN PER PILE.
2. PILES TO BE DRIVEN IN ACCORDANCE WITH STANDARD SS103-11 USING AN ULTIMATE GEOTECHNICAL RESISTANCE OF ... KN PER PILE BUT MUST BE DRIVEN BELOW EL ...
3. PILES TO BE DRIVEN IN ACCORDANCE WITH STRUCTURAL STANDARD DRAWING SS103-11 USING AN ULTIMATE GEOTECHNICAL RESISTANCE OF ... KN PER PILE BUT NOT BELOW EL ... WITHOUT APPROVAL OF THE ENGINEER.
4. PILES TO BE DRIVEN IN ACCORDANCE WITH STRUCTURAL STANDARD DRAWING SS 103-11 USING AN ULTIMATE GEOTECHNICAL RESISTANCE OF ... KN PER PILE BUT MUST BE DRIVEN BELOW EL ... AND NOT BELOW EL ... WITHOUT APPROVAL OF THE ENGINEER.
5. PILES TO BE DRIVEN TO BEDROCK.
6. PILES TO BE FITTED WITH ROCK POINTS AND DRIVEN INTO BEDROCK IN ACCORDANCE WITH OPSS 903.
7. PILES TO BE DRIVEN TO EL ...

The ultimate geotechnical resistance, given in notes (1) to (4) to be specified = 2 x (maximum factored design load at ULS).

When using notes (5), (6), and (7), do not give ultimate resistance as part of the pile driving note.

In a separate note, headed "PILE DESIGN DATA", the maximum factored design load at the ULS and at SLS should be given and identified as such, e.g., MAX. FACTORED LOADS: ULS 1450 kN, SLS 1055 kN.

SECTION 3 - DEEP FOUNDATIONS

3.3 Drilled Shafts

Drilled shafts are deep foundation elements. Drilled shafts, or caisson piles, are often a viable alternative to driven piles. The casing may be either permanent or temporary based on the recommendations of Foundation Investigation and Design Report (FIDR) depending on the design and/or the site conditions.

3.3.1 Design

The capacity of a drilled shaft may be dependent upon end bearing, side friction or a combination of the two. End bearing resistance is achieved by solid and full contact with the bottom of the rock socket. Drilled Shafts are considered where footing conditions are such that the structure's loads need to be carried to a rock formation and the following conditions exist:

- Shallow foundations and driven piles would not be feasible and economical due to the project specific constraints.
- There are concerns about pile driving vibrations, noise, or overhead clearance.

Completed diameters of caisson piles are based on available drilling equipment in Ontario and 650, 750, 880, 1000, 1180, 1300, 1500, 1800 and 2000 mm sizes are commonly available from the deep foundation contractors.

Design of caisson pile shall be a reinforced concrete column. The amount of reinforcing steel in a cage must satisfy all structural requirements, taking into account combined stresses of axial load, lateral load and bending moment. There should be sufficient clear space between the longitudinal bars as well as the transverse bars or spiral loops to allow free passage of concrete through the cage. Typical reinforced concrete column design requires tight pitch spacing for spirals and concrete placement into the caisson pile becomes often problematic. In order to minimize any caisson installation issues, 15M spiral reinforcement may be replaced by 15M hoops (circular ties) or discrete 20M hoops with spacing 50% larger than 15M bars. In either any case, the spiral pitch or hoop spacing cannot exceed 6 times the diameter of the longitudinal reinforcement, nor 150mm.

3.3.1.1 Permanent Steel Casing

Permanent casing is usually steel pipe having the same nominal diameter as the completed shaft. The permanent casing becomes a form to contain the concrete between the top of the shaft and the rock socket. Permanent casing is left in place after concrete placement. Sealing the permanent casing within the rock socket is important during concrete placement because concrete may leak through any gaps.

3.3.1.2 Temporary Steel Liner

Temporary liners are heavy-walled pipes that are usually driven, screwed, or vibrated into the earth. Drilling may occur either before or after the casing is in place. Temporary liners of a larger diameter and partial length may be used in addition to permanent casings, to reduce the friction of soil when the permanent casing is installed.

SECTION 3 - DEEP FOUNDATIONS

3.3.1.3 Concrete Placement

The method used to place the concrete falls into one of two categories, wet pour, or dry pour. The location of the water table and the type of materials that make up the underlying geology control whether the excavation is considered a "wet" shaft or a "dry" shaft. For wet shafts, the concrete must be placed through a tremie or pump extension so that the groundwater and contaminated concrete is pushed up by a head of concrete inside the tremie as it is being placed in the excavation. ~~For wet shafts, the concrete must be placed through a tremie or pump extension so that the water does not mix with the concrete as it is being placed in the excavation.~~ A wet pour requires the use of a permanent casing.

3.3.1.4 Rebar Cages

A drilled shaft rebar cage is comprised of longitudinal bars that are normally arranged in a uniform spacing circumferentially to form a cylinder that is concentric with the drilled shaft. Transverse reinforcing is placed around and attached to the longitudinal bars.

The most common types of transverse reinforcement in drilled shafts are spirals. 15M spirals are typically used for caisson piles and readily available from rebar fabricators. The tight pitch spacing on spiral reinforcement can often result in constructability issues with concrete flow through the rebar cage. When a design pitch spacing of 15M spirals is less than 80 mm, use of circular ties or hoops with either same bar size and/or larger bar size can allow an increase in the bar spacing as shown in Figure 3.3.1. Welded hoops shall not be permitted. Another solution ~~which can be designed by~~ is to detail 20M spirals with spacing 50% larger than 15M spirals. The designer shall confirm the availability of 20M spiral reinforcement from the rebar fabricators, as they are generally not available in Ontario. In any case, the spiral pitch cannot exceed 6 times the diameter of the longitudinal reinforcement, nor 150mm. Mixing of spirals and hoops within the same section is not permitted.

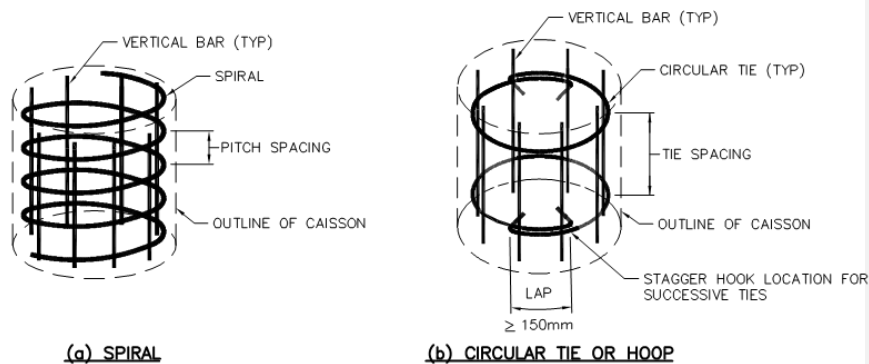


FIGURE 3.3.1 – TRANSVERSE REINFORCING DETAILS FOR CAISSON PILE

SECTION 3 - DEEP FOUNDATIONS

For long cages and cages with large diameter, bracing elements are normally provided by the Contractor to prevent permanent distortion of the cage as a result of stresses induced by lifting and placing, and for safety.

3.3.2 Materials

Normal 30 MPa concrete is used for caisson piles. Reinforcing steel for rebar cages shall be grade 500W.

3.3.3 Drawings

Standards notes for drilled shafts shall be listed on the drawings. The following are typical drilled shaft notes for various conditions.

1. CAISSONS ARE Mm-mm NOMINAL DIAMETER AS SHOWN AND SHALL BE DRILLED AND SOCKETED INTO BEDROCK.
2. MAXMUM COMBINED FACTORED LOADS:
SLS kN PER CAISSON
ULS kN PER CAISSON
3. STEEL CASINGS FOR CAISSONS SHALL CONFORM TO ASTM A252 GRADE 3 MODIFIED (345 MPa). SEAMS OF WELDED CASING SHALL BE SEAMLESS HELICAL BUTT AND COMFORM WITH THE REQUIREMENTS OF CSA W59.
4. CAISSON LENGTHS SHOWN ARE THEORETICAL LENGTH BELOW TOP OF CAISSON AND ARE BASED ON ESTIMATED TIP ELEVATION. FINAL LENGTHS SHALL BE DETERMINED ON SITE FROM DRILLING REPORTS.
5. CAISSON SPACING IS MEASURED AT THE UNDERSIDE OF FOOTINGS.

3.4 Pile Caps

The pile cap is a reinforced concrete slab or block which interconnects a group of piles and acts as a medium to transmit the load from wall or column to the piles. A rigid pile cap shall be assumed rigid in a routine design for narrowly spaced piles so as to distribute the forces equally correspondingly on the piles of a group. In general, it is designed like a footing on soil but with the difference that instead of uniform reaction from the soil, the reactions in this case are concentrated either point loads or distributed. No portion of the dead load is transferred to the soil directly from the pile cap that there is no contact stress. On the other hand, when piles are widely spaced in a pile cap, the compression of the soil below the pile toe level shall be considered as per CHBDC.

The thickness of the pile cap is typically established to resist shear without need for shear reinforcement and should be sufficient for the bars projecting from the piles and the dowel bars for the columns to be developed.

Where a pile cap meets the definition of a deep beam according to the CHBDC, the pile cap shall be designed using a strut-and-tie model.

SECTION 4 - SHALLOW FOUNDATIONS

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SECTION 4 - SHALLOW FOUNDATIONS

4 SHALLOW FOUNDATIONS**4.1 General**

A spread footing is a type of shallow foundation which may be a more suitable option than any deep foundations for specific project circumstances.

Prior to the initiation of the foundation engineering investigation, the structural Engineer shall provide the geotechnical Engineer with a conceptual layout of the proposed structure foundations.

The subsequent Foundation Investigation and Design Report (FIDR) produced by the geotechnical Engineer shall contain information and recommendations with respect to shallow foundations. When spread footings are selected, the final design shall be developed through consultation between the structural Engineer and the geotechnical Engineer.

In the Foundation Investigation and Design Report, the geotechnical Engineer shall provide recommendations for:

- i) Type of material supports and founding elevations for spread footings, e.g., spread footings can be founded on bedrock, on mass concrete placed on either subsoil or bedrock, or granular pad placed on subsoil.
- ii) Factored axial and horizontal resistances at ULS where the resistances are based on the geotechnical resistance of the footings.
- iii) Axial and horizontal resistances at SLS for identified settlements or displacements and corresponding subgrade reactions.

4.2 Design Criteria**4.2.1 Working Slabs/Mass Concrete or Granular Pads**

If required, recommendations for using working slab/mass concrete or granular pad shall be stated in the Foundation Investigation and Design Report. When the founding soils must be covered with the mass concrete within a limited time after exposure to prevent deterioration, the time limit should be noted on the footing drawing.

4.2.2 Foundations in Rock

Without special rock excavation procedures, over excavation may occur periodically. The over excavation can result in either approximately vertical side surfaces or sloping side surfaces.

When footing stability is a concern, the designers shall satisfy themselves about the provision of adequate sliding resistance on rock so that the footing will perform as intended, i.e., resistance against sliding shall be provided by bearing against

SECTION 4 - SHALLOW FOUNDATIONS

approximately vertical side rock surface, or by using steel reinforcement dowels between the rock and the footing or by the rough sloping rock surface provided that:

- a) In the case of over excavation with approximately vertical side surfaces, the over excavation is replaced with concrete of the same Specified 28-day Compressive Strength as the footing concrete;
- b) In the case of over excavation with sloping (greater than about 30° to the vertical) side surfaces, the sloping surfaces are rough (25 mm + deep depressions with jagged edges evenly spaced over about 50% of the sloping surface area) or if they are artificially roughened to the same roughness as above and the over excavation is replaced with concrete of the same Specified 28-day Compressive Strength as the footing concrete.

Since OPSS 902 currently states that over excavation is to be replaced by "a material suitable for the particular application and approved by the Contract Administrator", it is essential that the drawings state that over excavation of rock be replaced by the footing concrete.

Where it is necessary to key the footings into rock the footing drawing shall contain the following notes:

- i. FOOTING(S) SHALL BE SET ...mm INTO SOUND BEDROCK;
- ii. ROCK SURFACES IN OVER EXCAVATED AREAS SHALL BE SUBJECT TO APPROVAL BY THE CONTRACT ADMINISTRATOR;
- iii. OVER EXCAVATION SHALL BE REPLACED WITH CONCRETE OF SAME SPECIFIED 28-DAY COMPRESSIVE STRENGTH AS FOOTING CONCRETE.

4.2.3 Minimum Frost Protection to Footings

The frost protection depth to the underside of structure footings shall be according to the recommendations of the Foundation Investigation and Design Report. Where this is not available the following shall be used:

- OPSD 3090.100 - Contours of frost depths for Northern Ontario;
- OPSD 3090.101 - Contours of frost depths for Southern Ontario.

Rock fill and rock protection shall count for half of their thickness in determining the depth of cover provided.

Footings bearing on sound rock or well drained rock fill on sound rock do not require frost protection; but if the rock fill is on soil, the minimum frost protection depth applies to the soil.

RSS (MSE) wall concrete leveling pads require embedment to provide stability and protection from surface drainage scour as per CHBDC and RSS Design Guidelines.

SECTION 4 - SHALLOW FOUNDATIONS

4.2.4 Scour Protection

When scour of footings is a concern, the designer shall consider and show on design drawings the scour protection requirements of the footings as recommended by the Hydrology Report and/or Foundation Investigation and Design Report.

The minimum depth of embedment in soil or soft rock, e.g., weathered shale, should be 1.2 m. This depth is provided only as a general guide for minor structures where a Hydrology Report is not considered necessary by the hydrologist and where scour is not likely to be a problem.

SECTION 5 - ABUTMENTS, WINGWALLS AND RETAINING WALLS

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SECTION 5 - ABUTMENTS, WINGWALLS AND RETAINING WALLS

5 ABUTMENTS AND EARTH RETAINING SUBSTRUCTURES**5.1 General**

MTO has a strong preference to use rigid frame bridges or integral abutment bridges due to improved long term durability and having fewer components to inspect and maintain in the future (e.g., no bearings). Integral abutments shall be used when possible. If not possible, semi-integral abutment bridges are the next preferred since leakage of the expansion joint does not lead to deterioration of the abutments.

5.1.1 Integral Abutments

Integral abutment bridges are single span or multi-span bridges with a movement system composed primarily of abutments on flexible integral pile foundations and approach slabs, in lieu of movable deck expansion joints and bearings at abutments. The effect of the longitudinal forces in the structure due to temperature, shrinkage and creep is minimised by making the abutment foundations flexible and less resistant to longitudinal movements.

Integral abutment bridges are well-suited for the concrete slab-on-girder type of superstructures for total bridge length of 150 m with thermal movements of a maximum of 75 m length going to one side, and skew angle of less than or equal to 20 degrees. To minimise the effect of soil pressure and resistance to abutment movements, the total height of the abutment wall and length of wingwall should not exceed 6 m and 7 m respectively. The abutment should be supported on relatively flexible piles such as H-piles. Where the load-bearing strata is near the surface or where the use of short piles less than 5 m in length is planned, the site may not be suitable for integral abutment bridges. Integral abutments may be constructed on concrete filled-tube piles, concrete caissons, or on columns supported on spread footings (e.g., spill-through abutments) where these systems can provide the flexibility needed to accommodate the movements from the superstructure.

Short span bridges have been designed with integral abutments supported on narrow spread footings capable of providing a small amount of rotation, similar to the abutments of a ~~cast-in-place rigid frame~~ **cast-in-place rigid frame, and shall be designed as rigid frames**. The behaviour of the structure and its durability is greatly influenced by the movement required and detail of the footing. ~~It is therefore restricted to structures less than 40 m in length.~~

Bridges shall have integral abutments as required by Section 2.1.2. The structure length and skew limitations can be exceeded subject to approval by the Structural Section. Reference should be made to the Structural Office Report # SO-96-01, "Integral Abutment Bridges".

5.1.2 Semi-Integral Abutments

Semi-integral abutment bridges are single span or multi-span continuous deck type bridges with rigid, non-integral foundations and movement system composed primarily of reinforced concrete end diaphragms, approach slabs, movement-accommodating bearings and horizontal joints at the superstructure and abutment interface. This

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arrangement shall only be considered where an integral abutment arrangement cannot be used.

Reference should be made to Bridge Office Report BO-99-03, "Semi-Integral Abutment Bridges".

5.1.3 Conventional Abutments

Conventional abutments are built with deck expansion joints which are suitable for bridges with large longitudinal movements and where use of Integral or semi-integral abutments is restricted by design constraints. Conventional abutments shall be detailed with sufficient space between the ballast wall and the deck end for inspection and maintenance of joints and bearings, as required by Section 13.

5.2 Abutment Design

Beneficial effects of compression in reinforced concrete abutment components shall not be taken into account in the design.

5.2.1 Abutment Wall and Retaining Wall**5.2.1.1 Earth Pressure on Abutment Wall and Retaining Wall**

Backfill arrangements for abutment and retaining walls shall be according to OPSD 3101.150 or OPSD 3101.200.

Conventional abutments and retaining walls should normally be designed for the active earth pressure. This, of course, is contingent on the use of free draining granular backfill and on the type of compaction equipment used within the restricted zone behind the wall. Abutments and retaining walls founded on unyielding material or on short piles may generate earth pressures exceeding the active earth pressure. Integral abutment bridges shall be designed for earth pressure commensurate with the movement of the abutments into the soil from thermal movements of the superstructure.

The 0.8 m height equivalent live load surcharge does not need to be applied to most abutments, with approach slab supported on the abutment ballast wall. Where the length of the approach slab carrying live loads is less than the height of the abutment, the live load surcharge effects shall be considered.

When earth pressure is acting as a resistance, the effects shall be neglected when there is a possibility of the soil being removed, or when the soil is within the frost penetration depth.

5.2.1.2 Abutment Wall and Retaining Wall Drainage Design

Drainage of the granular fill behind retaining walls and abutments should be provided as follows:

SECTION 5 - ABUTMENTS, WINGWALLS AND RETAINING WALLS

- a) For perched abutments "150 mm diameter perforated subdrains" behind abutments and within the granular limits;
- b) For other types of abutments and for retaining walls "150 mm perforated sub-drains and/or wall drains";
- c) Where there is a sidewalk in front of an abutment or retaining wall, drainage should be provided by some means (e.g., sheet drain such as Miradrain) other than wall drains if possible;
- d) Wall drains shall be shown on the drawings by reference to OPSD 3190.100 or, if this is not appropriate, as "75 mm dia. non-metallic wall drains at 3000 mm c/c, elevation to be determined by the Engineer";

The part of the note above concerning the elevation should be used only if the drain elevations are not shown on the drawings. They should be shown if possible. Generally, elevations should be set as low as possible, but at least 300 mm above the level of the ground or normal water level in front of the wall.

A pocket of "open graded 19.0 mm clear stone in accordance with OPSS 1004" should always be shown 0.05 m³ in volume around the inlet to each drain. The drains should be shown level;

- e) Perforated subdrains must be shown on the preliminary and final versions of the general arrangement, on the elevation only. The length, outlets and connections should not be shown.

It is not generally necessary to drain the back of a wall below the elevation of soil or water in front of the wall. Backfill which is not free draining requires additional considerations and requires approval by the Structural Section.

5.2.1.3 Abutment Wall Stem Thickness

In some cases, the required gap between ballast walls and superstructures, and minimum ballast wall thicknesses, leads to excessively thick abutment stems. Where the abutment stem of a tall abutment exceeds 1.2 m in thickness, the back face of the abutment shall be corbelled below the bearing seat ledge to reduce the thickness of the lower portion of the abutment wall.

5.2.1.4 Abutment Wall Vertical Construction Joint

Relatively long and thick abutment walls are prone to vertical cracking due to restrained shrinkage and thermal effects. For abutment walls placed on spread footings or pile caps and when the length of the abutment wall exceeds 12 m, vertical construction joints shall be specified to construct the wall in shorter lengths to control cracking due to restrained shrinkage. Smaller diameter reinforcing steel at tighter spacing is more effective to control cracking than equivalent quantity of steel with larger bars at larger spacing. The reinforcing horizontal steel at the front face of the abutment shall have a spacing of not more than 150 mm. Concrete with a shrinkage ~~compensating-reducing~~ admixture ~~should~~ may be specified for ~~thick-massive~~ walls.

SECTION 5 - ABUTMENTS, WINGWALLS AND RETAINING WALLS

Walls which are placed in one pour and not subject to restraint from adjacent concrete members, as is typically the case with integral abutments, construction joints are usually unnecessary.

5.2.1.5 Abutments to Approach Slabs Dowels

It is important that the stainless steel dowels used to tie approach slabs to abutment ballast walls should be in line with the bottom steel of the approach slabs. If the dowels are near the top of the slab, settlement of the slab could cause a tension crack at the top of the slab that could precipitate a shear failure. In addition, dowels should be hooked so that they do not project beyond the back face of the abutment and obstruct backfill compaction.

The dowels provided shall be S15M @ 150.

5.2.1.6 Staining of Concrete Abutments by Weathering Steel

Where aesthetics is important, the Structural Section may recommend a sealer to prevent rust staining of concrete abutments from exposed steel beams during construction. This may occur during prolonged construction delays, such as a winter shutdown, before placing the deck and expansion joints.

All exposed faces of the abutments encompassing the bearing seats, front and side faces below the bearing seat should be treated before the erection of the structural steel. Preceding the application of the sealer, surface preparation should be carried out as per the manufacturer's instructions. As an alternative to a sealer, a clear curing compound may be applied after the concrete has been cured. Some rust stains may be inevitable, and provision should be made for washing the stains from the concrete after the deck has been cast.

MTO had successfully used "Dekguard system sealer" manufactured by Fosroc Construction Chemicals of Guelph, Ontario as a suitable sealer on several projects.

5.2.1.7 Traffic Barrier in Front of Abutment or Retaining Walls

When abutments or retaining walls are close to travelled lanes, a traffic barrier should be provided in front of the wall face. Possible applications of this treatment should be identified at the preliminary planning stage and discussed and decided by the Project Delivery Section, as it could affect the structure span. Where the wall is within the Clear Recovery Zone and not able to withstand the Vehicle collision load of CHBDC Clause 3.15, the wall should be located outside of the Zone of Intrusion (see Section 2.5.1 for further information).

5.2.2 Abutment Seats and Provision for Jacking

Abutment seats shall be detailed with discrete pedestals for each bearing location. The concrete surface between the pedestals must slope at 5% min. and shall be detailed to avoid potential leakage from draining onto the front face of the abutment.

SECTION 5 - ABUTMENTS, WINGWALLS AND RETAINING WALLS

All bridges supported on bearings shall have provision for jacking. It is feasible to jack from a 5% slope, using shims. The gap provided for jacks should be 250 mm minimum in height.

The unfactored dead load and live load reactions (for jacking purposes) and the permissible location for jacking points should be shown on the drawings. If it is anticipated that the bridge cannot be closed for jacking, live load reactions should be included. MTO's Bearing Replacement and Jacking Guidelines (BRO-062; ISBN 978-1-4868-2356-7) should be referenced for further details.

5.2.1.85.2.2.1 Bearing Pedestals on Abutment Seats

Bearings should be supported on concrete pedestals that are at least 150 mm above the top surface of the abutment wall except for side-by-side concrete boxes with integral abutment bridges, this should be at least 250 mm to allow integral abutment concrete to flow under the boxes. ~~The concrete surface between the pedestals must slope towards the front face at 5% min. under a sealed joint and at 1 in 3 min. under an open joint. In the direction parallel to the front face, this surface should be horizontal for simplicity.~~

5.2.2.2 Rust Stain Control for Steel Girder Bridges

Structural steel bridges require special treatment to prevent rust staining of piers and abutments. ~~For a standard detail on piers, see Section 6.2.3.~~ For a standard detail on abutments, see Figure 5.2.1.

SECTION 5 - ABUTMENTS, WINGWALLS AND RETAINING WALLS

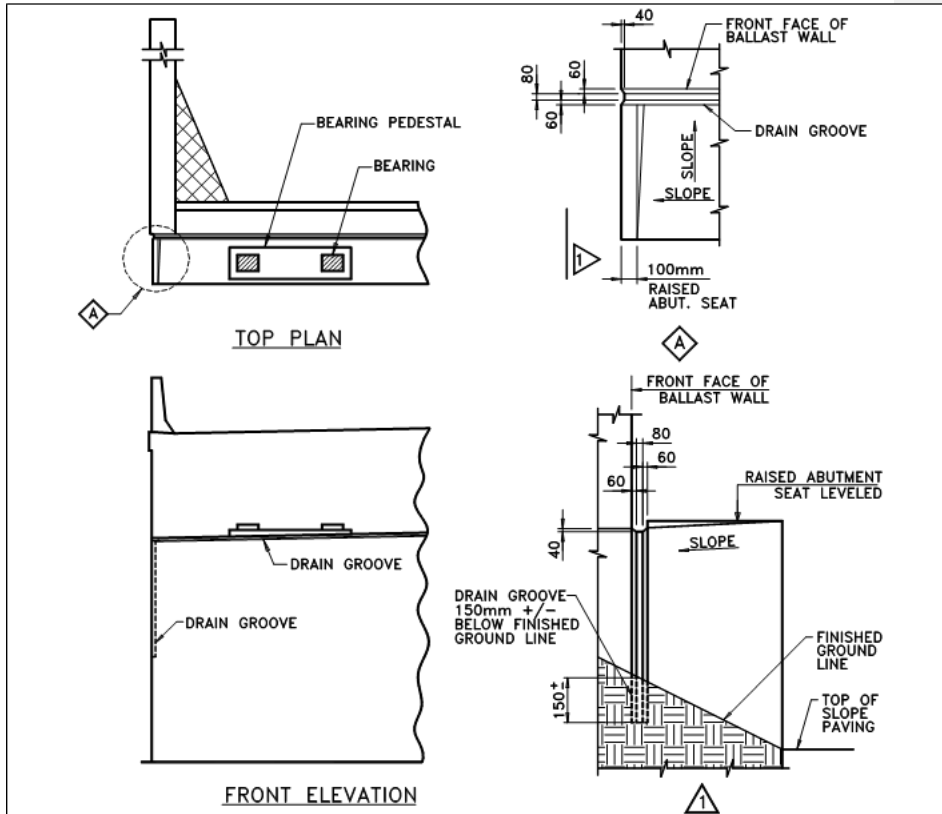


FIGURE 5.2.1 – STANDARD DETAIL FOR RUST STAIN CONTROL AT ABUTMENT

~~All bridges supported on bearings shall have provision for jacking. It is feasible to jack from a 5% slope, using chims. The gap provided for jacks should be 250 mm minimum in height.~~

~~The unfactored dead load reactions (for jacking purposes) and the permissible location for jacking points should be shown on the drawings. If it is anticipated that the bridge cannot be closed for jacking, live load reactions should be included. MTC's Bearing Replacement and Jacking Guidelines (BRO 062; ISBN 978-1-4868-2356-7) should be referenced for further details.~~

SECTION 5 - ABUTMENTS, WINGWALLS AND RETAINING WALLS

5.2.25.2.3 Ballast Walls**5.2.2.15.2.3.1 Ballast Wall Dimensions and Elevations**

Unless required otherwise by design, ballast walls which are not more than 1200 mm high, measured above the bearing seat ledge, shall have a minimum thickness of 380 mm. For greater heights, a minimum thickness of 450 mm shall apply. It is essential however that sufficient space be available between the superstructure end block-out and the ballast wall block-out to accommodate the expansion joint armouring on both sides (see Section 13.2.1). The desirable width of the concrete block-out at each side of the expansion joint is 500 mm to provide space for placing the concrete and for the bent "hairpin" rebars.

There has been some confusion regarding the calculation of elevations to be shown on the top of ballast wall on contract drawings. Standards, such as the OPSD 3370.100 for bridge deck waterproofing, show a 5 mm dimension from the top of ballast wall to the top of adjacent asphalt. The 5 mm represents a local increase in asphalt thickness to account for subsequent compaction by traffic and should not be taken into account when calculating the elevations to be shown on the top of ballast wall on contract drawings.

5.2.2.25.2.3.2 Ballast Wall Construction Joints

- a) For steel and precast concrete girder bridges, the horizontal construction joint between the abutment and the ballast wall should be shown level with the bottom of the bearing seats. Dowels from the abutment will then project up above the bearing seat.
- b) For post tensioned bridges, dowels may interfere with the stressing operation and the joint must generally be stepped down below the bearing seat. This may not be necessary if the anchorages are more than a lap length above the bearing seat.

5.2.35.2.4 Wingwalls

Wingwalls are provided to retain the roadway embankment at the approach of a bridge. The length and the height of the bridge wingwall should be adequate to retain the roadway embankment with an allowable slope and to eliminate the risk of spill through of the abutment backfill. The Ministry allows alignment of the bridge wingwalls either parallel to the roadway or at any angle to the abutment. Wingwalls aligned parallel to the roadway are very common because they confine the approach roadway effectively and reduce the risk of approach settlement. This alignment is also suitable for integral abutment bridges and can be used for supporting the traffic barriers over the wingwalls.

The Ministry had implemented design aids for cantilever type wingwall design in the Structural Manual in 2008. These design aids demonstrate all the applicable loads for the wing wall design for Ministry bridges, however barrier loading was limited to TL-4. Additionally, the Ministry developed and made available one standard structural drawing, SS105-2, for wingwall design with non-integral bridges.

The Ministry has implemented a policy to specify 500W steel reinforcement for new structures and all future designs shall be based on steel reinforcement yield strength of 500 MPa. Therefore, the current design aids have been updated with 500W steel

SECTION 5 - ABUTMENTS, WINGWALLS AND RETAINING WALLS

reinforcement and include traffic impact loading on both TL4 and TL5 barriers. Also, the previous standard structural drawing was renumbered as SS105-11, and a new standard structural drawing SS105-12 for wingwall with integral abutment bridges has been developed.

5.2.3-5.2.4.1 Wingwalls Length

The length of a wingwall, L_w , as illustrated in Figure 5.2.2 is measured from the front face of the abutment to its end. The length of the wingwall must be sufficient so that the slope of the approach embankment along the wingwall meets the back face of the abutment below the elevation of the bridge bearing seats. The length of the wingwall shall be rounded up in 500mm increments. When the wingwalls are aligned parallel to the roadway (U shape), an additional 500mm length must be provided so that the top of the approach slope meets 500mm away from the end of the wingwall to protect against erosion. The Ministry recommends using cleats at the interface of abutment and wingwalls with a minimum dimension of 500mm along the abutment and 1000mm along the wingwalls to minimise concrete shrinkage cracking due to the rapid change in the wall thickness, and to facilitate placement of concrete. The design length of a wingwall with cleat, L is measured from the end of the cleat to the end of the wingwall. Typical wingwalls aligned parallel to the roadway are illustrated in Figure 5.2.2.

The following equations in combination with Figure 5.2.2 can be used to calculate the minimum length of the wingwalls aligned parallel to the roadway:

Length of wingwalls from the front face of the abutment for non-skew bridges,
 $L_w = (E_T - E_L)S + 500mm$

Length of wingwall from the front face of the abutment for skew bridges,

$$L_w = \frac{(E_T - E_L)S}{\cos \theta} + 500mm \quad \text{at the abutment acute corner}$$

$$L_w = (E_T - E_L)S + 500mm \quad \text{at the abutment obtuse corner}$$

Design Length of Wingwalls

$$L = L_w - D - \text{Length of the cleat along the wingwall}$$

Here,

L_w , L , E_T , E_L , D and θ are illustrated in Figure 1.

S = Longitudinal slope of approach embankment along the wingwall as shown in Figure 5.2.2

SECTION 5 - ABUTMENTS, WINGWALLS AND RETAINING WALLS

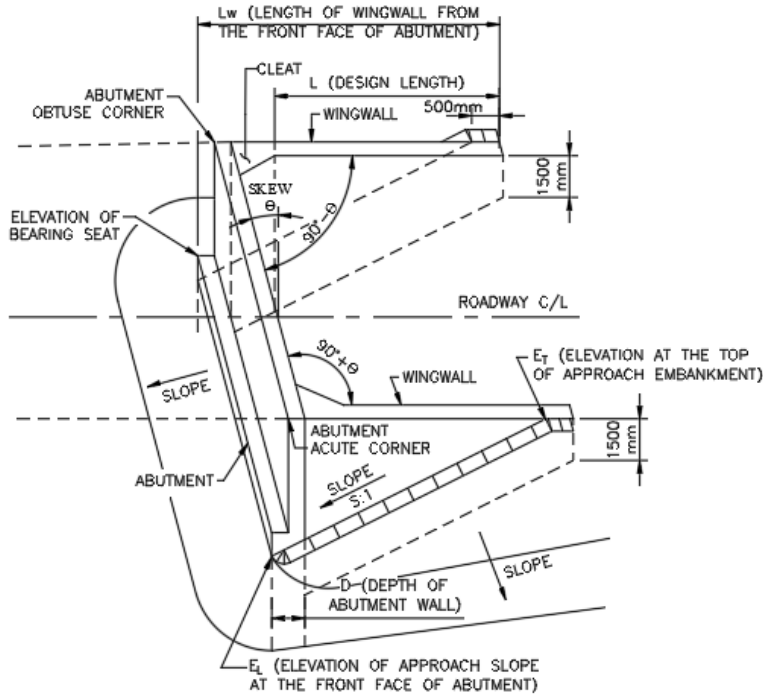


FIGURE 5.2.2 – GEOMETRY OF WINGWALLS ALIGNED PARALLEL TO ROADWAY

5.2.3.25.2.4.2 Height of the Wingwall

The wingwalls must have enough height above the ground level to retain the approach embankment and enough height below the ground level to ensure they are founded below the frost depth. Cantilever wingwalls are not susceptible to frost action provided they are supported over free draining fill above the normal water level. However, Ministry recommends providing a minimum wingwall height of 1500mm for a wingwall as illustrated in Figure 5.2.2 to retain the abutment backfill and provide sufficient height to transfer the collision load from the traffic barrier supported on it.

5.2.3.35.2.4.3 Loads on Wingwall

Wingwalls are designed for lateral active earth pressure, live load surcharge and compaction surcharge. When the bridge traffic barrier is supported on the wingwall, an equivalent static lateral live traffic load on the barrier as specified in CSA S6, must be included in the wingwall design. Lateral earth pressure for the wingwall design shall be calculated using soil parameters provided in the Foundation Investigation Design Report. Where the soil parameters are not available, the Ministry recommends using an equivalent fluid pressure,

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Ka $\gamma = 7.0$ kPa in the lateral earth pressure calculation. Figure 5.2.3 illustrates design loads on a wingwall.

5.2.3.45.2.4.4 Calculation of Forces in the Cantilever Wingwall

A cantilever type wingwall shall be analysed as a free cantilever wall suspended from the abutment. The maximum moment and shear in the cantilever wall occur at the interface of the wall to the cleat. The equations provided in the Design Aids can be used to calculate the maximum moment and shear for cantilever wingwall design for applicable loads. Caution must be exercised when determining the force in intensity when wingwalls do not extend for the full height.

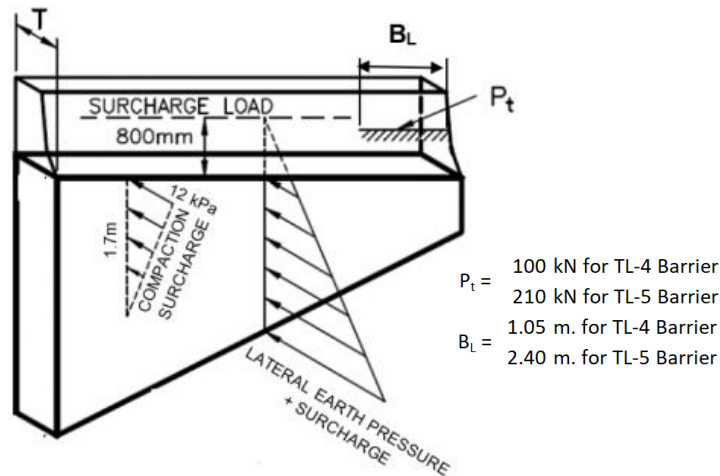


FIGURE 5.2.3 – DESIGN LOADS APPLIED ON WINGWALLS

5.2.3.55.2.4.5 Standard Design Aids for Wingwall

Design Aids are provided in Division 3. The design values provided in the design aids are only applicable for the particular shape of wingwall shown in the design aids.

5.2.3.65.2.4.6 Standard Structural Drawing (SSD) for Wingwall

Two Structural Standard Drawings exist for wingwall. One with integral abutment bridges (SS105-12) and one for wingwall with non-integral bridges (SS105-11).

5.2.3.75.2.4.7 Wingwall Extensions

For self standing structures independent of the abutment (e.g., MSE (RSS) wingwalls, separate retaining walls), the frost cover shall be as specified in Section 4.2.3. The above criteria for length, height and loads shall apply.

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5.3 Mechanically Stabilized Earth (MSE) Systems (RSS)**5.3.1 General**

Mechanically Stabilized Earth (MSE) Systems (as known as Retained Soil System, RSS) are structural systems that retain generally horizontal soil loads. They may employ either strip or grid-type, metallic or polymeric tensile reinforcements in the soil mass, and a discrete modular precast concrete facing, which is either vertical or nearly vertical. There are also other forms including interlocking soil-filled timber, reinforced concrete or steel modules that do not contain strip or grid reinforcement, but these are rarely used adjacent to structures and thus the terms MSE and RSS are used interchangeably in this manual.

MSE may be considered for use where conventional gravity, cantilever, or concrete retaining walls with counterfort are considered, and particularly where substantial total and differential settlements are anticipated. Economic and foundation conditions determine the most suitable type for a particular location.

The contract documents should include a Foundation Investigation Report, and constraints for the retaining structure, consisting of alignment, profile, and cross-sectional space constraints. MSE are categorised in terms of three attributes, application, performance, and appearance.

- For application, the categories are: true abutment, false abutment, wall/slope, road base embankment.
- For performance, the categories are: high, medium, low.
- For appearance, the categories are: high, medium, low.

These requirements are more fully detailed in the MTO RSS Design Guidelines (2008).

The MSE systems approved for use are listed in the Designated Sources of Materials (DSM) Listing, DSM #9.70.52, #9.70.53, #9.70.56 and #9.70.59. All design, construction and fabrication drawings and specifications required to complete this work are obtained by the contractor from the supplier of the proprietary MSE system (RSS) listed on the DSM.

5.4 The Use of MSE (RSS) Walls Adjacent to Structures**5.4.1 MSE (RSS) Wall Design History**

Early MSE (RSS) walls were generally separate from bridge structures until the introduction of integral abutment and semi-integral abutment bridges in the mid-1990s. MSE design was left entirely to the contractor's MSE suppliers, and MSE walls were often specified in locations that were not ideal. MSE (RSS) Design Guidelines were developed to capture previous experience and require that the foundation specialist engineer do adequate investigations to ensure that MSE is suitable for the site. The guidelines provide direction and details on the use of MSE in false abutments, wingwalls, and some other

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common uses including walls separated from the structure. [MSE \(RSS\) under water or in flood plain applications requires approval from the Foundation Section.](#)

The RSS Design Guidelines limit the maximum SLS settlement to 25mm for walls that are abutting a rigid bridge structure. Despite the 2008 Guidelines, settlement continues to be a problem. The magnitude of this settlement depends on consolidation of the underlying soil and of the MSE fill, both of which depend largely on the height of the embankment. For false abutments, excessive settlement can lead to unsightly gaps, as well as potential exposure of the piles. For wingwalls and retaining walls, settlement can lead to noticeable differences in traffic barriers 'on bridge vs. on MSE wing walls' top elevation problems with utilities in barrier conduits, roadway settlement, drainage problems and leaking, as well as cracks caused by concrete surfaces leaning against each other. For barriers on MSE walls with moment slabs (see Section 10.5.2), settlement of the barrier causes geometric discontinuity and a potential snag hazard between this barrier and the barrier on the bridge. This detail has also caused voids to open between the approach slab and the moment slab, leading to undermining and drainage issues.

In conventional bridges, it is relatively easy to build up the approach pavement if settlement occurs. With MSE walls and integral abutments, the movement of the walls makes settlement difficult to control and correct and creates the need for extensive maintenance or rehabilitation prior to the first planned rehabilitation.

As stated in the RSS Design Guidelines, consideration should be given to whether there are adequate quantities of MSE walls to justify their use, along with the need for a separate sub-contractor to do the work. In situations where there is minimal opportunity for cost savings, the potential settlement problems will outweigh cost savings. Another consideration is to move the MSE walls away from the structure so that the differential settlements can be tolerated. If this is done, the abutment would have conventional cast-in-place cantilever wingwalls, plus MSE walls away from the bridge substructure - either parallel to the abutment, curved, or flared at an angle. MSE walls under the front of the abutment, and/or used as wingwalls with the barrier and moment slab on MSE walls, should only be used where potential for settlement is very small, or where absolutely required for property or other reasons.

This following policy (March 2019) was developed to specify a preference for the MSE walls being separated from the structure, thus reducing the consequences of settlement on the performance, behaviour, and aesthetics of the bridge.

5.4.2 Design Requirements

The suitability of the site for MSE walls shall be determined with adequate investigations by the foundation specialist engineer.

When MSE is suitable:

1. The MSE wall shall be separated from the structure, both in front and to the sides, like what is shown in Figure 5.5.1 for integral abutment bridges or Figure 5.5.2 for bridges with bearings at abutments.

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- a) If the bridge has bearings, consideration shall be made for inspection of the bearings. If the bearings are accessible from a lift (cherry picker, scissor lift) below, the MSE wall may be kept tight to the abutment as shown in the RSS Design Guidelines. However, if the inspector is expected to access the bearings by foot, a 1.2 m wide platform is required, along with potential inspector safety railings depending on the MSE height.
 - b) If the MSE wall extends roughly parallel to the bridge, there shall be adequate slope and clear zone between the roadside barrier (concrete or steel beam guiderail) and the MSE wall to accommodate barrier deflection. Consideration shall also be given to preventing de-icing salts from reaching and causing deterioration of the steel straps within the MSE backfill by the use of curbs, or barriers to lead the drainage longitudinally away from the MSE wall.
2. The MSE wall under the front of the abutment, and/or used as wingwalls with the barrier with moment slab on MSE, may only be used when approved by the Structural Section, and with consultation with the RSS Committee. The following situations generally should be satisfied to be considered for approval:
- a) The anticipated settlement is very minimal (less than 10 mm); or,
 - b) The backfill and MSE fill is less than 5 m height; or,
 - c) The barrier on MSE begins beyond the approach slab or the barrier wall is supported directly on the approach slab; or,
 - d) If a unique situation dictates the need to minimize the MSE backfill mass.

5.5 Approach Slabs

- a) 6000 mm approach slab with 90 mm asphalt and waterproofing shall be used on structures on all paved roads and on roads that are to be paved in the near future.
- b) For locations with sleeper slabs, the length of ~~wingwall approach slab is preferably should be~~ extended so that the sleeper slab drainage is beyond the ~~extent end~~ of the wingwalls. The approach slab reinforcement may need to be increased ~~if the slab becomes too long~~.
- c) Approach slabs waterproofing is shown on Structural Standard Drawing SS 105-15 & SS 105-16, SS105-17, and OPSD 3370.100.
- d) Based on historical performance and considering MTO compaction practices, it is acceptable to design the approach slab as a simply supported structure over 60% of the length of the slab, ~~somewhat less than~~ instead of the 75% suggested in CHBDC Clause C1.7.2.
- e) If the approach slabs are not to be included in the bridge contract, the approach slab drawing should still be included in the contract drawings but must not bear the W.P. nor contract no. The general arrangement drawing must also carry the note:

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"THE APPROACH SLABS ARE NOT PART OF THIS CONTRACT".

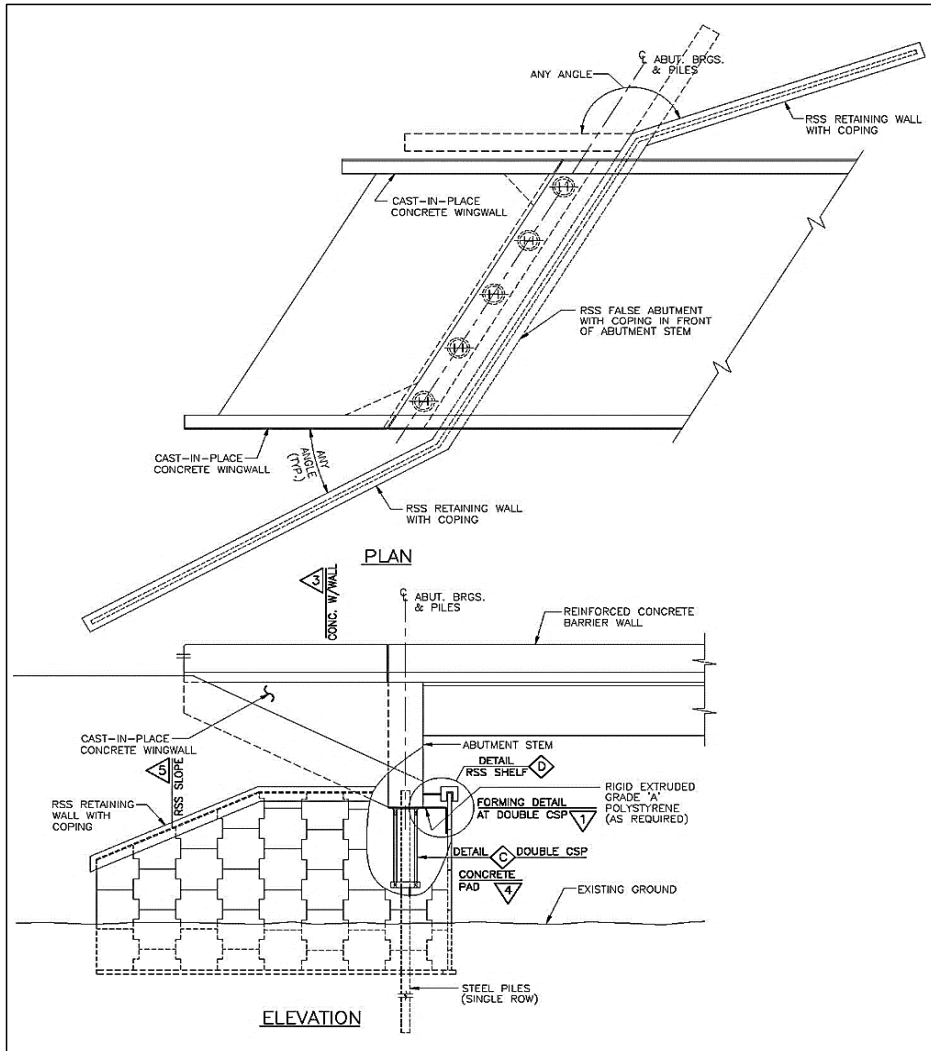


FIGURE 5.5.1 – MSE (RSS) FALSE ABUTMENT (IN FRONT OF INTEGRAL ABUTMENT STEM) WITH FLARED MSE WALLS

(From Figure 4.5 RSS Design Guidelines, 2008)

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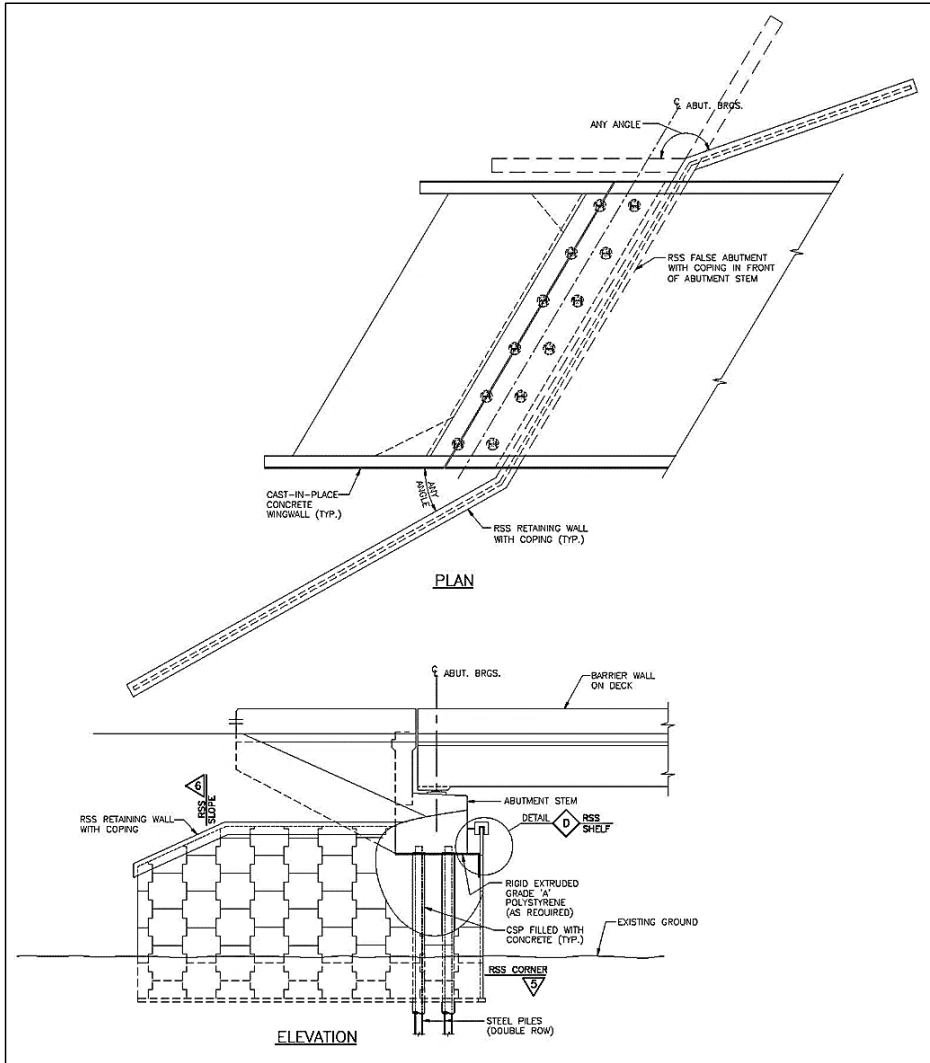


FIGURE 5.5.2 – RIGID PILED BRIDGE WITH MSE (RSS) FALSE ABUTMENT AND FLARED MSE WALLS

(From Figure 4.10 RSS Design Guidelines, 2008)

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6 PIERS**6.1 Design**

Preference is to provide integral piers for prestressed girder bridges in lieu of shallow bearings. These bridges require cast-in-place concrete in the pier, so this adds no additional construction operations, it improves structural redundancy and reduces the number of elements requiring future inspection and maintenance. For more than 2 spans, multiple integral piers are possible, however the piers must be able to accommodate the superstructure thermal movements.

Steel girders bridges typically are not made continuous with piers, although they may be designed that way with careful attention to details at the pier.

6.1.1 Round Column Diameters

Contractors traditionally rent their steel, round column forms as listed below for basic series of metric sizes of round column forms.

- 1000 mm, 1200 mm, 1350 mm, 1500 mm, 1800 mm, 2100 mm, 2400 mm.

These traditional steel forms can be used for both new and rehabilitation design. However, currently Sonotube forms are readily available for any metric sizes of columns for new design. Sizes other than above listed may be considered with a confirmation of Sonotube size availability.

6.1.2 Expansion Piers

Expansion piers shall be proportioned based on the appropriate shear rate when laminated elastomeric bearings are used and based on a sliding coefficient of friction according to CHBDC Table 11.4 when spherical or pot bearings are specified. Fixed supports, both piers and abutments, shall be designed to resist cumulative forces from expansion location, as required.

6.1.3 Pier Column Reinforcing Details in Earthquake Zones

Pier column reinforcing details in earthquake zones shall be in accordance with the provisions contained in CHBDC Section 4.7.

Reinforcing details shown in Figure 6.1.1 are to be used in seismic performance category 1. They provide sufficient flexibility to construct and do not compromise the integrity of the column at the interface of the footing.

Figure 6.1.2 shows the reinforcing details to be used for seismic performance categories 2 and 3 to prevent buckling of the longitudinal steel and to provide confinement for the core of the column. A continuous spiral with continuous longitudinal reinforcement is preferable. A break in spiral at locations 1, 2 and 3 is optional. The column reinforcing should be supported during the placement of the footing, splicing of longitudinal reinforcing or spiral outside the middle half of the column is not allowed.

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The spiral embedment shall be the greater of $D/2$ or 400 mm. The plastic hinge zone shall be the greater of $H/6$, D or 500 mm or the length over which the moment exceeds 80% of the maximum moment (see Figure 6.2.1(b)).

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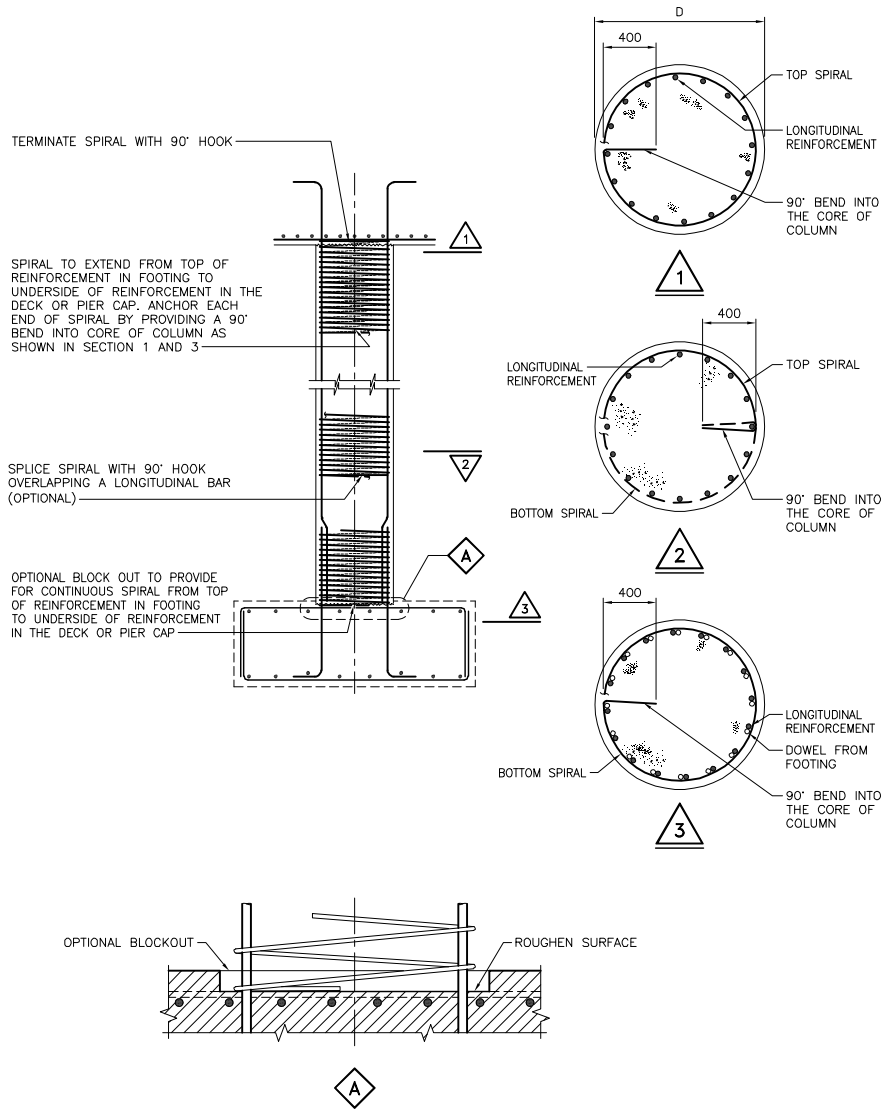


FIGURE 6.1.1 – PIER COLUMN REINFORCING DETAILS – SEISMIC PERFORMANCE CATEGORY 1

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It is intended that the details on Figure 6.1.1 or Figure 6.1.2, adapted to the specific bridge, shall be shown on the contract drawings to avoid debate during construction.

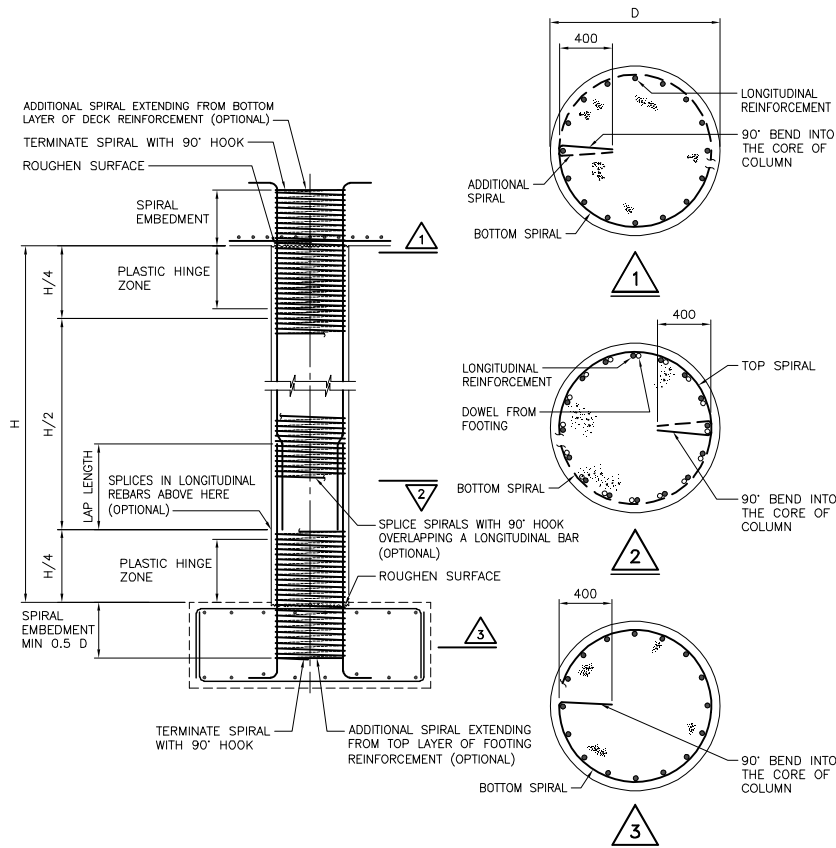


FIGURE 6.1.2 – PIER COLUMN REINFORCING DETAILS – SEISMIC PERFORMANCE CATEGORY 2 & 3

6.1.4 Protection of Bridge Pier Columns from Collision Loads

- a) New bridge pier columns located within 10.0 m of the nearest travelled lane (measured from the face of the column) shall be designed for the collision loading as specified in the CHBDC, regardless of whether they are protected by a roadside barrier. Piers and even some abutments, adjacent to railways, also have requirements for crash

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~~protection walls, and for crash load. These vary by railway owner and must be confirmed during bridge design.~~

- b) New Bridge piers with only one or two columns shall be designed for the CHBDC Collision load regardless of their distance from the edge of the travelled lane.
- c) Existing Piers: When new traffic lanes are added around an existing pier that encroach within 10m of an existing bridge pier, or a bridge is rehabilitated where the existing lanes are already within 10m of the pier, the Regional Structural Section shall assess the structural adequacy of the pier columns to withstand the collision loading according to the CHBDC. The Regional Structural Section shall consider factors including pier vulnerability, pier structural capacity, and remaining life of the bridge to determine whether strengthening of the pier, and/or protection of the pier columns is required.
- d) All piers accessible to impact at higher speeds should be protected by a barrier to reduce ~~excessive~~ loading and damage to ~~on~~ the pier, and to protect the vehicle occupant of the roadside hazard.
- e) Piers and some abutments, adjacent to railways, also have requirements for crash protection walls and for crash load. These vary by railway owner and must be confirmed during bridge planning phase.

For existing pier columns that were not designed for collision loads, effective measures for protecting the pier columns from collision loads include the following:

- a) An embankment; or,
- b) A structurally independent 1370 mm high barrier meeting the requirements of TL-5 as specified in CHBDC Clause 12.4.3.4.2; or,
- c) A structurally independent 1050 mm high barrier meeting the requirements of TL-5 as specified in CHBDC Clause 12.4.3.4.2 provided the barrier is a minimum of 3 meters from the component being protected.

If none of these measures is feasible to implement at a bridge site, the Regional Structural Section may choose to do a site specific detailed study considering site-specific truck traffic volumes, highway speed, highway alignment and grade, separation distance to the existing pier, structure redundancy, consequences of failure, acceptable probability of failure consistent with the requirements of CHBDC Clause 3.5.1, etc., in order to come up with an alternative measure of protection in terms of barrier type, shape, height or separation distance to the structure's components being protected.

6.2 Miscellaneous Details

6.2.1 Pier Nosing for River Piers

Steel nosing should not be provided unless ~~specifically called for in the Structural Design Report or in subsequent correspondence approved~~ by the Head of Structural Section for angular piers in rivers with heavy ice floes.

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6.2.2 Pier Bearing Seats

If the bridge deck over a pier is not continuous, the requirements of Section 6.2.3~~5.2.2~~ concerning the sloping of the bearing seat ledge apply ~~also to the pier~~.

The requirements of Section 5.2.2 concerning provisions for jacking also apply ~~also~~ to piers.

6.2.3 Rust Stain Control for Steel Girder Bridges

Piers below steel girders shall be detailed with dams to avoid leakage onto the piers and drained off at discrete vertical drain grooves. Rust dams should be at least 75 mm wide, and the trough shall be at least 75 mm. Piers are to be detailed so that The bearing seats ~~are shall be detailed~~ above the dams. The slope should match the deck crossfall. Add a counter slope to direct staining away from the ends. This slope should be of equal magnitude (max. 4%) but opposite direction to the deck cross-fall.

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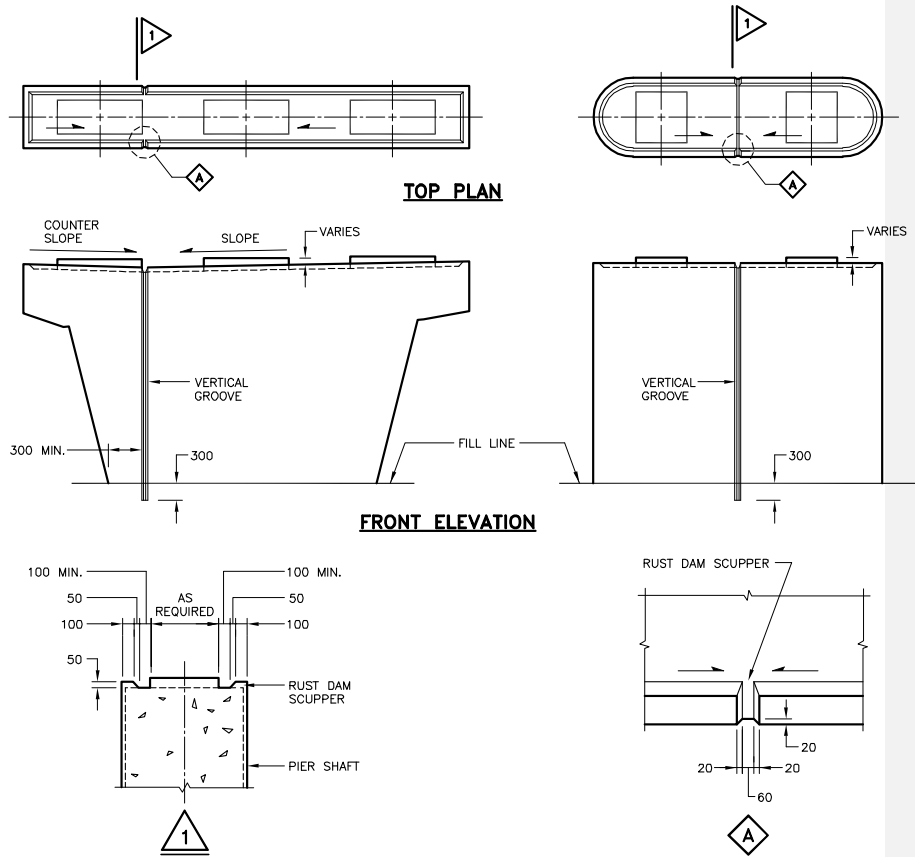


FIGURE 6.2.1 – RUST STAIN CONTROL – PIER SHAFTS

The vertical groove should clear any radius of shaft ends by at least 300 mm in pier shafts (see Figure 6.2.1) and clear the column by at least 300 mm in pier caps on columns (see Figure 6.2.2). Pier shapes vary; those shown are for guidance purposes only. Place scuppers and vertical grooves at all low points. Position the scupper to take advantage of any damming action from the outside bearing seat.

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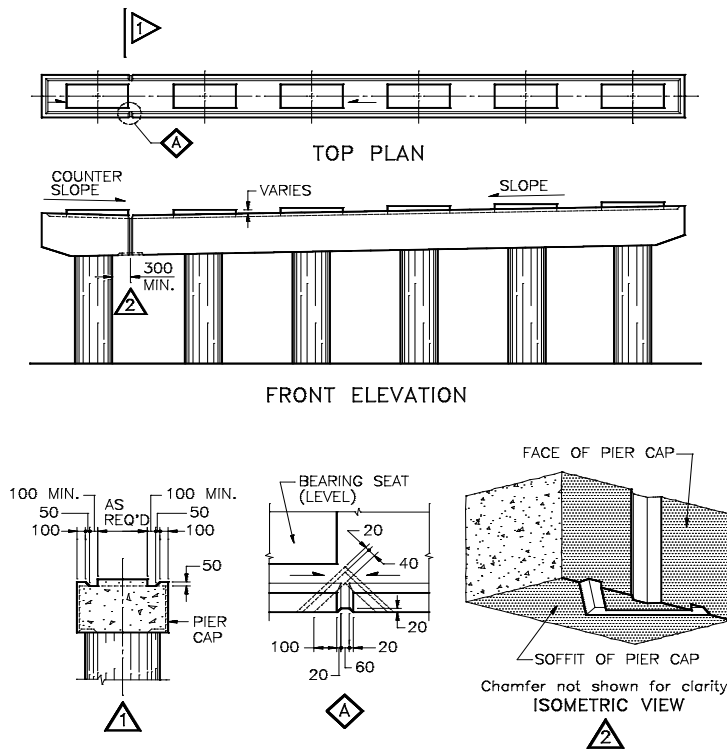


FIGURE 6.2.2 – RUST STAIN CONTROL – PIER CAPS ON COLUMNS

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SECTION 7 - PRESTRESSED CONCRETE

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SECTION 7 - PRESTRESSED CONCRETE

7 PRESTRESSED CONCRETE**7.1 General**

This section on prestressed concrete describes the requirements for pretensioned girders (Subsection 7.2) and post-tensioned decks (Subsection 7.3).

7.1.1 Background

The design and construction of precast prestressed concrete girders has evolved over time due to advancements in material quality, precasting production methods, and prestressing and concrete technologies. The Ministry first used prestressed girders in bridges in the mid to late 1950's, before the use of prestressing beds, meaning the first generation of girders were post-tensioned. In the early 1960s, the American Association of State Highway and Transportation Officials (AASHTO) girder sections with concrete strength of about 35 MPa were introduced. Prestressed concrete box girders, typically separated with a thin deck spanning between girders, was also introduced at that time. In the mid to late 1970s, the Ministry switched from AASHTO girders to Canadian Precast Prestressed Concrete Institute (CPCI) girders with concrete strength of 40-50 MPa and prestressing strands of 13 mm (0.5") diameter. Most recently, in the early 2010s, the Ministry introduced the Nebraska University (NU) girder shapes for use in bridge designs while still maintaining the use of CPCI girders. Since 2019, NU girders have been used exclusively, with CPCI being retaining only for widening of existing bridges.

Nu Girders are advantageous because:

- The constant size top and bottom flange and standard bottom flange configurations allow for easier reuse of forms in various size girders for easier fabrication;
- The wide top flange allows for shorter deck slab spans and hence less deck formwork between girder top flanges;
- The wide and thick bottom flange allows for more efficient placement of strands and increased strand capacity as well as increased stability during transportation and post installation;
- The ability to achieve longer spans (>45 m); and,
- The ability to have fewer numbers of girder lines at larger spacing with approximately the same girder depth, or shallower girders placed at approximately the same spacing.

All concrete I-girders on MTO projects shall be NU girder shapes. CPCI can only be used for special cases with approval of the Structural Section (such as widening of existing bridges or replacement of a damaged CPCI girder). Prestressed Box girders may also be used, although currently, side-by-side box girders which minimize forming requirements, are preferred over separated boxes.

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7.2 Precast, Prestressed Girders**7.2.1 General**

a) Girders should be pretensioned rather than post tensioned, unless transportation problems dictate post-tensioning (see Section 16.2), or the girders are to be spliced.

b) Capacity of all girders in the cross-section shall be identical. Side-by-side box girders, and girders carrying any unusual additional dead loads, may be designed differently with approval of the Structural Section

~~a)~~

~~b)~~

c) The design of prestressed concrete members shall be based on a 50 MPa Specified 28-day Compressive Strength for Box girders and solid slabs, and 60 MPa for NU girders, respectively.

Based on past Ministry experience, girder compressive concrete strength of 50 MPa or more should provide adequate durability for most exposures and situations. This assumes cover requirements are met, temperature limits are observed, and other specification requirements are complied with.

d) The minimum concrete strength required at transfer of prestress, f_{ci} , shall be specified to the nearest 0.5 MPa. It is very important for the production cycle of precast and prestressed components that the designer specifies the minimum value of strength at transfer that the CHBDC's SLS design requirements at transfer will permit. For NU girders, the concrete strength specified at transfer should be kept to 40 MPa or less and shall not exceed 45 MPa. The concrete strength at transfer shall not be greater than 38 MPa for concrete box girders and 35MPa for solid slabs.

Note: The availability of GU and HE cement will be severely restricted in late 2024 as the cement industry in Canada moves exclusively towards GUL and HEL Portland-limestone cements to reduce environmental impacts. Therefore, in the near future it will no longer be possible to achieve 45 MPa transfer strength with a single day with HEL turnaround because the limestone does not contribute to accelerated strength gain.

e) Deflected strands shall be fanned out at the ends of the girders to obtain the most uniform stress distribution. The designer should keep the centroid of the deflected strands as low as design parameters will allow, while attempting to spread the strands uniformly at girder ends throughout the web.

Typically, precast, pre-stressed girders are designed with both deflected and straight strands. However, to facilitate production, all precast, pre-stressed concrete box girders 900 mm deep or shallower shall be designed with straight strands only.

f) In calculating stresses, the transformed area of the strands may be used to compute section properties.

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- g) Single hold-down forces greater than 80 kN (18 kips) should be avoided since this is the limit for some precasting beds. Generally, an individual strand should not be deflected steeper than a slope of 1 to 6.

The design of precast, prestressed girders may require the debonding of a number of straight strands at girder ends. In this case, they shall be grouped according to CHBDC provisions and rigid debonding sleeves shall be specified on drawings.

The strands to be deflected shall be designated into groups with the hold-down location of each group clearly indicated on the drawings. A 1.0 m spacing between hold-down points of different groups of deflected strands is normally used.

- h) At the ends of the girders, the deflected strands are usually flared vertically in the web in multiples of 100 mm to avoid stress concentration and hence longitudinal cracking in webs at girder ends.
- i) The CHBDC prestressing stress limit for prestressing stress immediately prior to transfer of $0.75 f_{pu}$ does not account for stresses added by the fabricator to compensate for chuck sitting, form shortening, bulkhead rotations, friction at hold-downs, ambient temperatures, thermal effects, or relaxation which occurs between the time of stressing and when the strands are cut to release the force into the girder.

Precasters are reluctant to stress the strands above $0.80 f_{pu}$ due to the risk of breakage. This risk is higher for deflected strands. Consequently, the designer shall limit the specified prestressing stress prior to transfer to $0.74 f_{pu}$.

- j) Details of the positive moment connection over piers are given on [SS107-15 and SS107-24](#) ~~OPSD-3310-150~~. In integral and semi-integral abutment situations, the connection of the girder to the abutment, is achieved by the use of projected L-shaped reinforcement bars or by bent projected strands from the girder.
- k) All diaphragms shall be cast integrally with the deck slab pour, without construction joints. Diaphragms shall completely encase strands at the ends of girders to achieve the minimum cover required by the CHBDC.
- l) The girders' 'stirrups' projection above the top of girder must be calculated and specified so that it accounts for girder hogging (upward deflection) and the vertical curve of the roadway profile. It shall be verified and revised, if necessary, to ensure that the girder stirrups are projecting a minimum of 25 mm above the bottom reinforcement mat of the deck slab. The stirrups at the girder ends shall have a minimum spacing of 75 mm to avoid reinforcement congestion. Particular attention shall be given to the spacing and arrangement of stirrups at the vicinity of the rectangular dowel holes. In the case of skewed bridges, a plan detailing the arrangement of the stirrups and the dowel holes shall be included on the drawings.
- m) The distance from the end of the bearing to the end of the girder shall be specified as 100 mm.

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- n) Shear design shall assume 13 mm size of aggregate. OPSS 909 requires that 19.0 mm aggregate be used for precast girders but the precaster may request the use of 13.2 mm aggregate size for girders with congested reinforcement.
- o) When premium reinforcement is required for stirrups in precast girders, it shall be of stainless steel according to Section 12 of this manual. GFRP stirrups shall not be used.

7.2.2 Precast Prestressed Girder Sizes

The available metric girder sizes that the ministry currently uses are:

- a) NU Girders 900, 1200, 1400, 1600, 1800, 1900, 2000, 2400.
- b) Box Girders 700, 800, 900, 1000.
- c) Solid Slabs 300, 400, 500.

7.2.3 Debonded Strands

Debonding of strands shall be accomplished with rigid polymer sheathing with a wall thickness of not less than 1 mm.

As it is possible for moisture to penetrate around debonded strands from the ends of girders, debonded strands shall not be used at the ends of girders at expansion joints.

7.2.4 Precast, Prestressed Solid Slabs (Precast Planks) and Voided Girders

Precast sections of less than 700 mm depth shall not have voids. The addition of voids adds greatly to the complexity and saves relatively minimal dead load on the girder.

When used with spaces between the units, these members shall be regarded as precast girders and the deck slab shall be designed in accordance with normal composite deck slab requirements.

CHBDC Clause 8.21 gives requirements for interconnection of units placed side by side in contact.

Non-standard precast sections may be used in the construction of cast-in-place bridges with an approval of Structures Office (e.g., for the deck of a rigid frame bridge over railway). Precast sections with a span of ~~more~~-greater than 10 m shall be prestressed.

7.2.5 Temporary Bracing

In order to ensure that prestressed precast concrete NU girders are provided with adequate safety and stability provisions at the time of installation, the ministry has created minimum bracing requirements to prevent any possible movements, overturning or tipping of the installed girders until the time when the contractor can implement their formwork design. As a minimum, contractors are required to brace the girders by attaching the

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girder ends at each support location to the faces of abutments and piers and also provide in-span bracings within each span.

The ministry has created a Structural Standard Drawing (SSD) SS 107-9 for this purpose. Designers of bridges that have prestressed precast concrete NU girders are required to include Structural Standard Drawing SS 107-9 in the structural drawings package.

The deck formwork designer must not rely solely on the temporary restraints provided by these temporary bracing that are used solely for the stability of the girder. Additional or different temporary struts; bracing, tie bars/cables and other devices shall be used for temporary restraint, as required by formwork design, in order to carry all loads imposed during each stage of construction. When working platforms are provided between the bottom flanges of the girders they should be stabilized together appropriately.

Temporary struts, bracing, tie bars/cables and other devices used for temporary restraint shall be removed upon completion of the structure.

7.2.6 Side by Side Precast Prestressed Concrete Box Girders Type Bridge

Bridge decks made of side-by-side precast prestressed concrete box girders with an integral 150 mm topping cast in place concrete slab do not require shear keys between adjacent boxes nor transverse stressing. Such units shall have a steel plate welded connection tying them together following erection as shown on Structural Standard drawings to:

- a) Ensure their stability during construction;
- b) Prevent differential deflection during placement of the concrete deck, which would cause cracking of partially setup concrete; and,
- c) Prevent exterior girders from differentially deflecting and/or separating from the adjacent girders when they are loaded in torsion during the deck placement due to the deck cantilever overhang, or sidewalk that is detailed to be poured at the same time as the deck slab, or if the bridge is on a large skew.

For side-by-side box girder bridges that are designed without a topping slab, the designer shall design and detail shear keys between neighbouring boxes to fully carry the shear between the boxes. In this case, the thickness of the top slab of the box itself is usually much thicker than that of a standard box used with a topping slab. Waterproofing and paving are usually applied directly over the shear-keyed side-by-side boxes.

7.2.7 Establishing Screed Elevations

CHBDC Clause 8.13.3.4 allows designers to estimate long-term deflections by multiplying instantaneous deflections by appropriate factors. These factors, or "multipliers" are given in the code commentary Clause C8.13.3.

This Multipliers method originated from many documents published by the Precast/Prestressed Concrete Institute (PCI) and the Canadian Precast prestressed

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Concrete Institute (CPCI). However, they do caution the reader regarding the applicability of these multipliers when it comes to bridge girders where a large cast-in-place deck slab is made composite with them, as is usually the case in Ontario.

Screed elevations shall not be established using long-term deflections. The designers shall establish the screed elevations by including allowances for the roadway profile and the deflection due to the weight of the wet concrete slab and superimposed dead loads. The deflection due to the weight of the wet concrete slab and superimposed dead load shall be multiplied by a factor of 1.10.

7.2.8 Establishing the Required Undercut and Stirrup Projection

Although the “final” deflection multipliers specified in Table C8.8 of the commentary to the CHBDC should not be used in calculating the long-term deflections for precast prestressed girder type bridges, the deflections and rotations at the time of erection correlate closely to the use of the “at erection” multipliers specified in the commentary table referenced above. This can be explained by the fact that, at this stage, the time dependant effects are acting on the prestressed member alone and not on the composite section. As a result, the “at erection” multipliers are providing accurate predictions of deflections and rotations due to prestress and self-weight of the girder. At the erection stage, the deflections and rotations due to the weight of the wet concrete are instantaneous and do not require a multiplier (i.e., the multiplier is 1.0).

For composite action, according to the code, the stirrups must extend sufficiently into the deck to engage the bottom mat of reinforcing steel. The top of the stirrups must also be at least 100 mm below the top of the deck if black steel reinforcing bars are specified. The projection of the stirrups above the top of the girder shall be established for the erection stage. Stirrups along the length of the member can have different projection lengths specified when required by the design.

The purpose of the bearing soffit undercut is to ensure proper contact between the girder and the elastomeric bearing when all the dead loads have been applied. In calculating the undercut, the structure grade ‘G’, camber due to prestress ‘C’ as well as deflections due to the girder self weight and the wet concrete ‘D’ should be considered (camber and deflections in this case refer to the resulting rotations). At the low end, dimension ‘b’ shown on the Structural Standard Drawing ~~SS-107-X-containing the girder details~~, is a function of +G-D+C. At the high end, dimension ‘c’ ~~shown on the Structural Standard Drawing SS-107-X~~ is a function of -+G-+D+-C. Therefore ‘b’ and ‘c’ may differ.

The maximum undercut dimension (‘b’ or ‘c’) that is allowed, in order to maintain proper cover to the reinforcing steel, is 18 mm. When the undercut dimensions ‘b’ or ‘c’ exceed 18 mm, the embedded bearing plate detail shown on the MTO Standard drawings should be used.

Requirements

1. The multipliers specified in Table C8.8 of the commentary to the CHBDC shall be used when calculating the deflections and rotations for the girder erection stage.

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2. The projection of the stirrups above the top of the girder shall be established for the erection stage. Stirrups along the length of the member can have different projection lengths specified when required by the design.
3. The bearing soffit undercut shall be established using the "at erection" multipliers when calculating the rotations due to prestress and girder self weight and a multiplier of 1.0 when calculating the rotation due to the weight of the wet concrete.

7.2.9 Prestressed Girders Notes

Precast beams are generally detailed on Structural Standard Drawings which have the notes pre-printed. If Structural Standard Drawings cannot be used, copy the appropriate notes from them.

A) NOTES (On DECK DETAILS drawing, prestressed girder superstructure.)

1. SCREED ELEVATIONS ARE TO TOP OF CONCRETE DECK.
2. SCREED ELEVATIONS SHOWN IN TABLE INCLUDE AN ALLOWANCE FOR VERTICAL CURVE, WEIGHT OF DECK SLAB AND SUPERIMPOSED DEAD LOAD.
3. CONCRETE IN DECK SLABS AND DIAPHRAGMS SHALL BE RETARDED USING A TYPE B OR D ADMIXTURE TO ENSURE THAT THE CONCRETE REMAINS PLASTIC FOR THE DURATION OF EACH PLACEMENT.
4. CONCRETE IN BARRIER WALLS OR SIDEWALK SHALL NOT BE PLACED UNTIL ALL CONCRETE IN DECK SLAB HAS REACHED A STRENGTH OF 20 MPa.
5. FALSEWORK FOR THE CANTILEVER PORTIONS OF THE DECK SLAB (FOR A LENGTH OF 2.0 m) SHALL NOT BE REMOVED UNTIL THE CONCRETE AROUND THE INSTALLED EXPANSION JOINT HAS REACHED A STRENGTH OF 20 MPa (TYP. AT ALL FOUR CORNERS OF THE BRIDGE).
6. ALL DIAPHRAGMS SHALL BE CAST INTEGRAL WITH DECK SLAB, AND CONCRETE IN DIAPHRAGMS SHALL BE VIBRATED THOROUGHLY.

7.3 Post-Tensioned Concrete Decks**7.3.1 General**

- a) Post-tensioned superstructures, which are solid or voided by means of round tubes, must be transversely prestressed throughout their length with reinforcing steel reduced to a minimum. Transverse stressing is not mandatory for box section decks except as required by (c) and (d) below.
- b) For skew angles in excess of 20°, transverse prestressing cables and reinforcement reinforcing steel should be square to the deck except over skewed supports.

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- c) Transverse moments over piers and abutments shall be resisted by transverse prestressing rather than reinforcing steel.
- d) Wherever possible, the cantilever portion of cast-in-place, post-tensioned cross-section shall be greater than 1.6 m. Deck cantilever overhangs exceeding 3.2 m in length shall be prestressed.
- e) For decks with circular voids, the length of the solid section at the anchorages shall be at least equal to the distance from the centreline of the outside anchorage to the edge of the deck measured perpendicular to the longitudinal centreline of the deck.
- f) Post-tensioned decks shall be designed using 35 MPa concrete except, unless a higher strength is needed for web crushing in rectangular voided sections or compressive resistance of the bottom slab at piers.
- g) To control internal concrete temperature during casting without supplementary cooling measures, the thickness of solid slabs shall be limited to 1.0 m for 35 MPa concrete and 0.7 m for 50 MPa concrete. This limit does not apply to solid end zones of voided slabs or box sections.
- h) The location of the outlets for cable duct vents and drains shall not be shown on the drawings.
- i) The location and limits of construction joints in box girders (e.g., between slabs and webs) shall be clearly defined. This often requires that the joints be shown in plan or elevation as well as in cross-section.
- j) Chamfer strips shall be detailed around the perimeter of the concrete that constitutes the bottom slab of rectangular voided post-tensioned decks.
- k) In order to limit and contain spalling-type stresses, resulting from the shear caused by bearing reactions, the distance from deck end to centreline of bearing shall not be less than 0.8 m. This end distance for longitudinal cables may need to be even greater, depending on skew of bridge, size of tendon, size of bearing, and depth of section.
- l) Dead-end and live-end anchorages (including local zone components such as anchorage plates, spirals, etc.) are to be fully detailed and supplied by the post-tensioning supplier. Dead-end anchorages shall be detailed the same as live end anchorages on design drawings. Bond-head anchorages shall not be used unless approved the Head of Structure Section.
- m) Anchorage recesses should be detailed based on the largest available anchorage devices from listed suppliers.
- n) The spacing of stirrups should be such that, wherever possible, a stirrup or row of stirrups is located at each tendon ordinate location.
- o) For box girders, longitudinal tendons in the webs, at their lowest point, shall be detailed to above the top reinforcement of the bottom slab.

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- p) Post-tensioning couplers should be avoided. Instead, construction joints in long post-tensioned bridges should be detailed with tendons lapped across blisters in the web. If couplers are subjected to net tensile fatigue stresses, dynamic testing of the couplers shall be specified.
- q) Tendon curvature shall be limited to a 10 m radius for 19 strand tendons and larger, except for loop tendons where smooth steel duct is required. Round tendon ducts, excluding loops, shall have a minimum radius of 10 m.
- r) Clearances around tendon anchorage shall be designed to allow stressing with multi-strand jacks except for transverse tendons in flat ducts. Tendons should be designed to be stressed horizontally, and preferably at deck level.
- s) The diameter of a duct, or an equivalent diameter of a non-circular duct, shall not exceed 40% of the least gross concrete section thickness at the location of the duct.

7.3.2 Circular Voided Decks

For spans of 28 to 43 m, the span-to-depth ratio of circular voided decks shall be no greater than 26. A solid section may be more appropriate for spans less than 30 m, and rectangular voids are more appropriate above 40 m.

Corrugated metal pipe void forms shall be used to form all round voids. Corrugated metal pipe is available in any size over 150 mm diameter; but the following standard diameters should be used in specifying the void sizes (mm): 600, 700, 800, 900, 1000, 1200, 1400, 1500.

The arrangement of void forms and longitudinal tendons must provide, between all void forms, one unobstructed vertical passage of at least 100 mm in width as recommended in the CHBDC Commentary to Section 8.

Standard Drawings showing void drainage (OPSD 3333 series) and hold down details (OPSD 3332.100) shall be included in the design. Stirrups should be shaped so that void forms can be lowered into place after the longitudinal cables have been placed.

7.3.3 Box Girder Voids and Access Locations

Access openings for the inspection and maintenance of concrete box sections shall be provided where the cross-sectional depth is 1.8 m or greater. Box girders deeper than 2.4m shall have access openings through diaphragms between spans. The minimum clearance for walking through intermediate or support diaphragms inside box girders shall be 600 mm horizontally and 1200 mm vertically.

The number of access openings and their locations shall be as follows:

- a) For concrete box girders with solid diaphragms: One opening per box cell;
- b) For concrete box girders with walk-through diaphragms: Two openings per box girder, one located adjacent to each end diaphragm.

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The access openings shall be located to avoid or minimise interference with traffic during inspection and should be easily accessible to inspectors.

7.3.4 Post-Tensioning Tendons, Ducts, Strands, Detailing

Grade 1860, Size Designation 15 (15.2 mm or 0.60" diameter), low relaxation strand shall be specified for all tendons. The nominal area of size designation 15 strand is 140 mm².

~~Most post-tensioned superstructures, whether solid slab or circular voided post-tensioned structures or trapezoidal voided, do not require tendons larger than size designation 19/15 (containing 19, Size 15 strands). In the case of trapezoidal voided and box girders girder post-tensioned structures, large tendons are beneficial to reduce the width of the web and/or increase the eccentricity of the strand, if larger tendons are required, and tendons up to size designation 37/15 may be used. Tendons anchored and stressed from within a box girder shall be no larger than 19/15 to limit the size and weight of jacks.~~

Rigid ducts shall be detailed for the full length of longitudinal tendons in all cast-in-place post-tensioned concrete bridges.

High strength bars may be used for vertical and transverse post-tensioning. Cold rolled high strength threaded bars larger than 46 mm are not permitted.

7.3.5 Anchorage Slip for Post-Tensioning Systems

The CHBDC requires the magnitude of the anchorage slip to be either as required to control the stress in the prestressing steel at transfer, or as recommended by the manufacturer of the anchorage, whichever is greater. Stressing from two ends shall not be specified when the calculated elongation is less than the length of the wedge grip.

The anchorage slip specified on the plans for strand post-tensioning systems shall not be less than 8 mm for anchorages with up to 7 strands, 10 mm for anchorages with 8 to 12 strands, and 12 mm for anchorages with more than 12 strands. For post-tensioning bars, the slip allowed for in design shall not be less than 1.5 mm.

On short tendons when anchorage slips losses are a major contributor to the total prestress loss of the tendon, the designer may consider specifying power seating and low slip values upon consultation with suppliers.

7.3.6 Post-Tensioning Tendons and Duct Sizes

Design of post-tensioning shall be done with commonly stocked tendon sizes, and standard plastic duct sizes as shown in Table 7.3.1. Duct sizes are established to ensure the inside cross-sectional area of the duct is at least 2.5 times the net area of the strand. Duct diameters given are nominal and actual diameters can vary by ± 3 mm. OPSS 910 requires the use of plastic ducts and steel duct dimensions are provided for reference.

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TABLE 7.3.1

No. of 15 mm Strands	Plastic Dust-Duct I.D./O.D.	Steel Duct I.D./O.D. (mm)
5	48/59	55/60
7	59/73	65/70
12	76/91	85/90
19	100/116	105/110
27	115/135	125/130
37	130/151	135/140

Tendons for transverse post-tensioning of deck slabs may use flat anchorages with up to four strands and flat duct. Where flat ducts are used for transverse tendons in deck slabs, internal dimensions are 25 x 76 mm and outer dimensions are 40 x 90 mm.

7.3.7 Post-Tensioning Tendon Duct and Web Details

Duct arrangements and corresponding reinforcement layout and web dimensions are shown in Figure 7.3.1. Dimensions are based on 27/15 tendons.

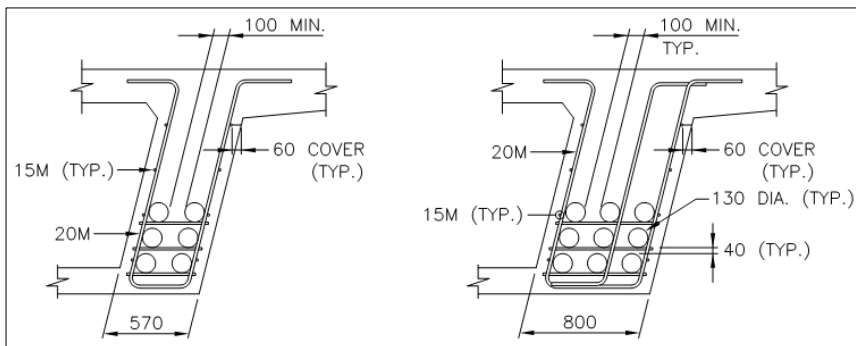


FIGURE 7.3.1 – DUCT ARRANGEMENT FOR RECTANGULAR VOIDS

7.3.8 Additional Reinforcing in Exterior Webs Adjacent to Tapered Voids

In order to prevent tendons breaking into the tapered voids, additional reinforcing is required to resist the radial forces caused by horizontally curved tendons. A detailed plan and cross-section through tapered portions of the voided section must be shown on contract drawings. No splices of duct sheathing shall be allowed along the tapered portion of void, and within one metre from either end of it, to reduce the risk of tendon kinking.

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7.3.9 Post-Tensioned Deck Notes

The following are standard notes of the type shown below the title block on the appropriate drawing. The notes must be worded to cover the requirements specific to the project and should only be used if applicable. Other notes may be required in special circumstances. In specifying the 28-day Compressive Strength of concrete, different components may be itemised together if the Specified 28-day Compressive Strengths are the same. For clarity the wording of the notes is shown in upper case (capital) lettering. Explanations shown in brackets in lower case lettering are not part of the notes.

A) General Notes

1. ALL TENDON ANCHORAGES SHALL BE INTERNAL TYPE AS SHOWN ON THE DRAWINGS.
2. DUCT GROUTING VENTS SHALL BE PROVIDED AT HIGH POINTS AND AT BOTH ENDS OF ALL TENDONS. DRAINS SHALL BE PROVIDED AT LOW POINTS.
3. MIN. CLEAR COVER TO TENDONS DUCTS:

TOP	± ... LONGIT.
	± ... TRANSV.
ELSEWHERE	80 ± 10
4. WELDING IS NOT PERMITTED WITHIN 3000 mm OF ANY TENDON OR TENDON DUCT.
5. ALL TENDONS SHALL BE STRESSED IN THE NUMERICAL ORDER SHOWN ON THE DRAWINGS.
6. SPECIFIED 28-DAY COMPRESSIVE STRENGTH: 35 MPa.
7. CONCRETE STRENGTH BEFORE STRESSING: 25 MPa.
8. ALL ANCHORAGE RECESSES SHALL BE ABRASIVE BLASTED AND COATED WITH AN APPROVED CEMENT PASTE PRIOR TO CASTING OF 35 MPa CONCRETE IN THE RECESSES. CONCRETE IN RECESSES SHALL BE COLOUR MATCHED WITH ADJACENT CONCRETE.
9. THE POST-TENSIONING SUPPLIER SHALL DETAIL AND SUPPLY LIVE-END AND DEAD-END ANCHORAGES INCLUDING SPIRALS, THE METHOD OF SUPPORT AND ASSOCIATED REINFORCEMENT.
10. PRESTRESSING TENDONS SHALL HAVE THE NUMBER OF STRANDS AS INDICATED IN THE TABLE(S).
11. PRESTRESSING STEEL SHALL BE LOW RELAXATION SEVEN WIRE STRAND, SIZE DESIGNATION 15, GRADE 1860, AND SHALL MEET THE REQUIREMENTS

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OF ASTM A416/A416M. MINIMUM BREAKING STRENGTH SHALL BE 260 kN PER STRAND.

- 12. SHEATHS FOR POST-TENSIONING DUCTS SHALL BE CORRUGATED PLASTIC ~~WITH A WALL THICKNESS NOT LESS THAN 1 mm.~~

B) Longitudinal Tendons

- 1. TENDON SIZE, REQUIRED ELONGATION, AND JACKING FORCE SHALL BE AS SHOWN IN THE TABLE BELOW. (Use "STAGE" column in table if superstructure is staged construction.)

ELONGATIONS SHOWN ARE APPROXIMATE AND MAY BE ADJUSTED BY MTO AFTER THE ACTUAL STRESS-STRAIN PROPERTIES OF THE STRAND TO BE USED ARE AVAILABLE.

ASSUMED $E_p=200\ 000\ \text{MPa}$, $K=0.001$, $\mu=0.14$

STAGE	LOCATION	TENDON SIZE	TYPE	DUCT OD (mm)	JACKED FROM	JACKING FORCE (kN)	ELONGATION (mm)	REQUIRED SLIP (mm)

(Jacking data should be given for simultaneous jacking from both ends and also for jacking at one end followed by final take up jacking at the other.)

- 2. DUCTS FOR PRESTRESSING STEEL SHALL BE SECURELY FASTENED IN PLACE TO PREVENT MOVEMENT UNTIL CONCRETE IS PLACED AND HARDENED. DUCTS SHALL BE SUPPORTED AT INTERVALS NOT EXCEEDING 600 mm OR AS SHOWN ON THE DRAWINGS.

C) Transverse Tendons

- 1. TENDON SIZE, REQUIRED ELONGATION, AND JACKING FORCE SHALL BE AS SHOWN IN THE TABLE BELOW.

ELONGATIONS SHOWN ARE APPROXIMATE AND MAY BE ADJUSTED BY MTO AFTER THE ACTUAL STRESS-STRAIN PROPERTIES OF THE STRAND TO BE USED ARE AVAILABLE.

ASSUMED $E_p=200\ 000\ \text{MPa}$, $K=0.001$, $\mu=0.14$

LOCATION	TENDON SIZE	TYPE	DUCT OD (mm)	JACKED FROM	JACKING FORCE (kN)	ELONGATION (mm)	REQUIRED SLIP (mm)

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(Jacking data should be given for simultaneous jacking from both ends and also for jacking at one end followed by final take up jacking at the other).

2. DUCTS FOR PRESTRESSING STEEL SHALL BE SECURELY FASTENED IN PLACE TO PREVENT MOVEMENT UNTIL CONCRETE IS PLACED AND HARDENED. DUCTS SHALL BE SUPPORTED AT INTERVALS NOT EXCEEDING 500 mm OR AS SHOWN ON THE DRAWINGS.
3. FOR GENERAL NOTES, SEE DWG. ...

D) NOTES (On DECK DETAILS drawing, post-tensioned superstructure.)

1. NO ALLOWANCE IS REQUIRED FOR DEAD LOAD DEFLECTION.
2. THE SCREED ELEVATIONS SHOWN ARE TO BE ADJUSTED FOR FALSEWORK DEFLECTION ONLY, BEFORE THEY ARE USED FOR SETTING SCREEDS.
3. ALL ELEVATIONS ARE TO TOP OF CONCRETE AS SHOWN.

The following notes are applicable only to trapezoidal-voided post-tensioned box-girders:

4. DRAINAGE TUBES TO BE LOCATED AT LOW POINT OF VOID CROSS-SECTION.
5. FORMWORK USED TO FORM VOIDS SHALL BE REMOVED AFTER CONCRETE STRENGTH HAS REACHED 20 MPa.
6. FOR ACCESS HATCH DETAILS, SEE DWG. ...

The following note shall be included on the Deck Detail drawing for post-tensioned deck bridges constructed in stages:

THE CALCULATED APPROXIMATE VERTICAL DEFLECTION BETWEEN THE TIP OF THE CANTILEVER AND SUBSEQUENT STAGE, IS AS FOLLOWS:

STAGE 1: ...
STAGE 2: ... etc.

THE CONTRACTOR SHALL VERIFY THE DEFLECTION AND ADJUST THE SOFFIT FORMS, AND/OR FALSEWORK, TO PROVIDE A SMOOTH TRANSITION FOR A MINIMUM DISTANCE OF 1.2 m FROM THE END OF THE PREVIOUS STAGE.

E) Sequence of Deck Construction (On DECK DETAILS drawing, for solid or circular -voided post-tensioned superstructure.)

1. PLACE CONCRETE IN DECK EXCEPT AT (SIDEWALKS, MEDIAN, CURBS AND BARRIER WALLS) PRESTRESSING ANCHORAGE RECESSES, AND EXPANSION JOINTS.

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2. STRESS ALL TRANSVERSE TENDONS AT PIERS AND ABUTMENTS, THEN IN SPANS, WHEN CONCRETE IN DECK HAS REACHED A STRENGTH OF 30 MPa.

3. ~~PLACE CONCRETE AT TRANSVERSE ANCHORAGE RECESSES.~~

4. STRESS ALL LONGITUDINAL TENDONS AFTER CONCRETE IN TRANSVERSE ANCHORAGE RECESSES HAS REACHED A STRENGTH OF 30 MPa.

~~5. PLACE 35 MPa CONCRETE AT RECESSES FOR ALL LONGITUDINAL TENDONS.~~

~~6.~~ GROUT ALL TRANSVERSE AND LONGITUDINAL TENDON DUCTS.

~~5.6. PLACE 35 MPa CONCRETE AT RECESSES FOR ALL LONGITUDINAL AND TRANSVERSE TENDONS.~~

7. PLACE CONCRETE IN SIDEWALKS, MEDIAN, CURBS AND BARRIER WALLS.

F) Sequence of Deck Construction (On DECK DETAILS drawing, for trapezoidal -voided post-tensioned superstructure.)

1. PLACE CONCRETE IN BOTTOM SLAB (ACCESS HATCH IN POSITION).

2. PLACE REMAINING CONCRETE IN DECK EXCEPT AT (SIDEWALKS, MEDIAN, CURBS AND BARRIER WALLS) PRESTRESSING ANCHORAGE RECESSES AND EXPANSION JOINTS.

3. STRESS ALL TRANSVERSE TENDONS AT PIER(S) AND ABUTMENTS WHEN CONCRETE IN DECK HAS REACHED A STRENGTH OF 30 MPa.

4. PLACE CONCRETE AT TRANSVERSE ANCHORAGE RECESSES.

5. STRESS ALL LONGITUDINAL TENDONS AFTER CONCRETE IN TRANSVERSE ANCHORAGE RECESSES HAS REACHED A STRENGTH OF 30 MPa.

6. PLACE 35 MPa CONCRETE AT RECESSES FOR ALL LONGITUDINAL TENDONS.

7. GROUT ALL TRANSVERSE AND LONGITUDINAL TENDON DUCTS.

8. PLACE CONCRETE IN SIDEWALKS, MEDIAN, CURBS AND BARRIER WALLS.

(The notes above apply to single stage construction. If the deck is cast and stressed in multiple stages, the order of steps 6 and 7 should be reversed).

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8 STRUCTURAL STEEL**8.1 Design****8.1.1 General**

The Structural Manual builds on current design and detailing requirements of the CHBDC. Since the 1960s, MTO has moved towards atmospheric corrosion resistant (ACR) steel to reduce long-term maintenance costs of steel bridges.

The design of an uncoated ACR steel bridge needs to consider the general geography in which the bridge is located (macro-climate), as well as the local, site-specific characteristics of its location (micro-climate). To achieve the performance expected of uncoated ACR steel, prolonged exposure to wetness without opportunity to dry, and exposure -and/or- to high levels of chlorides (Cl⁻), ~~without the opportunity to dry,~~ needs to be prevented.

Time of wetness may be a quantitative measure of the amount of time the relative humidity is greater than 80% and the temperature is above freezing, expressed in hours per year. Prolonged wetness may occur when there is:

- High humidity due to either the general climate or local effects such as surface water or vegetation growing against the bridge;
- Roadway or marine salts ~~that-which~~ slow the drying process and accelerate corrosion, ~~effectively reducing relative humidity to 60%~~; and,
- Debris that traps moisture.

The goal of design and detailing is to prevent the conditions above from occurring. Generally, the structure details should be approached with the mindset of promoting self-cleaning and easy discharge of water to the extent possible. The requirements of Section 8 will permit the use of uncoated ACR steel for girders on most highway bridges in Ontario.

For locations with past performance deficiencies of ACR steel (a stable patina has not developed, and the corrosion product flakes off), replacement with uncoated ACR steel is not recommended. For these locations, enhanced corrosion protection (i.e., metalized or A1010 trapezoidal box girders) may be specified by the Structural Section.

8.1.2 Structural Steel Design Requirements

- a) All structural steel used in highway structures shall be weldable and notch-tough steel. Steel shall conform to CSA standard G40.20/G40.21 Type AT or WT and shall be specified on the contract drawings. The notch-tough requirements for fracture critical and primary tension members shall be 27 Joules and the test temperature shall be dependent on the service temperature and as specified in Table 10.14 and Table 10.15 of the CHBDC.

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Rolled sections shall conform to CSA Standard G40.20/G40.21, ASTM specification A588 or A709. ASTM A709 atmospheric corrosion resistant steel grades shall be designated with a W suffix.

(Note: ASTM A588 may be substituted for G40.21 Grade 350AT steel when the notch-toughness requirements are verified by the submission of test documentation.)

- b) Section 10 of CHBDC defines primary tension members as members, or portions of members, including attachments, other than fracture-critical members and secondary components, that are subject to tensile stress. Girders and attachments fabricated from plate material shall be made from notch-tough grade steel.
- c) All primary members as well as secondary members designed for the forces they attract, as would be the case for curved or highly skewed bridges, shall be Type AT or WT.
- d) Secondary members of straight bridges or bridges with a skew less than 20 degrees may be specified as Type A or W.
- e) Secondary members of curved bridges or highly skewed bridges shall be designed for the forces they are subjected to and shall have the same notch-toughness requirements as the primary members.
- f) ~~All girders shall have the same cross-section. Interior and Exterior longitudinal girders not have less capacity than interior girders, shall have the same cross-section.~~
- g) The availability of the required widths and thicknesses of steel plates should be confirmed before finalising the design. Available lengths are sometimes of importance in relation to the positioning of field splices. Longitudinal splices in web plates are not permitted when depths are less than 3.8 m.
- h) Plate girders shall be specified rather than specifying welded wide flange (WWF) and welded reduced flange (WRF) designations.
- i) ASTM F3125, Grade A325M, Type 1 galvanized bolts shall be used on coated steel and Type 3 shall be used for all connections of uncoated ACR steel. Bolts shall be 22 mm diameter, except that where large numbers are required, in restricted spaces, 1" diameter may be used. For bridges with spans greater than 50 m, bolts shall be 1" diameter.

(Note: ASTM A490M Type 3 bolts may be substituted for ASTM F3125, Grade A325M Type 3 bolts on atmospheric corrosion resistant steel, but galvanized A490 Type 1 bolts are not permitted because of delayed fracture due to hydrogen embrittlement.)

- j) Shear stud connectors shall be 22 mm diameter, not larger, due to limitations of the welding equipment used by fabricators.

~~t) Huck bolt fasteners are acceptable if proposed as an alternative to high strength bolts by the contractor but should not be shown as such on contract drawings.~~

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- k) Any temporary or permanent field welding on all primary and secondary members is not permitted except at the bearing shoe plate connection and for field shear connector installation. If field welding is unavoidable, a written approval from the Head, Structural Section is required. All field splices shall be designed with bolted connections.
- l) Jacking points and loads must be shown.
- m) Uncoated exterior girders shall be checked for structural adequacy assuming a 1.5 mm loss of steel section for:
 1. Girder webs and bottom flanges adjacent to longitudinal median joints, which are not protected from splashing from the roadway above by a solid barrier as shown in Figure 8.1.1(a). Narrow, longitudinal joints are not preferred from a durability perspective, see 9.2.1.
 2. Girder webs adjacent to open railings for bridges with steel box-girders as shown in Figure 8.1.1(b);
 3. Girder webs and bottom flanges adjacent to open railings for bridges with steel I-girders as shown in Figure 8.1.1(c).

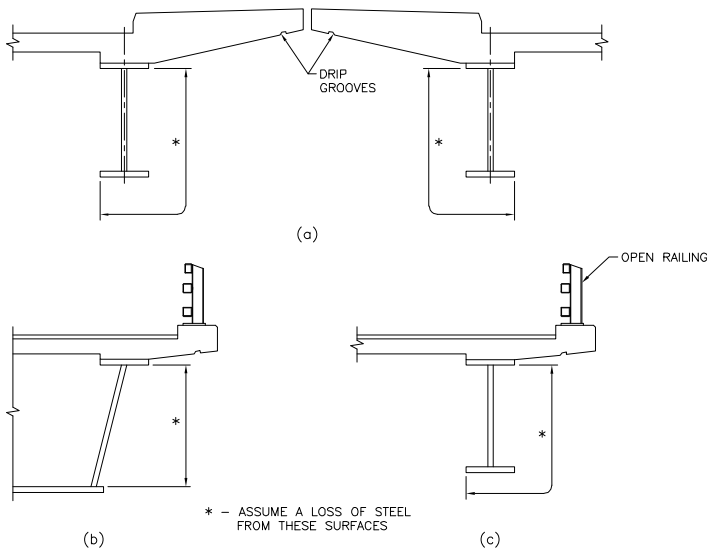


FIGURE 8.1.1 – STEEL GIRDERS BELOW LONGITUDINAL JOINT AND OPEN RAILINGS

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8.1.3 Protection of Steel

An approved shop applied coating is required to protect structural steel at abutments and deck expansion joints from salt laden water run off. The coatings should be applied as follows:

- Except for integral and semi-integral abutment bridges, all structural steel surfaces including diaphragms and bracing, but excluding surfaces in contact with concrete and the contact surfaces of bolted joints shall be coated for a distance of 3000 mm from the ends of the girders.
- For integral abutment bridges all structural steel surfaces, except diaphragms buried in concrete, shall be coated for a distance of 700 mm as follows: from the face of the abutment 100 mm towards the ends of girders and 600 mm towards the centre of girders. The colour of the finish coat shall be indicated on the drawings (see Section 8.7).
- For semi-integral bridges all structural steel surfaces, except the areas of girders and diaphragms encased in concrete, shall be coated as follows: from the ends of the girders to 600 mm beyond the front face of the abutment. The colour of the finish coat shall be indicated on the drawings (see Section 8.7).
- Field splices, at the outside face of exterior girders of I-girder bridges, are susceptible to deterioration at the bottom flange where debris accumulate and trap moisture from wind-driven rain and runoff. Field splices of exterior I-girders shall be coated per Section 8.7. This phenomenon does not occur with box girders, and they shall not be coated at field splices.
- Lapped surfaces of sign and light posts are susceptible to crevice corrosion. Contact surfaces at lap joints shall be coated.

8.1.4 Use of Steel I-Girders Over Roadways

Uncoated ACR steel I-girders shall only be used for the ranges of posted speeds and vertical clearances shown in Figure 8.1.2, along with the protection specified in this manual. Where I-girders are not permitted, trapezoidal box girders shall be used.

Partially coated ACR steel I-girders may be used where metallizing is permitted within Figure 8.1.2.

Fully coated steel I-girders (hot dip galvanized, metallized with a DSM seal coating, or a paint coating system) are permitted within the coated and uncoated range of Figure 8.1.2.

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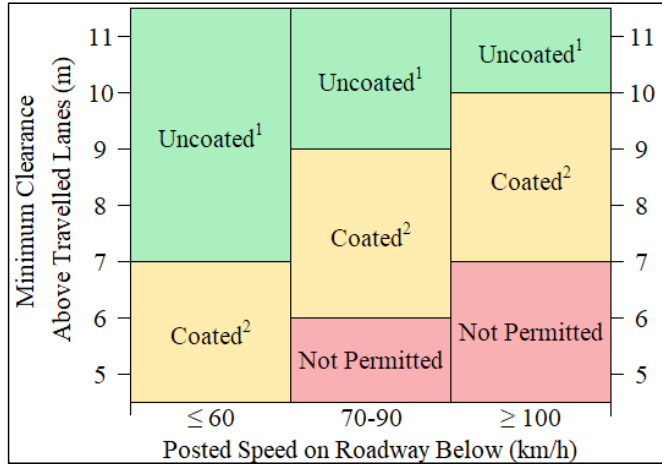


FIGURE 8.1.2 – USE OF ACR I-GIRDERS OVER HIGHWAYS

Notes:

1. ACR steel girders may be uncoated except as required by Section 8.1.3 at girder ends and Section 8.7 at splices of exterior girders.
2. Coated ACR steel I-girders shall have a paint coating of at least the following limits (although a greater extent may be specified for convenience):
 - Exterior girders for the entire length of the bridge with protection of the exterior top flange, exterior web, entire bottom flange, and 500 mm up the inside face of the web (Figure 8.1.3a); and,
 - Interior girders, over travelled lane(s) for a length 10m beyond the edge of any future travelled lane(s), with protection of the bottom flange and 500 mm up each face of the web (Figure 8.1.3b); and,
 - As required by Section 8.1.3 at girder ends and Section 8.7 at splices of exterior girders.

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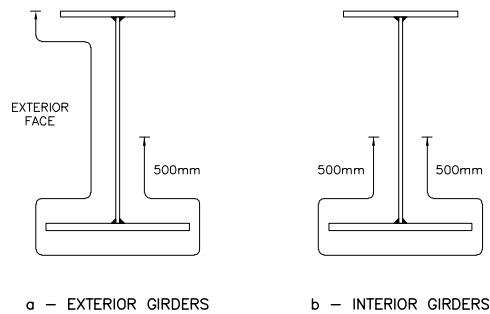


FIGURE 8.1.3 – PAINT COATING MINIMUM LIMITS

Steel I-girders which are not fabricated from ACR steel shall be Type WT, and shall be coated for the full section and length with:

- a) Hot dip galvanizing; or,
- b) Metallizing with a DSM seal coat.

Even where I-girders are permitted, steel trapezoidal box girders are preferred because they have a much lower exposed surface area. Box girders are also less likely to enable bird nesting and entrap salt and debris and are easier to inspect. I-girders are appropriate for most railway and water crossings. Connecticut Department of Transportation findings suggest that stray current from electrified rail line crossings may affect the corrosion rate in existing bridges so proper electrical insulation from electrified rail is recommended.

In recent years, MTO has built a few bridges using ASTM A1010 steel (now included in ASTM A709 as Grade 50CR). This steel has a high (11%) chromium content (which is just below the threshold to be considered a stainless steel) and could also be considered acceptable for locations where ACR I-girders are not permitted. Approval is required from the Structural Section for the use of A1010 steel until standard material and fabrication specifications are developed.

Deviations from the policy above require approval from the Structures Office.

8.1.5 Tunnelling Effects

Vertical clearance requirements were introduced to prevent the splash and spray of traffic from increasing local corrosion rates in the structure. However, sites with limited horizontal and vertical clearances can produce a micro-climate often referred to as a tunnel effect. Steel girder bridges that are determined to be subject to tunnelling effects shall be fully coated. Increased local corrosion is frequently observed on the bottom flange of girders directly over travel lanes of grade separation structures, and the fascia of the inbound girder. Sheets of unbonded patina often develop in these locations which may fall off the structure and pose a risk to traffic below.

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Factors and their values resulting in a tunnel effect have not been definitively identified and research is ongoing. Factors hypothesized to influence tunnelling effects include a combination of:

- Vertical and horizontal clearance, combined with variation in roadway elevation (smaller clearance is worse);
- Width of overpassing structure (wider is worse);
- Proximity of the abutment/pier to the travel lane (closer is worse);
- Average daily traffic (related to quantity of de-icing agents; higher is worse);
- Local wind conditions.

In the absence of high amounts of de-icing agents, low clearances do not generally appear to be detrimental to the performance of uncoated ACR steel. However, the designer should consider the possibility of tunnelling effects when there is a combination of the factors above. The structure's micro-climate may be assessed by referencing the National Steel Bridge Alliance (NSBA) Uncoated Weathering Steel Reference Guide, 2022.

8.1.6 Use of ACR Steel Girders Over Water Crossings

Uncoated ACR steel girders shall not be used over water crossings where prohibited by Table 10.1 of the CHBDC.

Higher humidity areas result in condensation on the structure that accelerates corrosion and doesn't permit the protective patina to develop. Limited vertical clearance over water can cause a localized increase in time of wetness relative to the surrounding macro-climate.

If the macro-climate of the site does not contain a high time of wetness (i.e., does not exceed 2500 hours/year at greater than 60% relative humidity), and minimal vegetation is present within 3m of the structure, there are no concerns with using uncoated ACR steel.

8.1.7 Vegetation

Vegetation in, or nearly in, contact with uncoated ACR steel shall be prevented where doing so is not detrimental to the surrounding environment and environmental regulations permit doing so. Growth of vegetation may be considered during the maintenance of a bridge, but the design should provide a suitable environment. A 3 m clear zone of rock fill or scrub/grasses shall be provided at each side of the wingwalls to permit sunlight to shine on the superstructure.

8.1.8 Standardised Nomenclature for Structural Steel Shapes

The designations for various structural sections are standardised as listed below.

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TABLE 8.1.1 – DESIGNATIONS FOR VARIOUS STRUCTURAL SECTIONS

Shape Designation	Structural Section	Example
C	Standard Channels	C230x30
HP	H-Pile Sections	HP310x79
HSS	Hollow Structural Sections	HSS101.6x50.8x3.18
L	Angles (Give nominal leg x leg x thickness.)	L203x102x13
M	Miscellaneous Shapes	M250x13.4
MC	Miscellaneous Channels	MC250x37
MT	Structural Tees. Cut from M Shapes.	MT100x24.3
S	American Standard Beams	S460x104
ST	Structural Tees. Cut from S. Shapes.	ST 150x37.3
W	Wide Flange. (Give nominal depth x mass per unit length.)	W410x67
WT	Structural Tees. Cut from W Shapes.	WT305x108.5
Z	Zeds	Z130x17.3

Standard designation for steel plate is thickness x width x length, e.g., PL14 x 510 x 3050. Specify all dimensions in mm.

8.1.9 Structural Steel Box Girders Temporary Bracing

For concrete deck slabs on steel girder bridges to be designed using the empirical method, the CHBDC requires that cross frames or diaphragms, at a maximum spacing of 8.0 m c/c, be provided throughout the full cross section width of the bridge, inside and between box girders.

When such diaphragms or cross frames are not provided, temporary bracing to prevent displacement or twisting of the girders may be required, particularly when the deck is placed. -The designer shall check the stability of the girders during the deck **placement**, and if temporary bracing is required, the requirements shall be indicated on the contract drawing. Tack welding the reinforcing steel to the studs is not allowed.

8.1.10 Identification of Tension Zones

To ensure that tension zones in structural steel girders are properly identified so that they can receive the required level of testing during fabrication, the designer shall identify the tension zones for weld testing for the top and bottom flanges on the girder elevation view of the structural steel drawing. See Figure 8.1.4 for example.

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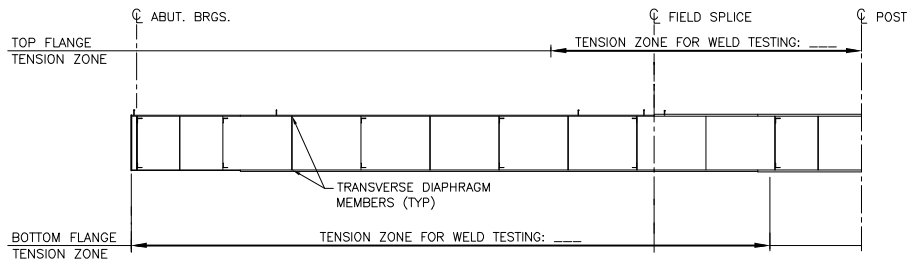


FIGURE 8.1.4 – DRAWING SHOWING TENSION ZONES FOR WELD TESTING

8.1.11 Construction Stage Design Requirements

The designer shall design the structure considering the non-composite structural steel construction stage (loads due to structural steel, formwork, cantilever brackets, wet concrete and construction live loads such as screed machine, equipment, personnel, etc.) as well as the subsequent stages and their contributions to the final stresses in the girder. Intermediate diaphragms and bracing shall be provided to ensure structural adequacy for all construction stages and, except for bridges over freeways, shall remain in place even if some can theoretically be removed after the concrete deck has attained its full 28-day specified strength.

The flanges of exterior girders shall be designed for the combined effect of longitudinal loading and transverse loading due to the wet deck, live load, and the curvature of the girders (if present). Intermediate bracing and connections shall be designed to resist the torsional and transverse moments on the exterior girder due to the cantilever overhang.

OPSS 919 requires screed rails be placed directly over the flange of the exterior girders within 0.1 m of the centreline of the web. In staged construction of the deck of steel girder bridge where only a single lane width of the bridge is constructed, this requirement is at odds with the requirement of OPSS 904 to use a mechanical screed machine to finish the deck, and it is necessary to place the screed rails at the outside of the deck cantilever overhang. When this is the case, the flanges shall also be designed for the transverse loading due to the screed loads acting on the cantilever overhang and if additional temporary bracing is required for the deck pour, the requirements shall be clearly noted in the Contract.

8.1.12 Concrete Cantilever Overhang

For girder depths ≤ 2 m, the overhang of the bridge deck over the steel girder shall be at least equal to the depth of the girder to avoid direct wetting by rain (Figure 8.1.5).

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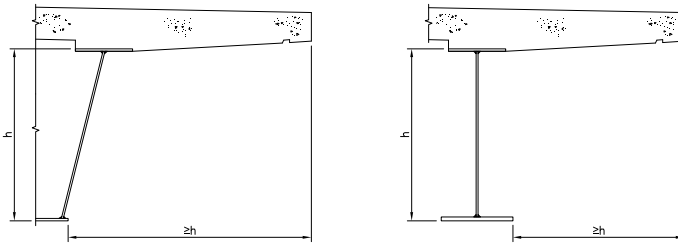


FIGURE 8.1.5 – MINIMUM CANTILEVER OVERHANG

8.1.13 Girder Flange Widths Between Splices

The plate width used for any one flange should be kept constant between field splices.

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.

8.1.14 Transitions of Box Girder Flange and Web Thicknesses

Except when the bridge is to be erected by launching, flange thickness transitions should be made so that a constant depth web plate is maintained. Flange thickness transitions made so that a constant web depth is maintained simplifies fabrication and results in economy.

Web thickness transitions should be made to maintain a flush outer box girder face. Web thickness transitions made so that a flush outer face is maintained, facilitates web splice details. If 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.

8.1.15 Bolt Grid

Bolt holes should be set on an 80 mm x 80 mm grid, or multiples of 80 mm, to facilitate the use of multi spindle drills (see Figure 8.1.6).

The use of a standard grid pattern simplifies fabrication and results in economies. The spacing shall be sufficiently for sealing of the plies to avoid crevice corrosion.

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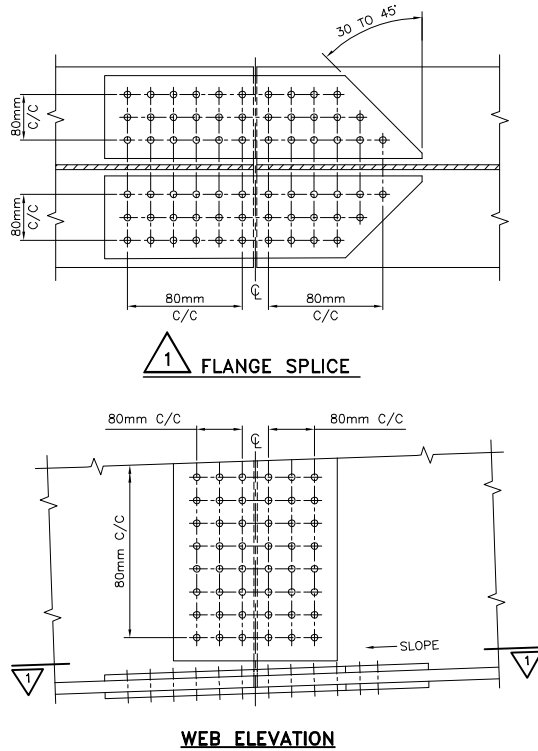


FIGURE 8.1.6 – POSSIBLE BOLT LAYOUT IN A FIELD SPLICE

8.2 Structural Steel Girders Profile

8.2.1 Camber

Camber is defined as the built-in deviation of a bridge member from straight, when viewed in elevation. It is intended to compensate for deflections due to all dead loads and usually includes the roadway profile.

Relaxed camber is the camber which compensates for deflection due to all dead loads, including girder + slab + superimposed dead loads, plus the roadway profile. This is necessary since in the process of fabrication, the self-weight of girders is not acting due to the method of girder support. The contract drawings must show the relaxed camber diagram. Girders are cambered to the values shown in the relaxed camber diagram. The camber cutting diagram, used by the fabricator to cut the web plates to shape for girder segments, is calculated by them based on the relaxed camber diagram, and shown only on the shop drawings. Unless the span length(s) is very short, relaxed camber diagram

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ordinates are given at the 10th point of the span. According to Section 10.7.4 of the CHBDC, for spans less than 25 m the girders need not be cambered. Variations in concrete haunch thickness can be used, in lieu of providing a camber, to achieve the required screed elevations. Where haunching in lieu of camber is used, variable shear stud lengths might be required.

Steel Plate Girders

Plate girders are fabricated with the web in a horizontal position. For plate girders, the relaxed camber diagram shown on the contract drawings is used to check that the required camber in the shop has been achieved within the allowable tolerances. Deflections for girder segments resulting from self-weight of girders are not required to be shown on the contract drawings.

Steel Box Girders

Box girders are generally fabricated with the webs in an upright position. Both webs are supported at close intervals during fabrication. For box girders, in addition to giving the relaxed camber diagram, it is necessary to show on the contract drawings the girder self-weight deflections for girder segments when simply supported at the field splice locations. This allows verification in the plant that the girder segments have been fabricated to the required camber (by subtracting ordinates for deflections for girder segments from the relaxed camber diagram ordinates).

8.2.2 Elevations

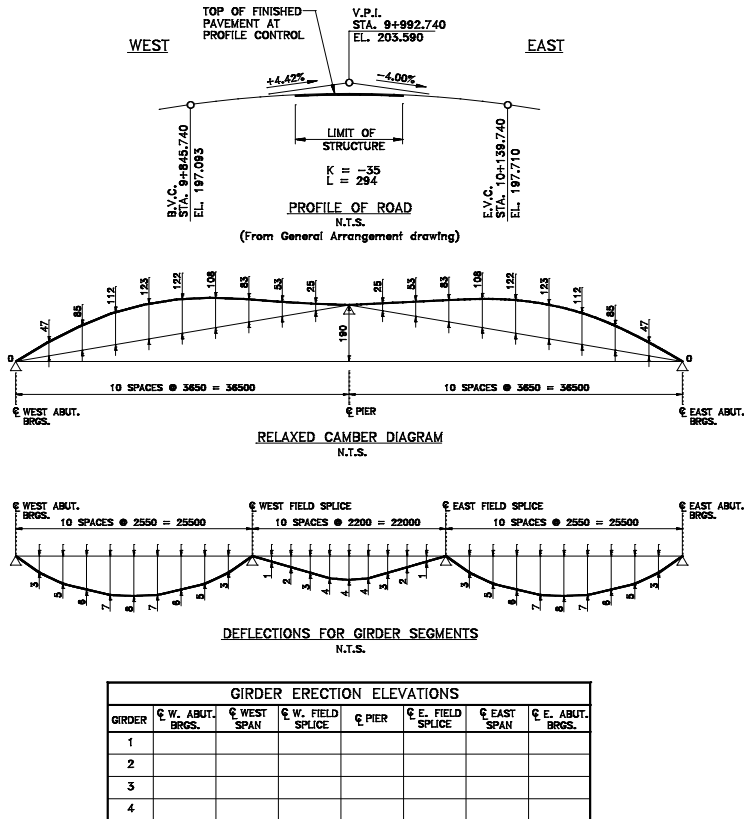
Structural Steel contract drawings shall show girder erection elevations at the top of girders with only the weight of the completely erected structural steel acting. These elevations are given at splice points, mid-span points, piers, and abutments. The elevations should always be given to the top surface of the girders, which will be to top of flange or top of splice plate. This must be clearly indicated on the drawings.

Top of girder elevations are required in order to verify that the girders have been erected within acceptable tolerances to the elevations given on the contract drawings. Prior to placing the deck, the height of the concrete deck haunches are set in order that the screed elevations shown on the contract drawings can be achieved. The as-constructed girder elevations, at the time the erection of all the girders is completed and accepted by the contractor, are permanently recorded.

8.2.3 Example: 2-Span Steel Box Girder

The sketches in Figure 8.2.1 illustrate the various diagrams required for steel box girders, to be shown on the structural steel contract drawings.

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NOTE:
ALL ELEVATIONS ARE TO TOP OF GIRDER AT TOP FLANGE OR TO TOP OF TOP SPLICE PLATE AT FIELD SPLICES, ALONG ☉ OF GIRDER.

Note: If the top flanges at any point are at different elevations due to geometry (horizontal curve of structure or skew of bridge) or different web heights (large deck crossfall), two girder erection elevations will have to be given, one at the top of each flange.

FIGURE 8.2.1 – STRUCTURAL STEEL GIRDERS PROFILE EXAMPLE FOR 2-SPAN STEEL BOX GIRDER

8.3 Details

A steel structure should not be permanently wet or damp. Even if the general environment is satisfactory, it is important to ensure good detailing to provide a natural drying process and prevent accumulation of moisture and debris.

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Water ponds and debris accumulate on horizontal surfaces and in corners formed by horizontal and vertical plates (re-entrant corners), fostering excessive corrosion. ~~For example, a girder bottom flange should not be oriented level in the transverse direction, even if the web needs to be out of plumb.~~

Avoid detailing which creates pockets and crevices. The most susceptible locations are bottom flanges, gusset plates for horizontal bracing, longitudinal stiffeners, bolted splices of horizontal and sloped members, and intersections of bearing and intermediate stiffeners with flanges and gusset plates.

8.3.1 Structural Steel Box Girders Bottom Flanges

The weld joining the bottom flange and the web should be detailed so that any projection of the flange outside of the web plate where salt could collect is minimized. A projection of 25 to 40 mm beyond the edge the web is sufficient.

Pier and abutment diaphragms shall be coped to clear continuous welding of the web-to-flange joint. At abutments, the copes shall have a mesh caulked over the copes to prevent the entry of rodents or birds into the girders (see Figure 8.3.1).

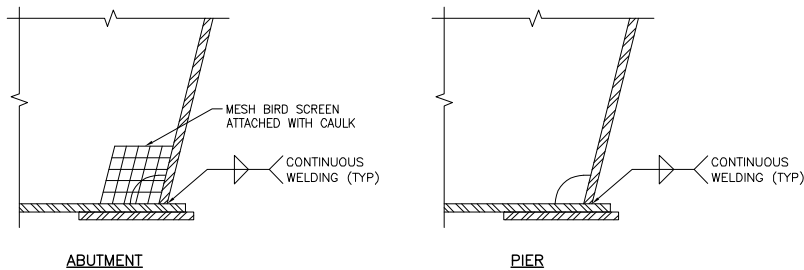


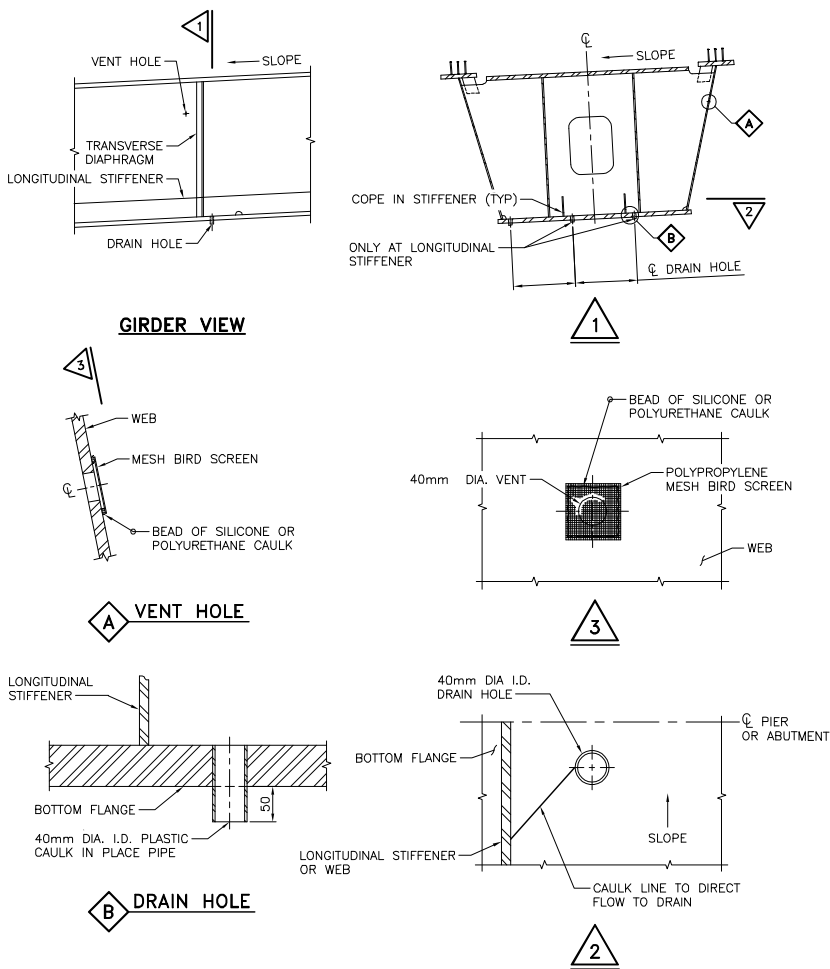
FIGURE 8.3.1 – BOTTOM FLANGE WELDING DETAILS

8.3.2 Box Girder Drainage and Ventilation

Drains are required through the bottom flanges of box girders wherever water can collect. These should be detailed to prevent water from running along the soffit and to keep out birds. As a minimum, drains should be located at each end of every span and at low spots. Drains shall project 50 mm below the bottom of the bottom flange. Cut outs in longitudinal stiffeners shall be detailed at drains as necessary. Drains shall be located to avoid staining of the substructure.

Adequate ventilation of the interior of box girders must be ensured to allow a draft. Two vent holes per span shall be provided on the interior web of each girder in each span. Vents shall be provided with screens attached with silicone caulking. Vent holes and drains shall be 40 mm diameter.

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Note: Multiple drain holes could be provided, or copes could be provided in stiffeners to channel runoff to one drain hole.

FIGURE 8.3.2 – BOX GIRDER DRAINAGE AND VENTILATION

8.3.3 Access to Box Girder Voids

CHBDC Clause 1.8.3.1.5 requires access openings for the inspection and maintenance of steel box sections having inside vertical dimensions of 1.20 m or greater.

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Box girders with access openings shall have a minimum inside clear height of 1200 mm. The minimum clearance, both vertically and horizontally, for walking through intermediate cross bracing and diaphragms inside box girders shall be preferably 600 mm and not less than 500 mm.

The number of access openings and their locations shall be as follows:

- a) For simple span steel box girders with walk through cross bracing: Two openings per box girder, one located near each end;
- b) For continuous steel box girders with walk through cross bracing and pier diaphragms: Two openings per box girder, one located near each end. For exceptionally long girders, the need for additional intermediate openings should be determined by the Structural Section and stated in the Structural Design Report.

The access openings shall be located to avoid or minimise interference with traffic during inspection and should be easily accessible to inspectors, but not to the general public. In Ontario, the access openings are usually located in the web near abutments. This would reduce the shear resistance of the web. Therefore, the access openings should not be located in the steel box area subjected to high shear forces and torsional moments. Web access hole should be added in the space between stiffeners. Access openings shall not be located on the exterior web of the girder where screens may be directly exposed to the elements.

Details of hatch for the access opening shall be as per OPSD 3311.900. The shear resistance of the web at the access openings shall be ~~checked~~designed along with transverse and longitudinal stiffeners around the opening, as required.

~~Vented access openings shall not be located on the exterior web of the girder where screens may be directly exposed to the elements.~~

8.3.4 Drip Tab Details

Drip tabs shall be used on steel beams and girders to prevent water along girders from being deposited on abutments and piers. Drip tabs divert water and debris from collection points, and to reduce or eliminate objectionable staining of the concrete substructure when the steel is ACR. When required, drip tabs shall be attached to all steel beams or girders in the shop, since staining may occur prior to slab placement.

Drip tabs should be shown both in plan and in elevation. They should be located such that the ends of the tabs are 1.0 m in front of the front face of the abutment wall, and 1.0 m to the side of a pier when the girder slopes upwards away from the pier.

Structures on crest curves shall have tabs in front of both abutments. Structures on a grade shall have tabs in front on the abutment on the low side of the bridge only. Structures with a sag curve, where the low point is away from the abutments, generally do not need drip tabs.

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Where water/debris may collect, such as when splice plates cannot be tapered or stiffeners cannot be provided with a cope (snipe/clip) according to Section 8.4.1, additional drip tabs should be provided.

Steel drip tabs falling within the coated zone specified in Section 8.7 note 13 shall also be coated.

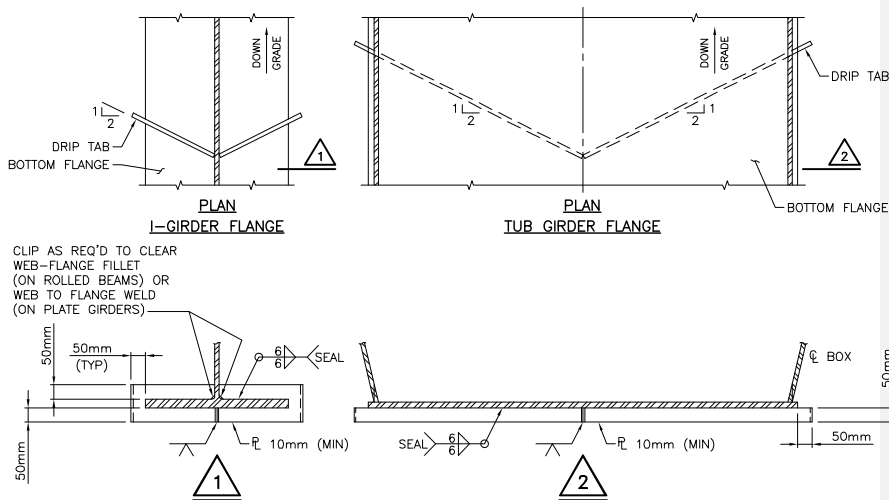


FIGURE 8.3.3 – DRIP TAB DETAILS FOR I-GIRDERS AND BOX GIRDERS

8.3.5 Girder Web Spacing

The spacing of girder webs between girders shall be no closer than the larger of 1.8 m or the depth of girder (Figure 8.3.4). Avoiding closely spaced girders provides adequate ventilation for drying.

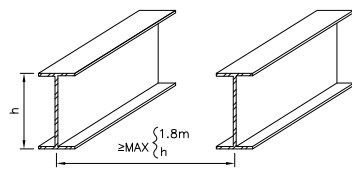


FIGURE 8.3.4 – GIRDER SPACING

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8.4 Stiffeners and Connection Plates

8.4.1 Coping of Stiffeners and Gusset Plates

Copes on transverse stiffeners shall be inverted J-clips, with at least the minimum dimensions shown in Figure 8.4.1. Copes on other details, such as longitudinal stiffeners and gusset plates, shall be quarter-round, not less than 50mm in radius (see Figure 8.4.2).

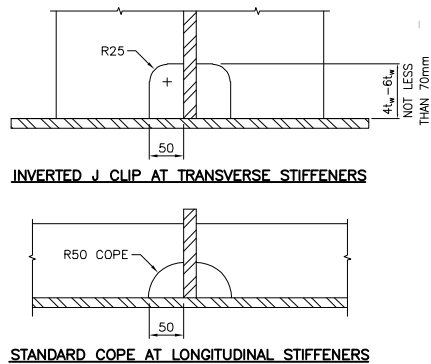


FIGURE 8.4.1 – COPE DETAILS

These larger copes are desirable for the following reasons:

- 1) They prevent the possibility of intersecting welds;
- 2) They reduce the high weld shrinkage strains associated with smaller copes; and,
- 3) They allow drainage without the buildup of debris.

At end diaphragms of box girders, copes shall be filled with weld or caulking to prevent entry of rodents or birds. This generally dictates a drain at the diaphragm.

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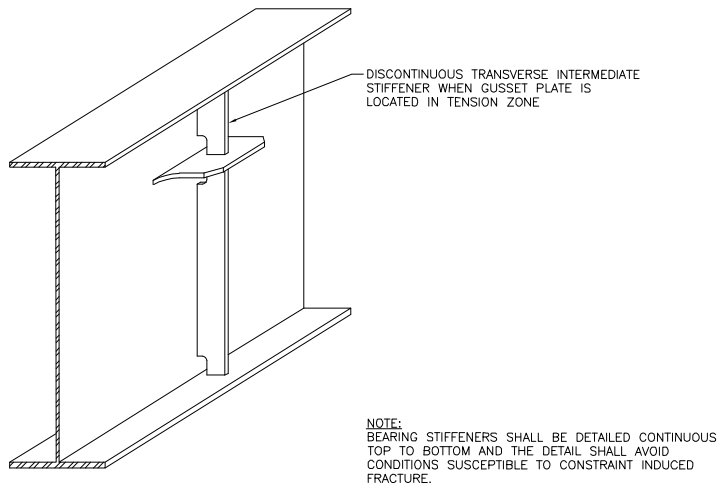
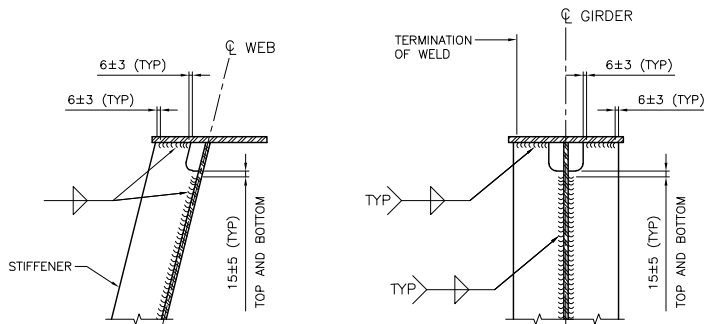


FIGURE 8.4.2 – COPING OF STIFFENERS AND LOCATION OF GUSSET PLATES

8.4.2 Fillet Weld Terminations

Fillet welds shall be detailed in such a way that the introduction of notch type deficiencies into the base metal is minimized. Due to the risk of creating notch type deficiencies if fillet welds are continued to the end of the joint or wrapped around the end of the joint, it is preferable to stop the weld short. An accepted practice is to stop welds short of the edge of the base metal by a distance equal to the designed fillet weld leg length +/- 3mm for shorter welds, such as those joining vertical stiffeners to flanges, and by 15mm +/- 5mm for longer welds, such as those joining stiffeners to webs. A weld termination detail shall be included in the structural steel drawings.

Fillet weld hold backs shall be indicated on the contract documents.



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FIGURE 8.4.3 – WELD TERMINATION DETAILS

Design details shall avoid the need to extend fillet welds to the edge of the base metal. In cases where fillet welds must reach the edge of the base metal and the joint will not be cut to length after welding, runoff tabs shall be used.

8.4.3 Lateral Bracing

Lateral bracing shall be provided only where required and shall be connected directly to the flange where feasible. When it is not feasible, lateral bracing may be connected to lateral gusset plates.

All gusset plates for lateral bracing should be fillet welded and be located a distance as required by the CHBDC and practical situations. The outer corners of the gusset plates should be left square unless fatigue design requires a radiused gusset plate. "Bridge Fatigue Guide, Design and Details" by J. W. Fisher should be consulted when determining the location of bolt holes. See also Figure 8.4.2.

Several factors should be taken into consideration in determining the position of lateral bracing gusset plates.

- 1) Access for fabricating and inspecting the gusset plate to web connection; and,
- 2) The fatigue performance; lateral bracing bolted directly to the flanges has superior fatigue performance, whereas gusset plates can be moved away from the flange into a lower stress region. For girders with a depth of up to 2.4 m, the bracing shall be connected to the top flange or connected to gusset plates installed close to the top flange.
- 3) Ability to form the deck. If precast panels are used, the bracing shall be connected directly to the flange.

8.4.4 Vertical Stiffeners

Bearing Stiffeners on plate girder bridges shall be true vertical under full dead load with the requirement noted on the contract documents (see 8.7(a)(7)). Intermediate stiffeners may be either true vertical, or perpendicular to fabrication work lines, depending on the fabricator's practice.

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.

Stiffeners should be detailed to avoid trapping water and debris (Figure 8.4.4) and shall be coped. Stiffeners consisting of closed sections such as a half pipe shall have copes filled with weld.

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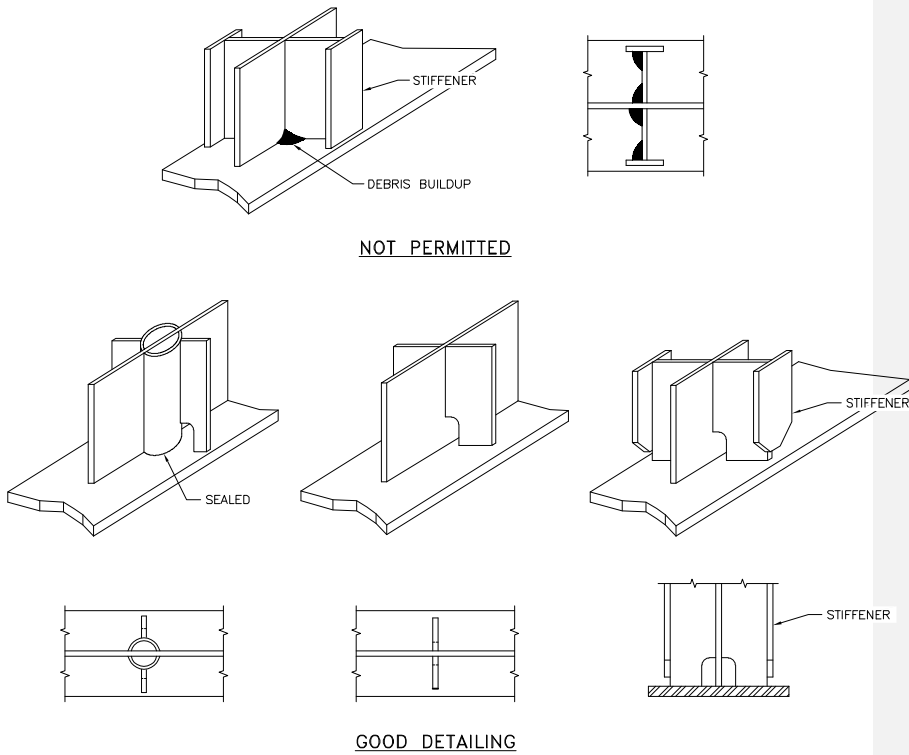


FIGURE 8.4.4 – CHOICE OF VERTICAL STIFFENERS

8.4.5 Bearing Stiffener to Flange Connection

Bearing stiffeners, irrespective of thickness, shall be fitted to bear and welded to both flanges at abutment locations as well as at interior supports, as shown in Figure 8.4.5. The size of the weld shall be specified on the contract drawings.

Because the load being transferred through the stiffeners may be too large to be transferred through the welds alone, bearing stiffeners are required to be fitted to bear in their contact to the inner surface of the flanges.

OPSS 906 describes the contact tolerance of “fitted” and “fitted to bear”.

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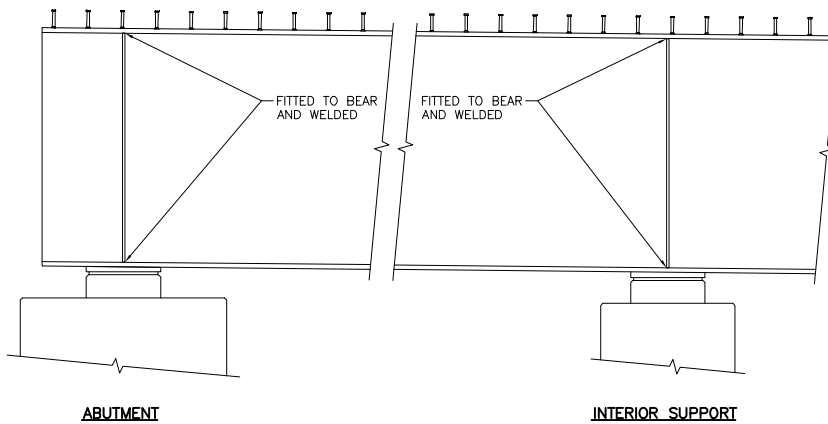


FIGURE 8.4.5 – BEARING STIFFENER TO FLANGE CONNECTION

8.4.6 Intermediate Stiffener to Flange Connection

Intermediate stiffeners that are used as connection plates on I-girders and on the inside of box girders shall be welded or bolted to both the tension and compression flanges depending on fatigue requirements.

Intermediate stiffeners that are not used as connection plates on I-girders and on the inside of box girders shall be welded to the compression flange and snug fit or welded to the tension flange depending on fatigue requirements.

8.4.7 Stiffener to Web Connection

All stiffeners shall be welded to the webs of the girders by continuous fillet welds which shall not be returned around the end of the stiffener. The size of these welds shall be the larger of the "minimum size" given on the contract drawings and the size which corresponds to a shear force of $0.0001 hF_y^{1.5}$ (N/mm) as given in the CHBDC.

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. The weld return about the end of the stiffener is undesirable as it aggravates the fatigue problem in the web between the stiffener and girder flange.

Longitudinal web stiffeners shall be plate to avoid trapping water/debris.

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8.4.8 Intersecting Longitudinal and Transverse Stiffeners

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

Intersection of stiffeners is sometimes unavoidable. Where longitudinal and transverse stiffeners intersect, the longitudinal stiffener should be cut short of the transverse stiffener. Cutting the longitudinal stiffener in tension regions results in a category E detail. This detail 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. Alternately, 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.

8.4.9 Box Girder Web Stiffeners

Web stiffeners on the inner and outer faces of box girders should be cut short of the bottom flange as shown in Figure 8.4.6 in order to allow use of automatic welding of the web-to-flange joint. This is necessary because the process of fabricating the box girders calls for the web stiffeners to be welded prior to welding the web to the flanges. The stiffener is then extended to the bottom flange by the attachment of a plate as shown in Figure 8.4.6. This plate shall be welded, bolted, or fitted to the bottom flange depending on its location (i.e., used or not used as connection plate) and fatigue requirements. The plate shall be welded to the flange where fatigue stress range permits. The connection of bracing to the outer faces of box girders shall be as shown in Figure 8.4.6 and should be opposed by an interior stiffener welded to the bottom flange.

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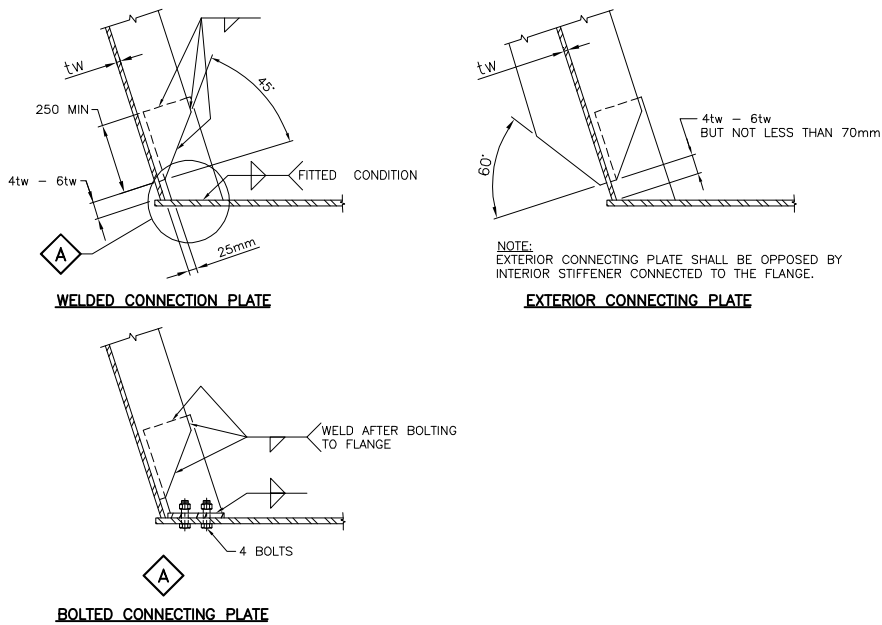


FIGURE 8.4.6 – BOX GIRDER WEB STIFFENERS

8.4.10 Transition Radius for Longitudinal Stiffeners and Gusset Plates

The ends of the transition radius for radiused gusset plates and longitudinal stiffeners shall be ground smooth (see Figure 8.4.7). The minimum radius (R) shall be based on the target fatigue category required in accordance with Clause 10.17 of the CHBDC. The radius should be 600 mm for Category B and at least 150 mm for Category C.

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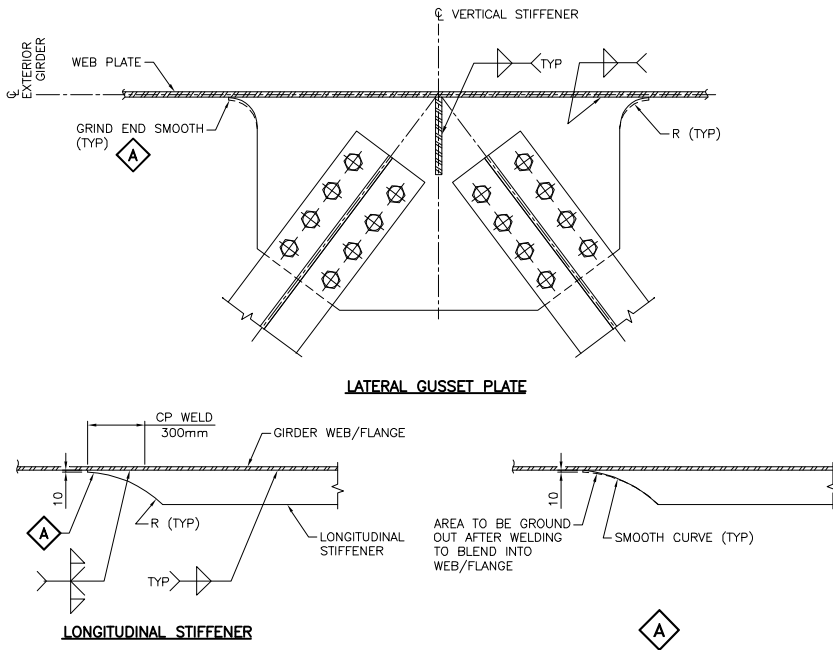


FIGURE 8.4.7 – TRANSITION RADIUS FOR GUSSET PLATES AND LONGITUDINAL STIFFENERS

8.4.11 Bottom Flange Stiffener Details

In terms of their structural function, wide flange "W" or "WT" sections are preferred for longitudinal bottom flange stiffeners. The sections should be spaced a minimum of 305 mm between flanges to allow the use of automatic welding equipment. For box girders fabricated from ACR, plate section should be used for longitudinal bottom flange stiffeners where practical due to better availability of 'AT' steel plate than rolled sections. 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 (see Figure 8.4.8).

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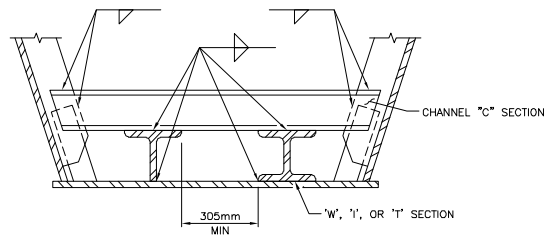


FIGURE 8.4.8 – BOTTOM FLANGE STIFFENER DETAILS

8.4.12 Clipping Connection Plates

Flange splice plates within a bolted field splice are a common water trap. If the girder is on a vertical grade, water flowing down the flange will get trapped by the flange splice plates. The bottom flange splice plates may be sniped at their leading and trailing ends to combine the function of a drip tab and facilitate water shedding. Where splice plates are clipped, it should be at an angle between 30 and 45 degrees (see Figure 8.1.6).

Clipping is not necessary for top flange splice plates embedded in a concrete deck.

8.5 Frames, Bracing and Diaphragms

8.5.1 Frames for Intermediate Bracing

Prefabricated frames should be used for intermediate bracing, in lieu of angle sections shipped loose to the site, and should be designed for shipping and erection as a single unit. All frames should be designed and detailed for fabrication from one side, eliminating the need for "turning over" during fabrication. A "K" brace angle system is preferred over "X" bracing when the girder spacing exceeds the girder depth, as it allows more space for workers and inspectors to pass through. Bracing connections need to be bolted connections with rectangular gusset plates. Oversized holes in the gusset plates are permitted when required.

Except for bridges over freeways, the preferred "K" brace system for use between girders consists 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 without the need to turn the frame, and when fewer pieces are handled in the field.

For bridges over freeways, intermediate bracing between tub girders shall be avoided altogether. Where absolutely necessary, frames consisting of angles shall be bolted or welded all around to gusset plates to seal the interface between angle and gusset plate to minimize crevice corrosion.

If "X" bracing is used, it shall be detailed with tightly fitting filler plates at the intersection of angles to minimize crevice corrosion.

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8.5.2 Box Girder Bracing

Unless required otherwise by design, L 102 x 102 x 9.5 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 and the additional attachments shall be subject to approval by the Ministry. The designer should ensure that the interior bracing can be welded to the web stiffeners (see Figure 8.5.1). In the case of X bracing if acceptable structurally, the intersection of the two bracing elements need not be connected.

The size chosen is adequate for the typical range of bridge spans built in Ontario.

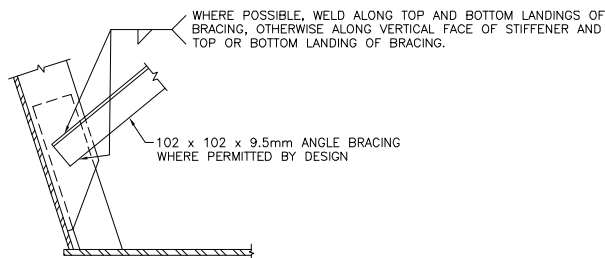


FIGURE 8.5.1 – BOX GIRDER BRACING

8.5.3 Intermediate Diaphragms in Shallow Girders

Constant depth intermediate diaphragms, in lieu of frame bracing, are preferred in I-girder bridges with a web depth of up to 1200 mm.

Diaphragms fabricated from channel, bent plate, or beam sections are preferred in shallow girder bridges over welded plate sections.

8.5.4 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 8.5.2(a)). Two possible solutions are shown.

Diaphragms at abutments are normally of a shallower depth to allow for deck end thickening details. In this case, the box girder flanges should be stabilised against rotation (see Figure 8.5.2(b)).

Diaphragms between box girders at piers and at abutments should be of constant depth and bolted to box girder web stiffeners (see Figure 8.5.2(c)). Oversized holes in diaphragms or stiffeners are permitted.

At piers and abutments with bearings, provisions for jacking within the width of the bottom flange should be provided for by design.

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The details as shown in Figure 8.5.2 were developed to meet design and fabrication needs.

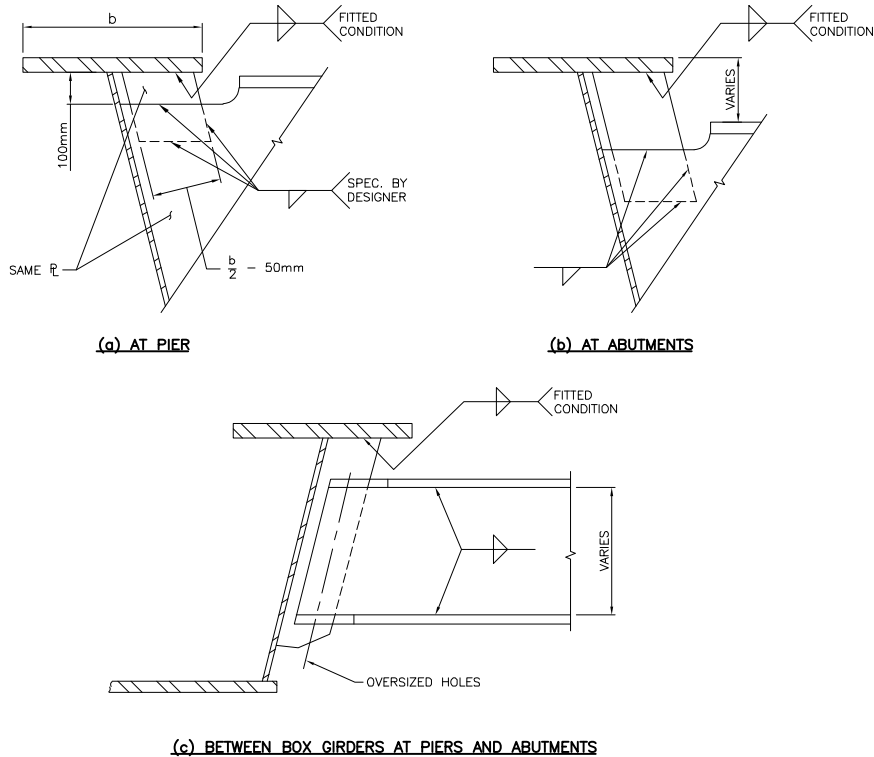


FIGURE 8.5.2 – BOX GIRDER DIAPHRAGMS

8.6 Weld Details

8.6.1 Transition of Web Thickness

Web splice details indicating a CJP weld shall be included in the structural steel drawings according to Figure 8.6.1. The plate preparation and specific weld details are provided by the fabricator according to their approved welding procedures.

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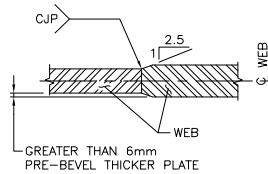


FIGURE 8.6.1 – WEB SPLICE DETAIL

8.6.2 Transition of Flange at Butt Welds

Transition of flange thickness at butt welds should be made in accordance with CSA W59 with a slope through the transition zone not greater than 1 in 2.5.

A slope of 1 in 2.5 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.

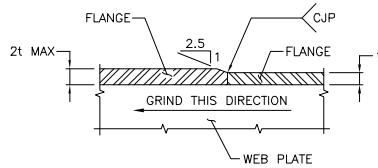


FIGURE 8.6.2 – FLANGE SPLICE DETAIL ELEVATION

Flange width transitions at butt welds shall be avoided. If absolutely necessary, they shall be detailed according to Figure 8.6.2.

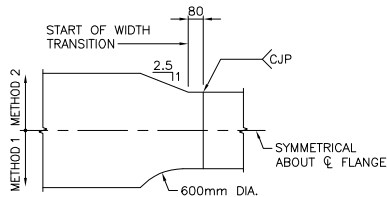


FIGURE 8.6.3 – FLANGE WIDTH TRANSITION PLAN

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8.6.3 Grinding of Butt Welds

In the following, "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 radiographic testing (RT) or ultrasonic testing (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

(a) Butt welds in webs of girders designed for tension in category B shall be "flush" for a distance of 1/3 the web depth from the tension flange.

(b) All other butt welds designed for tension in category B shall be "flush".

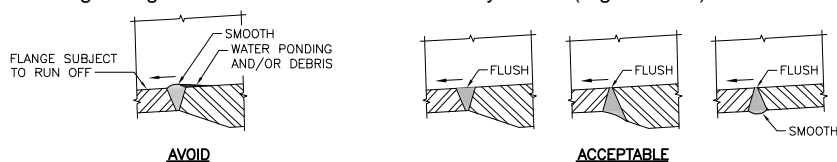
(c) Butt welds designed for compression only or for stresses in category C shall be at least "smooth".

These recommendations are in line 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 over-grinding 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.

In I-girders, the top surface of bottom flange butt welds shall be "flush". The designer shall consider grinding other butt welds where water may collect (Figure 8.6.4).



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FIGURE 8.6.4 – GRINDING FLUSH OF WELDS WHICH OTHERWISE FORM WATER TRAPS

8.6.4 Bearing Shoe Plate Welds

Bearing shoe plates should preferably be welded only longitudinally to plate girder flanges. If the shoe plates are to be welded in the field, they should be wider than the girder flanges to facilitate welding in the downwards horizontal position.

For box girders, unless design requirements dictate otherwise (e.g., skew), double bearings should be used at both piers and abutments. Where the bearing shoe plates are to be welded in the field, they should be positioned to allow longitudinal welds in the horizontal position between flange and shoe plate. In addition, short transverse welds, between flange and shoe plate, of approximately 150 mm in length, should be provided (see Figure 8.6.5). Sufficient access should be provided to execute these transverse welds.

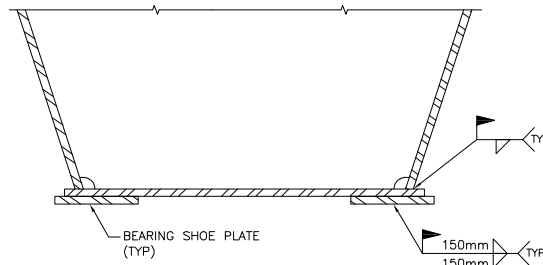


FIGURE 8.6.5 – BEARINGS AT PIERS AND ABUTMENTS

8.7 Structural Steel Notes

The following are standard notes to be shown below the title block on the appropriate drawing. The notes must be worded to cover the requirements specific to the particular project and should only be used if applicable. Other notes may be required in special circumstances.

For clarity the wording of the notes is shown in upper case (CAPITAL) lettering. Explanations shown in brackets in lower case lettering are not part of the notes.

A. General Notes

1. ALL STRUCTURAL STEEL SHALL CONFORM TO CSA G40.20/G40.21 GRADE 350AT. THE CHARPY IMPACT ENERGY REQUIREMENTS SHALL BE 27 JOULES AND THE TEST TEMPERATURE SHALL BE ... °C.

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(Note: For fully coated structural steel, Type AT shall be replaced with Type WT in the note above. Obtain the test temperature requirements from the appropriate table of CSA S6 for the service temperature given for the location).

ROLLED SECTIONS SHALL CONFORM TO CSA G40.20/G40.21, ASTM A588/A588M OR ASTM A709/A709M.

(Note: Add Charpy impact test requirements for primary tension members)

2. BOLTS ON ATMOSPHERIC CORROSION RESISTANT STEEL SHALL BE ASTM F3125/F3125M, GRADE A325M TYPE 3, M22. BOLTS ON COATED STEEL SHALL BE GALVANIZED GRADE A325M TYPE 1, M22. BOLT THREADS SHALL BE EXCLUDED FROM THE SHEAR PLANES.

(Note: ASTM F3125/F3125M, GRADE A490M Type 3 bolts may be substituted for ASTM F3125/F3125M, GRADE A325M Type 3 bolts on ACR steel, but galvanized A490 Type 1 bolts on coated steel are not permitted because of delayed fracture due to hydrogen embrittlement.)

3. STUD SHEAR CONNECTORS SHALL BE 22 mm DIA. AND CONFORM TO ASTM A108 AND CSA W59.
4. ALL LENGTHS SHOWN ARE IN THE HORIZONTAL PLANE AND MEASURED AT 20 °C.
5. GIRDERS SHALL BE CAMBERED TO VALUES SHOWN IN THE RELAXED CAMBER DIAGRAM.
6. RELAXED CAMBER ORDINATES INCLUDE AN ALLOWANCE FOR GIRDER SELF-WEIGHT, CONCRETE DECK, SUPERIMPOSED DEAD LOADS AND PROFILE OF HIGHWAY.

(Note: If the bridge is a multi-span steel box-girder structure, the following note should be included.)

ADJUSTMENTS SHALL BE MADE TO THE RELAXED CAMBER DIAGRAM TO COMPENSATE FOR THE DEFLECTION OF THE INDIVIDUAL GIRDER SEGMENTS.

7. THE ENDS OF GIRDERS AND BEARING STIFFENERS SHALL BE TRULY VERTICAL UNDER FULL DEAD LOAD.
8. ALL BUTT WELDS IN FLANGE AND WEB SHOP SPLICES SHALL BE FINISHED FLUSH OR SMOOTH AS INDICATED. MARKS LEFT BY GRINDING TO BE IN THE DIRECTION OF APPLIED STRESS. IF SHOP SPLICES ARE REQUIRED IN LOCATIONS OTHER THAN THOSE WHERE PLATE SIZES HAVE TRANSITIONS, THEIR LOCATION SHALL BE APPROVED BY THE ENGINEER.

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(Note: The weld finishes to be indicated on the drawing using the appropriate standard symbol (see Section 8.6.3)).

9. UNLESS OTHERWISE NOTED THE MINIMUM FILLET WELD SHALL BE AS FOLLOWS:

MATERIAL THICKNESS OF THICKER PART JOINED (mm)	MINIMUM SIZE OF SINGLE PASS FILLET WELD (mm)
TO 12 INCLUSIVE	5
OVER 12 TO 20	6
OVER 20 TO 40	8
OVER 40 TO 60	10
OVER 60 TO 120	12

10. FILLET WELDS BETWEEN STIFFENERS AND WEBS SHALL STOP SHORT OF THE EDGE OF THE BASE METAL BY 15 +/- 5 mm. ALL OTHER FILLET WELDS SHALL STOP SHORT OF THE EDGE OF THE BASE METAL BY A DISTANCE EQUAL TO THE DESIGNED FILLET WELD LEG LENGTH +/- 3 mm.
11. UNLESS OTHERWISE NOTED, LONGITUDINAL WEB STIFENERS SHALL BE CUT 25 mm SHORT OF THE TRANSVERSE WEB STIFFENERS.
12. BOLT HEADS IN FIELD SPLICES FOR BOX GIRDERS SHALL BE LOCATED ON THE EXTERIOR SURFACES.
13. ALL STRUCTURAL STEEL SURFACES, INCLUDING DIAPHRAGMS AND BRACING, BUT EXCLUDING SURFACES IN CONTACT WITH CONCRETE AND THE CONTACT SURFACES OF BOLTED JOINTS SHALL BE COATED FOR A DISTANCE OF 3000 mm FROM THE ENDS OF GIRDERS AT EXPANSION JOINTS. THE COLOUR OF THE TOPCOAT SHALL BE ...

(The colour of the topcoat or finishing coat to be inserted by the designer as follows:

- **10045 brown** According to Federal Standard 595C Colours (for ACR steel girder bridges); or,
 - **16307 grey** According to Federal Standard 595B Colours (for all other steel including any ACR steel used in the rehabilitation of carbon steel structures).
14. If the bridge is integral the following note shall be added:

ALL STRUCTURAL STEEL SURFACES SHALL BE COATED FOR A DISTANCE OF 700 mm AS FOLLOWS: FROM THE FRONT FACE OF THE ABUTMENT 100 mm TOWARD THE ENDS OF GIRDERS AND 600 mm TOWARDS THE CENTRE OF GIRDERS. THE COLOUR OF THE TOPCOAT SHALL BE ...

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(The colour of the topcoat or finishing coat to be inserted by the designer should be as shown in note 12.)

If the bridge is semi-integral the following note shall be added:

ALL STRUCTURAL STEEL SURFACES, EXCEPT DIAPHRAGMS, SHALL BE COATED AS FOLLOWS: FROM THE ENDS OF THE GIRDERS TO 600 mm BEYOND THE FRONT FACE OF THE ABUTMENT. THE COLOUR OF THE TOPCOAT SHALL BE ...

(The colour of the topcoat or finishing coat to be inserted by the designer should be as shown in note 12.)

15. CONTRACTOR SHALL ENSURE THE STABILITY OF ALL COMPONENTS DURING HANDLING, TRANSPORTATION AND ERECTION AND UNTIL THE STRUCTURAL STEEL IS IN ITS FINAL LOCATION WITH ALL PERMANENT BRACING, CONNECTIONS AND SUPPORTS IN PLACE AND THE CONCRETE IN THE DECK HAS REACHED 75% OF ITS SPECIFIED STRENGTH.

16. If the bridge is a multi-span steel box-girder structure, the following note 15 should be included:

ADJUSTMENTS SHALL BE MADE TO THE RELAXED CAMBER DIAGRAM TO COMPENSATE FOR THE DEFLECTION OF THE INDIVIDUAL GIRDER SEGMENTS.

17. The designer shall add the following note to the structural steel drawings at exterior ~~girder~~ field splice locations of I-girders, unless the entire exterior I-girder is coated:

ALL STRUCTURAL STEEL SURFACES OF EXTERIOR I-GIRDERS, INCLUDING SPLICE PLATES, BUT EXCLUDING SURFACES IN CONTACT WITH CONCRETE AND THE CONTACT SURFACES OF BOLTS JOINTS, SHALL BE COATED FOR A DISTANCE OF 2000 mm ON EITHER SIDE OF THE CENTRELINE OF A FIELD SPLICE.

B. Notes (on DECK DETAILS drawing, structural steel girder superstructure)

1. SCREED ELEVATIONS ARE TO TOP OF CONCRETE.
2. SCREED ELEVATIONS SHOWN IN TABLE INCLUDE AN ALLOWANCE FOR ROADWAY PROFILE, WEIGHT OF DECK SLAB AND SUPERIMPOSED DEAD LOAD.
3. CONCRETE IN DECK SLABS AND DIAPHRAGMS SHALL BE RETARDED USING A TYPE B OR D ADMIXTURE TO ENSURE THAT THE CONCRETE REMAINS PLASTIC FOR THE DURATION OF EACH PLACEMENT.

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4. CONCRETE SHALL REMAIN PLASTIC IN POURING OF SEGMENTS WITH THE SAME SEQUENCE NUMBER.
5. MINIMUM CONCRETE STRENGTH OF PREVIOUS DECK PLACEMENT SHALL BE 20 MPa BEFORE PROCEEDING WITH THE PLACEMENT.
6. CONCRETE IN BARRIER WALLS (AND SIDEWALK(S)) SHALL NOT BE PLACED UNTIL ALL CONCRETE IN DECK SLAB HAS REACHED A STRENGTH OF 20 MPa.
7. FALSEWORK FOR THE CANTILEVER PORTIONS OF THE DECK SLAB (FOR A LENGTH OF 2.0 m) SHALL NOT BE REMOVED UNTIL THE CONCRETE AROUND THE INSTALLED EXPANSION JOINT HAS REACHED A STRENGTH OF 20 MPa (TYP. AT ALL FOUR CORNERS OF THE BRIDGE).

8.8 Steel Truss and Arch Design

In the design and evaluation of steel trusses and arch structures, connections are assumed to be typically pinned. However, the effects of connection rigidity or seized pins shall also be considered during analysis to determine whether resulting bending stresses are acceptable and fatigue will not be an issue.

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9 BRIDGE DECKS**9.1 General**

This section covers requirements for bridge decks, including sidewalks and curbs.

9.2 Deck Geometry

The travelled width of the bridge deck typically matches that of the approach roadway, although some exceptions are possible depending on roadway class. Consideration is also made to future rehabilitation of the bridge. Requirements can be found in the MTO Design Supplement for TAC Geometric Design Guide (GDG) for Canadian Roads.

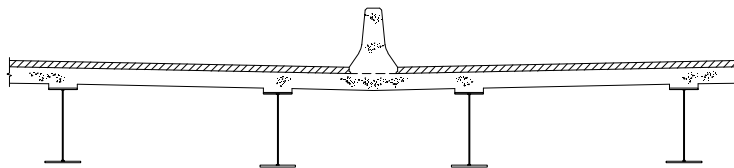
9.2.1 Bridge Deck Width

When traffic on a wide bridge is separated by a median curb or barrier, the bridge may be designed as a single wide bridge, or as twin bridges with longitudinal separation at the median. Closely spaced bridges lead to run-off and deicing salts on the cantilever overhangs between them and may promote a microclimate with high time of wetness. From the perspectives of durability and structural stiffness, for bridge deck widths up to 35 m, the preferred approach is to have a transversely continuous structure, see Figure 9.2.1.

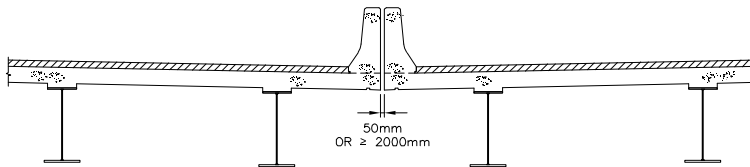
Where twin bridges are used, they shall be separated by less than 50 mm or more than 2 m apart. The 2 m minimum permits inspection with the Bridgmaster. Bridges carrying bidirectional traffic with a deck width less than 35 m shall be designed without longitudinal joints. Decks wider than 35 m and twin parallel bridges (typically with high skew or curvature) with cumulative width less than 35 m, require approval of the Structural Section.

As the bridge deck gets wider, transverse restraint due to thermal loads and shrinkage must be considered. The deck may require additional transverse reinforcement to control cracking.

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a) Transversely Continuous Bridge



b) Twined Bridge Configuration

FIGURE 9.2.1 – CLOSELY SPACED PARALLEL BRIDGES VS. CONTINUOUS SPAN CARRYING BOTH DIRECTIONS OF TRAVEL**9.2.2 Sidewalk Widths on Bridges**

Sidewalk widths on bridges shall comply with the *Accessibility for Ontarians with Disabilities Act*, S.O. 2005, c.11, and the Transportation Association of Canada. *Geometric Design Guide for Canadian Roads: 2017*, in conjunction with the MTO Design Supplement. For designs of new bridges with sidewalks or bridge rehabilitations where the sidewalks are rehabilitated, unless otherwise permitted in section 80.31 of *O. Reg. 413/12*, the minimum clear sidewalk width shall be 1500 mm. In addition, the minimum clear sidewalk width shall:

- Be maintained over the entire length of the bridge and transition smoothly to the approaches;
- Not be reduced below 1500 mm by appurtenances such as guiderail connections to the bridge;
- Apply to the clear surface of the sidewalk excluding surfaces of other elements such as curbs or tops of walls that are separated from the sidewalk by longitudinal joints.

The requirements above are illustrated in Figure 9.2.2 for greater clarity. These requirements are typically met using an 1800 mm wide sidewalk (measured at asphalt

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level and with the standard curb face slope of 50 mm horizontal to 150 mm vertical) using standard MTO details on the bridge approaches.

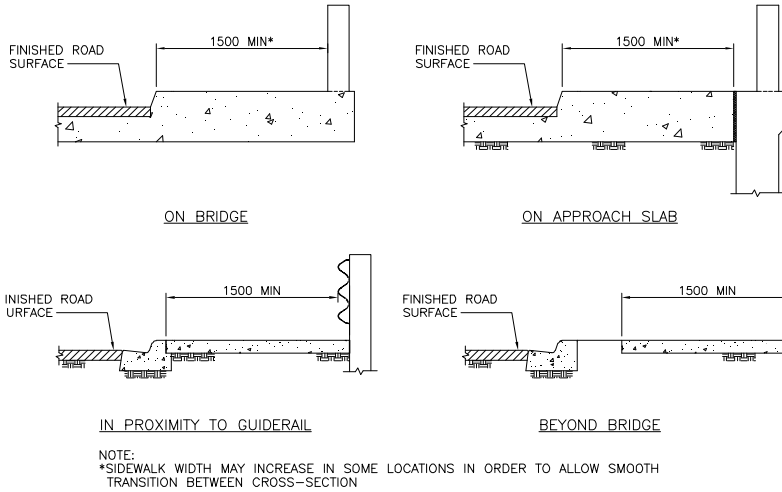


FIGURE 9.2.2 – SIDEWALK WIDTH

9.2.3 Deck Fascia Details

Fascia treatment is categorised by deck type and the presence or absence of a sidewalk as shown in Figure 9.2.3. Slab on girder decks with a sidewalk or curbs supporting open railings shall have a 500 mm high fascia. Decks without a sidewalk shall have a 350 mm high fascia with concrete barriers. Cast in place post-tensioned decks shall have fasciae from 350 to 600 mm high.

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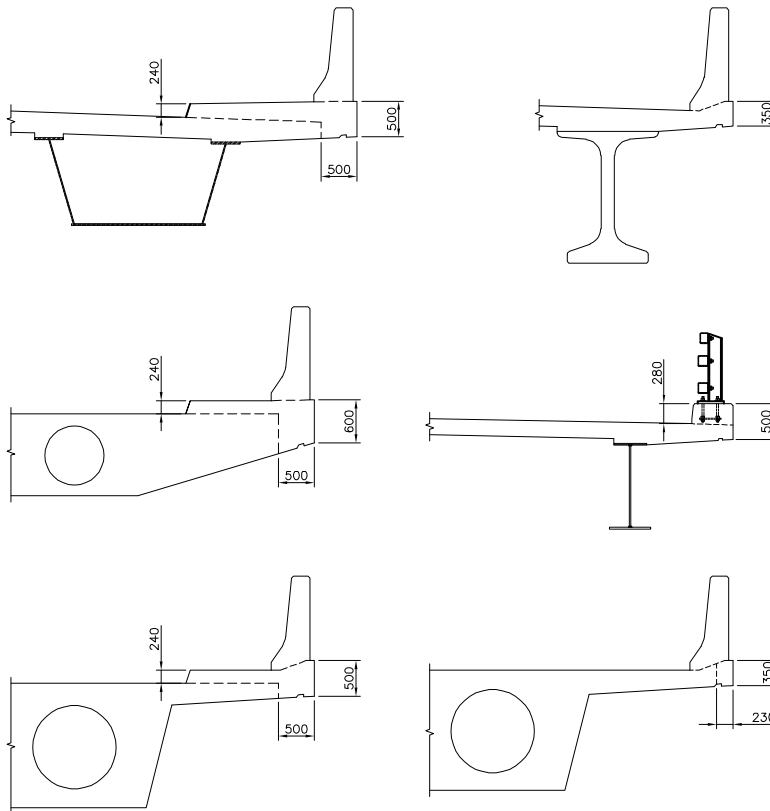


FIGURE 9.2.3 – DECK FASCIA DETAILS

9.3 Deck Slab Design

The ministry developed the empirical method of bridge design in the 1970's and it has been used for deck design since the introduction of the first Ontario Highway Bridge Design Code (OHBD) in 1979. This has been used continually since that time, with similar provisions being adopted by Section 8 of the CHBDC since the year 2000. Generally, for regular shape decks, all interior portions of decks use the empirical design method, which results in a 225 mm thick deck with a top and bottom mat of 15M reinforcement at 300 mm spacing in each layer in each direction.

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9.3.1 Concrete in Deck Slabs

To ensure adequate durability, the Specified 28-day Compressive Strength for decks shall be a minimum of 30 MPa.

9.3.2 Minimum Thickness of Deck Slabs

In order to permit placing four layers of reinforcing steel with the required cover and associated tolerances, the minimum thickness of deck slabs shall be 225 mm. Such slabs should be detailed so that the bars in the positive two layers of distribution (longitudinal) reinforcement are not vertically in line with one another. If possible, size 15M bars shall be used except for negative moment regions over piers or at deck cantilevers where larger diameter reinforcing steel bars would be required.

For deck slab made with full-depth precast deck panels, the minimum thickness of the panels shall be 200 mm.

9.3.3 Reinforcing Steel Below Barrier Walls

Cantilever portions of deck slabs tend to develop transverse cracks at barrier wall construction joint locations.

To control the cracks, 15M at 200 mm top and bottom longitudinal reinforcement shall be provided in cantilever slabs under barrier walls starting from the outside end of the slab and extending inward 500 mm from the inside face of the barrier wall (or to the outside girder flange, if it is closer than the 500 mm). See Figure 9.3.1.

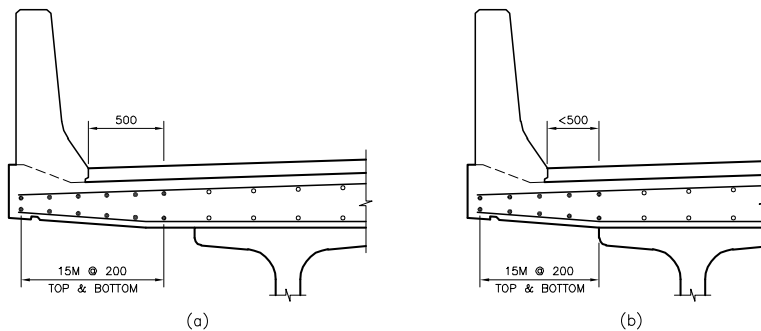


FIGURE 9.3.1 – LONGITUDINAL REINFORCEMENT BELOW BARRIER WALL

Cantilever portions of deck slabs with sidewalks do not require this additional crack-control reinforcement.

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9.3.4 Haunches

Concrete haunches of varying thickness may be used to fit the top of girders to a deck slab of constant thickness to achieve the proper screed elevation. The haunches shall be the same width as the top flange and shall not cover the sides of the top flange as shown in Figure 9.3.2(a). For deep haunches, the detail shown in Figure 9.3.2(b) is also acceptable but not preferred.

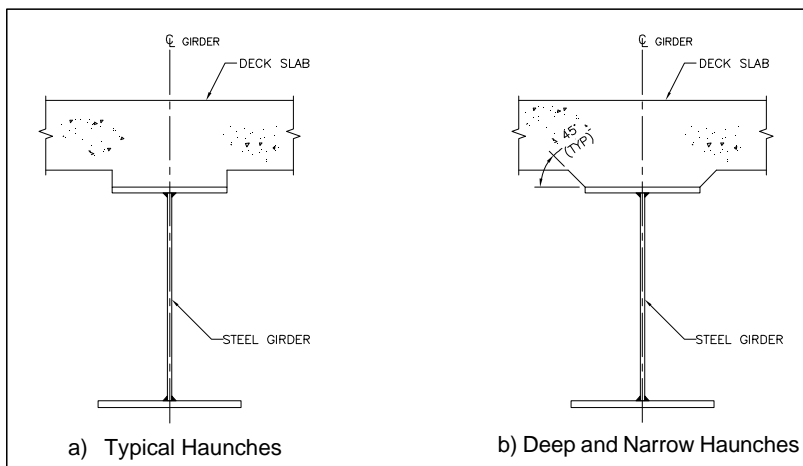



FIGURE 9.3.2 – HAUNCH DETAILING

Shear stud height in a steel girder shall project into the deck slab above the bottom transverse bars as required by the code or stirrup. Stirrup projection in a concrete girder shall extend to develop the strength of the bar across the interface between the top of the girder and the cast-in-place slab, unless a reduced strength of the bar is assumed in the calculation of interface shear. Nevertheless, the stirrups shall project a minimum of 25 mm above the bottom mat of bars, a minimum of 25 mm above the bottom mat of bars. Haunches shall not be reinforced unless the haunch depth above the flange exceeds 100 mm. Stirrup projections and shear stud height shall be designed as necessary to avoid additional flange reinforcement except when stirrup projections or shear stud length exceeds 300 mm, in which case haunches shall be reinforced to extend the bottom mat of reinforcing steel downwards into the haunch.

Typically, steel girders are fabricated to follow the roadway profile through built-in camber and a uniform haunch thickness is achieved along the girder length and in transverse direction, whereas concrete girders require a variable haunch to make up the difference between the highway profile and the deformed shape of the girder prior to casting the deck. Nevertheless, the actual haunch on site could vary from estimated haunch which may affect the stirrup projection in the deck. When stirrup projection turns out be less than

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~~25 mm above the bottom mat of deck reinforcement~~required by design, the haunch is required to be reinforced with transverse bars, usually in this shape  with standard hooks, to interlock with the girder stirrups or studs. Where stirrup projections or shear studs are too long and impede cover to the top of deck, consideration can be given to bending them to achieve the cover.

Additionally, it is important to ensure that the girders do not penetrate more than 25 mm into the nominal thickness of the deck slab. This may happen if an insufficient allowance has been made at mid span for the upward prestress camber of precast girders and also if the deck has a sag vertical curve. As a general rule, to achieve all of the criteria above, it is necessary to provide haunches raising the bottom of the deck above the top of precast beams or the top of steel beam flanges, with haunch heights at the supports as shown in Table 9.3.1. For steel beams spanning 80 m and above, a thicker haunch may be necessary.

TABLE 9.3.1 – TYPICAL HAUNCH HEIGHTS

Haunch Height (mm)	Concrete Girder	Steel Girder
50	Spaced box beams and NU 900, 1200	Steel girders with web depth < 1400 mm
75	NU 1400 or larger	Steel girders with web depth ≥ 1400 m

9.3.5 Stay-in-place Forms

Steel forms are considered to have many disadvantages and are therefore not to be used. They are or may become unsightly. They may cause maintenance problems and at the very least impede inspection. It does not appear that there is any economic advantage to their use. Stay-in-place Forms of any material shall not be used without approval of the Ministry.

9.3.6 Partial Depth Precast Deck Panels

As a result of research by MTO and others, precast, usually prestressed, concrete deck panels were introduced to Ontario in the 1990s as a means of accelerating bridge construction. The deck cantilevers are still formed conventionally, so the benefits of the panels diminish for narrow bridges with few girders, as well as for bridges constructed using staged construction where there are two additional temporary deck cantilevers. The use of the panels has been found to be advantageous for steel box girder bridges, or at least for the slab span within the steel box, to eliminate the difficult task of removing conventional formwork. The thickness of the deck panels shall be according to SS109-42 and SS109-44, and are topped with 135 mm of concrete. Tests have shown that stirrups connecting the topping slab to the panel are not required, although a few are added to the standard drawing to facilitate tying of the reinforcement for the topping slab. The design

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of partial depth deck panel deck is provided in the CHBDC, while details are contained in drawings SS109-42 and SS109-44.

This deck system requires continuity between the reinforcement in the deck panels. This is achieved, for the longitudinal reinforcement, by placing continuous reinforcement along the top of the panel. For bottom transverse reinforcement, strands or regular rebars must be developed over the girders. For NU girders, with their wide top flanges, lapping is easily achieved, while for steel girders the strands must be crossed over the adjacent panel.

For large spans with large negative moments in the deck, or bridges with large overhangs, adequate space between reinforcement may be difficult to achieve. For these aforementioned situations, a deck thicker than 225 mm may be required. For bridges on skew, the panels span perpendicular to the girders, and the triangular portions of deck near supports are formed conventionally with cast-in-place concrete.

9.3.7 Additional Reinforcement in Skewed Decks

The following detail (Figure 9.3.3), along with Table 9.3.2, shows the additional reinforcement required, unless analysis dictates otherwise, for the cantilevered portion of thin slab bridge decks with skews exceeding 20° but less than 45°, and with the cantilever span not exceeding 1.6 m. Skewed decks with cantilever spans less than 0.6 m do not require this additional reinforcement.

TABLE 9.3.2 – ADDITIONAL DECK REINFORCEMENT FOR DIFFERENT SKEWS

SKEW ψ	CANTILEVER	ADDITIONAL REINFORCEMENT
20° - 35°	0.6 m - 1.2 m	6 - 20M
20° - 35°	1.2 m - 1.6 m	6 - 25M
35° - 45°	0.6 m - 1.2 m	8 - 20M
35° - 45°	1.2 m - 1.6 m	8 - 25M

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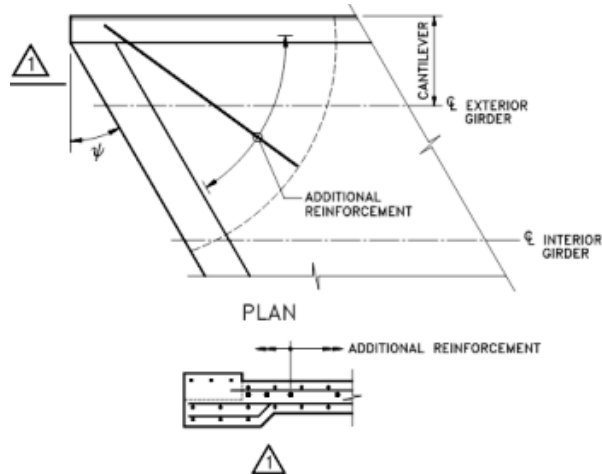


FIGURE 9.3.3 – ADDITIONAL REINFORCEMENT IN SKEWED DECKS

The additional reinforcement shall be placed directly under the top layer of deck reinforcement.

No additional radial reinforcement is required at the obtuse-angle corners of the bridge deck.

9.4 Construction

9.4.1 Deck Construction Joints

Construction joints across which force effects are assumed to be transmitted or which must resist leakage, require special treatment. Introduction of construction joints that are not shown on contract drawings is not allowed without approval.

It should be noted that waterstops in deck construction joints shall not be used.

9.4.2 Sequence of Deck Placements for Slab-on-Girder Bridges

No deck placing sequence should be specified when the deck can be cast in one continuous operation from one abutment to the other unless there are specific reasons, such as when the deck pour could exceed 12 hours in duration or where the structure type or structural articulation dictates otherwise (i.e., arch bridges, continuous multi-span girders with short end spans that could lead to lift off the abutment bearings when concrete is placed in the adjacent span, etc.).

When necessary, the deck placing sequence shall be shown on the drawings. Each deck placement should be as large as practical, taking into account structural considerations

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and the availability and volume of concrete. The following notes concerning strength of the previous placement before allowing the next placement should be given on the deck drawing:

CONCRETE STRENGTH OF PREVIOUS DECK PLACEMENT SHALL BE AT LEAST 20 MPa BEFORE PROCEEDING WITH THE NEXT PLACEMENT.

CONCRETE IN DECK SLABS AND DIAPHRAGMS SHALL BE RETARDED USING TYPE "B" OR "D" ADMIXTURE TO ENSURE THAT THE CONCRETE REMAINS PLASTIC FOR THE DURATION OF EACH PLACEMENT.

The deck placing sequence should be shown in numerical order.

NOTE: Simultaneous concrete placements should not be specified unless absolutely necessary, in which case the intent should be clarified on the deck slab drawing.

9.4.3 Screed Elevations on Bridge Decks

Screed elevations are the elevation to which the deck needs to be placed to achieve the final vertical profile after all dead load deflections occur. Screed elevations are achieved by the contractor by adjusting the height of the haunches as required.

9.4.3.1 Slab on Girder Decks

Screed elevations shall be given at the centreline of all exterior girders, the break points in the deck, and on the deck at the faces of curbs and barrier walls. Screed elevations shall be given at intervals not exceeding 3 m. ~~Screed elevations should include an allowance for long-term dead load deflection.~~

9.4.3.2 Post Tensioned Concrete Decks

Screed elevations shall be given at break points in the deck, and on the deck at the face of curbs or barrier walls. The screed elevations shall be given at intervals not exceeding 3 m. No allowance is required normally for dead load deflections provided that prestress and dead load are more or less balanced.

9.4.4 Construction Loading from Deck Cantilever Overhangs

Girders shall be designed to account for the load effects of the deck cantilever overhang during the deck pour.

OPSS 919 requires the screed rail to be placed above the centre of the web. In cases where this is not feasible based on the need to screed the deck to the full width (i.e., staged construction of a two-lane bridge), the girder design shall account for the load of the screed machine acting on the screed rail located 0.1 m outside of the deck fascia, and an NSSP shall be included to permit this condition.

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9.4.5 As-Constructed Elevations

Steel pins used to record as-constructed elevations of bridges shall be installed when called for in the Structural Design Report. The decision to install steel pins shall be made by the Head, Structural Section in consultation with the Design Section Head of the Structures Office. Structural Standard drawing SS116-40 shall be included in the contract when steel pins are to be used.

Steel pins shall only be required where long term settlement can be expected due to specific site conditions and on superstructures that are sensitive to long term creep effect, such as bridges of segmental construction, or cast in place post tensioned bridges of staged construction.

9.5 Durability**9.5.1 Bridge Deck Waterproofing**

Bridge decks detailed with an asphalt wearing surface must also be shown as being waterproofed. The drawings should state only "Asphalt and Waterproofing System, 90 mm total." (See Section 2.6.6).

9.5.2 Future Wearing Surface

All structures detailed with an exposed concrete surface shall be designed for 90 mm future asphalt wearing surface and waterproofing. To allow for wear of exposed surface, the cover to the reinforcing steel from the top surface shall be increased by 10 mm.

The heights of curbs and barrier walls should not be increased to accommodate the future wearing surface.

9.5.3 Waterproofing of Deck Cantilever of Steel Box-Girder Bridges

For steel box-girder bridges, the face of curb/sidewalk, if any, should preferably start beyond the exterior flange of the exterior girder. If this is not practical, hot poured rubberized asphalt waterproofing system shall be used to prevent salt-laden deck run-off from invading the joint between the slab and the sidewalk, filtering through the deck and into the unprotected steel box-girder, causing serious internal corrosion. This damaging potential is intensified on wide-deck bridges, and especially in the vicinity of piers, where the concrete decks may have numerous flexural cracks in the negative moment region. Hot poured rubberized waterproofing system shall be according to OPSS 914 except that protection board shall not be used. The concrete sidewalk shall be placed directly on the waterproofing membrane, a minimum of 1 day after the waterproofing has been placed.

This waterproofing should start at 150 mm in front of the face of the sidewalk, continue along the top of the deck/sidewalk construction joint ~~and be turned downwards along to~~ the vertical joint face under the barrier wall. The waterproofing system should extend the full length of the bridge.

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Whenever this requirement is used, additional dowels should be added between the deck/sidewalk interface to ensure continuity and the integrity of the deck cantilever to resist the traffic barrier loads given in Clause 3.8.8.1 of the CHBDC. The total area of dowels between the top of the deck and the sidewalk shall not be less than 2000 mm² per m along the length of the bridge.

9.5.4 Structure Deck Drainage

Requirements for deck drainage inlets must be established as part of the overall drainage design of the crossing. The size and number of drains shall meet the requirements of MTO's Highway Drainage Design Standards. Bridge deck drains shall be provided only where necessary. Structures having one or two lanes draining to one side, built with normal crossfall on a vertical crest curve not more than 120 m long, normally do not require deck drainage inlets.

For bridges on grades or sag curves, the roadway surface runoff shall be intercepted by catch basins or other means located on the approaches to prevent flow into the expansion joints or onto the bridge deck.

When bridge deck drainage inlets are required, they shall be used with an airdrop discharge. Water may not be discharged onto railway property, pavements, sidewalks, unprotected embankment slopes or waterways if environmental concerns prevail. When water is discharged onto other surfaces, where stability or appearance is a consideration, provision shall be made to prevent scour. The position and length of the discharge pipes shall be such that water falling at an angle of 45° to the vertical does not touch any part of the structure. Discharge pipes should project 400 mm below the bottom flange of adjacent girders to prevent splash. Consideration, however, must also be given to minimum vertical clearance requirements and aesthetics. Pipes need not be attached to adjacent girders if overall length is less than 2.5 m.

OPSD 3340.100 deck drainage inlets do not collect a significant quantity of water and should be used only to prevent local ponding. This is sometimes necessary when flat grades are unavoidable, or structures are subject to substantial permanent deflections that cannot be accurately predicted.

Catch basins are normally necessary just beyond the structure limits to intercept runoff from bridge decks. A continuous length of curb or gutter is provided to connect the bridge curb or barrier to the catch basin to prevent wash outs around the ends of wing walls. Detailing of the wing wall or approach slab should be such as to permit straight vertical junctions with approach curbs. The final grading drawings should be reviewed in conjunction with the structure drawings to ensure that this or an equally acceptable arrangement has been adopted.

Deck drainpipes shall be located outside of the tub or box girders and shall not cross through tub or box girders. Drainage analysis shall be performed at preliminary design stage to determine if deck drains can be avoided. If deck drains are required, the designer shall select a structural type and configure the cross-section accordingly.

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9.5.5 Drip Grooves

Continuous drip grooves are required along the soffit on both sides of all concrete decks. They should be provided on each side of the joint between abutting twin bridges, even if the joint is sealed.

The dimensions are shown in Figure 9.5.1.

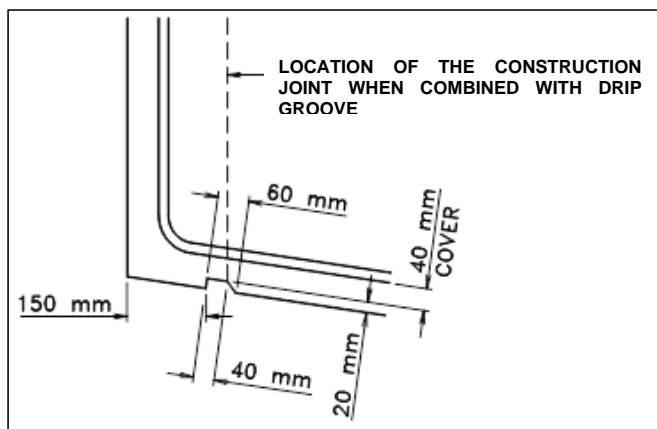


FIGURE 9.5.1 – DRIP GROOVES

For practical and aesthetic reasons, it may be desirable to combine the drip with construction joints that result from transverse prestress anchorage recesses, or decks with curbs or sidewalks. For this case the drip should not be located greater than 250 mm from the fascia (see OPSD 3390.100).

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10 BARRIERS AND RAILING SYSTEMS**10.1 General**

- a) Bridge railing shall be selected in accordance with the requirements of Section 12 of the CHBDC; that is, conforming to TL-2, TL-4, or TL-5, meeting the crash test requirements specified in the NCHRP Report 350, or the AASHTO Manual for Assessing Safety Hardware (MASH).

Only the railing appropriate for the test level of the bridge site should be used.

All bridge barriers adopted for use by the Ministry originate from the crash-testing programs carried out in the United States. The current standards used by the Ministry are based on crash tests carried out in accordance with the crash test procedures of:

- Michie, J. D. (1981). NCHRP Report 230: Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances. Washington: Transportation Research Board.; and/or,
- Ross, H. E., Sicking, J. L., Zimmer, R. A., & Michie, J. D. (1993). NCHRP Report 350: Recommended Procedures for the Safety Performance Evaluation of Highway Features. Washington: Transportation Research Board.; and/or,
- AASHTO. (2009). Manual for Assessing Safety Hardware (1st ed.). American Association of State and Highway Transportation Officials., or deemed equivalent.

MTO is in the process of moving towards the use of MASH 2016 barriers. Further information about crash tested barrier systems can be found in AASHTO publications, NCHRP reports, Transportation Research Record, and FHWA memorandums. The reference subsection of the commentary to Section 12 of the CHBDC gives a comprehensive listing of pertinent documents, and the following FHWA web site also gives actual details of approved railing systems and memorandums:

https://safety.fhwa.dot.gov/roadway_dept/countermeasures/reduce_crash_severity/listing.cfm?code=long

Some of the accepted bridge rail designs may also be found in the 1995 AASHTO AGC-ARTBA Joint Committee publication "A Guide to Standardized Highway Barrier Hardware".

- b) Slip forming of concrete barriers on structures is not permitted.
- c) When curbs are required, for sidewalks, raised medians etc., they shall be 150 mm high.

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10.2 Approved Traffic Barriers

In this section the various test levels and corresponding bridge railing types that have been successfully crash tested and used by the Ministry are described. Note that the railing systems identified in *italics* refer to the name of the railing system that has been crash tested and approved by the FHWA.

For bridges on low volume roads with an AADT, in both directions, of 400 or less, a Test Level lower than TL-2 is acceptable. MTO's criteria for selecting a lower test level for local low volume roads is provided in Division 1.

10.2.1 Test Level 2 (TL-2)

As none of the available crash tested TL-2 systems meet the minimum pedestrian height and other geometric requirements, none are suitable for sites where pedestrian protection is warranted. The barriers were crash tested to NCHRP 230, or AASHTO Guide Specifications for Bridge Rails, and were found to be NCHRP 350 compliant for TL-2 as per a 1997 FHWA memorandum.

The following barrier types are standards used by the Ministry for TL-2:

a) Box Beam Guide Rail – Side Mount (Structural Standard Drawing SS110-2)

The system in Figure 10.2.1 is based on the crash tested *California Type 115 Bridge Railing*. Its standard structural "W" and "HSS" shapes provide a relatively inexpensive, easy to fabricate and erect railing suitable for vehicular only warrants on low speed and/or low volume highways.

Its open configuration offers a low profile, see-through rail for maximum visibility, which is desirable in scenic and rural sites.

b) Thrie Beam Guide Rail – Side Mount (Structural Standard Drawing SS110-5)

The system in Figure 10.2.2 is based on the crash tested *Oregon Side Mounted Thrie-Beam Bridge Railing*. The Oregon side mounted system consists of standard "Thrie Beam" elements mounted on standard structural steel posts. It is ideally suited for cast in place and precast slab superstructures with at least 400 minimum slab depth. It should be noted that the "W" shape steel beam guide rail used in Ontario is not the same as the "thrie" beam section. Currently there are no crash tested bridge rail systems available incorporating the "W" shape steel beam.

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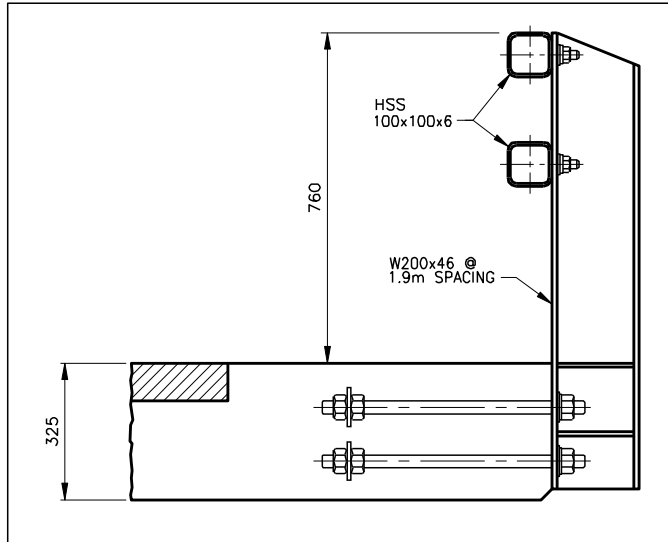


FIGURE 10.2.1 – BOX BEAM GUIDE RAIL – SIDE MOUNT (TL-2)

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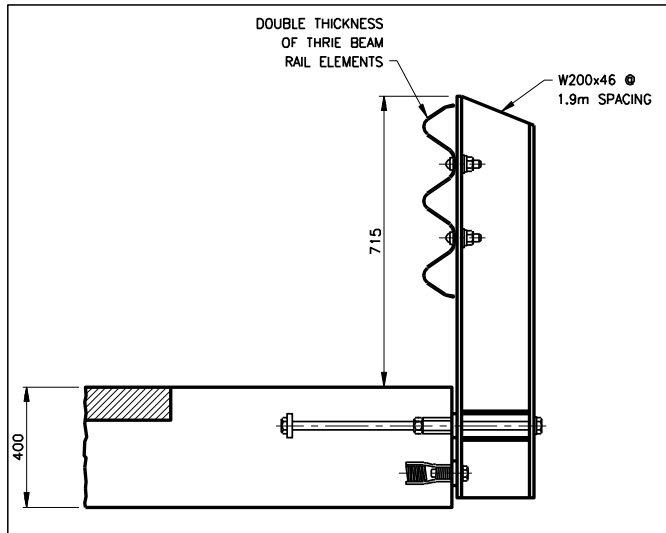


FIGURE 10.2.2 – THRIE BEAM GUIDE RAIL – SIDE MOUNT (TL-2)

c) Thrie Beam Guide Rail – Timber Deck (Structural Standard Drawing SS110-7)

The system in Figure 10.2.3, for longitudinal timber bridge decks, is based on the *Steel System-Thrie Beam on Steel Posts* bridge railing crash tested for the US Department of Agriculture Forest Service. This in turn is an adaptation of the *California Thrie Beam Bridge Rail* modified for timber bridge decks. It consists of a thrie beam connected to standard structural steel “W” shape posts and spacer blocks side mounted on the bridge deck. The steel system is connected to the deck with high strength bars. Material costs are more economical than longitudinal glulam and timber post alternatives.

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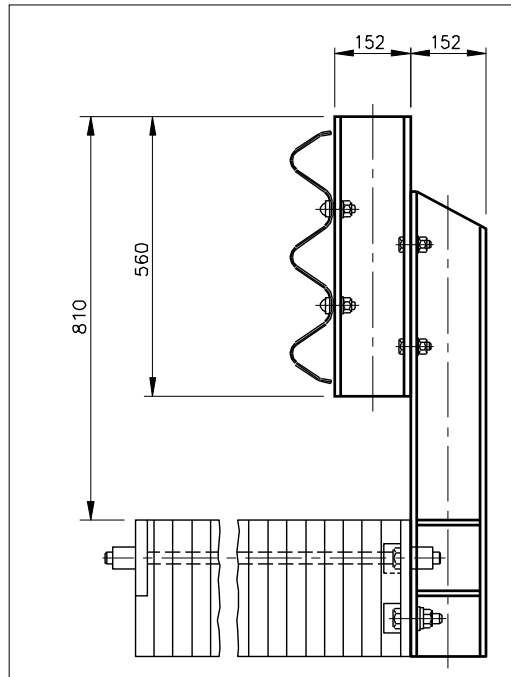


FIGURE 10.2.3 – THREE BEAM GUIDE RAIL – TIMBER DECK (TL-2)

10.2.2 Test Level 4 (TL-4)

The following barrier types are standards used by the Ministry for TL-4, each of which is based on an NCHRP 350 crash tested barrier:

a) Barrier Wall with Railing (Structural Standard Drawings SS 110-54/58/91)

The system in Figure 10.2.4 is based on the crash tested 32-inch (813 mm) *F-Shape Bridge Railing*. It is constructed of reinforced concrete and this type is the most common rigid traffic barrier in use today on both roadways and bridges. F-shape does not refer to the shape of the barrier but merely to the crash test designation. Its popularity is based on its generally effective performance as a barrier, particularly with its re-directional capabilities, and its low maintenance costs. The concrete barrier requires virtually no maintenance for most hits. The lower sloped face redirects vehicles without damage under low-impact conditions. During moderate to severe impacts, some energy is dissipated when the vehicle is lifted off the pavement. The loss of tire contact with the pavement also aids redirection. In crash tests, the F-shape has proven to be more successful than the New Jersey shape in preventing rollover for small vehicles. The barrier may redirect or contain heavy vehicles, but it was not

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designed for this purpose. Therefore, it is most suitable for highways carrying traffic with low heavy truck volumes.

A handrail is mounted on top to provide a combination railing for maintenance workers and occasional pedestrians on bridges without sidewalks. The handrail also provides some aesthetic benefit. The barrier wall is not to be used on bridges with sidewalks, as it provides no extra advantage when compared to a concrete parapet.

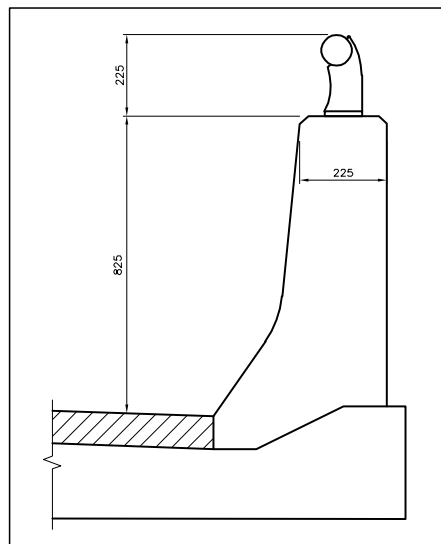


FIGURE 10.2.4 – BARRIER WALL WITH RAILING (TL-4)

**b) Parapet Wall with Railing
(Structural Standard Drawings SS 110-56/57/59/90/97/98/104/105)**

The system in Figure 10.2.5 is based on the crash tested 32-in (813-mm) vertical concrete parapet. It offers a simple to build reinforced concrete alternative to the F-shape railing. Vertical concrete walls do not have the energy management feature of the F-shape, but crash test have demonstrated that they perform acceptably as traffic barriers. Because vehicles are not lifted or tilted, all four wheels tend to stay on the ground and all the energy absorption upon impact goes into the crushing of the vehicle. Therefore, damage to a vehicle on impact with the parapet is likely to be more severe compared to the F-shape and redirection not as smooth. Potential rollover is minimised, however, with a vertical face.

A handrail is mounted on the top to provide a combination railing for maintenance workers and for bridges with sidewalks.

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Where width is a premium on bridge decks such as on rehabilitations the narrower width of the parapet provides an advantage over the F-shape.

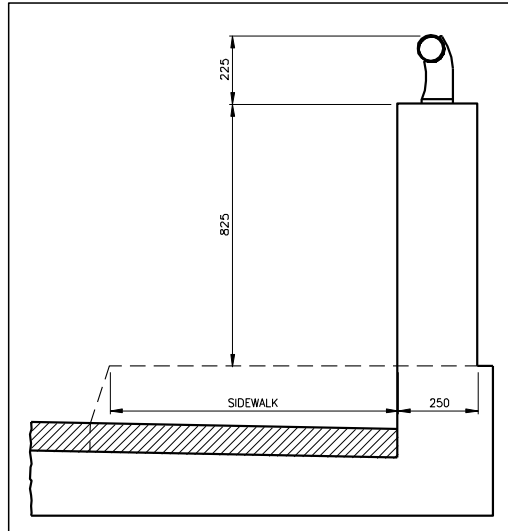


FIGURE 10.2.5 – PARAPET WALL WITH RAILING (TL-4)

c) **Box Beam Guide Rail on Curb (Structural Standard Drawings SS 110-37/39)**

The system in Figure 10.2.6 is based on the crash tested *Massachusetts Type S3 Curb Mounted Bridge Rail*. This system is constructed of standard structural steel W section posts and three HSS rails and offer a good performing lightweight and open (“see through”) rail alternative to the concrete barriers outlined above. Even though these rails are acceptable in most applications they are not recommended for use on high speed and limited access highways. For these cases the F-shape is recommended, as it is better at redirecting errant vehicles and requires less maintenance. The systems are usually used with concrete end posts for connection to the approach railing system. An alternative end connection between the box beams and approach steel thrie beam guide rail has been developed to accommodate user needs.

In general, these systems are suitable for vehicular applications, however, the Massachusetts system may be used for vehicular/pedestrian applications if pickets are included.

Because it offers one of the most open railings at this test level, it is most suitable for scenic and rural sites.

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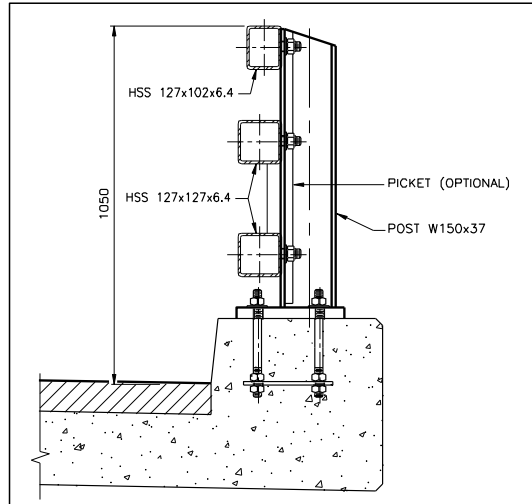


FIGURE 10.2.6 – THREE TUBE RAILING ON CURB (TL-4)

(d) Box beam railing on sidewalk (Structural Standard drawings SS110-46/49)

The system in Figure 10.2.7 is based on the crash tested *New England Transportation Consortium 4-Bar Sidewalk Mounted Bridge Rail*. It is constructed of standard structural steel W section posts and four HSS rails and offers a good performing lightweight and open (“see through”) rail alternative to the concrete parapet wall on sidewalk with railing outlined previously.

This system is suitable for vehicular/pedestrian applications. Because it offers one of the most open railings at this test level, it is most suitable for scenic and rural sites.

Although this railing satisfies all the current requirements of the CHBDC with regard to clear spacing between the rails, the ladder like orientation of the horizontal rails make it more inviting to climb for little children. Consequently, a different system should be used when the structure is located near public schools and where the anticipated pedestrian traffic includes little children.

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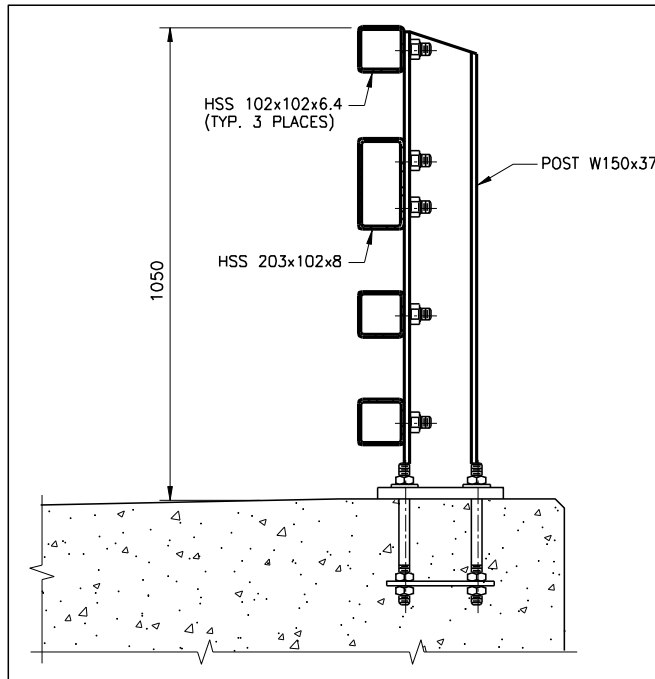


FIGURE 10.2.7 – FOUR TUBE RAILING ON SIDEWALK (TL-4)

10.2.3 Test Level 5 (TL-5)

The Ministry currently has two barrier/railing systems that satisfy the TL-5 requirements. An F-shape barrier without railing and a system consisting of box beam railing on a concrete parapet wall.

These railing systems provide the maximum level of protection, in the event of a collision, for which the Ministry has a standard. Barriers crash tested to NHCRP 350 or MASH or for special cases such as for heavy trucks are available, and details of these may be found on the FHWA web site.

a) Barrier wall without railing (Structural Standard drawings SS110-61/92/94/109)

The system in Figure 10.2.8 is based on the crash tested 42-inch (1.07 m) *F-Shape Bridge Railing*. This railing system is very similar to the F-shape TL-4 concrete barrier in that the front surfaces and its construction are identical except for its height. The crash test characteristics are similar to the F-shape TL-4 barrier except that the extra height reduces potential rollover for impacting vehicles.

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This railing system is not for use on sidewalks even though it meets the code requirements for pedestrians.

The system in Structural Standard Drawing SS110-92 is based on the crash test done by Ryerson University in 2011 at TTI on F-shape TL-5 barrier incorporating GFRP bars with anchor head. The crash test was performed in accordance with MASH TL-5.

The system in Structural Standard Drawing SS110-94 is based on the crash test done by Ryerson University in 2016 at TTI on F-shape TL-5 barrier incorporating GFRP bars with hook bar. The crash test was performed in accordance with MASH TL-5.

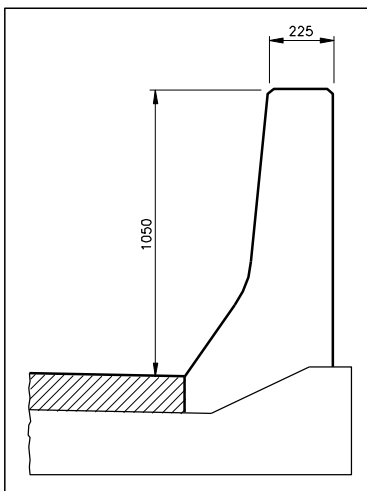


FIGURE 10.2.8 – BARRIER WALL WITHOUT RAILING (TL-5)

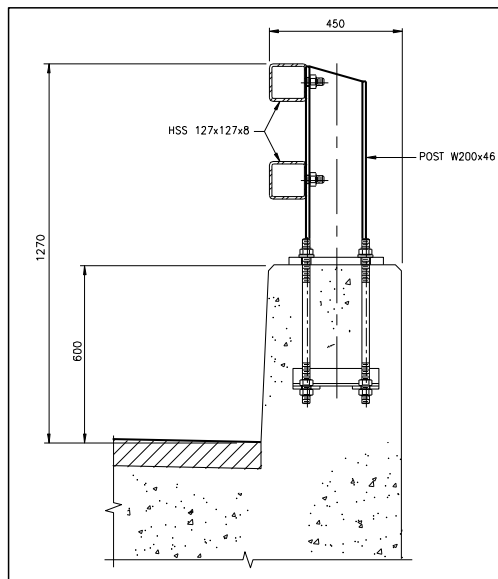


FIGURE 10.2.9 – PARAPET WALL WITH TWO TUBE RAILING (TL-5)

b) Parapet wall with two tube railing (Structural Standard drawing SS110-96)

The system in Figure 10.2.9 is based on the *PA Bridge Barrier* from Pennsylvania DOT. It was accepted by FHWA as a TL-5 barrier designation. It consists of a concrete parapet with metal railings mounted on top. The parapet facilitates transfer of post loads into the deck and the metal railings portion permit visibility through the railing. It offers a semi-open system alternative to the solid concrete barrier mentioned above.

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10.3 Combination Traffic/Bicycle Rail

This railing provides protection to both bicycles and vehicles when bicycles are travelling along the roadway. It shall be located on the outside face of bridges. The system shown in Figure 10.3.1 (Structural Standard Drawings SS 110-82/83/84/85) is developed based on the flush mounted combination traffic/bicycle rail system from Oregon DOT that was accepted by FHWA as a TL-4 barrier. It consists of a concrete parapet with two metal railings mounted on top.

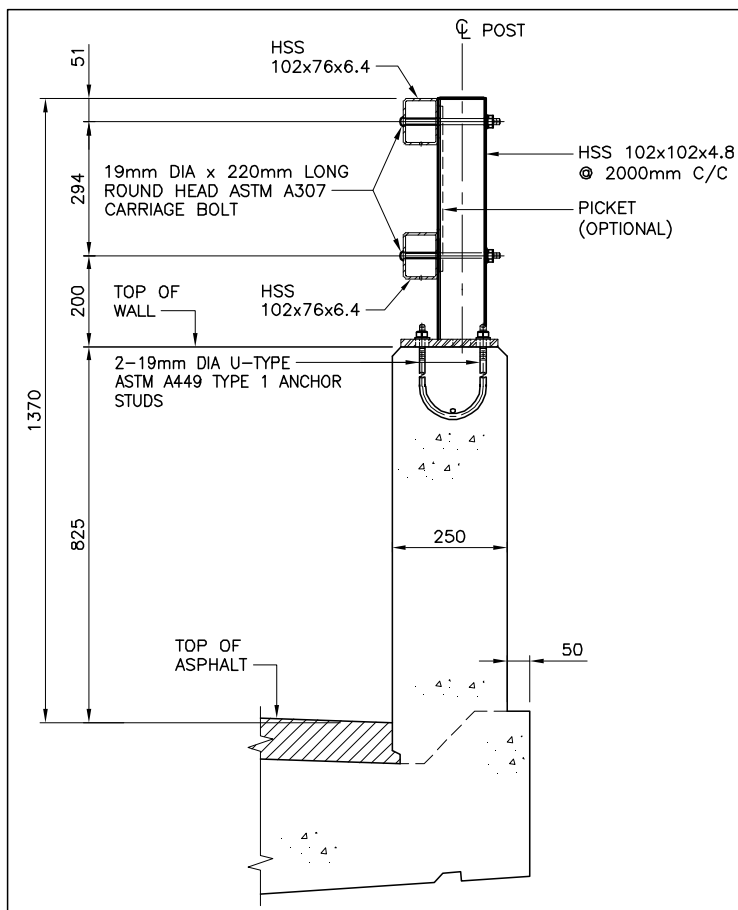


FIGURE 10.3.1 – PARAPET WALL WITH TWO TUBE COMBINATION TRAFFIC/BICYCLE RAIL (TL-4)

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Another system shown in Figure 10.3.2 (Structural Standard drawings SS110-34/36) is a TL-4 open railing which is developed based on a Maine DOT system. It consists of a steel post with 4 HSS metal railings mounted on a 225 mm curb. The overall height of both systems is 1370 mm. It meets the CHBDC requirements for bicycle barrier.

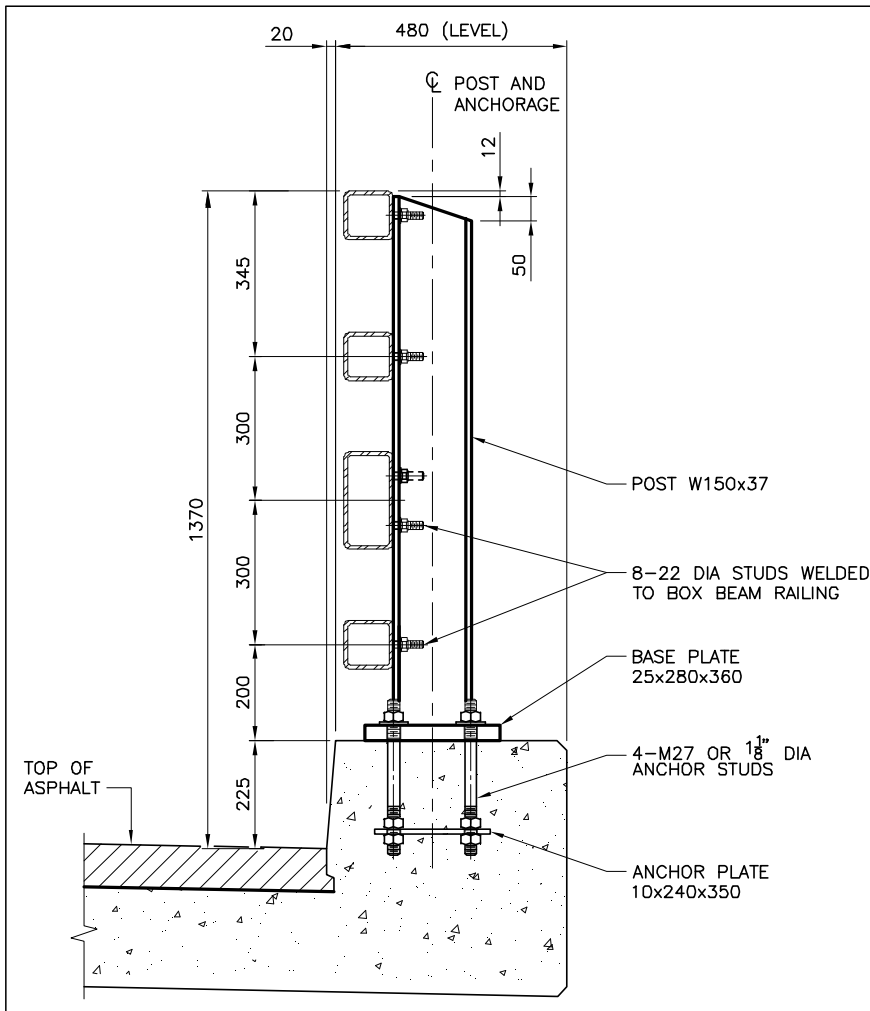


FIGURE 10.3.2 – FOUR TUBE COMBINATION TRAFFIC/BICYCLE RAIL (TL-4)

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10.4 Separation Barriers for Bridges with Multi use Pathways (MUPs)

A sidewalk is a raised platform for the passage of pedestrians and typically less than 2 m wide, while a Multi-Use Path (MUP) is intended for the joint use by pedestrians and bicyclists and are typically 3 m or in width or greater. When sidewalks or MUPs are used with a curb, the barrier on the outside edge non-traffic side of the sidewalk/MUP shall be a crash tested combination traffic/pedestrian or traffic/bicycle barriers respectively. These barriers should have a handrail at a minimum height of 1050 mm above the top of sidewalk and 1370 mm above the top of MUP. This handrail provides a safety “grip” for pedestrians in case they slip, makes it difficult for people to walk on top and adds some aesthetic value. The following is also noted:

- *The Barrier Wall Without Railing* TL-5 railing system is not for use on sidewalks even though it meets the code requirements for pedestrians.
- *The Four Tube Railing on Sidewalk* (Structural Standard Drawings SS110-46/49) and *Parapet Wall with Railing on Sidewalk* (Structural Standard Drawings SS110-57/97/98/105) are the MTO railing systems, currently available, for use on sidewalks.
- Currently no Structural Standard drawing exists for combination barriers (without separation barrier) on MUPs meeting bicycle height requirements.

For high-speed applications, as specified in CHBDC, a separation barrier is required between traffic and pedestrians/bicyclists. In lower speed application with very high pedestrian or multi-use usage, it may also be prudent to provide a separation barrier. This results in the following configuration:

roadway – separation barrier – sidewalk/MUP – pedestrian/bicycle barrier

The separation barrier shall satisfy the crash test and geometric requirements for the traffic, with the barrier height from the pedestrian or bicyclist side satisfying the minimum 600mm requirement of CHBDC. The barrier at the outside of the bridge is protected by the traffic separation barrier and must only be meeting loading and geometric design requirements for pedestrian or bicycle barrier for sidewalks and MUPs respectively. Structural Standard Drawing SS110-110 and SS110-111 show the design and details for the TL-4 and TL-5 separation barrier respectively (see Figure 10.4.1). Both separation barriers satisfy height requirements of CHBDC on the traffic side for the protection of the MUP user. No modification should be made to the separation barrier to increase the height or change the geometry. A TL-5 barrier is taller than TL-4, and it could be used where enhanced pedestrian/MUP user comfort is desired.

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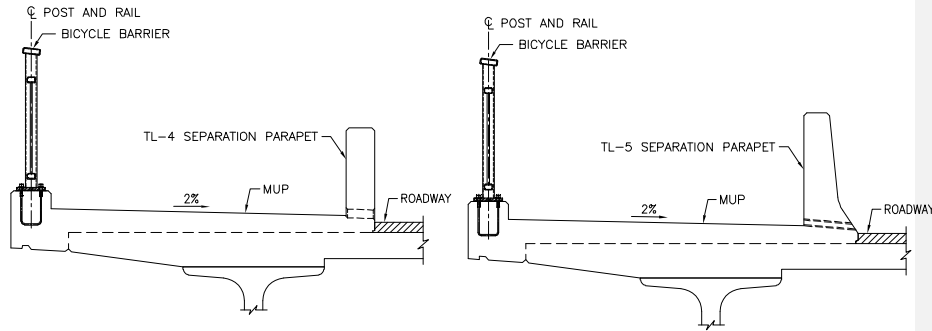


FIGURE 10.4.1 – SEPARATION BARRIER FOR TL-4 AND TL-5

Drawing SS110-22 shows the design of a bicycle barrier on the outside edge of the bridge (see Figure 10.4.2). Due to the high likelihood of the presence of bicycles on an MUP or sidewalk separated from traffic by a separation barrier, only a bicycle height barrier should be used in these circumstances.

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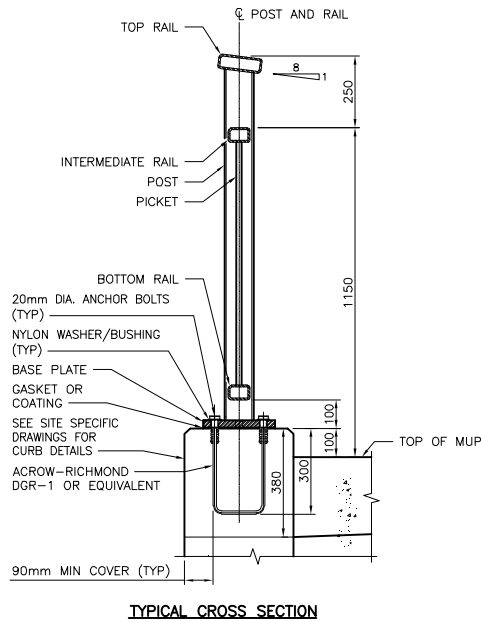


FIGURE 10.4.2 – BICYCLE BARRIER

10.5 Barrier Walls Beyond the Bridge Structure

10.5.1 Barrier Walls in Fill Piles

Because of the high cost of providing piles under supporting barrier walls on piles in back on-fill, especially to withstand the higher test level loads, the designer should investigate all possible alternatives, such as carrying the normal two-sided concrete highway barrier up to the end of the structure or retaining wall, with the standard transition, where required. If approach barriers are required to be supported on piles, the length of piles for barrier walls on fill shall be determined as follows:

- a) Piles located between the structure and first pavement expansion joint from the structure: Piles 1 m into existing ground, or minimum overall length 3 m and maximum overall length 6 m;
- b) Other piles under barrier walls: Piles 0.5 m into existing ground or minimum overall length 3 m, maximum overall length 5 m.

~~Because of the high cost of providing piles under barrier walls on fill, especially to withstand the higher test level loads, the designer should investigate all possible~~

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~~alternatives, such as carrying the normal two-sided concrete highway barrier up to the end of the structure or retaining wall, with the standard transition, where required.~~

10.5.2 Barrier Walls on MSE Structure

The CHBDC does not provide design details or guidelines for traffic barriers mounted on mechanically stabilized earth (MSE) System (RSS). Furthermore, the CHBDC has increased the traffic rail impact load requirements for all test levels compared to the earlier OHBDC. Designing for these new loadings, and in particular at the TL-5 level, without knowledge of their distribution through the barrier and transfer to a structural slab and wall system can result in costly or over conservative designs using conventional design methods. This problem has been recognised in the USA, and the Transportation Research Board has carried out a NCHRP project "Design of Roadside Barrier Systems Placed on MSE Retaining Walls" in 2010 (NCHRP 663). Design guidelines for the barrier system were developed based on finite element simulation, bogie vehicle tests and full-scale Test Level TL-3 crash test.

In Ontario the design of the current barriers on MSE walls has been based on the impact loads given in OHBDC. To avoid severe wall damage during vehicle impact, top mounted traffic barriers are connected integrally to continuous footings (normally called anchor slab or moment slab) that are independent of the MSE retaining walls (see Figure 10.5.1). So far, no unsafe performance or damage, in over 30 years of use on Ministry Highways, has been reported. Given that the TL-4 loadings in CHBDC are just marginally higher than the loading given in OHBDC, it was decided to use the CHBDC TL-4 loading, on an interim basis, for the design of both TL-4 and TL-5 traffic barriers on MSE retaining walls. Furthermore, in the USA, AASHTO generally accepts TL-4 loading for the majority of its applications on highways and freeways with a normal mixture of trucks and passenger vehicles and this also seems appropriate in Ontario for the majority of its highways. Therefore, the current Structural Standard Drawings (SS110-64/65/68/69/75) may continue to be used for barriers on MSE retaining walls.

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10.6 Other Barrier Types**10.6.1 Noise Barriers****10.6.1.1 Highway Noise Barriers**

The current practice for erecting noise barriers in the Ministry's Right-of-Way is as follows:

1. When noise barriers are required on roadways and are located beyond the clear recovery zone, then the noise barriers are erected simply between steel posts.
2. When noise barriers are required on roadways and are located within the clear recovery zone, then the noise barriers are erected either on top of or behind traffic barrier walls.

Noise Barriers adjacent to roadways are approved by Highway Design Office of MTO and follow DSM 5.50 and roadside safety requirements.

10.6.1.2 Noise Barrier on Bridges

On bridges and retaining walls, the noise barriers are usually located within the clear recovery zone and are generally attached either on top of or to the back face of barrier walls. According to the CHBDC, when the noise barrier is located within the zone of intrusion of the traffic barrier, the noise barrier in combination with the traffic barrier must meet the crash test requirements of NCHRP 350 or MASH. Alternatively, in lieu of crash testing, a noise barrier system meeting the requirements of CHBDC Clauses 12.4.7.1 to 12.4.7.4 may be used. Only noise barriers specified in DSM 5.50.40 shall be used on MTO bridges. Due to the limited number of crash-tested noise barriers available, MTO allows the use of a MASH tested TL-4 noise barrier, on TL-5 barrier wall, to be used for TL-5 conditions. For all other test levels, the noise barrier shall satisfy the crash-test requirements for the specific test level.

Recommendations

1. Before a noise barrier is considered for use on a bridge, a cost benefit analysis should be carried out. This study should consider, but not be limited to, the following:
 - That the noise reduction is significant enough to warrant the use of a noise barrier;
 - That there are a reasonable number of residents that are expected to benefit from the reduction in noise level as determined by MTO Environmental Guide for Noise Oct 2006;
 - Stopping the noise wall at the bridge and turning it obliquely (flanking) away from the highway towards the residential area. In effect avoiding noise barrier walls being erected on the bridge itself;
 - The effect of increased loading to the structure (from vertical, wind, torsion loads etc.). These effects may create the need for additional girders and cross bracing, and a refined structural analysis for verification;
 - Access for inspection of the bridge and the noise barrier components;

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- Cost of an approved traffic/noise barrier wall system and additional costs to the structure;
 - Possible snow accumulation; and,
 - Aesthetics.
2. Where it has been assessed that it will be beneficial to install a noise barrier wall on bridges and retaining walls or portions thereof, the design shall satisfy CHBDC, including requirements to contain or otherwise prevent debris that could endanger public safety, from falling.

10.6.1.3 Noise Barrier Design

Noise barriers shall be designed in accordance with the CHBDC as light slender structures not unusually sensitive to wind action, and CSA-Z107.9, Standard for Certification of Noise Barriers. Wind loads and ice accretion loads on all the noise wall elements shall be considered in the design as per the CHBDC. Reference wind pressures for a 25-year return period shall be used.

In the calculation of section properties and strength for cold formed steel members, for which the provisions of the CHBDC are not applicable, the requirements of CSA-S136 for ultimate limit state design shall apply.

In evaluating or designing a structure on which a noise barrier is to be mounted, the above criteria shall apply, except that the local reference wind pressure shall always be used and a gust factor of 2.0 is sufficient for the relatively rigid structures (e.g., retaining walls), to which noise barriers are generally attached.

10.6.2 Inspector Guards

Some areas exist where bridge inspectors and other maintenance staff are expected to approach a location where a fall hazard exists. This hazard can be mitigated by wearing appropriate PPE, or more preferably, by the use a railing. Site specific considerations shall be reviewed to assess whether there is a need for an Inspector Guard to protect against this, with the final decision made by the Head of the Structural Section. Installation of guards should be planned as part of the major capital program when work is planned on the structure. In some unique situations, guards may be provided as standalone installations.

If guards were found to be needed in the review of fall hazard risk assessment, the following policies shall apply:

1. Retaining walls within MTO's right-of-way in areas with no pedestrian access where the height of the wall exceeds 2.4 m shall be equipped with a guard or guardrail on top of the wall as shown in Figure 10.6.1. For sloped walls where the height tapers from zero to the full height of the embankment, the guard shall be placed starting from a height of 1.0 m as shown in Figure 10.6.1. Guards shall be designed to meet the requirements for guardrails of Industrial Requirements, *R.R.O. 1990, Reg. 851, s.14*. Those requirements include a top and middle rail, but do not require a kick plate.

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According to *R.R.O. 1990, Reg. 851, section 14. (2)* the loading and structural design shall be in accordance with *O. Reg. 332/12: Building Code*.

2. At bridges and culverts, where the fall hazard from the obvert exceeds 2.4 m, the headwall or top slab of the culvert shall be equipped with a guard on top of the wall conforming to Section 1 of this memo. Figure 10.6.2 illustrates the schematic locations of inspector guards in two typical culvert scenarios.

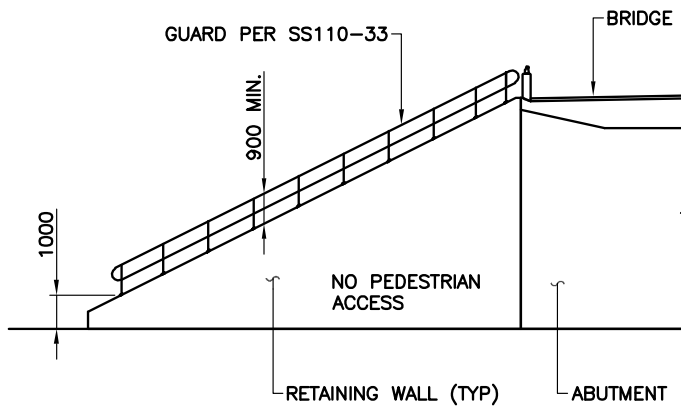


FIGURE 10.6.1 – GUARD REQUIREMENTS WITHIN MTO'S RIGHT-OF-WAY WITHOUT PEDESTRIAN ACCESS

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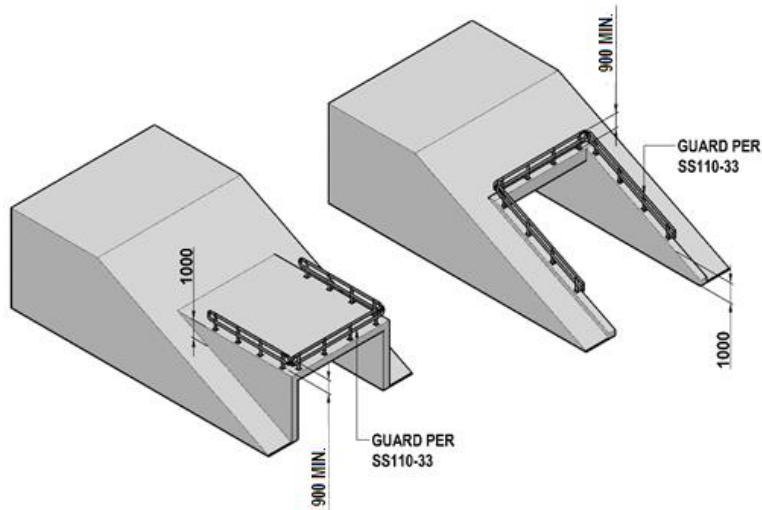


FIGURE 10.6.2 – GUARD REQUIREMENTS AT CULVERTS WHERE FALL HAZARD EXCEEDS 2.4M

The guards shall be detailed as follows:

1. The guards shall be detailed on the structural drawings within a Contract and shall include:
 - a. an elevation or plan to denote the limits of the guards;
 - b. details at expansion joints in the railings;
 - c. mounting details, and,
 - d. details of anchorage at base plates.
2. The guard shall be mounted on the wall, within 300 mm of the exterior face of the wall.
3. Posts shall be installed vertically.
4. Posts shall be mounted to the retaining wall with base plates and anchors designed to resist the loads imposed on the guard. Anchors shall be embedded into the retaining wall or anchored with epoxy. Given their history of problematic maintenance, posts shall not be embedded directly into the concrete, ~~nor shall anchors be used to affix base plates.~~
5. Guards shall be designed to avoid the accumulation of standing water at posts and connections, with positive drainage to the fill side of the wall where practical.
6. Guards shall be galvanized, and pipe shall have a wall thickness no less than 3.5 mm.
7. Pickets, when required, shall be solid bar, round, or plate.

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8. Connections shall be designed by the structural engineer. The structural engineer shall ensure that the underlying surface and component to which the guard is mounted, is adequate to resist the force applied to it from the post.
9. Drawings of guards shall be sealed per the requirements of the Structural Manual Division 1.

Structural Standard Drawings SS110-33, Inspector Guard Details, is designed to meet the design requirements above. Other Guards, meeting the requirements of the Industrial Regulations or Building Code, as needed, may be used for longer applications or other unique cases, where approved by the Head of the Structural Section. For Corrugated Metal Culverts the details shown on SS110-33 will require modifications to suit the placement and attachment to corrugated metal.

For locations that are more accessible to pedestrians, and with fall hazards as low as 600mm, a taller railing and/or one satisfying *Ontario Building Code, O. Reg. 332/12, s.3.3.1.17*, with loading as specified in *s.4.1.5.14* may be required.

10.6.3 Unreinforced Concrete Median Barriers and Shoulder Barriers

Unreinforced barrier walls are no longer used on Ministry bridges. The justification for the use of unreinforced concrete median barriers was based entirely on testing and a history of acceptable performance, rather than on their being able to meet structural design requirements. Outside barriers on structures have always had to meet structural design requirements.

Barrier walls are not permitted to be slip-formed on structures because the horizontal reinforcement may produce voids beneath themselves during the slip-forming process. Moisture could collect in these voids and cause problems when the moisture would freeze. Concrete median barriers on structures must be cast-in-place and shall not be placed directly on waterproofing membranes and must be provided with lateral restraint to prevent movement. If the membrane is stopped at the barrier, it must be turned up the vertical face at the barrier base and into a groove. With this arrangement, the barrier must be dowelled to the deck so that small relative movement does not rupture the membrane. Median barrier details shown on Structural Standard Drawings SS 110-62/63 meet these requirements and that of the CHBDC.

10.7 Miscellaneous Details of Barriers**10.7.1 Architectural Finish on Barrier Walls**

Bridge barriers are usually located where they are frequently seen by the public. An architectural finish on the wall surface provides a pleasant aesthetic while maintaining the proper structural function of the barrier.

A TL-5 concrete barrier wall with an architectural finish added to the outside face is shown in SS110-70/93/99. The surface finish can be selected from the four standard patterns given in Structural Standard Drawings SS110-71/72/73/74.

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10.7.2 Detailing of Standard Steel Tube Railing

The Structural Standard Drawings permit detailing using either of two methods:

- a) For structures on which the exact length of parapet or barrier railing can be readily established, e.g., individual structures on tangent, the spacing of the posts shall be shown on the drawings.
- b) For other structures, e.g., structures on horizontal curves; structures or retaining walls in groups connected by railing, on which the exact rail length may not be readily established, the post spacing shall be determined by the contractor and should not be shown on the plans. A note to this effect should be given on the drawings.

Unless otherwise specified in the standards, the post spacing with single tube hand railings shall not exceed 2.5 m or 3.5 m for aluminum and steel rails respectively. For other HSS steel railings, post spacing shall not exceed values specified on the respective standard drawings. The rail shall be supplied in length to be attached to a minimum of three rail posts and the rail splice shall be located within 600 mm from the post. The rail should be ordered in standard lengths satisfying the criteria mentioned above plus pieces for each side of the bridge of such a length as to give the exact rail length required.

If the curvature of the rail in position is sharper than 150 m radius, at least the first and last sections of the rails should be pre bent and a note to this effect should be given on the drawings.

10.7.3 Reinforcement Detail of Concrete Barrier Walls on Structures

The following requirements apply:

- a) Longitudinal reinforcement in concrete barrier walls must be continuous through construction joints;
- b) Unsupported ends of barrier walls (including those at expansion joints) must, for a distance of 1 m from the unsupported end, be provided with double the amount of reinforcement required for moment for the remainder of the wall. This is normally done by doubling up the number of bars at the unsupported end thereby reducing the spacing to one half of the normal spacing.

10.7.4 Ducts in Concrete Barrier Walls

The following must be considered for all bridges with ducts in the concrete barrier walls:

- a) A maximum of two ducts are permitted in a barrier;
- b) For all new bridges when electrical duct is required, the duct diameter shall follow electrical OPSD standards. Parapet walls shall be 300 mm wide to permit use of 75mm diameter ducts;

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- c) Ducts shall be no larger than 50 mm in parapet walls of 250 mm width. For existing bridges where a 300 mm width of parapet wall is not feasible, use of 50 mm diameter ducts in 250 mm wide parapet walls can be considered, with consultation with the project's electrical engineers to determine if the 50 mm diameter duct is adequate for the project-specific needs;
- d) Ducts shall be positioned to be tied to the reinforcement in the back face of the parapet or barrier, with the lower duct at 200 mm above the asphalt. Duct positioning shall allow concrete to flow around tension reinforcement and shall not impede the anchorage of the tension reinforcement;
- d) Junction boxes should not be located within 1 m from expansion joints, where the vertical steel is at 110 mm spacing;
- e) The deflection cavity for the duct (located at bridge expansion joints) is 143 mm dia. x 150 mm deep. This would interfere with reinforcement if the cover were at the maximum tolerance of 70 ± 20 mm (90 mm). A note should be added stating:

"ADJUST END BAR SPACING TO ACCOMMODATE DUCT DEFLECTION CAVITY FORMER.";
- f) On structures with expansion joint assemblies, especially joints such as modular joints (DSM 9.40.20), where the armouring is carried to the top of the barrier wall, provision must be made to allow the ducts and duct deflection cavity former to be placed.

10.8 Barrier Design Aid

Dead loads of standard permanent barriers and temporary barriers are provided in Division 3 in Design Aid 10-1 and Design Aid 10-2 respectively. For permanent barriers the dead load is tabulated for typical barrier sections to be used for the design of superstructure elements.

The temporary Barrier design Aid 10-2 contains the MTO's standard Precast temporary barriers along with proprietary steel barrier. Information of these proprietary barrier can be obtained from MTO's Roadside Design Manual.

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SECTION 11 - RIGID FRAMES

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SECTION 11 - RIGID FRAMES

11 RIGID FRAMES**11.1 General**

Rigid frame structures are ideal for short to medium span bridges. The high degree of structural indeterminacy allows redistribution of forces between the deck and the substructure contributing to resilience in the face of extreme events. The jointless nature of a rigid frame structure offers a sustainable structure with low maintenance efforts during the service life of the bridge.

The following sections provide discussion of key design parameters of rigid frame structures.

11.1.1 Backfill

Unless the abutment walls have been designed to withstand earth pressure without the deck in place, backfill should not be placed behind them until the deck is constructed. Therefore, in most cases a note stating this should be shown on the contract drawings. A standard note to be shown on the general arrangement drawing is given in Section 2.6.6.

The footings of rigid frame bridges exert a horizontal thrust upon the foundation, as well as a vertical thrust. If the footings are free to rotate the frame is said to have hinged supports. When the rotation of the footing is prevented, the supports are said to be fixed. The supports are actually rarely hinged or fixed, but the foundation conditions lie somewhere in the range between these two extremes: that is, the supports are restrained.

If the summation of the horizontal forces at the footing is not equal to zero (i.e., there is insufficient resistance against sliding) the remaining unbalanced horizontal force will cause a moment at the top of the rigid frame that should be accounted for in the design.

Normally, rigid frame structures are to be designed assuming simultaneous earth pressure on both sides. It is essential in such cases that the general arrangement should bear the standard note (given in Section 2.6.6) under "Construction Notes", on the right-hand side of the drawing.

The standard note, or a shorter note, e.g., from "Backfill" to "abutment", in the note above, must also appear on the preliminary version of the general arrangement.

When essential due to site access problems, a structure may be designed for earth pressure on one side. This must be stated as a requirement in the Structural Design Report. When a structure is designed for earth pressure on one side, the general arrangement shall bear the following note under "Construction Notes" instead of the standard note:

"BACKFILL BEHIND (designate, e.g., "the north", "either", etc.) ABUTMENT MAY BE COMPLETED BEFORE BACKFILLING BEHIND THE OTHER ABUTMENT".

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If the design of the abutment wall requires that it be supported during construction (e.g., for stability) a note to this effect should be shown on the general arrangement drawing under "Construction Notes".

11.1.2 Earth Pressure

Earth pressure distributions shall correspond to the anticipated movement at nominal temperature loads. Nevertheless, Rigid-rigid frames with spans up to 30 m, and other structures where the deflection of the abutment is prevented by the propping action of the deck, shall may be designed for the at-rest earth pressure.

11.1.3 Drainage

The provisions of Section 5.2.1.2. (b) to (e) inclusive apply to the legs of rigid frame structures.

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12 REINFORCING**12.1 General**

Reinforcing steel in structures shall be CSA G30.18 Grade 500W. Premium reinforcing steel shall be stainless steel according to OPSS 1440 or GFRP according to OPSS 1640. Designs shall use a yield strength of 500 MPa in proportioning size and spacing of all reinforcing steel in all components.

Structures shall be designed with a combination of black carbon reinforcement and premium reinforcement, as required by Section 12.2. Steel reinforcement and GFRP shall not be used in different orthogonal directions within the mat of reinforcement in a given area. At locations where a mat transitions from steel to GFRP reinforcement, the designer shall consider the implications for the structural behaviour (since the sectional behaviour may change from being tensioned controlled to compression controlled).

Examples of how to approach design with GFRP reinforcing bar include:

- ISIS Design Manual, No. 3, Reinforcing Concrete Structures with Fibre Reinforced Polymers, September 2007;
- ISIS Design Manual, No. 4, FRP Rehabilitation of Reinforced Concrete Structures, March 2008;
- Bank, Lawrence C. Composites for Construction: Structural Design with FRP Materials. 1st Ed. Hoboken: John Wiley & Sons, Inc., 2006;
- Nanni, Antonio, De Luca, Antonio, Jawaheri Zadeh, Hany. Reinforced Concrete with FRP Bars: Mechanics and Design. 1st Ed. London: CRC Press, 2014.

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However, these examples shall not be considered authoritative for MTO work. MTO published documents and the CHBDC shall govern.

For bridges with decks that are waterproofed, black (carbon) reinforcing bars shall be used in the deck except in the locations identified in Section 12.2, where Premium Reinforcing materials (see Section 2.4.8) shall be used.

12.1.1 Reinforcing Steel Bar - Identification

Reinforcing bars shall be designated as illustrated in the following example pertaining to black steel reinforcing:

15M @ 300 

which denotes size 15 metric bars spaced at 300 mm, centre-to-centre, with the indicated shape. M is the metric bar size identifier.

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Some examples of bar designation for black steel reinforcing, illustrating the above, are as follows:

15M

15M @ 300

20 - 15M @ 300

When the required bar shape is clear on the plan or section, the shape need not be given.

The bar size must always be given, but the bar spacing is not required to be given if they are specified as "equally spaced" within a clearly defined length or width on the contract drawings.

Where the length over which they apply can be calculated, or the distance is given, the number of bars is not required to be shown.

The quantity, size and spacing of bars should be given in situations when the extent of the bars is unclear, or it is simpler to do so.

The reinforcing schedules and placing drawings, which are prepared by the contractor, will contain bar marks for identification purposes.

12.1.2 Premium Reinforcing - Identification

Prefix identifiers, as specified below, shall be used with Premium Reinforcing bar designations. A prefix identifier shall be regarded as an inseparable part of the bar identification.

Stainless Reinforcing Steel:

Stainless steel reinforcing shall be prefixed by the letter S. For example, a metric, 15 mm diameter, stainless steel reinforcing bar would be denoted as S15M. The General Notes on the drawings shall indicate the Type of Stainless Steel (2205 Duplex or 316LN, or 2304 Duplex) to be used.

GFRP Reinforcing:

Only Grade III GFRP reinforcement shall be used in the design. The bars shall be identified by "G" for GFRP along with the nominal diameter (for example, a metric 15 mm nominal diameter, GFRP reinforcing bar would be denoted as G15). The suffix M is omitted from GFRP bar identifiers as the bar sizes do not conform to those of Canadian, metric steel reinforcement. Bar diameter designators and nominal bar areas are defined in CSA S807. Note that the nominal area values are not based on the designated diameter. Nominal area values from CSA S807 shall be used in design.

GFRP reinforcement (bar sizes and areas as per CSA S807) is available in nominal metric bars which are equivalent in area to the imperial sizes (e.g., size 15 has area of

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199 mm², size 25 has area of 509 mm²). For MTO projects, nominal diameters of 13, 15, 20, and 25 shall be used. The 10 mm nominal diameter is susceptible to damage during construction, and 30 mm diameter is difficult for performing QC and QA testing, and these sizes shall only be used with approval of the Structural Section.

12.2 Premium Reinforcing – Where Required

Premium Reinforcing shall be used in locations vulnerable to salt induced corrosion (as defined in Table 12.2.1). In those locations, bars that are within 100 mm (e.g., cover specified as 125 ± 25 mm) of the surfaces shall be of Premium Reinforcing unless:

- a) The surface is permanently covered by water; or,
- b) Footings permanently covered with at least 500 mm of soil and the concrete cover is specified as minimum of 75 mm (i.e., 100 ± 25); or,
- c) Buried structures where the surface is waterproofed.

Even for bridge decks that are waterproofed, those on busy highways have additional wear of the waterproofing due to heavy traffic while simultaneously tending to have rehabilitation and waterproofing replacement delayed due to the desire to avoid traffic disruptions. For these bridges, there are benefits to using Premium Reinforcement in the deck. Table 12.2.1 identifies bridge decks that require Premium Reinforcement.

For bridge rehabilitation:

- Where stainless steel is used, 2304 Duplex shall be specified. It is expected that Type 2304, with its lower Nickel content will be less expensive, but still adequate for the rehabilitation service life;
- Where the remaining service life of the bridge is less than 35 years, conventional concrete and black steel shall be used;
- Where the remaining service life of the bridge is between 35 and 45 years, Premium Reinforcement is only required in exposed surfaces being replaced as part of rehabilitation;
- Where the remaining service life of the bridge is above longer than 45 years, the extent of Premium Reinforcement, for components which require replacement, shall be the same as for a new bridge.

Where bridges are being widened, the reinforcement requirements shall be as specified by the Structural Section, with consideration to the remaining life of the widened portion, and whether it will survive beyond the remaining life of the existing components of the bridge. Premium Reinforcing is not required for selected components or structures that are covered by specific standards and that have a design service life less than 50 years, such as footings of sign support structures and footings of high mast pole structures.

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TABLE 12.2.1 – REINFORCING REQUIREMENTS FOR SURFACES WITHIN SPLASH ZONE

Component Surface		Reinforcement ⁶
Deck Top	<u>Deck Slab Top Mat</u>	<u>Stainless or GFRP on freeways with AADT > 50,000</u>
	<u>Closure pours between CIP deck stages</u>	<u>Stainless or GFRP on freeways with AADT > 50,000</u>
	Deck Closure pours between Precast components.	Stainless or GFRP
	Deck Top within 1.5 m of expansion joint gap.	Stainless
	Topping Slab within 1.5 m of expansion joint.	Stainless
Sidewalks and Barriers	Barrier and parapet walls.	Stainless or GFRP ¹
	Sidewalks, medians, and curbs.	Stainless or GFRP
Deck Soffit	See Figure 12.2.1, Figure 12.2.2 and Figure 12.2.3.	Stainless or GFRP
	Deck soffit within 1.5 m of expansion joint gap.	Stainless
	Soffit of Post-tensioned Bridges with AADT > 50,000 under bridge ⁸ .	Stainless or GFRP
	Precast deck.	Same as CIP deck.
Girders	Stirrups and perimeter bars from precast component (i.e., NU, CPCI, box) within 1.5 m of expansion joints.	Stainless ²
Abutment Areas	Front surface of ballast wall and top surfaces of bearing seats and pedestals exposed to roadway drainage or possible dripping ³ .	Stainless ²
	Surfaces of abutments, wingwalls, retaining and MSE(RSS) walls ⁵ that are exposed to roadway drainage or possible dripping ³ .	Stainless or GFRP
	Exposed faces of abutment walls, wingwalls, tunnels, retaining and MSE (RSS) walls ⁵ within 7 m horizontally and 5 m vertically of an existing or future roadway ⁴ .	Stainless or GFRP
Piers ⁷	Pier caps, bearing seats and sides of pier shafts and columns exposed to roadway drainage or possible dripping ³ .	Stainless

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	Side and end surfaces of pier shafts and columns within 10 m horizontally and 5 m vertically above an existing or future roadway ⁶ . All reinforcement in the column should be considered, including dowels from the footing.	Stainless
	Pier shafts and columns exposed to roadway drainage or possible dripping.	Stainless
Misc.	Top surfaces of expansion joint end dams including those at sleeper slabs supporting the end of the approach slabs.	Stainless ²
	Bars connecting approach slab to bridge and at approach slab end.	Stainless ²
	For semi-integral abutment bridges: All surfaces at and within 750 mm of the joint between the deck and wingwall. (See Figure 12.2.4 for examples).	Stainless ²
	Top and side surfaces of bases for lighting poles that are attached to bridges.	Stainless ²

Notes:

1. Where specified by the Structural Section, stainless steel may be required for TL-5 barriers on bridges with AADT > 100,000 and where deck/shoulder width is narrow, precluding a safe work zone if repair to the barrier wall is required. For other situations, use either Stainless Steel or GFRP.
2. GFRP shall not be used due to the excessive bend requirements of bars in these locations.
3. This includes surfaces below joints (sealed or unsealed), deck drain outlets, and overhanging features from which dripping may occur. "Below" should be taken to mean within a vertical cone of height 5m (measured from the bearing seat or drainage discharge) and having an apex angle of 45 degrees or likely to receive run-off from a surface within such a cone.
4. Distance measured from the edge of the nearest travelled lane.
5. The designer shall specify which MSE walls in the contract fall within these limits and require Premium Reinforcing in their facings. Premium reinforcing shall not be specified on contract drawings and shall be supplied according to approved DSM product drawings.
6. When AADT both on and under the bridge (if applicable) are between 400 and 2,000, Premium Reinforcement is only required in the barrier walls, curbs and sidewalks, with Black steel elsewhere. Where AADT < 400 both on and under the bridge (if applicable), Premium Reinforcement is not required, and Black steel shall be used throughout.

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7. Although pier caps are within 5 meters vertically of the roadway surface, they are on the outer periphery of the splash zone and historically have performed well with black steel.

8. The requirement for Premium Reinforcing applies only to portions of the soffit within 6 m of top of the roadway surface. For the purpose of evaluation and design, post-tensioned bridges have a design service life of 100 years.

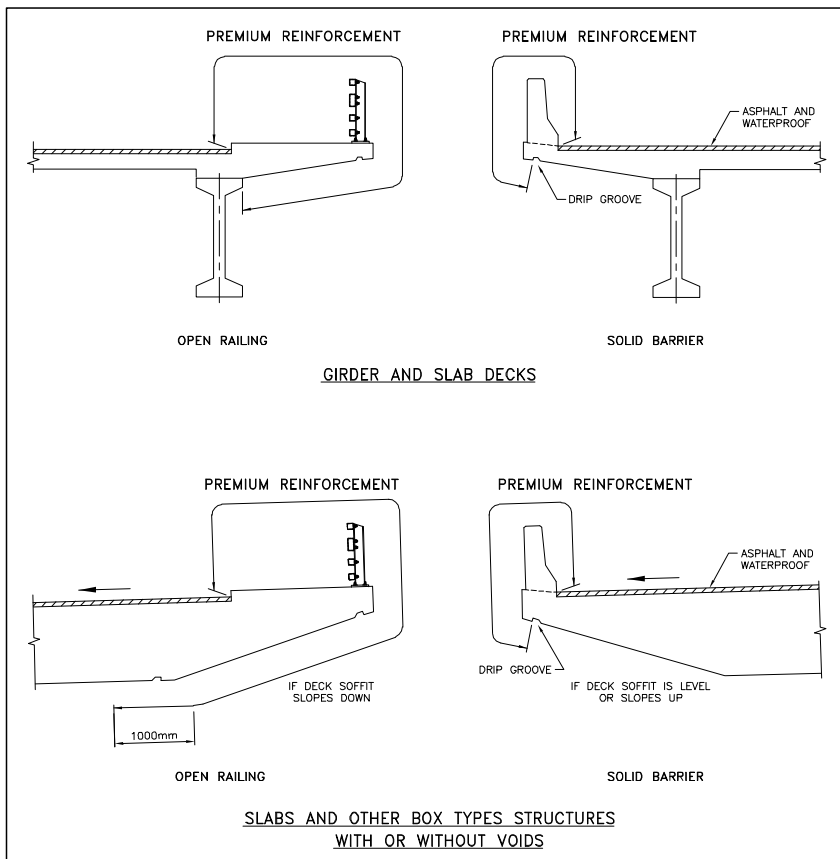


FIGURE 12.2.1 – PREMIUM REINFORCEMENT BELOW JOINTS AND RAILINGS

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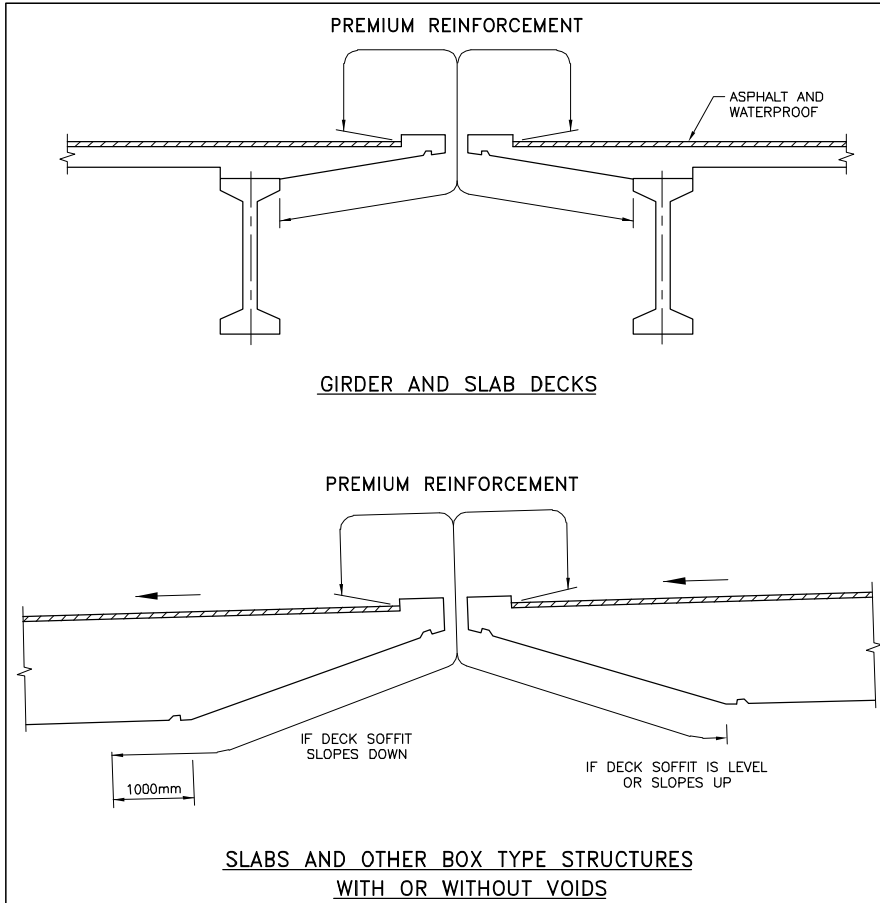


FIGURE 12.2.2 – PREMIUM REINFORCEMENT BELOW JOINTS AND RAILINGS

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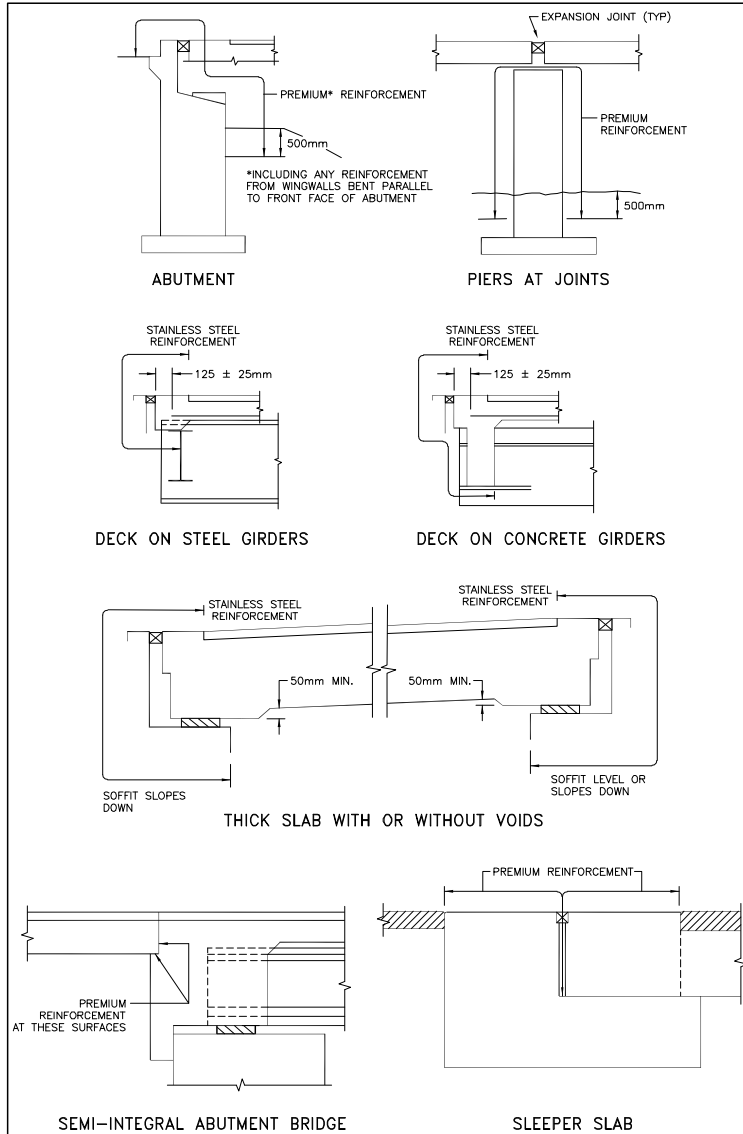


FIGURE 12.2.3 – PREMIUM REINFORCEMENT BELOW JOINTS AND RAILINGS

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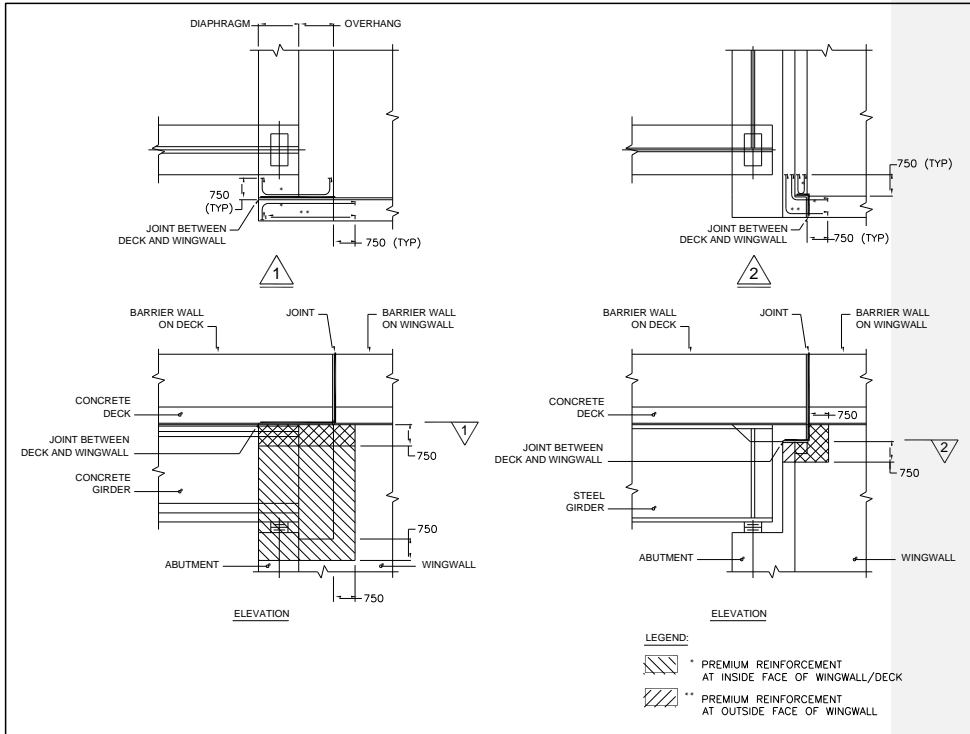


FIGURE 12.2.4 – PREMIUM REINFORCEMENT BELOW JOINTS AND RAILING

12.2.1 Reinforcing Steel for Concrete Culverts

Reinforcing steel quantities for all cast-in-place concrete culverts in a contract are combined and included in a separate lump sum tender item for each culvert. However, when the total quantity of reinforcing steel for all concrete culverts in a contract is less than 5 tonnes, the reinforcing steel will be included in the concrete item "Concrete in Culverts" and there will be no separate tender item for reinforcing steel.

12.3 Reinforcing Geometry

12.3.1 Reinforcing – Dimensional and Shape Limitations

12.3.1.1 Steel Reinforcing

The normal mill length for reinforcing steel bar stock is 12 m for size 10M, and 18 m for all other bar sizes. These lengths should never be exceeded for straight bars.

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The shipment by truck of bars longer than 15 m requires an uncommon type of vehicle and should be avoided unless there is an economic advantage.

Bent bars, when laid flat, should fit into a rectangle having dimensions not greater than 20 m by 2.4 m.

12.3.1.2 GFRP Reinforcing

Length and shipment limitations for straight GFRP reinforcing bars are similar to those of steel reinforcing bars. The process for forming and curing bent GFRP bars means there are general limitations on the types of bent bar shapes that can be manufactured, either by total length or effective plan area of shape. Due to differences in the manufacturing process, the longest leg length of bent bars varies between DSM listed suppliers. Consult DSM listed supplier documentation for further information.

In order to provide a GFRP design which all DSM suppliers are able to supply, the following detailing limits should be respected:

- For components detailed with bent bars, the bars should be the smallest practical size (13, 15 or 20).
- For 90° bends, the length of the sum of each leg should be less than 3 m.
- Stirrups should be designed with any assembly of shapes satisfying S6, each of them designed to fit within a 1.1 m by 2.2 m rectangle. If these cannot be met, stirrups and bent shapes should be detailed to fit within a 2.5 by 2 m rectangle.
- Shapes should have all bends in a single direction (i.e., all bends moving in clockwise direction)
- Spirals or hoops should be detailed with extra turns or laps for anchorage and are to be used only for structures classified in seismic performance category 1 per S6. Spirals are available for columns up to 1.2 m in diameter. Spirals with bends into the core of the column are not commercially available. Hoops may be used instead of spirals in the drilled shaft foundations when the pitch spacing of spiral becomes tight for concrete placement.

These limitations should be discussed with manufacturers if they affect the design.

Mechanical splices of GFRP reinforcing bar are not available, only lap splices shall be permitted.

12.3.2 Use of Size 10M Reinforcing Bars

Size 10M bars are too flexible for most applications, particularly in deck slabs and where they may be walked on. Size 10M bars shall not be used in cast-in place deck slabs; their use should be restricted to small sections, such as barrier walls and precast units, where supporting stirrups are close together.

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12.3.3 Availability of 45M and 55M Reinforcing Steel

Size 45M and 55M reinforcing steel bars are not commonly required within the reinforcing steel industry. Fabricators cannot economically justify stocking these bars for which there is a low demand. As a result, a premium may be paid, and delays are possible if these bars are specified. It is recommended that these bar sizes be avoided wherever possible.

12.4 Reinforcement Details**12.4.1 Reinforcing Schedules**

Rebar schedules shall not be prepared or provided by the designer in the contract documents. Contractors are expected to produce the rebar schedules and working drawings. The designer shall prepare the contract drawings so that there are sufficient rebar details and other relevant information to facilitate the preparation of rebar schedules and working drawings by the fabricator.

12.4.2 Detailing Reinforcing on Contract Drawings

1. The designer shall ensure that the contract drawings clearly show all required detailing information.
2. Reinforcing bars shall be identified according to Sections 12.1.1 and 12.1.2.
3. Reinforcement details shall include the size, shape, spacing and placing limits for individual bars, and if necessary, the number of bars. Detailing shall be adequate to eliminate ambiguity and misinterpretation. Dimensioned sketches of bars should be provided as required.
4. Where multiple layers of reinforcement exist, the bar sizes in the outer layer shall not be less than any inner layer, and the bar spacing (linear for rectangular sections and angular for circular columns) shall not be greater than in any inner layer.
5. Lap lengths for reinforcing steel, not indicated on the contract drawings, shall be Class B as per CHBDC. Other lap lengths, where required, shall be indicated on the drawings. For GFRP, these lap lengths shall be as modified according to the requirements of CHBDC.
6. Reinforcing steel bar hooks shall be as per Structural Standard Drawing SS112-1 unless indicated otherwise.
7. Reinforcing steel stirrups shall have hooks as per Structural Standard Drawing SS112-1, unless indicated otherwise.
8. Bent or curved GFRP reinforcing shall be fabricated with radii specified by the manufacturer. Designers should be aware of manufacturers' recommendations and ensure that they are accounted for in the design, uncommon bend radii may increase the cost of bent reinforcement as manufacturer's may need to purchase components

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for their bend forming apparatus.

Because of instances of poor quality GFRP reinforcing and code strength limitations at "bend" locations, straight bars should be specified where possible.

9. Reinforcing tender items are lump sum bid items and the quantity shall not be listed on the contract "Quantities-Structure" drawing.
10. The designer shall forward the quantities of reinforcing items for black steel reinforcing, stainless reinforcing steel and GFRP reinforcing to the Project Delivery Section for Ministry internal use.
11. The requirements of the notes 5 to 7 above should be given as part of the general notes on the general arrangement drawing of all projects.
12. Structural reinforcement shown on the contract drawings shall not be used for any electrical grounding purposes. Any electrical grounding, if required, shall be done using independent insulated grounding cables that could be buried in/through concrete, if preferred, and exiting the structural component to be connected outside to a copper grounding rod.

12.4.3 Hooks and Bar Bends

Structural Standard Drawing SS112-1 shall be attached to the full-size Structural Standard Drawings in a contract. It is intended for the contractor's use, in order to produce consistently accurate hooks and bar bends in black and stainless reinforcing steel.

The GFRP industry has standardized bent shapes using a bend-radius-to-bar-diameter ratio (r/d) of 4.0 and may not be able to readily manufacture bends with other r/d ratios. The bend strength of GFRP reinforcing bar is strongly correlated with the r/d ratio, which means that while the CHBDC (2019) permits the use of bars with a bend-radius-to-bar diameter ratio of 3.0, most of the MTO's DSM listed suppliers were qualified for bends a $r/d = 4.0$ as meeting bend strength requirements. As such, both the minimum r/d and standard r/d used on MTO projects shall be 4.0.

12.4.4 Spiral Reinforcement

Ontario reinforcement fabricators are only able to bend spirals of size 15M or smaller. If larger transverse reinforcement is required, it may be possible to use bundled spirals (2 nested 15M spirals that are touching, and then with a spiral pitch somewhat larger than what could be achieved with a single spiral. Hoops of size 20M or 25M are also possible.

12.4.5 Splicing of Reinforcing Bars

Design drawings shall clearly define splice type(s), locations, and force requirements. For example, lapped versus mechanical splices; Type 1 versus Type 2 mechanical splice.

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Lap splices are preferred for splicing of reinforcing bars due to their reliability and ease of inspection/verification during construction. Mechanical splices require material testing and assembly verification during construction and should be used where geometry necessitates, or where they provide benefits of reduced congestion of reinforcement or cost savings from materials reduction, such as:

- a) When the location of construction joints and provision for future construction dictates the use of mechanical splices to provide tensile continuity, e.g., staged replacement of bridge decks when projecting bars are not feasible.
- b) Splicing bars greater than 35M.
- c) Where spacing of rebar is insufficient to permit lapping of bars, or large amounts of reinforcement or larger bars may obstruct concrete flow. This situation is common in columns and caissons.
- d) Forming openings in tub girders to facilitate formwork removal for cast-in-place decks.

OPSS 905 lists and describes the following six groups of mechanical connectors (subject to DSM lists) for reinforcing steel bars:

- | | |
|----------------------------|---------------------------------------|
| a. Filled Sleeve Type | b. Sleeve Swaged Coupler Type |
| c. Threaded Coupler Type | d. Hot Rolled Thread Bar Coupler Type |
| e. Forged Bar Coupler Type | f. Form Saver Type |

The MTO groups of mechanical connectors is descriptive and a historical practice for the DSM. The categorization of all DSM mechanical connectors is tension-compression mechanical splices that can resist both tensile and compressive forces. The type of DSM mechanical connector may be Type 1 or Type 2.

Mechanical connectors of black steel and stainless steel shall be supplied in accordance with the latest active DSM at the time of contract awarded. Black steel bars shall not be coupled with stainless steel connectors, and vice versa.

Mechanical connectors are not available or acceptable for splicing of GFRP reinforcing.

Minimum cover requirements apply to the splices and mechanical connectors. The designer shall also consider geometric and clearance requirements for couplers (e.g., adequate access for swaged connectors, space between non-staggered couplers for concrete flow, etc.).

Where required, mechanical connectors and splices shall be designed for their specified fatigue loading.

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12.4.6 Anchor Headed GFRP Bars

Anchor heads are only permitted on GFRP reinforcing bars of size 15 and may be assumed to develop 100 kN of bar strength.

12.5 Anchors in Concrete

Anchors post-installed into concrete shall be adhesive bonded dowels. Mechanical, grouted bonded and screw types shall not be used.

12.5.1 Post-installed Adhesive Dowels in Concrete

~~Adhesive Dowels~~ ~~dowels~~ into concrete shall not be used in new structures, except for reinforcement installed through steel or precast concrete girders at integral abutments and piers, and ~~for the~~ connection of precast headwalls to precast culverts.

Dowels shall be detailed with holes drilled in downward, downward inclined, and horizontal orientations. Adhesive anchors upwardly inclined, including vertically overhead shall not be allowed. DSM List # 9.30.25 does not include any adhesives for use in overhead applications, and for these applications, the industry is moving towards a requirement that dowels be installed by personnel certified by an applicable certification program. The collapse of the suspended ceiling system of the I-90 connector tunnel in Boston on July 10, 2006, prompted significant research and advancements in epoxy technology and dowel installation requirements, explained in the FHWA publication T5140.34, NCHRP Report 639, NCHRP Report 757, and the ACI/CRSI "Adhesive Anchor Installer" certification program. Until approved products are included in the DSM and installer/inspector programs become more accessible in Ontario, to ensure public safety and long-term performance of the infrastructure, upwardly inclined dowels are not allowed.

Dowels into concrete shall not be larger than 25M.

Installation and acceptance testing shall be according to the relevant OPSS and SSP specification.

For all applications other than anchorage of barriers and expansion joint end dams, epoxy ~~anchors/dowels~~ shall not be relied on to provide more than 25% of the total required tensile resistance at the critical section, and epoxy ~~anchors/dowels~~ shall not be used for applications where sustained tensile load exceeds 25% of the allowable tensile capacity of the dowel based on bond. If a higher capacity is required, the design of post-installed adhesive ~~anchors/dowels~~ may be completed according to most recent edition of CSA A23.3 Annex D.

SECTION 13 - EXPANSION JOINTS AND BEARING ASSEMBLIES

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SECTION 13 - EXPANSION JOINTS AND BEARING ASSEMBLIES

13 EXPANSION JOINTS AND BEARING ASSEMBLIES**13.1 General**

This section describes the types of deck expansion joints and bearings used on MTO's bridges. The design and detailing requirements for both components are discussed throughout the section.

13.2 Expansion Joints**13.2.1 Expansion Joints in Decks**

Bridge expansion joints are designed to allow for continuous traffic across structures while accommodating movements due temperature variations of the structure, long-term effects of creep and shrinkage of concrete, and differential settlement of substructures. Expansion joints shall allow sufficient vertical movement to permit bearing replacement without the need to dismantle the bridge expansion joint.

The designer should make every effort to minimise the number of expansion joints along the length of structure and to locate expansion joints behind the abutments through integral or semi-integral abutment configurations.

In all cases, the expansion gap shall be detailed parallel to the skew of the deck, for the full width of the deck. In skewed bridges, the resulting exposed acute angled concrete edges shall be provided with 50 mm chamfers. Designers should avoid skew angles of 32 to 38 degrees, inclusive, to minimise the chances of snagging snow plough blades which are usually set at approximately 35 degrees.

In addition to longitudinal movement of the superstructure, displacements due to rotation of the structure under transient loads shall be considered.

The gap between the end of a deck and the ballast wall face must meet the requirements of the CHBDC. The gap between the underside of the deck and the bearing seat shall be large enough and accessible for cleaning. The drawings must show the gaps free of any material used to form them in construction.

When specifying an expansion joint movement rating, only those movements which can occur after the joint has been installed shall be considered. Thus, when specifying expansion joint movement ratings for rehabilitation projects, elastic shortening, hydration, shrinkage, and creep effects should be neglected. Likewise, for new prestressed concrete bridges, creep and shrinkage effects occurring prior to joint installation should be neglected. In prestressed concrete bridges, joint installation should occur as long after the completion of prestressing operations as the schedule permits and shall be indicated on the expansion joint standard.

Proper installation is critical in achieving the expected long-term performance of expansion joints. Important elements of the installation include proper alignment, installation after paving, and epoxy injection under the armouring. Proper installation of expansion joint

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assemblies accounts for compaction of the asphalt wearing surface, by traffic, adjacent to the concrete end dams.

Expansion joints may be installed prior to or after paving. The Ministry's preference has historically been to install deck joint assemblies after paving because this method provides a superior ride for vehicles, resulting in lower impact on the joint and its anchorage, and superior durability. Installing deck joint assemblies after paving often places the operation on the critical path of the construction schedule. Projects have the potential to be delayed and incur cost overruns if the designer and/or contractor does not fully account for this. It has been found that there are some instances where installation prior to paving is acceptable, such as when a reduced durability is acceptable over the remaining service life of the bridge.

The designer shall specify the method of deck joint assembly installation in all new bridge construction or bridge rehabilitation projects which require new deck joint assemblies as follows:

1. The designer may choose to specify installation prior to paving when all the following criteria are met:
 - a) The bridge is being rehabilitated;
 - b) The bridge is not skewed;
 - c) Average Annual Daily Traffic (AADT) is less than 2000;
 - d) The estimated working days for structure work, with installation prior to paving, is at least 10% less than with installation after paving.

The designer shall specify installation after paving for all other scenarios, unless written Approval is obtained by the designer from the Head, Structural Section.

2. The designer shall include Structural Standard Drawing SS113-38 in the Contract Documents and modify it to reflect the actual bridge and to show the specified method of installation.
3. The designer shall include fill-in standard special provision SP 920F03 in the Contract Documents, amending OPSS 920 to clearly specify the method of deck joint assembly installation for each bridge.

The designer shall consider that installation prior to paving reduces the durability and longevity of the expansion joint. A service life of 20 years shall be assumed for financial analysis.

The guidelines included above may also be used to assist the designer in evaluating a contractor's request to install deck joint assemblies prior to paving.

Concrete end dams detailed between an asphalt wearing surface and an expansion joint seal shall be wide enough to permit the placing of concrete through the gap between joint

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armouring and concrete dam armouring. Concrete end dams should not be less than 500 mm in width, either side of the joint seal. Weep holes through the deck are required at low points at these dams. Refer to OPSD 3349.100.

Straight reinforcing bars shall be placed parallel to expansion joints in the concrete end dams and shall consist of:

- a) for modular joints: two shorter bars lapped between the support boxes; and,
- b) for all joints: at least two shorter lapped bars which are to be weaved through anchor loops and through other reinforcing steel.

Joints should be specified on the drawings by reference to the appropriate "Designated Sources for Material List" number and, in the case of DSM 9.40 - Joints, by type.

When splicing of joint armouring is unavoidable, such as between construction stages, permissible splice locations must be shown on the drawings. Such locations should be at crown points, if possible, and in no case shall they be located near curbs, barrier walls, wheel paths or at any point where water is likely to pond. On skew bridges, when the approved splice is shown at a crown, the splice in the armouring should be shown parallel to the centreline of the traffic lane and not perpendicular to centreline of joint.

The material for strip seals shall be polychloroprene (neoprene). Natural rubber is not permitted because MTO lab tests have found that the effects of ozone and oil swell tests were unacceptable.

13.2.2 Approved Expansion Joint Assemblies

The expansion joint systems approved for use on new and/or rehabilitated structures are listed in Designated Sources for Materials List DSM 9.40 - Joints as follows:

DSM NO.	DIVISION/APPLICATION/PRODUCT
DSM 9.40.18	Expansion, Injection Systems for Armouring
DSM 9.40.20	Expansion, Modular
DSM 9.40.24	Expansion, Strip Seals Anchored in Concrete, Type A
DSM 9.40.27	Expansion, Strip Seals Anchored in Concrete, Type C
DSM 9.40.33	Expansion, Strip Seals in Preformed Retainer

Guidelines for the application of these lists are given in 13.2.1 to 13.2.5. The selection of the expansion joints for bridges shall be based on the total design movements at SLS and other requirements of this section. During the design process, the designer shall select which type should be used for each joint location on the bridge (see Table 13.2.1).

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TABLE 13.2.1 – EXPANSION JOINT SELECTION

Design Movement at SLS (mm)	Integral and Semi-integral Abutment Bridges	Conventional Abutment Bridges	Note
0 to 10	-	Type A or C	Section 16.6 or Section 13.2.2
11 to 40	Type C*	Type A or C	* at Sleeper Slab
41 to 80	Type C*	Type A or C	* at Sleeper Slab
76 to 120	-	Sliding Plate	-
81 to 400	-	Modular	Field splicing of modular joints is not permitted

13.2.3 Expansion Joint Assemblies in Skewed Bridges

A plan and profile must be provided on the drawings for skewed expansion joints.

Joint assemblies in bridges with skews should be carried through sidewalks, medians, and barrier walls without horizontal change in direction wherever possible. A 50 mm chamfer shall be detailed on all exposed acute angled concrete deck edges on skewed bridges.

Where this is not possible the horizontal change in direction should be affected at a location not less than 100 mm and not greater than 600 mm from the edge of the asphalt at the barrier wall. The change in direction shall be affected by means of a 600 mm radius bend. The exact location of the radius bend should appear on the plan view of the structure on the contract drawings and on the contractor's shop drawings. Locating the horizontal change as close as is practicable to the barrier wall reduces the modifications necessary to the substructure.

A horizontal change in direction shall NOT be located within 100 mm of a vertical change in direction since simultaneous bending of the seal in two planes is either physically impossible or likely to lead to leakage between the seal and the clamping device.

Structures having a skew of up to 15 degrees or over 45 degrees shall have joint armouring anchorage bars, on the deck side, detailed perpendicular to the expansion joint. Structures skewed from over 15 degrees up to 45 degrees shall use joint armouring anchorage bars, on the deck side, detailed 30 degrees offset from the perpendicular to the expansion joint. Anchorages on the abutment side of the joint shall be placed at right angles to the joint. The gap and blockout dimensions for the expansion joint on skewed bridges should be detailed perpendicular to the joint.

13.2.4 Expansion Joint Drainage System

Strip seal expansion joints and the sliding plate expansion joint assembly are available with a drainage system which allows runoff from the seal to be channeled through the barrier and down to the ground through a scupper and downpipe according SS113-14. The drainage system prevents local ponding where the joint seal is otherwise upturned into the barrier and flushes the joint seal of debris. The flushing is particularly beneficial for sliding plate joints where the strip seal is covered with steel plates. The use of the

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drainage system is recommended when freezing and/or ponding of water is a concern, specifically for bridges on flat profiles or close to a flat section of a vertical curve. The drainage system shall not be used in aesthetically sensitive locations and underpasses.

13.2.5 Expansion Joints at the End of the Approach Slab of Integral and Semi-Integral Bridges

Integral abutment bridges are single or multi-span bridges with the superstructure integrally connected to the abutments. Both Integral and Semi-Integral bridge approach slabs are integrally connected to and move together with the superstructure and the abutment.

The movement demand is proportional to the expansion length of the superstructure. Presently, a bridge can be made integral when the expansion length of the superstructure does not exceed 75 m (i.e., total length of 150 m). The specified treatment at the end of the approach slab should vary with the expected movement.

For integral and semi-integral abutment bridges where the expected movement at the end of the approach slab does not exceed 10 mm, the movement end of the approach slab shall be detailed as follows. Regardless of the movement calculated, the following detail shall be used on integral bridges of up to 40 m in total length.

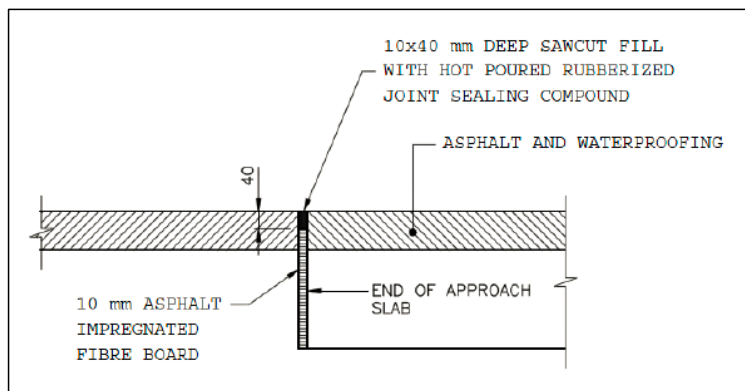


FIGURE 13.2.1 – EXPANSION JOINT DETAIL AT THE END OF APPROACH SLAB FOR SPAN LENGTH < 40M

For integral and semi-integral abutment bridges where the expected movement at the end of the approach slab is greater than 10 mm or for integral bridges longer than 40 m, Structural Standard Drawing SS113-36 shall be used. The approach end of the approach slab shall be supported on a sleeper slab and a Type C expansion joint shall be provided between the approach slab and the sleeper slab.

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13.2.6 Longitudinal Joints and Gaps

Longitudinal joints exposed to traffic shall be avoided unless approved by the Head of the Structural Section. When used, they shall be Type C expansion joints to accommodate movement demands from differential deflection and movement and have adequate durability to withstand the traffic loads placed upon them.

Twin bridges separated by a narrow longitudinal gap shall be sealed to prevent the accumulation and passage of debris, snow, water, and chlorides. These longitudinal gaps (typically between raised medians or parallel barrier walls) shall be detailed with a proprietary expansion joint seal according to MTO Policy Memo #SCB-SO-2021-02.

13.2.7 Completion of Structural Standard Drawings

The following note on the expansion joint Structural Standard Drawings must be completed by the designer:

"EXPANSION JOINT SHALL BE IN ACCORDANCE WITH THE DESIGNATED SOURCES FOR MATERIALS LIST DSM 9.40.^[Note 1] AND SHALL HAVE A RATED TOTAL MOVEMENT CAPACITY OF ... mm."^[Note 2]

Note 1: The designer shall insert the number of the appropriate DSM list or Lists which are acceptable. Where DSM 9.40 - Joints are shown, the acceptable type or types must also be given, except when all types are acceptable.

Note 2 The movement shown should be the required movement as determined by the Designer, calculated at Serviceability Limit States; this movement shall not exceed the rated movement of at least one of the joints in the type specified, and where possible, should not exceed the rated movement of at least one joint from each supplier in the type specified.

The movement ratings given in Designated Sources for Materials List DSM 9.40 were established from laboratory testing and to allow seal replacement at 25°C.

In completing the table of design requirements, the "J" dimension shall be determined by setting the joint seal to be closed to the minimum opening at the hottest effective temperature, increased by 5 mm where possible (i.e., when the full rated movement of the joint is not fully utilised). This will have the effect of providing slightly more opening for future replacement of the seal.

13.2.8 Force Due to Movement of Modular Joints

The force associated with the movement of modular joints shall be considered when designing abutments.

When a bridge contracts in the longitudinal direction due to a decrease in Temperature (as well as due to Creep and Shrinkage in prestressed concrete bridges), the modular joint responds by opening further to accommodate this movement. As the joint opens, the control springs which maintain equidistant openings across the seals are compressed, causing a corresponding resistance to the opening movement, and thereby transferring this resisting force as a horizontal load to the top of the ballast wall.

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Assuming a control spring stiffness of 0.15 kN/mm, and supports beams spaced at 800 mm centre-to-centre, the resulting unfactored horizontal force would be approximately 8 kN per metre length of ballast wall.

13.2.9 Strip Seal Joints Anchored in Concrete

The joints in this list are for bridges having total deck joint movements calculated at SLS, measured parallel to the centreline of the highway, and after joint assembly installation, of 80 mm or less but not greater than the rated movement of the joint specified.

DSM 9.40.24 and 9.40.27 is subdivided into several types based on the method of seal retention used. General guidelines showing the characteristics of each type and the applications for which it should be considered, are given on the following pages.

It is the responsibility of the designer to select the appropriate type or types of joint in accordance with those general guidelines. All types of expansion joints on the DSM List have been approved by the Ministry and should be considered for application on structures. Where further guidance is required the Head, Structural Section should be consulted.

Structural Standard Drawing SS113-11 covers anchorages and armouring for all types of expansion joints listed in DSM 9.40.24 and 9.40.27.

Strip Seals - Type A Steel Plate Clamping Device

Clamping Plate Subject to Direct Wheel Load

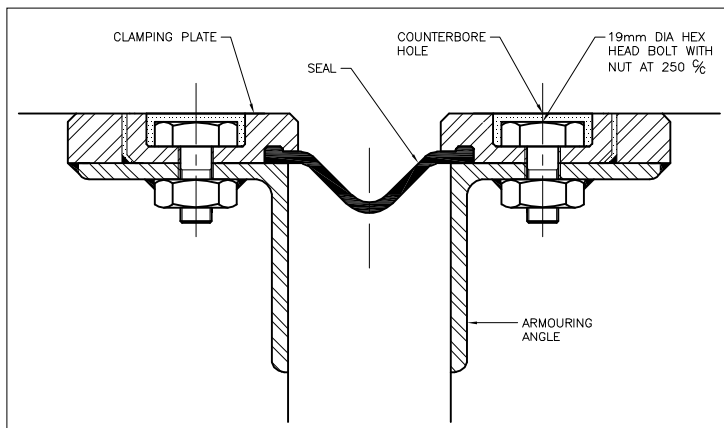
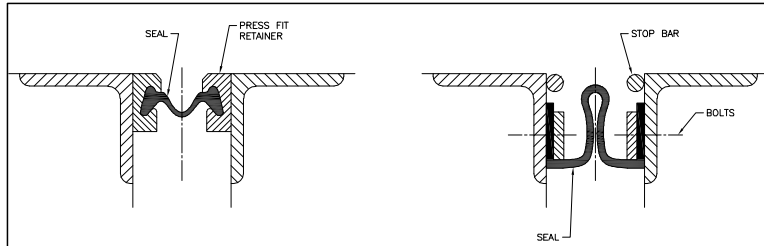


FIGURE 13.2.2 – STRIP SEALS – TYPE A STEEL PLATE CLAMPING DEVICE

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Characteristics

- Clamping plate directly supported.
- Few components.
- Excellent seal retention.
- Easy to install and replace seal.
- Direct access to clamping bolts and seal.
- Seal can be inspected completely and reused, if undamaged.
- Easy to modify for changes in elevation.
- Higher initial cost.

Strip Seals - Type C Retainers and Stop Bars**FIGURE 13.2.3 – STRIP SEALS – TYPE C RETAINERS AND STOP BARS**Characteristics

- Good seal retention.
- Can accommodate change in direction in plan.
- Seal cannot be inspected completely without removing the seal.
- Installation or replacement of seal may be difficult.
- Special tools and trained personnel required to install or replace seal for maintenance purposes.
- No direct access to clamping device or retainer and seal.
- Difficult to replace bolted clamping device or damaged retainer.

Application

Type A joints as per DSM 9.40.24 should be considered for all bridges and particularly for bridges which carry high volume freeway/arterial traffic (and especially when there is a high proportion of heavy vehicles). For bridges which, as determined by the Head, Structural Section, it is essential to minimise the frequency and duration of future lane closures, only Type A joints shall be specified.

Type C joints as per DSM 9.40.27 shall be specified for all bridges except those where only Type A joints are allowed. Expansion joints between sleeper slabs and approach slabs shall be Type C.

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13.2.10 Sliding Plate Joints

Sliding plate expansion joints (see Figure 13.2.4) and Structural Standard Drawings SS113-30 to SS113-35) shall be used for joints where the SLS design movement is greater than 75 mm and less than or equal to 120 mm. The sliding plate is designed to behave as propped cantilever. It is essential that the sliding plate be installed with the correct tolerance to the stop plate. During service, the joint is designed to function with the sliding plate either directly bearing on the sliding plate or up to a maximum of 3 mm above the sliding plate. At installation, the gap between the sliding plate and the stop plate should be no more than 1 mm.

Sliding plate expansion joints shall only be used at abutments of post-tensioned deck bridges and at abutments of girder type bridges. They shall not be used at pier locations. Sliding plate joints shall not be installed in bridges when the slope of the roadway at the joint exceeds 2% unless the bearings at that location are inclined to the same slope as the highway. When the deck is sloped and the bearings are installed level, translation due to expansion and contraction results in a change in the vertical gap between the sliding plate and the stop plate.

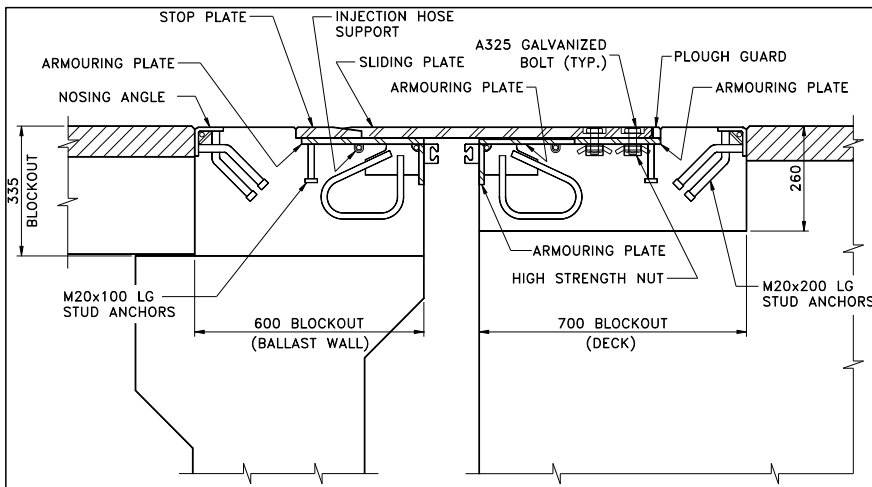


FIGURE 13.2.4 – SLIDING PLATE EXPANSION JOINT

13.2.11 Modular Joints

13.2.11.1 Modular Joint Use

Modular joints shall be used when total deck joint movements, after joint assembly installation and measured parallel to the centreline of the highway, are greater than 120 mm in one direction. With this type of expansion joint, wheels bear directly on the separation beams between the seals. Seals and separation beams form a watertight joint at the riding surface. Separation beams are supported by steel support bars each having

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its own support box at both ends. One end of the support bar is fixed in a short support box in the end dam, and the other end slides on engineered bearings inside a longer support box. Modular joints are classified by the number of longitudinal support bars and how they are connected to the transverse separation beams. In a multiple support bar system, each separation beam is supported on an individual support bar. The number of support bars therefore equals the number of separation beams and in a large movement joint, the number of support bars leads to a joint that is not easily accessed from below for maintenance. A single support bar system has all the separation beams supported on one large support beam, and the separation beams slide on bearings on the top face of the support beam. A single support bar system is therefore more accessible for maintenance. Due to past performance, the Ministry has shifted towards single support bar systems.

Modular joints shall not be field spliced.

In order to preserve the ride quality across a modular joint, the bearings supporting the girder end shall be installed at the same slope as the highway vertical profile, to allow the support bars to remain parallel to the highway at all range of movement.

Available Modular joints can have a minimum of 3 to a maximum of 8 seals. The maximum movement of any one seal, measured parallel to the centreline of the highway, must not exceed 60 mm.

The end dam over the ballast wall, the gap between the ballast wall and the deck end, and the end dam at the deck end shall be detailed to permit installation of any modular joint listed on DSM No. 9.40.20.

The absolute minimum distance between end dams, denoted **F**, is 250 mm at 15°C, to permit access to the joint components for maintenance. The distance between the ballast wall and the deck end, denoted **L1** shall be as specified in Table 13.2.2 to permit inspection and maintenance of the expansion joint from beneath. A corbel shall be introduced on the ballast wall side with length **D** to support the end dam. The corbel shall be detailed with a dimension **D** of 250 mm for 3 seals, 200 mm for 4 seals, and 150 mm for 5 or more seals. The corbel shall be designed and detailed to be increased by up to 150 mm to accommodate the specific joint supplier. Dimensions **F** and **D** are shown Figure 13.2.5.

For post-tensioned concrete bridges, **L1** should be calculated at 15°C for the required joint size and movements due to creep, shrinkage and temperature fall occurring after time of joint installation. In lieu of more accurate data, the designer may assume joint installation occurs at 90 days after stressing of prestressed concrete bridges.

The width of blockout **B1** and **B2** depend on the number of seals in the joint, while the depth of blockout **H** depends on the number of seals and on the supplier of the expansion joint. Dimensions **B1**, **B2** and **H** in Figure 13.2.6 shall be detailed according to Table 13.2.2.

The following note shall be added to the ABUTMENT drawing:

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"THE CONTRACTOR SHALL ADJUST CORBEL DIMENSION 'D' AND REINFORCING STEEL TO SUIT THE ACTUAL MODULAR EXPANSION JOINT FOR THE BRIDGE, UP TO A MAXIMUM OF 150 mm."

13.2.11.2 Modular Joint Selection

1. Calculate the Design Movement, due to the sum of Creep + Shrinkage + Temperature Range.
Creep and Shrinkage movements are from time of joint installation (e.g., t = 90 days) to joint replacement, assumed at a service life of 40 years.

Movements due to Temperature Range are from the sum of temperature fall to temperature rise.

2. Obtain the Serviceability Limit States design movement which is 80% of the above Design Movement.

If the SLS Design Movement < 120 mm, use strip seal (see Section 13.2.2 and Section 13.2.3 to Section 13.2.5) or sliding plate joints (see Section 13.2.6) as appropriate.

3. For Modular Joints, select the joint size (number of seals required) based on the SLS Design Movement as provided in DSM 9.40.20.
4. Detail the abutment ballast wall and deck ends to accommodate dimensions "B1", "B2" and "H".

TABLE 13.2.2 – MODULAR JOINT DETAILING TABLE

No. OF SEALS	2	3	4	5	6	7	8
B1 (mm)	590	675	760	835	915	995	1075
B2 (mm)	600	600	600	650	650	700	700
H (mm) MINIMUM	355	355	355	355	380	380	380
F @ 15°C (mm)	200	330	460	600	730	850	980
L1 @ 15°C (mm)	600	600	660	750	880	1000	1130

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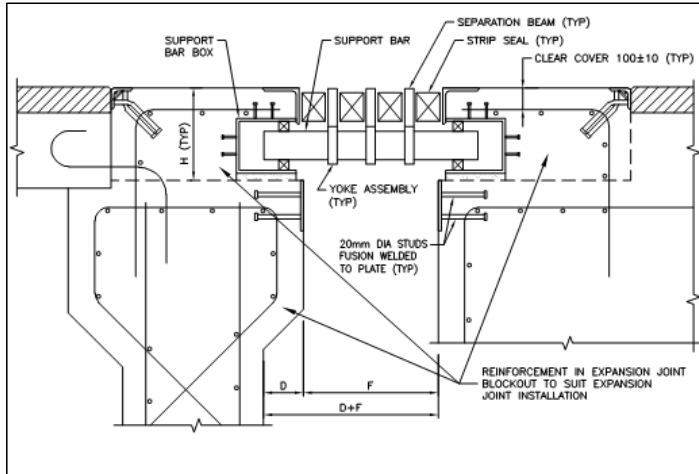


FIGURE 13.2.5 – MODULAR JOINT DETAIL – SECTION AT BOXES (SINGLE SUPPORT BAR SYSTEM)

13.2.11.3 Modular Joint Setting

Having selected an expansion joint whose Range of gap rating (max. gap minus min. gap) is greater than or equal to the SLS Design Movement, the "J" gap dimension must be calculated at the assumed 15° C construction temperature.

For prestressed concrete structures, "J" at 15°C equals $0.8 (CR + SH + T_{fall})$, where T_{fall} is the movement due to a temperature drop from 15°C to the Minimum Effective Temperature used in design, and CR and SH correspond to the anticipated creep and shrinkage of the bridge for 40 years after joint installation.

The maximum and minimum "J" dimensions shall be shown on the expansion joint drawing.

13.2.11.4 Abutment Width

Since large movements are expected when modular joints are used, the abutment should be proportioned based on the distances shown in Figure 13.2.6. The effect of skew and minimum seating requirements for earthquake should also be considered. L1 shall be a minimum of 600 mm for modular joints, to permit inspection and maintenance of the joint. For post-tensioned bridges, the minimum lengths of L1 and L2 shall be satisfied at 90 days after placing concrete.

L1 = Gap between ballast wall and end of deck

L2 = Distance between centreline of bearings and end of deck, as required to satisfy minimum seating length requirements,

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L_3 = Distance between centreline of bearings and face of abutment

t = Thickness of ballast wall (see Section 5.2.3.1 for minimum thickness)

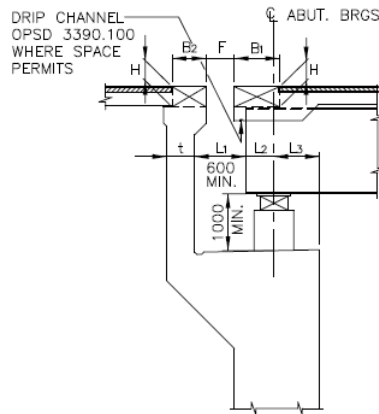


FIGURE 13.2.6 – MINIMUM WIDTH OF ABUTMENT

13.3 Bridge Bearings

13.3.1 Bearing Selection

MTO maintains standards for rotational bearings and for elastomeric bearings. The selection of the bearings should be based on the articulation requirements of the structure. The requirements of the bearings such as translations, rotations, vertical and lateral load capacities and any special requirements must be established during the design process. Design values provided in the tables on the drawings shall be the actual demands on the bearings, from the structure, and shall not include additional tolerances. Tolerances are added to the values specified in the design, as required by OPSS 1202 and 1203, which include the tolerances required by the CHBDC.

For reactions of up to 3000 kN at SLS1, the designer shall specify elastomeric bearings because of their long-term durability and cost-effectiveness, unless movement or rotation demands preclude their use.

For high loads and/or long movements, rotational bearings shall be selected. For the majority of bridges, all three types of rotational bearings will satisfy all the structural requirements. However, there will be some bridges where, for reasons such as rotational capacity or rotational stiffness, only one or two classes of bearings will be acceptable.

For bridges with more than one span, except for integral bridges where only plain elastomeric bearings are used and for prestressed girder bridges with elastomeric bearings, a bearing drawing shall be included in the design drawings for the bridge. The bearing drawing shall contain a plan layout of the bearing locations at every support, along

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with a symbol to indicate the bearing type and fixity. Figure 13.3.1 shows an example of a bearing layout, along with the standard symbols to be used for rotational and elastomeric bearings, and their associated restraints.

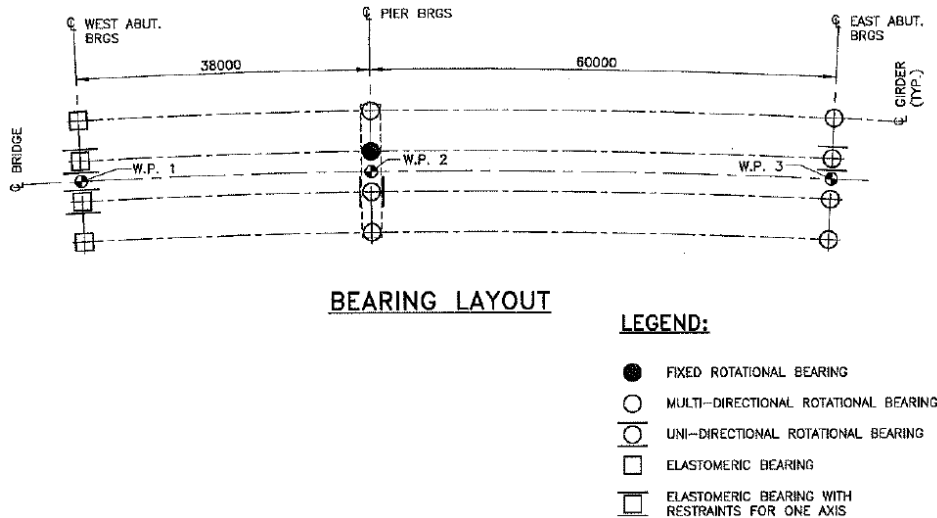


FIGURE 13.3.1 – BEARING LAYOUT AND LEGEND

Since the final bearing design is completed by the bearing supplier, the designer shall provide a schematic section on the bearing drawing to convey the main components of the bearing and the outer dimensions of the bearing assumed in the design. For elastomeric bearings, the height of the bearing shall be shown along with any details of positive attachment or plates above and below the bearing. For rotational bearings, a typical section as shown in Figure 13.3.5 (bearing section). For most cases, a section through the bearing transverse to the bridge profile is sufficient but a longitudinal section may be required for bearings with longer movements or highways with steeper grades.

13.3.2 Rotational Bridge Bearings

This Section provides a summary of the features of rotational bearings which could affect bearing selection and identify the bearing design data for rotational bearings that must be shown on the plans. They do not apply to plain or laminated elastomeric bearings.

The Designated Sources for Materials List DSM 9.15.70 for Bridge Rotational Bearings (except elastomeric bearings) will include three types of bearings divided into six classes:

- Class 1A – Pot Bearings Without Uplift Restraint Devices
- Class 2A – Disc Bearings Without Uplift Restraint Devices
- Class 3A – Spherical Bearings Without Uplift Restraint Devices

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These bearings may be equipped with sliding surfaces for translation and guides for lateral restraint.

Uplift restraint devices should be avoided and their use requires the approval of the Structures Office Manager. Uplift-restraint devices shall not restrict the function of a bearing and shall not limit the rotation capacity nor the displacement of the bearings under any load combination. OPSS 1203 and 922 do not include provisions for design of uplift-restraint bearings. When they are used, the contract shall include special provisions to cover their design.

Columns supporting expansion bearings made of lubricated, unfilled PTFE surfaces shall be proportioned based on a design coefficient of friction as given in the CHBDC.

13.3.2.1 Effect of Rotation on Eccentricity of Axial Load

When a bearing rotates about a horizontal axis there is a shift in the axial load from the centre of bearing and the load becomes eccentric. The shift in the axial load depends on the type of bearing.

For pot and disc bearings, the shift in the axial load from the centre of bearing depends on the properties of the elastomer and is difficult to calculate. Creep of elastomer has a beneficial effect in reducing the shift for disc bearings. Tests indicate that the shift can be 2% to 4% of the diameter of confined elastomer for pot bearings, and 2% to as much as 30% of the diameter of elastomer disc for disc bearings.

For spherical bearings, the shift "e", in the axial load, depends on the coefficient of friction " μ ", and the spherical radius of curvature, "R", and is expressed by:

$$e = \mu R$$

Given the values of coefficient of friction specified in the CHBDC, and that the spherical radius of curvature "R" is generally between 1.0 and 2.8 times the plan diameter of the PTFE curved surface, the shift in the axial load for spherical bearings can be 3% to 17% of the diameter of the PTFE curved surface. At Serviceability Limit State Combination 1 loads and maximum rotation, CHBDC limits the shift in the axial load from the centre of the bearing to the following values:

- a) 4% of the diameter of the confined elastomer for pot bearings;
- b) 10% of the diameter of the polyether urethane polymer compound for disc bearings;
- c) 10% of the plan diameter of the curved PTFE surface for spherical bearings.

The effects of the maximum permitted shift in the axial load must be considered in the design of the affected structural components above and below the bearing. Where it is difficult or costly to provide for the 10% shift, the design may be restricted to pot bearings.

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13.3.2.2 Translation and Rotation Capacity

Translational and rotational requirements about any horizontal axis and about the vertical axis through the centre of the bearing provided in the bearing design data table shall be as required by design.

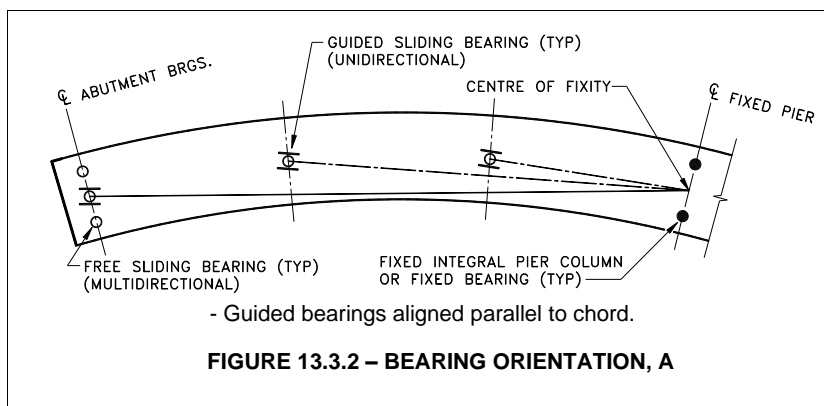
13.3.2.3 Orientation of Guided Bearings on Curved Structures

The designer should minimise the number of guided sliding bearings at each support (abutment or pier) location. One is usually adequate, and this should preferably be at the centre of the support. If multiple guided bearings are required at a support, the alignment should be the same for both and set to correspond to a point equidistant from each bearing.

In horizontally curved structures, two concepts generally exist when designing for horizontal movements. They are as follows:

- a) The structure is guided to a Centre of Fixity by aligning guided sliding bearings parallel to the chord drawn from the bearing to the theoretical point of fixity (see Figure 13.3.2). The Centre of Fixity is the point of zero movement of the superstructure for internally induced forces or movements due to temperature change, as well as creep and shrinkage if applicable. It is calculated by taking into consideration the combined stiffness of all supports including shear stiffness of bearings, flexural stiffness of supports (abutment or pier) and rotation of footings due to strain of piles or subsoil.

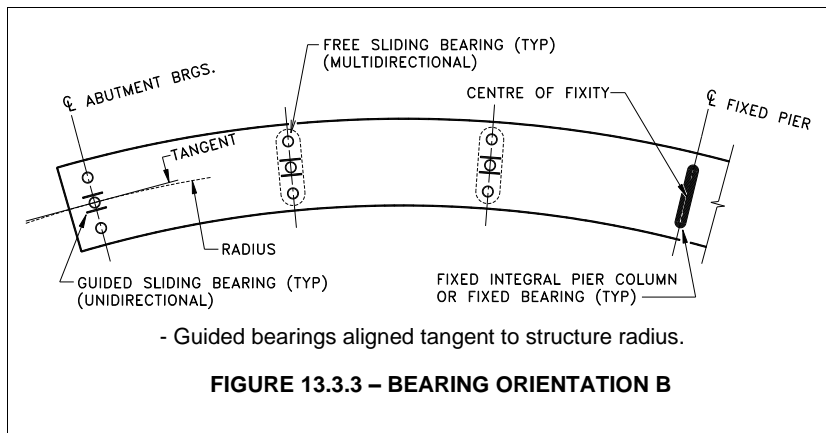
Bearing Orientation A tends to accommodate the direction of the superstructure movements without introducing horizontal forces to the substructure due to these movements. However, for sharp or compound curves, this can introduce distortion at the abutment expansion joint and misalignment of barrier walls. One way to avoid this is to place the expansion joint perpendicular to the centroidal chord, but this is not recommended since it will necessitate skewed abutments.



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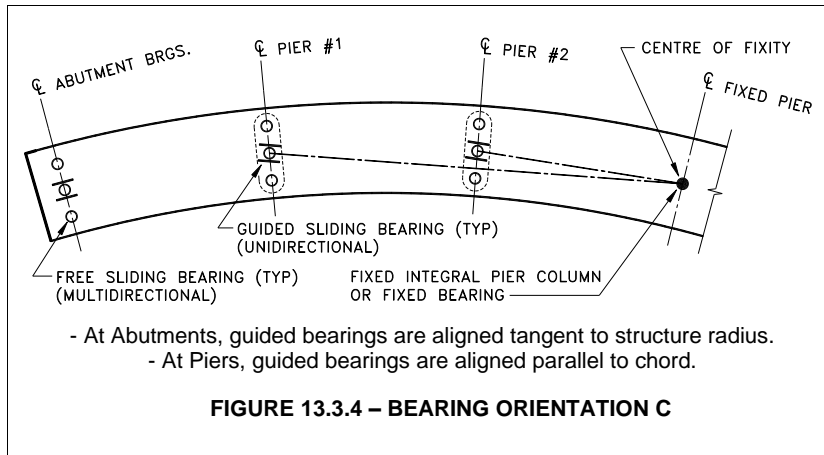
- b) The structure is guided tangentially by aligning guided sliding bearings tangent to the radius of the structure at the support (abutment or pier) location (see Figure 13.3.3).

Bearing orientation B relies on the guided bearings to force the horizontal movement in the direction of the longitudinal curved axis of the superstructure, resulting in horizontal forces being transferred to the substructure through these bearings.



Both methods of bearing orientation have been used successfully in the past, each with their attendant benefits and disadvantages. The choice of method to use is left to the individual designer. However, since it is always better to minimise the introduction of additional forces or deformations into a structural system, the guided bearings at piers are recommended to be aligned as per Method A, while the guided bearings at abutments are recommended to be aligned as per Method B (see Figure 13.3.4). As a result, because expansion joints at abutments are placed radially, they will expand and contract parallel to the abutment guided bearings without distortion and the abutments are generally better able to handle the lateral loads.

SECTION 13 - EXPANSION JOINTS AND BEARING ASSEMBLIES



Whichever method of bearing orientation is used in design, the designer is encouraged to use a refined method of analysis to determine the resulting horizontal movements and corresponding forces transferred to the substructure.

13.3.2.4 Lateral Load Capacity

Fixed and guided bearings shall be capable of resisting lateral loads in the restrained direction as required by the design and as given in the CHBDC. All lateral loads shall be resisted by a single bearing at a support line (pier or abutment line). However, for ULS-5 through ULS-8 (i.e., for Exceptional Loads), when more than one identical bearing with guides for lateral restraint are provided at a pier or abutment, one bearing shall be assumed to achieve its full lateral resistance while a second bearing shall be assumed to resist 50% of its full lateral capacity and any subsequent guided bearings shall not contribute to the lateral resistance. Other means of lateral restraint (i.e., a shear key) should be used to provide lateral resistance to the bridge superstructure when the factored lateral force exceeds 50% of the permanent reaction at the bearing.

13.3.2.5 Concrete Bearing Pressure

The average concrete bearing pressure used by the bearing manufacturer to proportion the bearing shall be based on a Specified 28-day Compressive Strength of 30 MPa.

The concrete bearing surfaces above and below the bearing should be proportioned and reinforced when necessary to withstand these pressures and lateral loads.

13.3.2.6 Installation

As the elevation at the top of bearings is critical especially for bolting diaphragms of steel box girders, the tolerances for the elevation at the top of bearings as specified in OPSS 922 are as follows:

SECTION 13 - EXPANSION JOINTS AND BEARING ASSEMBLIES

Concrete deck or girders	±5 mm
Steel plate girders	0 to +3.0 mm
Steel box girders	0 to 2.0 mm

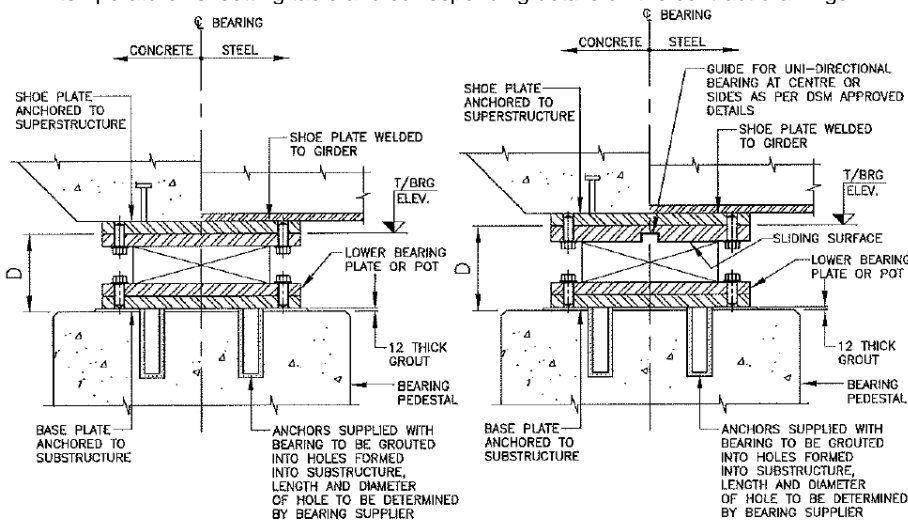
The Specification requires all bearings to be bedded over their entire area on an approved grout to achieve the theoretical elevations within these tolerances. The thickness of the bedding grout shall be 12 mm +/- 3 mm.

The height of bearing, 'D' (as shown in Figure 13.3.5) assumed in establishing the bearing seat elevation should include 12 mm for grout bedding and the drawings should include the following note:

"HEIGHTS OF BEARINGS INCLUDE 12 MM BEDDING GROUT AND EXCLUDE THE SHOE PLATE THICKNESS. HEIGHTS OF BEARINGS ASSUMED IN ESTABLISHING BEARING SEAT ELEVATIONS ARE AS FOLLOWS. THE CONTRACTOR SHALL ADJUST BEARING SEAT ELEVATIONS AND REINFORCING STEEL TO SUIT ACTUAL HEIGHTS OF BEARINGS.

ABUTMENTS ... mm
PIERS ... mm "

Precautions must be taken by the Designer, when there is the possibility of bearings being installed under extreme cold or hot temperatures and not at 15°C. Either the top plate of a sliding bearing should be made sufficiently large to accommodate all of the maximum expansion and contraction movements, or alternatively the Designer should specify a temperature vs. setting table and corresponding details on the contract drawings.



FIXED BEARING

SLIDING BEARING

SECTION 13 - EXPANSION JOINTS AND BEARING ASSEMBLIES

FIGURE 13.3.5 – BEARING SECTION

13.3.2.7 Bearing Design Data

The bearing design data provided on the plans shall be as indicated below. Applied horizontal loads shall be consistent with applied axial loads. Additional rows may be added if necessary for atypical bridges where one of many ULS combinations may govern. Any special requirements must also be specified.

ROTATIONAL BEARING DESIGN DATA

LOCATION	TYPE	LIMIT STATE	COMBINATION	AXIAL LOAD (kN)	MAX. HORIZ. LOAD (kN)		MAX. ROTATION (radians)		MAX. TRANSLATION (mm)	
					LONGIT	TRANS	ABOUT HOR. AXIS	ABOUT VERT. AXIS	LONGIT	TRANS
	[Fixed or uni-directional or multi-directional]	Serviceability	PERMANENT							
			PERMANENT + TRANSITORY MAX.							
			PERMANENT + TRANSITORY MIN.							
		Ultimate	PERMANENT							
			PERMANENT + TRANSITORY MAX.							
			PERMANENT + TRANSITORY MIN.							
			PERMANENT + EXCEPTIONAL MAX.							
			PERMANENT + EXCEPTIONAL MIN.							

The contract drawings shall include the following notes:

1. ROTATIONAL BEARINGS SHALL BE CLASS/CLASSES LISTED IN DSM 9.15.71, 9.15.75, and 9.15.80 UNDER THE HEADING "BEARINGS, BRIDGE (ROTATIONAL)".
2. THE CONTRACTOR SHALL ESTABLISH THE BEARING SIZE SUCH THAT CONTACT PRESSURE UNDER PERMANENT LOADS AT SLS IS NOT LESS THAN 25 MPa.
3. BEARING SUPPLIERS ARE REQUIRED TO PROVIDE ADDITIONAL ROTATIONAL CAPACITY OF 0.02 RADIANS ABOUT THE HORIZONTAL AXIS AND 0.02 RADIANS ABOUT THE VERTICAL AXIS IN ACCORDANCE WITH OPSS 1203 AND AS REQUIRED BY CHBDC.

SECTION 13 - EXPANSION JOINTS AND BEARING ASSEMBLIES

4. THE CONTRACTOR SHALL COORDINATE WITH THE BEARING SUPPLIER THE LOCATION OF THE BEARING ANCHORAGES IN THE PIERS AND/OR ABUTMENTS. HOLES TO RECEIVE ANCHORS SHALL BE FORMED INTO THE SUBSTRUCTURE. CORING IS NOT PERMITTED.

The additional rotations mentioned in note 3 shall not be included in the bearing design table.

For bearings subject to uplift, the maximum permitted separation of the bearing components must be specified.

The maximum required translation must be based on the assumption that, at the time of installation, the longitudinal and transverse centrelines of the bearing sliding plate will be set to coincide with the longitudinal and transverse centrelines of the bearing.

The coefficient of friction for PTFE sliding surfaces given in the CHBDC depends on whether the PTFE resin is "filled" or "unfilled". The type of PTFE resin assumed for design must be included with the bearing design data given on the contract drawings, specifically giving the following note: "UNFILLED PTFE, LUBRICATED DIMPLED SHEET HAS BEEN USED IN THE DESIGN FOR ESTABLISHING THE BEARING COEFFICIENT OF FRICTION." This produces the minimum coefficient of friction. If for some reason the designer wants to transfer a greater than minimum horizontal force to a particular sub-structure, other sliding surface treatments given in the CHBDC may be considered.

13.3.3 Elastomeric Bearings

13.3.3.1 General

Elastomeric bearings, plain and laminated, shall be natural rubber supplied in accordance with OPSS 1202. The following bearing sizes are recommended but other sizes may be available.

TABLE 13.3.1 – STANDARD LAMINATED ELASTOMERIC BEARING SIZES

Size (mm x mm)	Minimum Thickness (mm)	Target SLS1 Load (kN)
300 x 200	50	420
350 x 250	55	610
400 x 300	55	840
450 x 350	60	1100
500 x 400	65	1400
550 x 450	70	1730
600 x 500	80	2100
600 x 600	90	2520
600 x 700	90	2940
600 x 800	100	3360

SECTION 13 - EXPANSION JOINTS AND BEARING ASSEMBLIES

S6-19 limits the total shear strain on the bearing but does not limit the bearing pressure. OPSS 1202 further limits the compressive deformation of the bearing to approximately 0.07 of the effective elastomer thickness. Target SLS1 values correspond to a total pressure of 7.0 MPa. The target SLS1 load values included in the table have proven performance but may be exceeded provided the bearing is designed to satisfy S6-19 and OPSS 1202.

Higher aspect ratio (width to length ratio) bearings may be preferable when thinner bearings are required and rotation limitations are governing; they may be used when approval is obtained by the designer from the Head, Structural Section.

The rotational capacity of any bearing is a function of the effective elastomer thickness and is determined as per CHBDC.

13.3.3.2 Elastomeric Bearings Thickness

The thickness of plain elastomeric bearings specified on the contract drawings shall be between 15 mm and 25 mm. Plain elastomeric bearings shall be used only for temporary applications, for bedding of larger bearings, or in cases where they are encased in concrete.

Bearing thickness shall be chosen to limit shear strain due to shear displacement to 0.5.

The minimum thickness of laminated elastomeric bearings shall be according to Table 13.3.1. The minimum thicknesses in the table were established to accommodate a rotation of 0.002 rad due to transient loads in addition to the rotation tolerances in S6-19. As long as bearing is specified with a thickness equal to or greater than the value in the table, and loaded with at least 3 MPa under permanent loads, the bearing will meet the strain requirements (S6-19 clauses 11.6.6.3.2 to 11.6.6.3.4), lift-off requirements (Clause 11.6.6.8), positive attachment (11.6.6.6) for service temperature $T_s > -44^\circ\text{C}$ and elastomer-to-concrete interfaces, as well as deformation limits required by OPSS 1202.

For post-tensioned concrete structures, the use of a thicker elastomeric bearing may be a more economical choice than jacking the structure several months after stressing to release the shear deformation in the bearing.

Thicker bearings do not add significant cost. A taller bearing may be preferable to reduce the force transferred to the substructure, to avoid the need for positive attachment, or to accommodate larger rotations.

13.3.3.3 Strip Bearings for Precast Side-by-side Box Girders and Precast Slabs

Strip plain elastomeric bearings may be used for precast side-by-side box girders or precast slab bridges. The thickness of the strip bearing shall not exceed 25 mm.

13.3.3.4 Laminated Elastomeric Bearing with Slider Plate Assembly

For durability reasons, a slider plate shall not be used when a thicker bearing without a slider plate is feasible and shall not be used for a movement range of less than 50 mm.

SECTION 13 - EXPANSION JOINTS AND BEARING ASSEMBLIES

This type of bearing may be a reasonable choice for a long structure where the only other solution would be a rotational bearing (pot, disc, or spherical bearing). OPSS 922 does not cover the design provisions for laminated elastomeric bearings with sliding elements, and if used, details of the sliding surfaces should be provided in the Contract Drawings with appropriate NSSP's to cover their design and fabrication.

13.3.3.5 Supply of Additional Sample Bearings for Testing

To ensure the quality of the bearings supplied, standard special provision 922F01 requires that additional elastomeric bearings be supplied by the contractor for testing purposes.

The number of bearings requiring testing is based on the type of bearing, and the number of bearings of each size. The fill-in special provision for bearings provides guidance to the designer on the exact number of bearings to test. This special provision shall be included in the contract. Each structure requires at least one bearing of each size for testing. No samples for testing are required for bearings that are temporary and/or subsequently encased in concrete (e.g., bearings located at the integral supports of integral abutment type structures).

Additional bearings are not required to be supplied for all other approved bearings (i.e., disc, pot, spherical).

Twin structures shall be considered as separate structures.

13.3.3.6 Shear Rate of Elastomeric Bearings

When an elastomeric bearing undergoes shear deformation due to horizontal loading, a horizontal force develops which is transferred to the substructure. The magnitude of this force is a function of the shear stiffness (shear rate) of the bearing at the time of displacement and the amount of bearing deformation.

The amount of deformation (due to superstructure movement) depends on the combined effects of the following internally induced forces:

- a) Heat of hydration (of concrete);
- b) Elastic shortening (of concrete due to prestressing);
- c) Shrinkage (of concrete);
- d) Creep (of concrete under prestress and permanent loads);
- e) Temperature effects.

Note: a) to d) are applicable to concrete structures only, and e) is applicable to all structures.

The shear stiffness of any elastomeric bearing depends on the plan area and total thickness of the rubber, and the shear modulus of the elastomer. The shear stiffness is

SECTION 13 - EXPANSION JOINTS AND BEARING ASSEMBLIES

nearly constant for all temperatures above freezing but increases rapidly with decreasing temperatures due to crystallization of the rubber. At -40°C , the shear rate can be two times the stiffness at construction temperature.

In light of this, and since the various structure movements occur not only at its different ages (important when considering creep and shrinkage) but also at a variety of temperatures, the designer must use the bearing shear stiffnesses that correspond to each condition in order to design the substructure.

For bearing design data given on the contract drawings, the designer should give the shear stiffness rate that is assumed in the design of the substructure, at 20°C , denoted K_{20} . Bearing catalogues provide the shear rates for standard laminated elastomeric bearings sizes.

The force that is transferred to the substructure for which it must be designed shall be determined as follows.

For temperature drop, even though the bearing will experience a sustained shear deformation over an extended period of time during cold temperatures, the bearing stiffness starts at near K_{20} and increases at the lower temperatures after much of the deformation has already occurred. The arithmetic mean of the full range of K values should be used. For temperature drop, $K = 1.35 * K_{20}$ should be used.

For temperature rise, since such bridge movements occur at a temperature above 15°C , K_{20} should be used.

A) Shear Rate for Concrete Structures

Because shrinkage and creep occur throughout the year and gradually over many seasons, it is expected that the bearing will undergo cycles of cooling and warming. Any force that is induced during colder periods will be relieved to that calculated with K_{20} when the temperature rises. Therefore, the horizontal force due to creep and shrinkage is calculated assuming K_{20} .

For post-tensioned bridge, the superstructure may be jacked up (at approximately three months after construction) to relieve all horizontal shear deformation that has taken place in the bearing pad until that time, returning the bearing to its originally undeformed configuration. This relieves bearing movement due to heat of hydration, elastic shortening and a portion of the total shrinkage and creep. After jacking, the bearing returns to vertical at the assumed construction temperature and must be designed for the remaining portion of shrinkage and creep, and all displacement due to temperature.

13.3.3.7 Elastomeric Bearing Design Data

For elastomeric bearings, the minimum design data provided on the contract drawings shall be as indicated below. Any special requirements must also be specified.

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ELASTOMERIC BEARING DATA

LOCATION	ABUTMENTS	PIERS
TYPE	---	---
SIZE (mm)	__x__x__	__x__x__
NUMBER REQUIRED	---	---
SERVICEABILITY LIMIT STATES		
PERMANENT LOAD (kN)	---	---
TOTAL LOAD (kN)	---	---
MOVEMENT (mm)	± ____	± ____
ROTATION (radian)	---	---
SHEAR RATE at 20°C (kN/mm)	---	---

The rotation value in the table shall be the imposed rotation required by the design, plus the tolerances for rotations due to manufacturing and construction as required by the CHBDC but shall be not less than ± 0.009 radians. Rotation in the bearing data table corresponds to total load.

13.3.3.8 Installation of Elastomeric Bearings

Elastomeric bearings are to be installed directly on the bearing seats. No filler or grout is to be shown on the drawings.

Precautions must be taken by the designer when there is the possibility of bearings being installed below 0°C or above 20 °C. The translational capacity of the elastomeric bearing shall be greater than the anticipated maximum movement.

Positive attachment shall be detailed on the contract drawings where required by S6-19.

Laminated elastomeric bearings supporting steel bridges shall be detailed with two pintles between the shoe plate and the top lamina of the bearing. This prevents the structural steel from unseating from the bearing during construction. This is especially important for longer bridges where laminated elastomeric bearings cannot accommodate any movements through shear deformation until a greater axial load from the deck concrete is applied across the bearing to initiate frictional resistance to shear.

13.3.4 Provision for Future Bearing Replacement

Enough space, both vertically and horizontally, must be provided between the superstructure and substructure to accommodate the required jacks for replacing the bearings. The vertical clearance for jacks shall be no less than 200 mm. The designer shall detail the jacking locations in the contract drawings based on the MTO Bearing Replacement and Jacking Guidelines.

Connections (e.g., between bearings and shoe plates) must be bolted or make use of accessible screws and should only be welded when future access and removability can be guaranteed.

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Requirements and guidance on bearing replacement procedures and bridge jacking methods are provided in MTO document (BRO-062) – Bearing Replacement and Jacking Guidelines.

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SECTION 14 - CULVERTS

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SECTION 14 - CULVERTS

14 CULVERTS**14.1 General**

This section shall apply to:

- a) all culverts (buried structures) with spans 3m and greater; and,
- b) all cast-in-place culverts detailed in the MTO Concrete Culvert Design and Detailing Manual.
- c) All culvert sizes specified in OPSS 1821,
- d) all culverts contained in the future MTO Precast Concrete Box Culvert Manual.

On contract and working drawings the Culvert shall be identified as SPAN X RISE.

14.2 Concrete Culverts**14.2.1 Cast in Place Concrete Culverts**

All cast in place concrete culverts should be designed and detailed in accordance with the latest version of the Concrete Culvert Design and Detailing Manual. In this manual the designs meet the requirements of the CHBDC and contains information necessary to complete the standard drawings and quantities for concrete box and open footing culverts. The following culvert types are covered:

- a) Rigid frame open footing culverts with fill heights of 0.6 m to 6.0 m and sizes 2.0 m span x 1.25 m height to 6.0 m span x 4.0 m height;
- b) Rigid frame box culverts with fill heights of 0.6 m to 6.0 m and sizes 2.0 m span x 1.25 m height to 6.0 m span x 4.0 m height; and,
- c) Non-rigid frame box culverts with fill heights of 0.6 m to 5.0 m and sizes 1.25 m span x 1.25 m height to 1.5 m span x 1.5 m height.

Reduced scale copies of the standard drawings for all of the above culvert types are available in the "Concrete Culvert Design and Detailing Manual."

14.2.2 Precast Concrete Culverts

Currently in MTO's inventory, two categories of precast concrete culverts exist:

1. Culverts spanning 3m or less, which are fabricated and built according to OPSS 1821 "Material Specifications for Precast Reinforced Box Culverts and Box Sewers", and typically use Dry-Cast concrete. These culverts are generally used in drainage and sewerage applications. The design of these culverts, in specific span and height combinations to achieve maximum efficiency, is specified in the design tables which are included in OPSS 1821.

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The culverts of this category are reinforced with welded wire fabric (WWF). The material requirements are contained in OPSS 1340 for Concrete and OPSS 1440 for WWF steel reinforcement. The construction specifications of these culverts are covered in OPSS 422.

2. Culverts spanning greater than 3m are typically designed for larger scale application for example high volume of stream flow or to act as a small bridge for multi-use (e.g., pedestrian, bicycle) traffic. Current practice is that the designer is required to perform project specific analysis and design of the large culverts. These culverts are to be reinforced either with black steel reinforcement or with WWF. Construction and material requirements are specified in OPSS 912.

MTO has recently published construction specification OPSS 912 for Culverts with Spans MTO has also issued design tables for this category of culverts in Memo # SCB-SO-2021-01 which will enable designers to use the design information directly from the tables on the contract drawings. Precast Concrete Box Culvert Manual containing design guidelines will be published in the near future.

Irrespective of the categories, all precast box culverts shall be designed in accordance with the CHBDC.

14.2.3 Waterproofing of Culverts

Concrete culverts with spans 3.0 m and greater shall be waterproofed irrespective of fill height. Hot applied rubberized asphalt waterproofing system is the standard treatment for the top surface of culverts with spans 3.0 m or greater. For precast concrete culverts, self-adhering waterproofing membrane shall be applied at the joints prior to application of the hot applied rubberized asphalt waterproofing on the top surface of the culvert. General waterproofing and joint waterproofing shall extend a distance down from the top as specified in SSP 599S30. Waterproofing need not be placed on areas beyond the limits of embankment fill or under a distribution slab, as specified in the SSP. Protection board is also added to protect the waterproofing from damage from backfill.

Waterproofing shall be specified for concrete culverts with a span 3.0 m or less for following situations:

- a) where a 75-year service life is required;
- b) where additional durability is required;
- c) under high volume roads where culvert rehabilitation or replacement would be disruptive; or,
- d) which are expected to be used by pedestrians or snowmobilers during the winter (where icicles can pose a danger to the public).

Hot-applied rubberized asphalt waterproofing system is used by MTO for top surface waterproofing of all types of culverts. However, the self-adhering waterproofing membrane may also be used in special situations, such as in remote areas where labour

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and equipment to install hot- rubberized asphalt waterproofing membrane system becomes unavailable or deemed uneconomical. Use of self-adhering waterproofing system shall require the approval from the Structural Section in advance, prior to the start of waterproofing operation on site.

14.2.4 Earth Pressure

Culverts and other structures where the deflection of the side walls is prevented by the propping action of slabs shall be designed for the earth pressures specified in CHBDC.

14.2.5 Design of Concrete Culverts

Beneficial effects of compression in reinforced concrete culvert components should not be taken into account in the design.

Cast-in-place culverts and precast culverts with fill height less than 600 mm shall require reinforced concrete distribution slab above the culvert for distribution of live loads. The distribution slab shall be designed as per the CHBDC.

Design of culvert appurtenances such as headwalls, retaining walls or wingwalls are to be designed as per CHBDC. In the design, consideration should be given if the culvert appurtenances are cast-in-place or precast. For precast elements, design should consider ease of site installation of the element on the precast culvert with integral connection. Culvert cut-off wall or apron wall is considered a non-structural component.

14.3 Metal Culverts**14.3.1 Corrugated Metal Culverts**

Corrugated Steel Culvert (CSP) and Structural Plate Corrugated Steel Pipe (SPCSP) terms are often used interchangeably. When used separately, the former is used typically for the smaller sized made of a continuous helical corrugated plate, while the latter is used for typically larger diameters made up of several plates around the circumference. Steel is the most common material, although aluminum is also available, typically for more severe environments.

Structural Plate Corrugated Steel [Pipe Culverts \(SPCSP\)](#) are fabricated by using hot-rolled sheets or plates that are corrugated, curved to radius, custom hot-dip galvanized or has a thermoplastic copolymer coating, assembled, and bolted together to form steel culverts, pipes, arches, pipe-arches, and other shapes.

Environmental factors shall be considered while checking the suitability of site for using [SPCSP](#). The thickness of [SPCSP](#) and other steel components shall be determined according to CHBDC to ensure the culvert is structurally sound until the end of the design life. The design shall account for environmental conditions that exist at the site or are likely to exist during the design life of the structure, and the anticipated steel material loss during the design life. The MTO [Gravity Pipe Design Guidelines](#) and the CHBDC shall be referenced for guidance. Notwithstanding the requirements above, the thickness of the [SPCSP](#) shall not be less than 5 mm.

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All metal culverts irrespective of their span length must have adequate site preparation and compacted backfill material around them.

The use of pipe arches is restricted to sites at which round pipes cannot be accommodated. Other alternatives should also be considered. It is necessary that the installation of pipe arches make provision for the necessary backfill volume of engineered soils (granular 'A') on each side of the structure and that each installation have cut off walls or headwalls on each end.

MTO is currently working to create standards for metal culverts material and construction specifications. Until these standards are published designer must create non-standard specifications and seek for Structural Office's endorsement. For NSP preparation, MTO's Special Provision No. DBSP3271 could be referred to.

14.3.2 Corrugated Steel Pipe (CSP) Products Dimensioning

All dimensions of corrugated steel pipe products must be given in metric units, which are generally conversions of dimensions set in imperial.

14.3.3 Length of Structural Plate Pipes and Pipe Arches

The overall length of structural plate pipe and pipe arches shall always be detailed on the drawings and the dimension shown shall always be multiples of 610 mm, +100 mm.

e.g., 61,100 mm is acceptable.

This permits the use of an integral number of standard plate sizes, without cutting, and ensures that the hook bolts in the cut off walls will be aligned with the standard circumferential holes in the plates.

The spacing of hook bolts, if required, should be a multiple of 244 mm, to match the spacing of standard holes in the plates.

14.3.4 Durability

Joints, including seams, shall be constructed and treated to prevent leakage and infiltration. To prevent leakage through the culvert and associated passage of chloride contaminated water leading to corrosion, the culvert shall be covered by a waterproofing membrane. The membrane has a minimum thickness of 0.5 mm and is made of PVC, HDPE, or some other polymer materials. It is draped over the culvert to take water away from the culvert, with granular backfill between the liner and culvert to prevent punctures from bolts or other edges.

Structural Plate Corrugated Steel Pipe (SPCSP) and Corrugated Steel Pipe (Culverts) made of galvanized steel have shown signs of early corrosion in many environments in Ontario. Polymer-coated pipe and metallic coated bolts have been used in newer culverts. Structural Section and Structures Office shall be contacted to assist in determining the suitability of the metal pipe for the given soil and water conditions, as well as for requirements for the pipe system.

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SECTION 15 - WOOD STRUCTURES

15 WOOD STRUCTURES

15.1 Material Properties of Softwood

Properties of individual species are available, but for the purposes of design, some similar species and varieties of sub-species are grouped together such as Southern Yellow Pine (*Pinus spp.*), or the species combinations of CSA O86.

Hardwood is not commonly used for structural lumber but is very similar to softwood.

15.1.1 Structure of a Tree

Wood is an anisotropic, cellulosic, semicrystalline, cellular material. The material complexity may be simplified by understanding that wood is a biological structure that serves three functions in living plants:

1. To conduct water and nutrients from the roots to the leaves;
2. Mechanical support of the plant body;
3. Storage of biochemicals.

These functions have resulted in cells that are structured and interconnected in ways to meet the needs of the plant, which has implications on the properties and design of wood. The trunk is the useable portion of a tree for wood used in structures which may be divided generally into heartwood and sapwood (Figure 15.1.1).

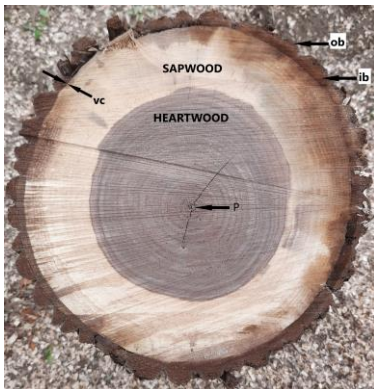


FIGURE 15.1.1 – MACROSTRUCTURE OF TRUNK (BOLE)

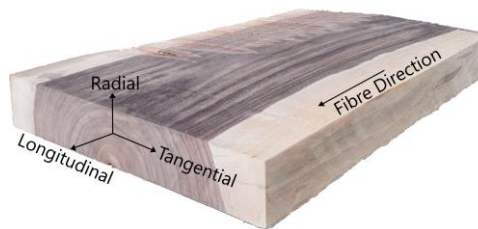


FIGURE 15.1.2 – PRINCIPAL AXES OF WOOD

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Sapwood is the active conduction portion of the stem with live, metabolically active cells. The pores are more open and there is a higher moisture content. The sapwood is more prone to shrinkage and cracking, and is more susceptible to decay, but is easier to treat with preservatives.

The heartwood functions as long-term storage of biochemicals (extractives) which occludes pores and makes it more difficult to treat. The cells are dead, but it is more naturally decay resistant than sapwood and is denser, dryer, and stronger than sapwood (acts as the central, supporting pillar of the tree).

A review of the microstructure of the wood shows that discrete cells, typically many times longer than they are wide, are interconnected in two separate systems of cells, i.e., an axial system and a radial system. The axial system acts primarily as long-distance water movement, so it has the most free water in the cell lumina and provides the bulk of mechanical strength to the tree. The radial system provides lateral transport for biochemicals and in many cases performs a large fraction of the storage function in wood. More detail on the structure and properties of wood may be found elsewhere, such as Forest Products Laboratory, *Wood Handbook, Wood as an Engineering Material* (FPL-GTR-282).

The cell structure directly affects the properties of wood as a building material. As such, the mechanical properties (elastic, strength, and rheologic) exhibit strong orientation effects (axial and radial system) and are complicated by the addition of growth irregularities (e.g., branches are seen as knots in sawn lumber). Mechanically, clear wood obeys the laws of elastic orthotropic materials, and its failure characteristics are well described by strain energy of distortion-type theory.

Because cell structure is predictable, wood may generally be described as orthotropic. The material properties vary according to the longitudinal, radial, and tangential axes with respect to fibre direction (

Figure 15.1.2).

15.1.2 Strength Properties

Common mechanical properties are measured and represented as strength properties for species grades in CSA O86 and are represented as parallel, at an angle to, or perpendicular to fibre direction (grain).

The reported properties reflect test mode and the gradation, shape, and size of test specimen. Remanufacture (ripping, resawing or dressing) of graded dimension lumber and timber should be avoided. Remanufacture can affect the rated properties of the member, voids the NLGA grade stamp and contractors are generally not in a position to re-grade the pieces. Specifically desired properties or shapes may be fabricated with engineered wood products.

15.1.3 Moisture

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Wood exchanges moisture with the surrounding environment. The water content depends on the relative humidity and temperature of air and the current amount of water in the wood.

Shrinkage occurs when the water content drops below the fibre saturation point of the wood species. Average values for shrinkage vary for different wood species when drying from fibre saturation to service dry conditions, but there may also be a different percentage of moisture movement in each fibre direction for the same species (longitudinal direction of the stem / axial), radial direction (across annular rings) and tangential direction (along annular rings).

The ratio of tangential to radial shrinkage generally represents the uniformity of shrinkage of a species. The volumetric shrinkage indicates the magnitude of shrinkage. Longitudinal shrinkage is effectively zero. Shrinkage is roughly twice as fast in the end grain as the side grain.

At equilibrium moisture content, the wood is neither gaining nor losing moisture (or swelling or shrinking). Service dry wood (19%) is about the typical moisture content expected of in-service open decks (16-17%) and should shrink in a linear relationship to moisture content. Higher moisture contents exhibit a non-linear relationship.

To accommodate movement, connections need to consider orientation of wood with respect to adjoining wood, and the in-service moisture content expected at the site.

15.2 Design

Under suitable conditions, wood will give centuries of service. Unsuitable conditions include abiotic and biotic factors and result in degradation. Wood must be protected during processing, merchandising and in use.

In an unprotected environment, wood is susceptible to nonliving or physical agents such as heat, abrasion, ultraviolet light, and strong chemicals, which generally act slowly to decrease wood strength. However, the greatest hazard to timber bridges results from living or biotic agents such as decay fungi, bacteria, insects, and marine borers. Most biotic agents that enter and decay wood require four basic conditions for survival:

1. Free oxygen;
2. Suitable temperature (generally a range of 10-30 degrees Celsius. Decay effectively stops at less than 2 and greater than 38 degrees);
3. Sufficient water (a moisture level in the wood above the fibre saturation point which varies by wood species. On average, fibre saturation may be considered 25-30% and 28% for Canadian softwood species); and,
4. A food source.

Elimination of any one condition above can prevent wood decay, however oxygen supply and temperature cannot reasonably be controlled in the environment of a bridge structure.

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The food source may be poisoned with wood preservative treatments. Preservative treatment is the addition of a registered product to the wood and treatments come in two primary varieties based upon their composition. Preservative treatments can be applied through pressure-treatment, injection and surface coatings and affects a relatively shallow depth of wood members. Since preservative treatment could fail during the life of a structure, it should be combined with the elimination of as much water exposure as is practical.

Durability and longevity of outdoor treated wood articles is dramatically impacted by exposure of bright untreated wood by machining post treatment. Durability of wood is dependent upon limiting exposure of bright untreated wood to the elements. A durable wood design uses treated wood, deflects water from components, provides good drainage, and facilitates member drying.

15.2.1 Species

The availability of wood species which may be used in design is practically limited by what is available to post-treat wood after fabrication.

15.2.1.1 Permitted Species and Species Combinations for Sawn Wood

Only individual species and species combinations specified in CHBDC Table 9.15 are permitted for sawn wood.

TABLE 15.2.1 – TREATMENT USE CATEGORY FOR WOOD SPECIES

Species Combination	Treatable Species ¹	Treatment ²	
		UC 4.1	UC 4.2
Douglas Fir-Larch	Douglas Fir ^{3,4}	ACQ, ACZA, CA-B, CCA, CR, CR-PS, CR-S, PCP-C	ACZA, CCA, CR, CR-S, PCP-C
	Western Larch ³	ACZA, CCA, CR, CR-PS, CR-S, PCP-C	CR, CR-S, PCP-C
Hem-Fir	Western Hemlock ³	ACQ, ACZA, CA-B, CCA, CR, CR-PS, CR-S, MCA, MCQ, PCP-C	ACQ-C, ACQ-D, ACZA, CA-B, CCA, CR, CR-S, MCA, PCP-C
	Amabilis Fir ³		
Spruce-Pine-Fir (SPF)	Lodgepole Pine	CR, CR-PS, CR-S, PCP-C	No
	Jack Pine		
	White Spruce		
	Engelmann Spruce		
	Black Spruce	CR, CR-PS, CR-S, PCP-C	No

1 Bold species are specifically identified in CSA S6-19, Table 9.15 as treatable species for sawn wood. Western Hemlock and Lodgepole Pine may be treated with some difficulty.

2 CSA O80 Series Sawn Product treatment options for the Use Category specified in OPSS 1601.

3 Coastal species are identified separately under NLGA Grading Rules but are not a separate species combination for strength properties according to CSA O86. The differences between interior and coastal species affect the ability to treat wood with preservative.

4 Preferred wood species for structural characteristics but interior species are difficult to treat. Consider interior not treatable and only specify coastal Douglas Fir.

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	Red Spruce		No
	Alpine Fir	ACQ-C, ACQ-D, CA-B, CR, CR-PS, CR-S, PCP-C	No
	Balsam Fir	CR, CR-PS, CR-S, PCP-C	No
Northern Species	Ponderosa Pine	ACQ, ACZA, CA-B, CCA, CR, CR-PS, CR-S, MCA, MCQ, PCP-C	ACQ-C, ACQ-D, ACZA, CA-B, CCA, CR, CR-S, MCA, PCP-C
	Red Pine		
	Western Red Cedar	No	No
	White Cedar	No	No
	Yellow Cedar	No	No
	Grand Fir	ACQ, ACZA, CA-B, CCA, CR, CR-PS, CR-S, MCA, MCQ, PCP-C	ACQ-C, ACQ-D, ACZA, CA-B, CCA, CR, CR-S, MCA, PCP-C
	Eastern Hemlock		
	Eastern White Pine		
	Western White Pine	ACQ, CA-B, CR, CR-PS, CR-S, MCA, MCQ, PCP-C	ACQ-C, ACQ-D, ACZA, CA-B, CCA, MCA
	Western White Spruce	ACQ-C, ACQ-D, CA-B, CR, CR-PS, CR-S, PCP-C	No

15.2.1.2 Permitted Species and Species Combinations for Glulam

Glulam is primarily produced in Canada as D.Fir-L or Spruce-Pine. Hem-Fir GLT may be available in certain areas and local fabricators should be consulted for product availability.

The post-gluing treatment options with oil-based preservatives limits availability of GLT products to Coastal Douglas Fir and Western Hemlock (Section 15.2.3.1 and Table 15.2.2).

15.2.2 Dimensions

Structural members can generally be fabricated to any size or shape. However, treatment with preservatives practically limits members to that which can be handled and will fit in the pressure treatment vessel, which varies by treatment plant. Weight of the treated member may be a limiting factor for some treatment plants.

To be generally treatable, individual members should be:

- Less than 12m in length;
- Up to 1m in depth;
- Less than 8000 kg.

The designer shall contact treatment plants for treatment ability for all curved shapes or when the dimensions or weight above are exceeded. (Note: the largest known member dimensions which may be treated in Ontario are 15m length and ~1.5m depth).

15.2.2.1 Sawn Timber

There are many different nominal and actual dimensions of sawn timber depending on the machining processes and moisture conditions. To avoid any ambiguity, all contract drawing dimensions shall be actual dimensions, in millimetres (e.g., 191x191), and standard sizes for sawn lumber or sawn timber.

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Sawn timber shall not be specified as dressed.

Sawn timber is generally available up to 4.85 m length in sections up to 241 x 241 mm. For any CSA O86 sizes above these dimensions, the timber may be a special order and the designer should first contact a mill or consider glulam. Custom sizes not listed in CSA O86 should not be used.

15.2.2.2 Glulam

Beams are typically fabricated in standard finished widths of 80, 130, 175, 215, 265, 315 and 365 mm, corresponding with CSA O86 selection tables. Additional widths up to 515 mm are available according to CSA O122 but members wider than 365 mm are by special order.

Glulam member depth shall be specified in 38 mm increments. Glulam member depth may be specified in 19 mm increments for curved elements when permitted in CSA O86.

15.2.3 Pressure Preservative Treatment for Wood

All wood shall be pressure treated with preservatives according to CSA O80 Series and the accompanying American Wood Protection Association (AWPA) preservation standards in OPSS 1601.

Preservatives in CSA O80 are current at the time of publication and are subject to the product label. The AWPA specification U1 is updated annually and may have more up-to-date preservative treatments available than CSA O80. However, the production and use of wood preservatives in Canada is regulated by the Pest Management Regulatory Agency (PMRA) under the Pest Control Products Act (PCPA). Specific preservatives listed in CSA O80 or AWPA U1 may be restricted or may not be available during design. The designer shall check with preservative treatment suppliers about product availability before specifying treatments in the contract documents.

Inorganic Boron (SBX), also known as Disodium Octaborate Tetrahydrate (DOT) requires dual treatment where the SBX treated product is treated separately with an oil-borne preservative (typically Creosote). The oil-borne treatment is required to seal in the SBX which depletes and vapourizes when the moisture content exceeds about 22%.

Elements shall be specified in the drawing notes under the use category system. Each element shall identify the wood species and size, use category, and type of preservative (see Section 15.4).

15.2.3.1 Preservative Treatment of Glulam

Glulam shall be treated with an oil-based preservative (Table 15.2.2). The glulam laminations are individually kiln-dried and are effectively sealed during gluing to a service-dry moisture content. Adequately protected glulam can generally be considered to be in service-dry condition. Incising and treatment with water-born preservatives increases the moisture content of the product to a wet condition and may distort element dimensions and any machined details, impacting fit-up.

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TABLE 15.2.2 – GLT POST-GLUING PRESERVATIVE TREATMENTS

Treatable Species	Stamp Identification	Treatment	
		UC 4.1	UC 4.2
Coastal Douglas Fir	D FIR(N)	CR, CR-PS, PCP-A, PCP-C	CR, CR-PS, PCP-A, PCP-C
Western Hemlock	W Hem(N)		

There is currently an issue with procuring oil-based preservative treated glulam:

- The sole North American supplier of Pentachlorophenol (PCP) shut down production in December 2021 and the PRMA product registration has ended, effectively banning the use of PCP to treat any new products.
- There are three Creosote preservative treatment plants in Canada, limiting availability. Wood may also be creosote treated in the USA.
- Creosote Petroleum Solution is not available in Canada. Wood may be treated in the USA.
- Copper Naphthenate (CuN) in hydrocarbon solvent is currently specified to UC 4.1 only. The CuN supplier has submitted information to the CSA O80 technical committee for review and may become a future alternative.
- 4,5-Dichloro-2n-Octyl-4-Isothiazolin-3-One (DCOI) is under review by PMRA as a potential future alternative to PCP.

15.2.3.2 Component Treatment Use Category Designation

Components shall be specified to the Use Category in Table 15.2.3.

TABLE 15.2.3 – WOOD COMPONENT USE CATEGORY

Component	CSA O80 Use Category
Foundation elements	UC-4.2
Structure elements (except barrier)	UC-4.2
Structure barrier	UC-4.1
Guide Rail Posts	UC-4.1
Guide Rail Blocks	UC-4.1
Ground Mounted Sign Posts	UC-4.1
Fence Posts	UC-4.1

15.2.3.3 Fire Retardant Treated Wood

Pressure preservative treated processes are available in CSA O80 Series for fire-retardant-treated lumber, timber, and plywood. It is not expected that fire retardance of wood through chemical treatment will be required because:

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- The occurrence of fires near or under bridges is relatively rare; and,
- Bridge members are generally large timbers that will have an inherent fire resistance because of the slow burning rate of large timbers and the insulating effects of the char layer.

Information about calculating fire resistance of wood members and assemblies may be found in American Wood Council Technical Report No. 10.

15.2.4 Decks

Decks on wood girders shall be glulam unless approved by the Head of Structural Section.

Wood-concrete composite decks shall not be used unless approved by the Head of Structural Section. MTO performance experience with wood-concrete decks has varied but poorly performing ones all experience rot at the wood/concrete interface and can only be 'repaired' by deck replacement. The detail in Figure 9.4 of CSA S6-19 shall not be used; it contains angled shear key reinforcing nails installed top down at the wood/concrete interface facilitating water penetration deeper into the wood laminates.

15.2.5 Traffic Barrier and Railing

TL-4 barrier crash tested to NCHRP Report 350 may be designed. References for design of this timber TL-4 barrier are:

Faller, R. K., Ritter, M. A., Rosson, B. T., Fowler, M. D., & Duwadi, S. R. (2000, April 3-5). Two Test Level 4 Bridge Railing and Transition Systems for Transverse Timber Deck Bridges. Transportation Research Record, 1(1696), 19.

Duren, J. T., Yosef, T. Y., Rosenbaugh, S. K., Faller, R. K., Bielenberg, R. W., & Steelman, J. S. (2023). Crash-Tested Bridge Railings and Transitions for Wood Bridges – Phase IIA. Forest Products Laboratory. U.S. Department of Agriculture.

15.3 Fabrication

All structural components shall be designed to be prefabricated and post-treated with preservatives. Machining and fastening post-treatment into bright wood is a significant contributor to deterioration of wood bridges and reduces service life below a 75-year target.

15.3.1 Shop Trial Assembly

The designer shall specify in the contract documents, girders and other main components requiring shop trial assemblies before and, if required, after preservative treatment.

A typical single span simply supported girder bridge may not require a shop trial assembly of fabricated elements prior to preservative treatment. However, wood bridges are not regularly constructed on MTO highways so pre-assembly should be considered for all

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glulam and dowel laminated (DLT) structures until there is general familiarity with them in design and construction.

Oil-based preservative treatments are more dimensionally stable and will typically not need a second trial assembly after preservative treatment.

15.3.2 Details**15.3.2.1 Changes in Cross Section**

Always provide taper cuts at notches to reduce abrupt differential stiffness change. Taper cuts shall be slope cut not less than 1:12 to a maximum length of 300 mm or a maximum depth of $D/4$, where D is the depth of the member being cut. Taper cuts in glulam shall not be permitted in compression zones of the member.

Notches or abrupt changes in cross section introduce stress raisers which can promote cracking to bright wood. Notches and abrupt changes in cross section shall not be permitted unless a detailed assessment of the stress concentration effect has been made. Notches on the tension side shall also be verified for fracture shear resistance at the notch in accordance with CSA O86.

15.3.2.2 Accommodating Shrinkage and Expansion

Before fabrication of connector joints, members should be seasoned to a moisture content corresponding as nearly as practical to that which they will attain in service. The moisture content shall be indicated on the contract drawings.

15.3.2.3 Connections

The designer shall design all connections.

Connections of members and steel plates connecting members with their grain perpendicular to each other shall be provided with slotted holes.

Connections should consider shrinkage and swelling, notching effects, decay prevention and lateral restraint at supports. Examples of unacceptable connections that promote cracking, and preferred alternatives, are provided in CSA O86 and American Institute of Timber Construction (AITC) 104-2003, *Typical Construction Details*.

All holes should be pre-bored during fabrication so that they are effectively protected by post-treatment with preservatives. Through holes shall be horizontal. Fastener and hole information may be found in OPSS 907.

Vertical fasteners installed top down in structural elements or that penetrate or expose bright wood shall not be permitted. Figure 15.3.1 shows an example of bottom-up deck fastening that accommodates shrinkage. All through vertical fasteners should be avoided, but where used shall be in pre-machined, treated holes.

Flat washers shall be installed between the timber surface and the nuts of bolts, or between the timber surface and spring washers where spring washers are used.

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Gauge lines for horizontal nailing shall be provided.

Installation torque of lag screws shall be based on the specific fastener, diameter and depth, and wood species specific gravity.

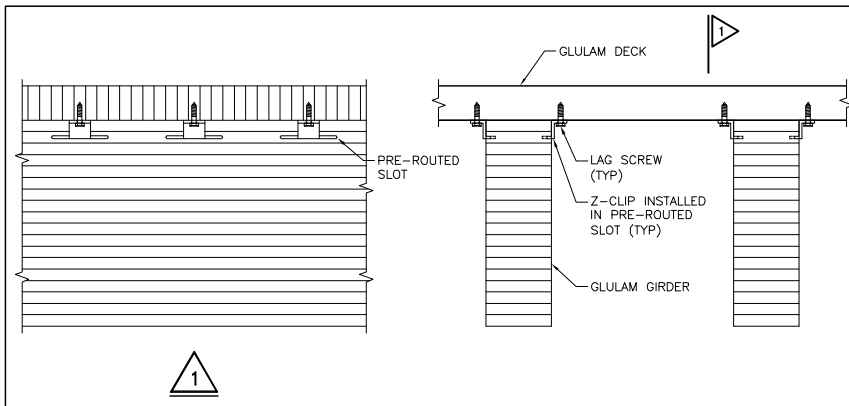


FIGURE 15.3.1 – GLULAM DECK FASTENING

Fasteners shall be the same material as the piece they are connecting and compatible with the preservative treatment used. E.g., copper nails with copper flashing; stainless steel nails with stainless steel flashing. Stainless steel type 304 or type 316 shall be used when fasteners are driven into wood treated with ACQ, CA, CCA or MCA.

Electrolytic corrosion is most often considered where dissimilar metals are in direct contact. However, the metals do not need to be in direct contact, they only need to be electrically connected. While wood is generally a good electrical insulator, under certain conditions wet wood may form an electrical connection between two dissimilar fasteners, leading to an anode/cathode relationship. If the fasteners are embedded, acidic and basic chemicals may accumulate at the node and cathode respectively. These chemicals may degrade the wood surrounding the fasteners. Fasteners should be the same material as embedded materials nearby.

15.3.3 Appearance

Appearance grade of glulam shall be industrial. Commercial grade glulam may be considered for elements within 5 m of a pedestrian's view and shall be clearly identified on the contract drawings when used.

Note: CWC appearance grade designations are different from APA classifications. If suppliers from the United States are anticipated because of species/treatment/size requirements, the appearance grade should be industrial or framing. Architectural grade may be considered for elements exposed to view.

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15.3.4 Coatings

Wood with a minimum nominal dimension greater than 51 mm shall not be painted. Coatings shall not contain more than 29% solids. Stains are preferred to paint.

Coatings with greater than 29% solids have been historically used to protect rail elements with added benefit of visual appeal. These high-solid paints tend to crack over time as the wood expands and contracts, allowing moisture to infiltrate the member. The moisture becomes trapped under the paint causing areas of high moisture content, and often causing the paint to delaminate from the member. Thus, high solids coatings can often promote rather than prevent decay.

15.4 Wood Notes

The following are standard notes of the type shown below the title block on the appropriate drawing. The notes must be worded to cover the requirements specific to the particular project and should only be used if applicable. Other notes may be required in special circumstances.

For clarity the wording of the notes is shown in upper case (CAPITAL) lettering. Explanations shown in brackets in lower case lettering are not part of the notes.

A. General Notes

1. ALL WOOD SHALL CONFORM TO CSA O86.
2. ALL SAWN LUMBER AND TIMBER SHALL CONFORM TO CSA O141. SAWN LUMBER AND TIMBER SHALL CONFORM TO THE NLGA "STANDARD GRADING RULES FOR CANADIAN LUMBER".
3. If the bridge uses glued-laminated timber, the following note shall be added:

ALL GLUED-LAMINATED TIMBER (GLT OR GLULAM) SHALL CONFORM TO CSA O122. GLUED-LAMINATED TIMBERS SHALL BE INDUSTRIAL APPEARANCE GRADE UNLESS INDICATED OTHERWISE ON THE CONTRACT DRAWINGS.
4. If the bridge uses mechanically laminated timber, the following note shall be added:

ALL MECHANICALLY LAMINATED TIMBER (MLT) SHALL BE DOWEL LAMINATED TIMBER (DLT) AND SHALL CONFORM TO CSA O125.
5. MOISTURE CONTENT (BY WEIGHT) OF LUMBER AND TIMBER SHALL NOT EXCEED 19% AND GLULAM SHALL NOT EXCEED 15% AT THE TIME OF INSTALLATION. ALL WOOD SHALL BE ADEQUATELY PROTECTED DURING ALL STAGES OF CONSTRUCTION TO ENSURE MOISTURE CONTENT REMAINS BELOW THE SPECIFIED LIMIT.

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(This note is about the maximum moisture content permissible of the wood product, this does not represent the expected moisture content of the site.)

6. NO CHAINS, HOOKS OR PEAVIES SHALL BE USED IN HANDLING TREATED WOOD.
7. FIELD DRILLING OR CUTTING NOT SHOWN IN THE CONTRACT DRAWINGS SHALL NOT BE PERMITTED. FIELD MODIFICATIONS SHALL NOT BE PERMITTED WITHOUT PRIOR APPROVAL FROM THE OWNER.

(Field drilling and cutting into bright wood is a significant contributor to deterioration of a wood structure. If field drilling and cutting cannot be avoided, notes 8 and 9 shall always be included with note 7.)

8. ALL FIELD-DRILLED HOLES SHALL BE TREATED TO REFUSAL/SATURATION WITH COPPER NAPHTHENATE PRESERVATIVE ACCORDING TO OPSS 1601. TREAT HOLES AS SOON AS POSSIBLE AFTER DRILLING. ENSURE HOLES ARE CLEAN AND FREE OF SAWDUST OR DEBRIS PRIOR TO TREATING.
9. FIELD-CUT SURFACES SHALL BE TREATED TO REFUSAL/SATURATION WITH COPPER NAPHTHENATE PRESERVATIVE ACCORDING TO OPSS 1601. END GRAIN SHALL BE COATED WITH ANCHORSEAL® PARAFFIN SEALANT. TREAT BEAMS AS SOON AS POSSIBLE AFTER CUTTING. ENSURE CUTS ARE CLEAN AND FREE OF SAWDUST OR DEBRIS PRIOR TO TREATING.
10. If the bridge uses prefabricated deck panels, the following note shall be added:

DECK PANELS MAY SHRINK OR EXPAND AFTER FABRICATION. DURING INSTALLATION, PERIODICALLY CHECK THE CUMULATIVE DISTANCE FROM THE END AND ADJUST GAP SIZE AS NEEDED. ORIENT ANY PANEL CURVATURE IN THE SAME DIRECTION TO MINIMIZE GAPS.

B. Connection Notes

1. ALL STEEL CONNECTION HARDWARE SHALL BE HOT DIP GALVANIZED ACCORDING TO ASTM A123. CONNECTOR PLATES SHALL BE FASTENED ACCORDING TO THE MANUFACTURER'S RECOMMENDATIONS TO DEVELOP THE FULL CAPACITY OF THE CONNECTOR.

(Fasteners used with hot-dip galvanized plates shall also be hot-dip galvanized. If connector plates are stainless steel, fasteners shall also be stainless steel and the note above shall be modified.)

2. NO CHECKING OR SPLITTING SHALL BE PERMITTED AT AREAS TO BE BOLTED OR LAGGED.
3. A STANDARD FLAT WASHER SHALL BE PROVIDED WHERE A BOLT OR LAG SCREW HEAD OR NUT BEARS DIRECTLY ON WOOD. WASHERS SHALL BE

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HOT DIP GALVANIZED ACCORDING TO ASTM A123, OR STAINLESS STEEL ALLOY 304 OR 316 ACCORDING TO ASTM F594.

4. BOLT HOLES SHALL BE ACCURATELY ALIGNED AND DRILLED 1-2mm LARGER THAN THE BOLT DIAMETER. OVERSIZED HOLES SHALL NOT BE PERMITTED UNLESS INDICATED ON THE CONTRACT DRAWINGS.

5. If the bridge uses bolted connections, the following note shall be added:

BOLTS FOR TIMBER CONNECTIONS SHALL BE TIGHTENED ONLY TO A SNUG-TIGHT FIT. LOCK WASHERS SHOULD BE FLATTENED AND THE NUT SHALL BE TURNED AN ADDITIONAL $\frac{1}{4}$ TO $\frac{1}{2}$ TURN.

WOOD SHALL NOT BE DEFORMED UNDER WASHERS OR STEEL PLATES DUE TO OVERTIGHTENING. UNLESS SPECIFIED OTHERWISE IN THE CONTRACT DOCUMENTS, BOLTED CONNECTIONS SHALL HAVE A MAXIMUM TORQUE OF 40 N-m.

AFTER BOLTS ARE TIGHTENED, BURR OR EPOXY 3-5 THREADS ON THE CLEAN END OF THREADS. THE MINIMUM NUMBER OF CLEAR THREADS PAST THE NUT SHALL BE 3.

6. If the bridge uses lag screw connections, the following note as well as note 7 and 8 below shall be added:

DRILL A PILOT HOLE TO A DEPTH EQUAL TO THE DISTANCE FROM THE INSIDE OF THE LAG SCREW HEAD TO THE SHOULDER OF THREADS (DOES NOT INCLUDE THE TIP).

FOR REDUCED-BODY DIAMETER LAG SCREWS, THE DIAMETER OF PILOT HOLE ACCOMMODATING THE THREADED PORTION OF THE LAG SCREW (E.G. PARENT WOOD SEGMENT) SHALL BE 90% OF SHANK DIAMETER IN HARDWOODS OR 65% OF SHANK DIAMETER IN SOFTWOODS. THE DIAMETER OF PILOT HOLE NOT ACCOMMODATING THE THREADED PORTION (E.G. LAG SEGMENT) SHALL BE OVERSIZED BY 1.6mm OVER THE SHANK DIAMETER.

FOR FULL DIAMETER LAG SCREWS, THE DIAMETER OF PILOT HOLE ACCOMMODATING THE THREADED PORTION OF THE LAG SCREW SHALL BE 90% OF THREAD OUTER DIAMETER IN HARDWOODS OR 65% OF THREAD OUTER DIAMETER IN SOFTWOODS. THE DIAMETER OF PILOT HOLE NOT ACCOMMODATING THE THREADED PORTION SHALL BE OVERSIZED BY 1.6mm OVER THE THREAD OUTER DIAMETER.

7. SUITABILITY OF PILOT HOLE DIAMETER FOR LAG SCREWS SHALL BE CONFIRMED ON SITE WITH A TRIAL HOLE. INSTALLATION SHALL NOT RESULT IN CLEAVING.

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8. LAG SCREWS SHALL BE INSTALLED BY TURNING WITH A WRENCH. DRIVING WITH A HAMMER SHALL NOT BE PERMITTED. PETROLEUM-BASED LUBRICANT SHALL BE USED TO FACILITATE INSERTION OF LAG SCREWS. POWER-ASSISTED WRENCHES MAY BE USED FOR PRIMARY INSERTION OF THE LAG SCREW, OR IF A CALIBRATED WRENCH IS SET TO THE TORQUE VALUE SPECIFIED IN THE CONTRACT DOCUMENTS.

FINAL TORQUING OF LAG SCREWS SHALL BE TO ... N-m, COMPLETED WITH A TORQUE WRENCH OR SUITABLE TORQUE MEASURING DEVICE.

(Torque level is to be based on thread diameter, pitch, length, and species of wood. Insert the required torque value for each fastener specified).

9. WHEN USING NAIL GUNS, INCREASE THE NAIL SIZE TO ACHIEVE THE REQUIRED DIAMETER.

C. Wood Preservatives Notes

1. ALL STRUCTURAL WOOD ELEMENTS SHALL BE PRESSURE TREATED WITH PRESERVATIVES ACCORDING TO OPSS 1601 AND CSA O80 SERIES TO THE USE CATEGORY SPECIFIED IN OPSS 1601.

(If elements will be incised, modify the note as below)

ALL STRUCTURAL WOOD ELEMENTS SHALL BE INCISED AND PRESSURE TREATED WITH PRESERVATIVES ACCORDING TO OPSS 1601 AND CSA O80 SERIES TO THE USE CATEGORY SPECIFIED IN OPSS 1601.

(If elements will be treated with products anticipated to only be available in the United States, include references to AWPA standards, e.g.)

STRUCTURAL WOOD ELEMENTS PRESSURE TREATED WITH CREOSOTE SHALL BE TREATED IN ACCORDANCE WITH CSA O80 OR AWPA U1 AND AWPA T1 TO USE CATEGORY UC4.2 OR UC4B.

2. ALL MACHINING (E.G., CUTTING AND DRILLING) SHALL BE COMPLETED PRIOR TO PRESERVATIVE TREATMENT.
3. WOOD COMPONENTS SHALL BE:

(List the unique wood components and specify their parameters. E.g., component, element size, timber species, Use Category and treatment preservative)

DECKING – 38x140, LPP (LODGEPOLE PINE), UC4.2, PCP-C
GIRDERS – 315x1712 GLT, D FIR (COASTAL DOUGLAS FIR), UC4.2, PCP-C
POSTS – 140x140, RED PINE, UC4.1, CA-B

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15.5 Maintenance Schedule

The designer shall prepare and submit a maintenance schedule that identifies:

- Required periodic maintenance;
- Inspection requirements;
- Component replacement.

Required periodic maintenance may include, but is not limited to the following:

- Time of first maintenance of galvanized components;
- Fastener tightening intervals. There should be at least one interval during the time units are coming to moisture equilibrium (normally during the first 2-3 years of service life);
- Post-tensioning re-tightening intervals;
- Cleaning moisture-trapping dirt and debris;
- Component replacement.

15.6 Sources of Information**15.6.1 Extra Design Information**

Extra design information may be found in the AASHTO LRFDDBS-9, *LRFD Bridge Design Specifications*, 9th Edition and the American Wood Council ANSI/AWC *National Design Specification for Wood Construction*.

15.6.2 Historical MTO Research References

The ministry directed its continuous bridge testing program towards wood structures in 1973 and produced research publications about wood structures until 1992. References are provided for convenience below.

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- Taylor, R J, B deV Batchelor, and K van Dalen. 1983. *Prestressed Wood Bridges*. Paper, Structures Research Office, Ontario Ministry of Transportation, Downsview: Queen's Printer for Ontario, 23.
- Tharmabala, T, and B Bakht. 1986. *Steel-Wood Composite Bridges*. Paper, Structures Research Office, Ontario Ministry of Transportation, Downsview: Queen's Printer for Ontario, 108.

15.6.3 Other Information

Other publications which have not been referenced above and may be informative, but shall not be taken as authoritative for the purposes of design of wood for MTO contracts may be found from the following sources:

- Ontario Wood Bridge Reference Guide (2017);
- The Canadian Wood Council;
- WoodWorks;
- The Engineered Wood Association;
- The American Wood Protection Association;

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- The American Institute of Timber Construction;
- USDA Forest Products Laboratory.

SECTION 16 - MISCELLANEOUS

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SECTION 16 - MISCELLANEOUS

16 MISCELLANEOUS**16.1 Utilities****16.1.1 Services Carried Through or Under Bridges**

Fluid carrying pipelines are not normally allowed to be carried through or under bridges, unless approved. This includes oil and gas pipelines, sanitary and storm sewers, and water mains.

Electrical power lines may be carried through, under, or over bridges provided the voltage does not exceed 44,000 volts.

The following details show approved details for the accommodation of non MTO utilities on bridges. Bridges shall be used for this purpose only as a last resort and after the proponent has carried out a cost benefit analysis. Schemes other than those shown below may be considered, subject to the approval of MTO.

Utility plant is not allowed in sidewalks or to be directly suspended from thin deck slabs. Also, utility hardware must not be placed in a location which prohibits routine inspection of structural components. In any case, it must not be located below the underside of girders for slab on girder bridges, or the underside of deck for slab type bridges.

Steel components that make up the duct support system for bridges having an all-concrete superstructure or for bridges having painted steel girders, must be stainless steel, galvanised, metallised, or painted. For bridges having ACR steel girders, all non-embedded components may be fabricated of the same steel. Attachments shall not be welded to flanges of superstructure girders.

Where necessary, the designer must also provide special details to allow the utility ducts to pass through the abutment ballast walls and superstructure diaphragms. These details must ensure:

1. That the ducts will be able to accommodate all prescribed structure movements including deck jacking for bearing repairs;
2. That fill material behind the ballast walls is not washed out; and,
3. Any settlement that may occur in the fill material behind the ballast wall does not impose undue stresses on the ducts. When the utilities are run through a steel box girder, the designer must consider the location of intermediate cross braces, vertical stiffeners in pier diaphragms, access openings in pier diaphragms, etc. All utilities shall be designed with provisions to allow for future bridge maintenance.

Examples of how ducts could be accommodated through or carried by a structure may include the following cases:

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A. Duct location for structures with accessible cavities or spaces located within the limits of the of the superstructure.

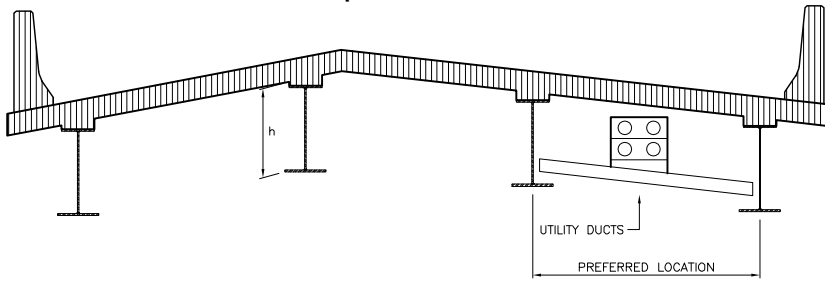


FIGURE 16.1.1 – SLAB ON CONCRETE/STEEL I TYPE GIRDERS

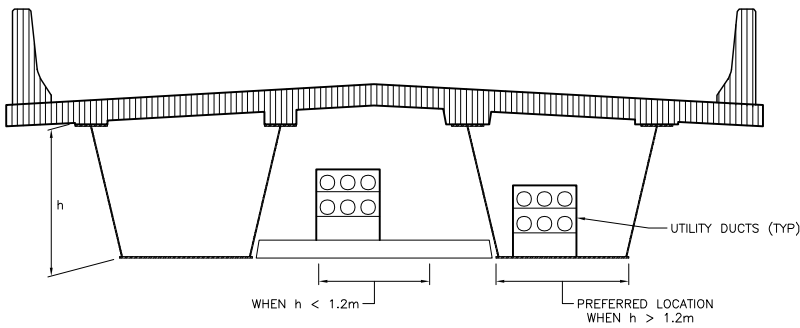


FIGURE 16.1.2 – SLAB ON CONCRETE/STEEL BOX GIRDERS

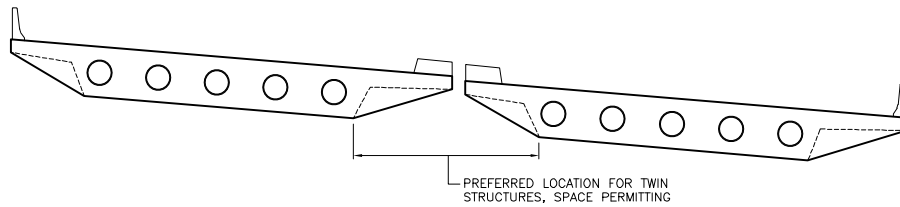


FIGURE 16.1.3 – TWINNED ROUND VOIDED POST TENSIONED DECKS

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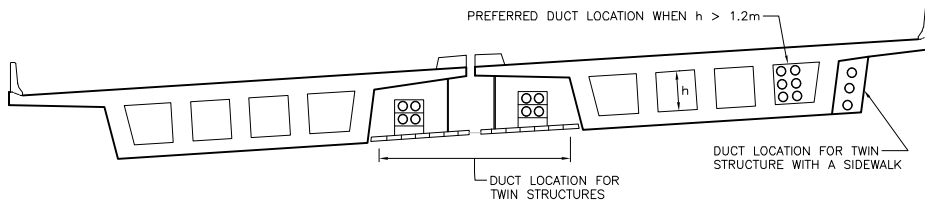


FIGURE 16.1.1(D) POST TENSIONED DECKS WITH TRAPEZOIDAL VOIDS

B. Duct location in slab type structures, (superstructures without accessible cavities).

Projects covered by this criterion require the construction of a non-structural utility corridor outside of and adjacent to the barrier wall. Structure types included in this category include rigid frames, culverts, and solid or round voided post tensioned concrete structures. This scheme is best suited for structures without sidewalks or situated in non-urban locations because of the possibility of pedestrians climbing up onto the utility corridor.

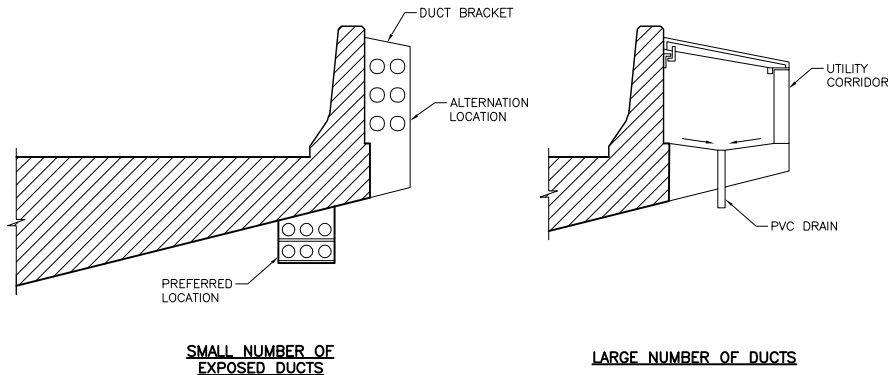


FIGURE 16.1.4 – CANTILEVERED UTILITY CORRIDOR

Electrical lighting ducts (including MTO utilities) shall not exceed 50 mm diameter, except when installed in barrier walls, and shall be placed in accordance with the following criteria:

1. In abutments, ballast walls, slab piers and rigid frames, members must be greater than 500 mm in thickness and spacing of adjacent ducts shall be not less than 6 m.
2. In round columns:
 - Maximum of one electrical duct, outside diameter not exceeding 50 mm, may be placed at centre of columns having a diameter of 1000 mm or more.
 - Junction boxes are not permitted in columns.

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- Entrance/exit of duct at the bottom of columns to be through the footing.
 - Entrance/exit of duct at the top of columns:
 - for fixed-fixed columns (no bearings at top) the duct shall pass straight through the deck column interface.
 - for columns with bearings at top, duct may pass through side of column, provided it does not interfere with reinforcement or bearing assembly and is kept clear of the zone of high stress near the bearings.
3. In post tensioned decks:
- Ducts and junction boxes may be placed in post tensioned decks provided they are located in solid portions of the deck adjacent to supports; they must be placed above the bottom mat of reinforcement (and transverse post tensioning, when present).
 - Longitudinal ducts placed in post tensioned decks may be located inside deck voids, but in any case, shall not be located in web portions of the deck.

When ducts are to be placed in such elements, positive drainage of the embedded duct work must be assured, and the ducts must be located behind the steel reinforcement. The structural Engineer shall also give consideration to the location of junction boxes necessary in such installations and to the potential effects of duct bursting on the structural integrity of the component(s). Electrical ducts left unused would need to be grouted as is required with unused post tensioning ducts.

Other services may be provided for only as a result of specific approvals and agreements. These, and approval for exemption from above restrictions, should be referred to the Assistant Deputy Minister of Operations.

16.1.2 Electrical Grounding of Structures

Railings attached to a structure must be electrically grounded if the structure carries electrical power conductors or provision for them. The Electrical Engineering Section details the grounding.

Bridges having lighting ducts require grounding but those carrying only telephone ducts or cables do not.

High level bridges may require lightning protection. Such structures should be referred to the Traffic Management and Engineering Office, Electrical Engineering Section.

16.2 Transportation and Fabricator Handling of Structural Components

Prefabricated, indivisible structural components that exceed (including the vehicle) any of the following limitations of:

- Length 19.0 m
- Width 3.5 m
- Height 2.6 m
- Weight 30,000 kg

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shall require special oversize/overweight hauling permit(s) in order to be transported by highway carrier over King's highways.

For the following two categories: (A) Routine oversize/overweight loads, and (B) Non routine oversize/overweight loads, pre-approval for King's highways transportation is not required. The length/width/height/weight limitations for both categories are detailed in SSP No. 109F16, to which reference should be made.

Components exceeding the limitations of categories (A) or (B) require that the designer of the load obtain transportation approvals from the Weight and Load Engineer, Carrier safety and Enforcement Branch, MTO and other authorities, all as required by SSP No. 109F16.

As early as possible in the design stage, written request for road transportation shall be submitted to the Oversize / Overweight Permits Office for an in principle special permit approval. The request shall indicate the number of components, dimensions, weights, travel origin and destination, duration of transportation and required travel frequency. The transportation approvals may take up to four (4) weeks and may not be granted if transportation is deemed to be unsafe.

16.3 Modular Bridges**16.3.1 Temporary Modular Bridge (TMB)**

The use of temporary modular bridge (TMB) (such as Acrow, Bailey, Mabey, AIL, etc.) is primarily for installations as detours during new bridge construction or for emergency situations with existing bridges. The use of a TMB as a permanent structure shall be subject to the following:

- Consideration of the volume of traffic at the bridge site Fatigue design as per the CHBDC; and,
- The required traffic barrier crash test level (e.g., TL-3, TL-4, etc.).

The responsibility and control of the supply, maintenance, and inventory of all TMB components under the jurisdiction and ownership of MTO is governed by the following Policy, Planning & Standards Directive PHM-B-22:

"Temporary Modular Bridges":

- (A) Issuing Priorities*
- (B) Supply, Maintenance and Inventory Control*
- (C) Retrieval, Rental and Disposal*

A summary of the contents of this document is given in this subsection.

16.3.2 MTO's TMB Policy for Municipalities and other Agencies

The Ministry will only supply TMB to municipalities when there is a real emergency. The Ministry will not supply TMB to municipalities for detour purposes when new bridges are being constructed or for bridge rehabilitations.

Municipalities that need TMB components in an emergency shall initially attempt to obtain their requirements from the private sector. If the private sector is unable to supply the

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components within one week from the date of the emergency, the municipality may request them from the Ministry. When submitting a request to the Ministry, the municipality must provide evidence that they were unable to obtain the TMB components in a timely manner. The procedure to follow when making an emergency request and details of a legal TMB rental/purchase agreement that must be entered into is given in Directive PLNG-B-007.

The Ministry will not supply TMB to any other jurisdictions.

16.3.3 Policy for Issuing Priorities of TMBs

The Regional Director for the region in which the TMB is required is responsible for approving the release of TMB. The Manager of Operations, Northern Region, North Bay is responsible for issuing TMB. Issuing priorities will be in the following order of precedence:

1. Emergency needs on the Ministry's Provincial Highway System.
2. Emergency needs in municipalities.
3. Detour needs on the Ministry's Provincial Highway System.

16.3.4 Emergency Needs Policy for TMB's

To ensure a responsible supply of TMB components sufficient to meet most anticipated emergencies, the Ministry maintains and services an emergency stock of TMB components at a stockyard in Northern Region.

An emergency may exist due to failure of any bridge component or collapse of an existing bridge that leads to a permanent closure of the bridge. An emergency requiring the issue of TMB components is deemed to exist when one or more of the following conditions are satisfied:

- a) An existing structure has been damaged beyond immediate repair;
- b) The road is completely closed to traffic due to damage to the bridge;
- c) The shortest detour that exists exceeds what would normally be considered reasonable for purposes of emergency response (i.e., fire, police, and ambulance);
- d) The time required to make repairs to the existing structure is considered excessive;
- e) Alternative solutions, other than the supply of TMB components are not viable.

The approval procedure for emergency requests at Ministry and municipality sites, prior to the release of TMB components, shall be as detailed in Directive PLNG-B-007.

16.3.5 Policy for Detours and Non-Emergency Needs

At present, the Ministry maintains and services a portion of the existing inventory of TMB components for use as detours at Ministry construction sites. This stock is kept separate

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from that intended for emergency use. However, this detour stock may be depleted by attrition with time and the supply of TMB components for operational needs will eventually be obtained from the private sector.

The need for a TMB and sufficient information for design must be clearly stated in a Structural Planning Report. This will be the Structural Planning Report for a structural project for which a TMB detour is required or, if the TMB is not part of such a project, a separate report.

TMB drawings, quantities, and appropriate tender items are to be prepared by the Regional Structural Section responsible for the project. Site numbers should be assigned according to Section 2.5.1, e.g., 34E-2221/B0 in the title block.

During the preliminary stages of design, the Regional Structural Section shall obtain confirmation from the Manager of Operations, Northern Region that the required quantities of TMB components will be available when needed. If this is not possible then consideration shall be given to alternative designs from the private sector.

The approval procedure, bills of materials and requests for the release of TMB components shall be as detailed in Directive PLNG-B-007.

16.3.6 Use of Bailey Bridges as TMB

Bailey Bridges have been used in Ontario for over a half century. Due to their modular components, Bailey bridges can be assembled in numerous ways, with multiple truss panels either adjacent or on top of each other. Similarly, the floor system is made up of a number of adjacent stringer sets (depending on the width of bridge) and a number of transverse floor transoms, usually 2 or 4 per 10 ft (3 m) bay.

For Bailey Bridges with two floor transoms per bay, an evaluation according to the CHBDC found that the load carrying capacity of the floor system was deficient. Subsequent load testing by the MTO Structures Office confirmed that the floor system was incapable of carrying full CHBDC CL-625-Ont loading.

Consequently, all new Bailey Bridge installations shall be specified with four transverse transoms per bay when load posting is not desirable and when two transverse transoms are specified, the bridge shall have a triple load posting of 25 tonnes, 40 tonnes, and 55 tonnes for a single, two unit and multi-vehicle train respectively.

16.4 High Mast Lighting Poles**16.4.1 Design of High Mast Lighting Pole**

The design of high mast lighting poles (HMLP) and their foundations shall be based on the requirements of the CHBDC. A reference wind pressure for a 50-year return period shall be used. AASHTO's "Standard Specifications for Structural Supports for Highway Signs, Luminaries and Traffic Signals" shall also be used for design requirements, for example fatigue design, not specified by the CHBDC. However, where there is a conflict the CHBDC shall govern.

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On Ministry projects pole heights of 25, 30, 35, 40 and 45 m are currently used and the design for these high mast lighting supporting structures has already been carried out. MTO issued standard drawings showing details of these structures. General details are shown on the following MTO standards:

- OPSD 2450.011 HMLP 25, 30 & 35 m 8-Sided Pole;
- OPSD 2450.021 HMLP 40 & 45 m 12-Sided Pole.

The installation of high mast poles is covered by OPSS 630 "Construction Specification for the Installation of High Mast Poles" and related special provisions.

High mast lighting supports have fixed based support systems that do not yield or break away upon impact. The large mass of the poles and the potential safety consequences of them falling to the ground necessitate a fixed base design. Since fixed base systems are rigid obstacles, they should not be used in the roadside clear zone unless protected by a barrier or approved by the ministry.

16.4.2 Foundations of High Mast Lighting Pole

An anchorage assembly detailed on the following standards connects the high mast lighting support to a concrete caisson foundation:

- OPSD 2218.010 HMP Anchorage Assembly Placement;
- OPSD 2456.011 HMLP Anchorage Assembly Details.

The design of foundations shall be based on the requirements stated in Section 16.4.1.

Foundations for high mast poles may be designed according to the Structures Office publication BRO-009, "Guidelines for the Design of High Mast Pole Foundations, third edition," which is based on the requirements of Section 16.4.1. This guide presents the design of concrete caisson type foundations in a variety of soil conditions including rock and layered soils. The designs and design methods given in this document are modeled on short piles with a rotational limit of 0.005 radians at the ground surface, and the theoretical analyses given in papers by Bengt, Broms and others.

The following Structural Standard Drawings for high mast pole foundations that are ground or median mounted are available for use:

- SS116-50 High Mast Lighting Pole Footing Ground Mounted;
- SS116-51 High Mast Lighting Pole Footing Median Tall Mounted (Symmetrical);
- SS116-52 High Mast Lighting Pole Footing Median Tall Mounted (Asymmetrical).

The construction of foundations for high mast poles is covered by OPSS 631 "Construction Specification for Concrete Footings and Maintenance Platforms for High Mast Lighting Poles" and related special provisions.

16.5 Pedestrian, Bicycle and MUP Bridges

This section provides guidelines for the design of pedestrian and bicycle bridges.

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Pedestrian, Bicycle and MUP bridges shall be designed following CSA S7-23 – Pedestrian, cycling, and multiuse bridge design guideline. CSA S7-~~25-23~~ relies heavily on CSA S6 (CHBDC). In addition to those requirements, the requirements of Section 16.5.1 shall be followed.

16.5.1 Loads

Where the clear width of the bridge is ≥ 2.4 m and < 3 m, the bridge shall be designed for a load equal to 50% of the maintenance vehicle load specified in CSA S7, while for bridges with clear width ≥ 3.0 m, they shall be designed for 100% of the maintenance vehicle load.

16.6 Ethylene Vinyl Acetate (EVA) Foam**16.6.1 Overview**

EVA foam is a flexible cellular expanded rubber foam product, usually composed of Ethylene-Vinyl Acetate/Polyethylene. It is typically used for providing a separation between adjacent concrete elements.

OPSS 920 has historically classified EVA foam only as a joint seal in accordance with ASTM D1056. It has been used as a joint seal between twin structures, but its use has recently been more common in other bridge details such as semi-integral conversions. ASTM D1056 allows for many different types, classes, and grades of EVA foam. There is a need to better define the EVA foam material on MTO bridge projects to ensure it is supplied and installed to meet the performance requirements of the design.

Designers shall include Standard Special Provision SSP 109S18 in MTO bridge projects. The SSP amends OPSS 904 and provides material and construction requirements for EVA foam.

Designers shall designate the type and grade of EVA foam according to guidance described below.

16.6.2 Material Properties and Classifications

ASTM D1056-14 gives the requirements for EVA foam, and classifies the material based on Type, Class, and Grade. There are two main Types of EVA foam which can be specified. These include:

1. **Type 1: Open-Cell Rubber** - A product whose cells are not totally enclosed by its walls and open to the surface, either directly or by interconnecting with other cells;
2. **Type 2: Closed-Cell Rubber** - A product whose cells are totally enclosed by its walls and hence not interconnecting with other cells.

Both types are further classified into four different Classes of material. All Classes are made from either natural, synthetic, reclaimed, or rubber-like materials, but each have various oil and/or temperature interaction requirements. These Classes according to ASTM D1056-14 are:

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- Class A: Specific resistance to the action of petroleum base oils is not required.
- Class B: Specific requirements for oil resistance with low mass change.
- Class C: Specific requirements for oil resistance with medium mass change.
- Class D: Specific requirements for extreme temperature resistance (-75 to 175°C), but specific requirements for petroleum-based oils is not required.

Types and Classes of EVA foam are then further divided into different Grades, which specify the range of stiffness for the material based on compression of 25% of the thickness. This is an important property of the material which the designer must consider when calculating forces acting on the ballast wall due to thermal movements of a semi-integral downturn. These Grades (according to ASTM D1056-14), as well as the calculated unfactored elastic modulus values are:

Grade	Compression-Deflection Range (kPa)	Avg. Modulus of Elasticity, E_{avg} (kPa)	Max. Modulus of Elasticity, E_{max} (kPa)
0	0 - 13.8	27.6	55.2
1	13.8 - 34.5	96.6	138
2	34.5 - 62.1	193	248
3	62.1 - 89.6	303	358
4	89.6 - 117.2	414	469
5	117.2 - 172.4	579	690

The average modulus of elasticity is based on the midrange value of the compression deflection range, which is used for estimating deflections across the material. The maximum modulus of elasticity is based on the upper bound value in the range, which is used for determining design forces transferred to an adjacent element across the EVA foam. Both values are based on 25% compression, since they are calculated from the specified compression deflection ranges given in ASTM D105614.

16.6.3 MTO's Best Practices and Preferences

MTO preference is to use Type 2 (Closed Cell) and Class A (no specific oil or temperature requirements) EVA foam on bridge projects. When EVA foam is not explicitly classified, Contract special provisions will require the Contractor to supply Type 2, Class A, and either Grade 2 or 3 EVA foam by default. The designer may specify a different grade, but this should be limited only to areas where a specific grade is required.

It is not economical to specify numerous types, classes, and grades of EVA foam on bridge projects. Since Type 2 or 3 and Class A is preferred, designers should make an attempt to limit the number of grades required.

It is preferred to use a single thickness of EVA foam rather than layering different sizes together. There are many thicknesses available, but the most common are 20, 25, 30, 40, 50, 75, and 100 mm. If greater than 100 mm is required, the designer should specify a

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total thickness that can be built up using multiple layers of one of the most common thicknesses. The width and height are usually cut from sheets, so these dimensions can be prescribed as required.

16.6.4 Design Considerations for EVA Foam in MTO's Projects
EVA Foam Supporting Dead Loads

The designer should consider the stiffness of EVA foam when it is used to support dead loads such as approach slabs resting on wingwalls.

The compressive deflection of EVA foam supporting dead loads should not exceed 10% of its specified thickness as a best practice. The following table gives the maximum amount of dead load that each grade of EVA foam can support, based on the calculated average modulus of elasticity (E_{avg}) and a maximum strain of 0.1:

Grade	Maximum Dead Load (kPa)
0	2.8
1	9.7
2	19.0
3	30.0
4	41.0
5	58.0

EVA Foam Accommodating Movements

Design forces transmitted to structural components through EVA foam need to be considered in design. The maximum modulus of elasticity should be used in calculating the applied forces as follows:

$$\sigma = E_{max} \times (\Delta_c/t)$$

Where:

$$\Delta_c = \text{Imposed displacement across the EVA foam}$$

$$t = \text{Thickness of EVA Foam}$$

A sufficient thickness of EVA foam shall be provided such that the applied strain (Δ_c/t) does not exceed 0.25 (25% of the specified thickness).

Design Example #1

A semi-integral abutment detail experiences 13.4 mm of thermal contraction. A total of 60 mm thickness of EVA Foam Grade 2A2 is provided with a height of 300 mm. Calculate the unfactored applied force on the ballast wall as a result of this movement.

Solution:

$$\sigma = E_{max} \times (\Delta_c/t); \sigma = (248 \text{ kPa}) \times (13.4 \text{ mm} / 60 \text{ mm}); \sigma = 55.4 \text{ kPa}$$

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For a unit length of ballast wall: $P = (55.4 \text{ kPa}) \times (300 \text{ mm})$; $P = 16.6 \text{ kN/m}$

Design Example #2:

Re-calculate the result in Example #1 with 80 mm thickness of EVA Foam grade 2A1 provided instead.

Solution:

$$\sigma = E_{\max} \times (\Delta_c/t); \sigma = (138 \text{ kPa}) \times (13.4 \text{ mm} / 80 \text{ mm}); \sigma = 23.1 \text{ kPa}$$

For a unit length of ballast wall: $P = (23.1 \text{ kPa}) \times (300 \text{ mm})$; $P = 6.9 \text{ kN/m}$

16.6.5 Presentation of EVA Foam on Contract Drawings

When the default classification of EVA foam is appropriate (Type 2, Class A, Grade 2 or 3), the designer shall use the following notation in the contract drawings:

“X” mm EVA FOAM

Where:

X = Thickness of EVA Foam

The designer may specify a different classification of EVA foam where necessary. Type 2 and Class A EVA foam should always be specified, and only the Grade should be changed. The designer shall use the following notation in the contract drawings when a specific grade is required:

“X” mm EVA FOAM, GRADE “TCG”

Where:

X = Thickness of EVA Foam,

T = Type (2),

C = Class (A), and

G = Grade (0 through 5)

For example, for 40 mm thick of Grade 4 EVA Foam, the designer would show it on the contract drawings as “40 mm EVA FOAM, GRADE 2A4”.

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17 COMPUTER APPLICATIONS - NOT USED

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SECTION 18 - SIGN SUPPORTS

18 SIGN SUPPORTS**18.1 General**

The design and detailing of standard overhead sign support structures is to be carried out in accordance with the latest version of the Sign Support Manual (SSM). Roadside sign supports are covered in Ontario Traffic Manual – Book 3, Ground-Mounted Sign Support and Installation, 2022.

The SSM contains information needed to prepare the contract drawings, tender quantities and special provisions for standard sign supports. The design of the cantilever, tri-chord, monotube and variable message sign (VMS) support structures meets the requirements of the CHBDC.

All non-standard sign support structures that do not meet the assumptions stated in the SSM must be custom designed, sealed and signed by two engineers.

The standard sign support structures covered by the SSM are:

18.2 Overhead Sign Supports**18.2.1 Cantilever Static Sign Supports**

A sign support which cantilevers from the roadside over the closest driving lane and shoulder for static signs. Two types of cantilever sign supports, single cantilever and butterfly, are currently available in the SSM. This sign support is suitable for total signboard sizes up to and including 48 m², depending on the sign type, class, reference wind pressure and the eccentricity of the centreline of the signboard. The static signs range in depth from 1525 mm to 2745 mm inclusive. The structure is constructed of galvanised structural steel and consists of an overhead truss supported on a single leg column with a concrete caisson type foundation.

18.2.2 Tri-Chord Static Sign Supportsa) Simply Supported Type:

This is a sign support structure that crosses the highway with spans ranging from 14 m to 36 m inclusive. These sign supports are suitable for carrying total signboard areas of up to and including 45 m². The static signs range in depth from 1525 mm to 3965 mm inclusive, depending on the structure type. The structure is constructed of galvanised structural steel and consists of a three-chord overhead truss supported on a single leg column on each side with concrete caisson type foundations.

b) Cantilever Type:

A sign support which cantilevers from the roadside over the closest driving lane and shoulder. This sign support is suitable for total signboard sizes up to and including 26.7 m². The static signs range in depth from 1525 mm to 2745 mm inclusive. The

SECTION 18 - SIGN SUPPORTS

structure is constructed of galvanised structural steel and consists of a three-chord overhead truss supported on a single leg column with concrete caisson type foundation.

18.2.3 Monotube Sign Supports

A sign support that crosses the highway with spans ranging from 13.5 m to 24 m inclusive for lane designation signs. This sign support is suitable for signboard sizes up to and including 1200 mm by 1200 mm. The structure is constructed of galvanised structural steel and consists of four tapered, octagonal monotubes with concrete caisson type foundations.

18.2.4 VMS Sign Supportsa) Overhead truss:

A sign support that crosses the highway with spans ranging from 17.6 m to 34 m inclusive for variable message sign systems. This sign support is suitable for sign systems up to and including 40 m². The structure is constructed of an overhead aluminum truss supported on galvanised structural steel columns with concrete caisson type foundations.

b) Pole mounted:

This VMS sign support is fabricated from a galvanised structural steel pole with concrete caisson type foundation. The pole is carrying the VMS digital panel. The maximum VMS size is 3000 mm in depth by 5000 mm in width.

18.2.5 Bridge Mounted Sign Supports

A sign support mounted on bridges for static signs ranging in depth from 1525 mm to 2745 mm inclusive, and width as required. The structure is constructed of aluminium struts and arms connected to the bridge by stainless steel anchors.

18.3 Ground Mounted Sign Supports

Ground mounted sign supports are available with break-away supports, or rigid supports when located beyond the clear recovery zone or adequately behind a roadside barrier. These sign supports are no longer contained in the MTO's Sign Support Manual and are in Ontario Traffic Manual - Book 3 (Ground-Mounted Sign Support and Installation).

SECTION 19 - BRIDGE REHABILITATION

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19.1.1 MTO's Structure Rehabilitation Manual (SRM)..... 2

SECTION 19 - BRIDGE REHABILITATION**19 BRIDGE REHABILITATION****19.1 Rehabilitation of Bridge Structures**

The overall structural design and detailing for the rehabilitation and repair of bridge structures and components shall be according to the Structural Manual and CHBDC. Methods, procedures, and detailed technical information related to bridge rehabilitation are covered in the MTO Structure Rehabilitation Manual (SRM).

Additionally, other MTO manuals, guidelines, policies, and standards that are applicable to bridge rehabilitation projects should also be referred to.

19.1.1 MTO's Structure Rehabilitation Manual (SRM)

The SRM contains information on performing detailed site investigations, selecting the appropriate methods and scope of rehabilitation, information on contract preparation, and other guidance as it relates to bridge (and other structure) rehabilitation.

DIVISION 3 - DESIGN AIDS

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None.

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None.

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None.

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None.

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None.

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None.

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None.

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None.

18 SIGN SUPPORTS

None.

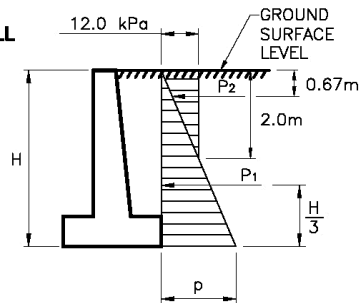
SECTION 1 - INDEX OF DESIGN AIDS

SECTION 2 - GENERAL DESIGN AND DRAFTING

DESIGN AID 2-10

ACTIVE EARTH PRESSURE WITHOUT SURCHARGE

q = EQUIVALENT FLUID PRESSURE (kPa)
 p = UNIT PRESSURE AT DEPTH H METRES
 P = TOTAL FORCE PER LINEAR METRE OF WALL
 M = OVERTURNING MOMENT (kN-m)



AT SERVICEABILITY LIMIT STATES

$\alpha_E = 1.0$
 $q = 7.0 \times (H)$

AT ULTIMATE LIMIT STATES

$\alpha_E = 1.25$
 $q = 1.25 \times (7.0) \times (H)$

$\phi = 30^\circ$
 $P = P_1 + P_2$

- RESULTS INCLUDE THE EFFECT OF COMPACTION SURCHARGE

SLS			
H (m)	P (kPa)	P (kN)	M (kN-m)
1.0	7	13	6
1.5	11	19	14
2.0	14	26	25
2.5	18	33	40
3.0	21	44	59
3.5	25	55	84
4.0	28	68	114
4.5	32	83	152
5.0	35	100	197
5.5	39	118	251
6.0	42	138	315

ULS			
H (m)	P (kPa)	P (kN)	M (kN-m)
1.0	9	16	8
1.5	13	24	18
2.0	18	33	31
2.5	22	43	50
3.0	26	55	74
3.5	31	69	105
4.0	35	85	143
4.5	40	104	190
5.0	44	125	246
5.5	48	148	314
6.0	53	173	394

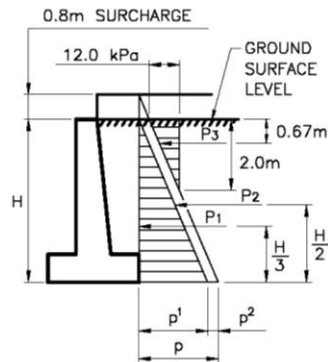
THE METHOD OF EQUIVALENT FLUID PRESSURES IS LIMITED TO A MAXIMUM HEIGHT OF 6.0m. FOR RETAINING WALLS WITH HEIGHTS > 6.0m, THE EARTH PRESSURE DISTRIBUTION SHALL BE ESTABLISHED BY A GEOTECHNICAL ENGINEER.

SECTION 2 - GENERAL DESIGN AND DRAFTING

DESIGN AID 2-11

ACTIVE EARTH PRESSURE WITH 800 mm SURCHARGE

q = EQUIVALENT FLUID PRESSURE (kPa)
 p = UNIT PRESSURE AT DEPTH H METRES
 p¹ = UNIT PRESS. DUE TO BACKFILL AT DEPTH H
 p² = UNIT PRESSURE DUE TO 0.8m SURCHARGE
 P = TOTAL FORCE PER LINEAR METRE OF WALL
 M = OVERTURNING MOMENT (kN-m)



AT SERVICEABILITY LIMIT STATES

$\alpha_E = 1.0$
 $q = 7.0 \times (H)$

AT ULTIMATE LIMIT STATES

$\alpha_E = 1.25$
 $q = 1.25 \times (7.0) \times (H)$

$\phi = 30^\circ$
 $P = P_1 + P_2 + P_3$
 $p = p^1 + p^2$

– RESULTS INCLUDE THE EFFECT OF COMPACTION SURCHARGE

SLS			
H (m)	P (kPa)	P (kN)	M (kN-m)
1.0	13	18	9
1.5	16	28	20
2.0	20	37	37
2.5	23	48	58
3.0	27	60	85
3.5	30	75	118
4.0	34	90	159
4.5	37	108	209
5.0	41	128	268
5.5	44	149	337
6.0	48	172	417

ULS			
H (m)	P (kPa)	P (kN)	M (kN-m)
1.0	16	23	11
1.5	20	35	25
2.0	25	46	46
2.5	29	60	73
3.0	34	75	106
3.5	38	94	148
4.0	43	113	199
4.5	46	135	261
5.0	51	160	335
5.5	55	186	421
6.0	60	215	521

THE METHOD OF EQUIVALENT FLUID PRESSURES IS LIMITED TO A MAXIMUM HEIGHT OF 6.0m. FOR RETAINING WALLS WITH HEIGHTS > 6.0m, THE EARTH PRESSURE DISTRIBUTION SHALL BE ESTABLISHED BY A GEOTECHNICAL ENGINEER.

SECTION 2 - GENERAL DESIGN AND DRAFTING

DESIGN AID 2-12
CONTINUOUS BEAMS (EQUAL SPANS)

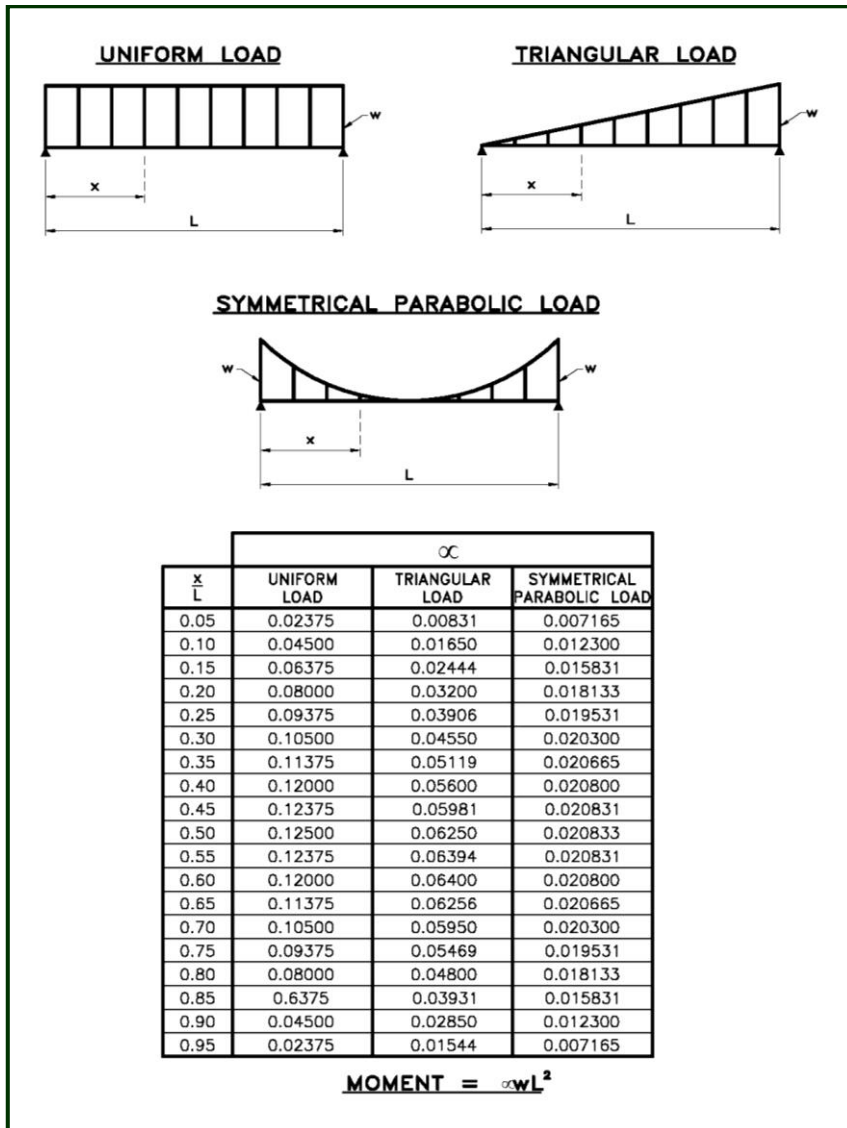
<p>BENDING MOMENT = coefficient x w x L² SHEAR FORCE = coefficient x w x L w = U.D.L. L = SPAN</p>	
<p>COEFFICIENTS FOR MAXIMUM BENDING MOMENT AND SHEAR</p>	
<p>DEAD LOAD (ALL SPANS LOADED)</p>	
<p style="text-align: center;">BENDING MOMENT</p>	<p style="text-align: center;">SHEAR</p>
<p>LIVE LOAD (SEQUENCE OF U.D.L. LOADED SPAN(S) TO GIVE MAX. BENDING MOMENT OR SHEAR)</p>	

SECTION 2 - GENERAL DESIGN AND DRAFTING

DESIGN AID 2-13

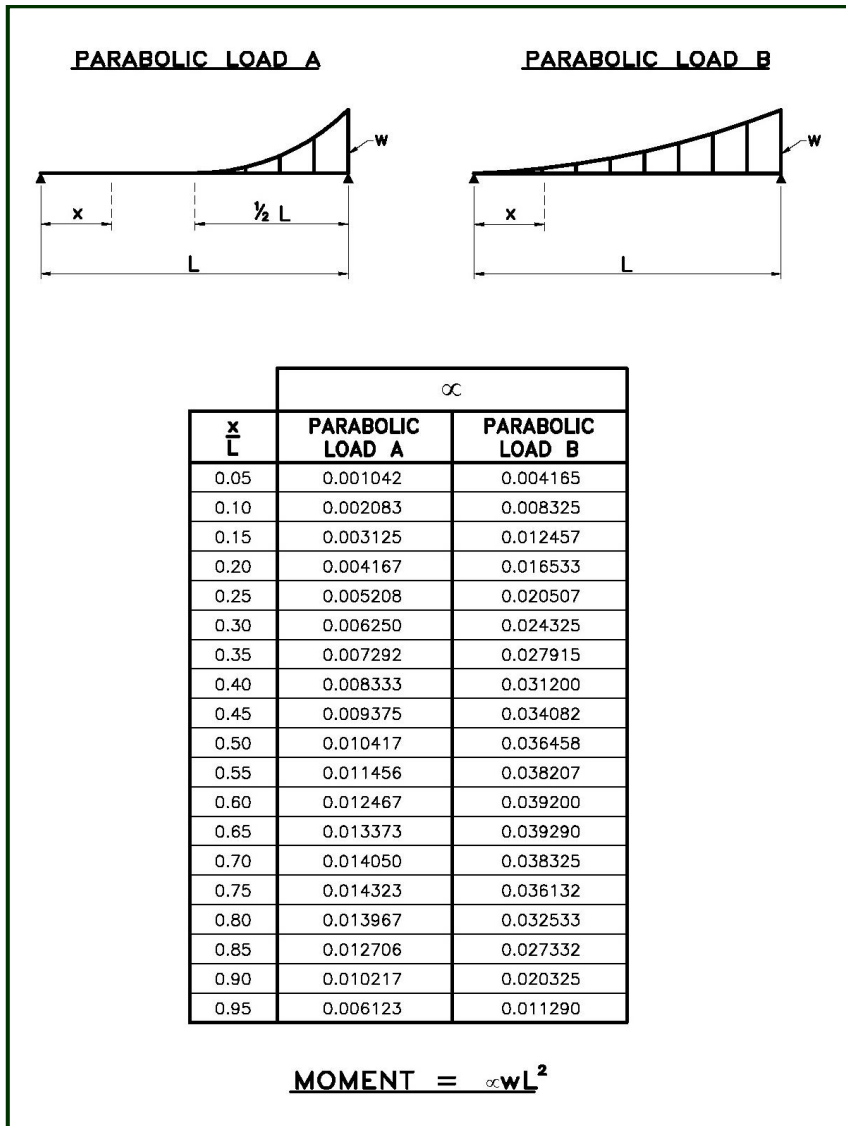
SIMPLE BEAM MOMENT COEFFICIENTS

(Uniform, Triangular, and Symmetrical Parabolic Loads)



SECTION 2 - GENERAL DESIGN AND DRAFTING

DESIGN AID 2-14
SIMPLE BEAM MOMENT COEFFICIENTS
(Parabolic Load A and B)



SECTION 2 - GENERAL DESIGN AND DRAFTING

DESIGN AID 2-15
 FIXED END MOMENTS FOR PRISMATIC BEAMS
 (Simply Supported / Fixed End)

SIMPLY SUPPORTED	FIXED END
	$\frac{3}{16} PL$
	$\frac{Pa}{2} \left[1 - \left(\frac{a}{L} \right)^2 \right]$
	$1.5 Pa \left(\frac{b}{L} \right)$
	$\frac{PL}{3}$
	$\frac{9}{32} PL$
	$\frac{15}{32} PL$
	$\frac{19}{48} PL$
	$\frac{Pa}{8} (n^2 - 1)$ n=number of spaces
	$\frac{Pa}{8} (n^2 + 0.5)$ n=number of loads
	$\frac{PL}{8}$
	$\frac{Pc}{8} \left(2 - \left(\frac{c}{L} \right)^2 \right)$
	$\frac{Pc}{8} \left[2 - \left(\frac{c}{L} \right)^2 \right]$
	$\frac{3}{4} Pc \left(1 - \frac{2}{3} \frac{c}{L} \right)$
	$\frac{Pa}{8L^2} \left[4b(a+L) - c^2 \right]$
	$\frac{3}{16} PL \left[1 - \frac{1}{3} \left(\frac{c}{L} \right)^2 \right]$
	$\frac{PL}{7.5}$
	$\frac{7}{60} PL$
	$\frac{PL}{6.4}$
	$\frac{3}{16} PL$
	0.15 PL
	$\frac{PL^2}{8} \left[1 - 2 \left(\frac{a}{L} \right)^2 - \left(\frac{a}{L} \right)^3 \right]$
	$\frac{M}{2} \left[1 - 3 \left(\frac{a}{L} \right)^2 \right]$
	$\frac{3EI}{L^2} \Delta$

SECTION 2 - GENERAL DESIGN AND DRAFTING

DESIGN AID 2-16

FIXED END MOMENTS FOR PRISMATIC BEAMS

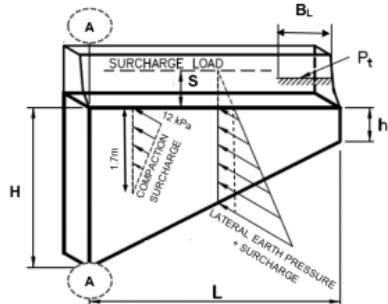
(Fixed End / Fixed End)

Ma	FIXED END	TYPE OF LOADING	FIXED END	Mb
$\frac{PL}{8}$				$\frac{PL}{8}$
$P_a \left(\frac{b}{L}\right)^2$				$P_b \left(\frac{a}{L}\right)^2$
$P_a \left(\frac{L-a}{L}\right)$				$P_a \left(\frac{L-a}{L}\right)$
$\frac{2}{9} PL$				$\frac{2}{9} PL$
$\frac{3}{16} PL$				$\frac{3}{16} PL$
$\frac{5}{16} PL$				$\frac{5}{16} PL$
$\frac{19}{72} PL$				$\frac{19}{72} PL$
$\frac{P_a}{12} (n^2 - 1)$ n=number of spaces				$\frac{P_a}{12} (n^2 - 1)$ n=number of spaces
$\frac{P_a}{12} (n^2 + 0.5)$ n=number of loads				$\frac{P_a}{12} (n^2 + 0.5)$ n=number of loads
$\frac{PL}{12}$				$\frac{PL}{12}$
$\frac{P_c}{2} \left[\frac{2}{3} \times \frac{c}{L} - \frac{1}{2} \left(\frac{c}{L} \right)^2 \right]$				$\frac{P_c}{2} \left[1 - \frac{4}{3} \times \frac{c}{L} + \frac{1}{2} \left(\frac{c}{L} \right)^2 \right]$
$\frac{P_c}{2} \left[1 - \frac{4}{3} \times \frac{c}{L} + \frac{1}{2} \left(\frac{c}{L} \right)^2 \right]$				$\frac{P_c}{2} \left[\frac{2}{3} \times \frac{c}{L} - \frac{1}{2} \left(\frac{c}{L} \right)^2 \right]$
$\frac{P_c}{2} \left(1 - \frac{2}{3} \frac{c}{L} \right)$				$\frac{P_c}{2} \left(1 - \frac{2}{3} \frac{c}{L} \right)$
$\frac{P_a}{12L^2} [12ab^2 + c^2(L-3b)]$				$\frac{P_a}{12L^2} [12a^2b + c^2(L-3a)]$
$\frac{PL}{8} \left[1 - \frac{1}{3} \left(\frac{c}{L} \right)^2 \right]$				$\frac{PL}{8} \left[1 - \frac{1}{3} \left(\frac{c}{L} \right)^2 \right]$
$\frac{PL}{15}$				$\frac{PL}{10}$
$\frac{PL}{10}$				$\frac{PL}{15}$
$\frac{PL}{9.6}$				$\frac{PL}{9.6}$
$\frac{PL}{8}$				$\frac{PL}{8}$
$\frac{PL}{10}$				$\frac{PL}{10}$
$\frac{wL^2}{12} \left[1 - 2 \left(\frac{a}{L} \right)^2 + \left(\frac{a}{L} \right)^3 \right]$				$\frac{wL^2}{12} \left[1 - 2 \left(\frac{a}{L} \right)^2 + \left(\frac{a}{L} \right)^3 \right]$
$M \frac{b}{L} (2 - 3 \frac{b}{L})$				$M \frac{a}{L} (2 - 3 \frac{a}{L})$
$\frac{6EI}{L^2} \Delta$				$\frac{6EI}{L^2} \Delta$

SECTION 5 - ABUTMENTS, WINGWALLS AND RETAINING WALLS

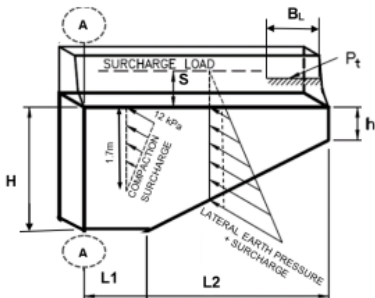
WINGWALL DESIGN AID

Moments and Lateral Thrust for Cantilever Wingwalls



A - A = Interface of the Wingwall to the Cleat
 W = Equivalent Fluid Pressure, γK (kPa)
 P_t = Transverse Live Load on Barriers (kN)
 B_L = Distribution Length of P_t (m)
 S = Surcharge Height (m)

Loadings	Total Moment at A-A	Total Lateral Thrust
Lateral Earth Pressure	$M_E = \frac{WL^2}{24} (H^2 + 2Hh + 3h^2)$	$P_E = \frac{WL}{6} (H^2 + Hh + h^2)$
Surcharge	$M_S = \frac{WL^2}{6} S(H + 2h)$	$P_S = \frac{WL}{2} S(H + h)$
Compaction Surcharge	$M_{CS} = 5.1 L^2$	$P_{CS} = 10.2 L$
Live Load on Barrier	$M_L = P_t(L - \frac{B_L}{2})$	$P_L = P_t$



A - A = Interface of the Wingwall to the Cleat
 W = Equivalent Fluid Pressure, γK (kPa)
 P_t = Transverse Live Load on Barriers (kN)
 B_L = Distribution Length of P_t (m)
 S = Surcharge Height (m)

Loads	Total Moment at A-A	Total Lateral Thrust
Lateral Earth Pressure	$M_E = \frac{W}{24} (6L_1^2H^2 + 4L_1L_2(H^2 + Hh + h^2) + L_2^2(H^2 + 2Hh + 3h^2))$	$P_E = \frac{W}{6} (3L_1H^2 + L_2(H^2 + Hh + h^2))$
Surcharge	$M_S = \frac{W}{6} S(3L_1^2H + 3L_1L_2(H + h) + L_2^2(H + 2h))$	$P_S = \frac{W}{2} S(2L_1H + L_2(H + h))$
Compaction Surcharge	$M_{CS} = 5.1 (L_1 + L_2)^2$	$P_{CS} = 10.2 (L_1 + L_2)$
Live Load on Barrier	$M_L = P_t(L_1 + L_2 - \frac{B_L}{2})$	$P_L = P_t$

SECTION 5 - ABUTMENTS, WINGWALLS AND RETAINING WALLS

DESIGN AID 5-1

WINGWALL DESIGN TABLE (T = 400 mm)

T = 400mm

f_c = 30 MPa
Concrete Cover = 70 mm
f_y = 500 MPa

P_t = 100 kN for TL-4 Barrier
210 kN for TL-5 Barrier

B_L = 1.05 m. for TL-4 Barrier
2.40 m. for TL-5 Barrier

Equivalent Fluid Pressure, W = α k = 7 kPa

SHEAR AND MOMENT AT A-A

$$M_{A-A} = \left\{ \frac{WL^2}{24} (H^2 + 2Hh + 3h^2) + \frac{WL^2}{6} S(H + 2h) + 5.1L^2 \right\} \alpha_E + P_t \left(L - \frac{B_L}{2} \right) \alpha_L$$

$$P_{A-A} = \left\{ \frac{WL}{6} (H^2 + Hh + h^2) + \frac{WL}{2} S(H + h) + 10.2L \right\} \alpha_E + P_t \alpha_L$$

Design Table of Wingwall with TL-4 Barrier

L (m)	Vf at A-A (kN/m)	Mf at A-A (kN.m/m)	Sr (mm)					
			15M	20M	25M	30M	35M	
2.00	104	133	200	300				
2.50	105	171	150	225				
3.00	108	207	125	175	300			
3.50	112	244	100	150	250			
4.00	116	282		125	225	300		
4.50	121	320		100	200	275		
5.00	127	360		100	175	225		
5.50	133	402			150	200	300	
6.00	140	446				125	175	250
6.50	147	492				100	150	225
7.00	155	541				100	150	200

Design Table of Wingwall with TL-5 Barrier

L (m)	Vf at A-A (kN/m)	Mf at A-A (kN.m/m)	Sr (mm)						
			15M	20M	25M	30M	35M		
2.00	178	147	175	275					
2.50	173	217	125	175	300				
3.00	170	281		125	225				
3.50	169	341		100	175	250			
4.00	169	399			150	200	300		
4.50	171	454			125	175	250		
5.00	173	509				100	150	225	
5.50	177	564				100	125	200	
6.00	181	620					125	175	
6.50	186	677	Use Thicker Wingwall						
7.00	192	735	Use Thicker Wingwall						

Legend:

- L** Design length of winwall
- Vf** The factored shear force due to lateral pressure from earth and traffic loads at the fixed end (U.L.S.) per unit height of the wall
- Mf** The factored moment due to lateral pressure from earth and traffic loads at the fixed end (U.L.S.) per unit height of the wall
- Sr** Spacing of Principal Reinforcement (mm)
- TL-4, TL-5** Traffic barrier test level
- P_t** Transverse traffic loads
- B_L** Barrier Length for P_{t force}
- $\alpha_{t}^{ULS} = 1.25$ $\alpha_L = 1.70$

NOTES: THIS TABLE TO BE READ IN CONJUNCTION WITH DRAWINGS SS105-11 & SS105-12

SECTION 5 - ABUTMENTS, WINGWALLS AND RETAINING WALLS

DESIGN AID 5-2

WINGWALL DESIGN TABLE (T = 450 mm)

T = 450mm

$f_c = 30 \text{ MPa}$
 Concrete Cover = 70 mm
 $f_y = 500 \text{ MPa}$

$P_t = 100 \text{ kN}$ for TL-4 Barrier
 210 kN for TL-5 Barrier
 $B_L = 1.05 \text{ m}$ for TL-4 Barrier
 2.40 m for TL-5 Barrier

Equivalent Fluid Pressure, $W = \gamma k = 7 \text{ kPa}$

SHEAR AND MOMENT AT A-A

$$M_{A-A} = \left\{ \frac{WL^2}{24} (H^2 + 2Hh + 3h^2) + \frac{WL^2}{6} S(H + 2h) + 5.1L^2 \right\} \alpha_E + P_t \left(L - \frac{B_L}{2} \right) \alpha_L$$

$$P_{A-A} = \left\{ \frac{WL}{6} (H^2 + Hh + h^2) + \frac{WL}{2} S(H + h) + 10.2L \right\} \alpha_E + P_t \alpha_L$$

Design Table of Wingwall with TL-4 Barrier

L (m)	Vf at A-A (kN/m)	Mf at A-A (kN.m/m)	Sr (mm)				
			15M	20M	25M	30M	35M
2.00	104	133	225				
2.50	105	171	175	275			
3.00	108	207	150	225			
3.50	112	244	125	175	300		
4.00	116	282	100	150	275		
4.50	121	320		125	225		
5.00	127	360		125	200	275	
5.50	133	402		100	175	250	
6.00	140	446		100	150	225	
6.50	147	492			150	200	275
7.00	155	541			125	175	250

Design Table of Wingwall with TL-5 Barrier

L (m)	Vf at A-A (kN/m)	Mf at A-A (kN.m/m)	Sr (mm)				
			15M	20M	25M	30M	35M
2.00	178	147	200				
2.50	173	217	125	200			
3.00	170	281	100	150	275		
3.50	169	341		125	225	300	
4.00	169	399		100	175	250	
4.50	171	454			150	225	300
5.00	173	509			125	200	275
5.50	177	564			125	175	250
6.00	181	620			100	150	200
6.50	186	677			100	125	200
7.00	192	735				125	175

Legend:

- L** Design length of wingwall
- Vf** The factored shear force due to lateral pressure from earth and traffic loads at the fixed end (U.L.S.) per unit height of the wall
- Mf** The factored moment due to lateral pressure from earth and traffic loads at the fixed end (U.L.S.) per unit height of the wall
- Sr** Spacing of Principal Reinforcement (mm)
- TL-4, TL-5** Traffic barrier test level
- P_t** Transverse traffic loads
- B_L** Barrier Length for P_{t force}
- $\alpha_{t}^{ULS} = 1.25$ $\alpha_L = 1.70$

NOTES: THIS TABLE TO BE READ IN CONJUNCTION WITH DRAWINGS SS105-11 & SS105-12

SECTION 5 - ABUTMENTS, WINGWALLS AND RETAINING WALLS

DESIGN AID 5-3

WINGWALL DESIGN TABLE (T = 475 mm)

T = 475mm

$f_c = 30 \text{ MPa}$
 Concrete Cover = 70 mm
 $f_y = 500 \text{ MPa}$

$P_t = 100 \text{ kN for TL-4 Barrier}$
 $210 \text{ kN for TL-5 Barrier}$
 $B_L = 1.05 \text{ m. for TL-4 Barrier}$
 $2.40 \text{ m. for TL-5 Barrier}$

Equivalent Fluid Pressure, $W = \alpha k = 7 \text{ kPa}$

SHEAR AND MOMENT AT A-A

$$M_{A-A} = \left\{ \frac{WL^2}{24} (H^2 + 2Hh + 3h^2) + \frac{WL^2}{6} S(H + 2h) + 5.1L^2 \right\} \alpha_E + P_t \left(L - \frac{B_L}{2} \right) \alpha_L$$

$$P_{A-A} = \left\{ \frac{WL}{6} (H^2 + Hh + h^2) + \frac{WL}{2} S(H + h) + 10.2L \right\} \alpha_E + P_t \alpha_L$$

Design Table of Wingwall with TL-4 Barrier

L (m)	Vf at A-A (kN/m)	Mf at A-A (kN.m/m)	Sr (mm)				
			15M	20M	25M	30M	35M
2.00	104	133	250				
2.50	105	171	200	300			
3.00	108	207	150	225			
3.50	112	244	125	200			
4.00	116	282	100	175	275		
4.50	121	320	100	150	250		
5.00	127	360		125	225	300	
5.50	133	402		100	200	275	
6.00	140	446		100	175	250	
6.50	147	492			150	225	300
7.00	155	541			125	200	275

Design Table of Wingwall with TL-5 Barrier

L (m)	Vf at A-A (kN/m)	Mf at A-A (kN.m/m)	Sr (mm)				
			15M	20M	25M	30M	35M
2.00	178	147	225				
2.50	173	217	150	225			
3.00	170	281	100	175	275		
3.50	169	341		125	225		
4.00	169	399		100	200	275	
4.50	171	454		100	175	225	
5.00	173	509			150	200	300
5.50	177	564			125	175	250
6.00	181	620			125	150	225
6.50	186	677			100	150	200
7.00	192	735			100	125	200

L Design length of wingwall

Vf The factored shear force due to lateral pressure from earth and traffic loads at the fixed end (U.L.S.) per unit height of the wall

Mf The factored moment due to lateral pressure from earth and traffic loads at the fixed end (U.L.S.) per unit height of the wall

Sr Spacing of Principal Reinforcement (mm)

TL-4, TL-5 Traffic barrier test level

P_t Transverse traffic loads

B_L Barrier Length for P_{t force}

$\alpha_E^{ULS} = 1.25$ $\alpha_L = 1.70$

NOTES: THIS TABLE TO BE READ IN CONJUNCTION WITH DRAWINGS SS105-11 & SS105-12

DESIGN AID 5-4

SECTION 5 - ABUTMENTS, WINGWALLS AND RETAINING WALLS

WINGWALL DESIGN TABLE (T = 500 mm)

T = 500mm

$f_c = 30 \text{ MPa}$
 Concrete Cover = 70 mm
 $f_y = 500 \text{ MPa}$

$P_t = 100 \text{ kN for TL-4 Barrier}$
 $210 \text{ kN for TL-5 Barrier}$

$B_L = 1.05 \text{ m. for TL-4 Barrier}$
 $2.40 \text{ m. for TL-5 Barrier}$

Equivalent Fluid Pressure, $W = \gamma k = 7 \text{ kPa}$

SHEAR AND MOMENT AT A-A

$$M_{A-A} = \left\{ \frac{WL^2}{24} (H^2 + 2Hh + 3h^2) + \frac{WL^2}{6} S(H + 2h) + 5.1L^2 \right\} \alpha_E + P_t \left(L - \frac{B_L}{2} \right) \alpha_L$$

$$P_{A-A} = \left\{ \frac{WL}{6} (H^2 + Hh + h^2) + \frac{WL}{2} S(H + h) + 10.2L \right\} \alpha_E + P_t \alpha_L$$

Design Table of Wingwall with TL-4 Barrier

L (m)	Vf at A-A (kN/m)	Mf at A-A (kN.m/m)	Sr (mm)				
			15M	20M	25M	30M	35M
2.00	104	133	275				
2.50	105	171	200	300			
3.00	108	207	175	250			
3.50	112	244	125	200			
4.00	116	282	125	175	300		
4.50	121	320	100	150	275		
5.00	127	360		125	225		
5.50	133	402		125	200	300	
6.00	140	446		100	175	250	
6.50	147	492		100	175	225	
7.00	155	541			150	200	300

Design Table of Wingwall with TL-5 Barrier

L (m)	Vf at A-A (kN/m)	Mf at A-A (kN.m/m)	Sr (mm)				
			15M	20M	25M	30M	35M
2.00	178	147	250				
2.50	173	217	150	250			
3.00	170	281	125	175	300		
3.50	169	341	100	150	250		
4.00	169	399		125	200	300	
4.50	171	454		100	175	250	
5.00	173	509		100	150	225	
5.50	177	564			150	200	275
6.00	181	620			125	175	250
6.50	186	677			100	150	225
7.00	192	735			100	150	200

NOTES: THIS TABLE TO BE READ IN CONJUNCTION WITH DRAWINGS SS105-11 & SS105-12

L Design length of wingwall
Vf The factored shear force due to lateral pressure from earth and traffic loads at the fixed end (U.L.S) per unit height of the wall
Mf The factored moment due to lateral pressure from earth and traffic loads at the fixed end (U.L.S) per unit height of the wall
Sr Spacing of Principal Reinforcement (mm)
TL-4, TL-5 Traffic barrier test level
P_t Transverse traffic loads
B_L Barrier Length for P_{t force}
 $\alpha_{E}^{ULS} = 1.25$ $\alpha_L = 1.70$

SECTION 5 - ABUTMENTS, WINGWALLS AND RETAINING WALLS

DESIGN AID 5-5

WINGWALL DESIGN TABLE (T = 550 mm)

T = 550mm

$f_c = 30 \text{ MPa}$
 Concrete Cover = 70 mm
 $f_y = 500 \text{ MPa}$

$P_t = 100 \text{ kN for TL-4 Barrier}$
 $210 \text{ kN for TL-5 Barrier}$

$B_L = 1.05 \text{ m. for TL-4 Barrier}$
 $2.40 \text{ m. for TL-5 Barrier}$

Equivalent Fluid Pressure, $W = \alpha k = 7 \text{ kPa}$

SHEAR AND MOMENT AT A-A

$$M_{A-A} = \left\{ \frac{WL^2}{24} (H^2 + 2Hh + 3h^2) + \frac{WL^2}{6} S(H + 2h) + 5.1L^2 \right\} \alpha_E + P_t \left(L - \frac{B_L}{2} \right) \alpha_L$$

$$P_{A-A} = \left\{ \frac{WL}{6} (H^2 + Hh + h^2) + \frac{WL}{2} S(H + h) + 10.2L \right\} \alpha_E + P_t \alpha_L$$

Design Table of Wingwall with TL-4 Barrier

L (m)	Vf at A-A (kN/m)	Mf at A-A (kN.m/m)	Sr (mm)				
			15M	20M	25M	30M	35M
2.00	104	133	300				
2.50	105	171	225				
3.00	108	207	175	275			
3.50	112	244	150	250			
4.00	116	282	125	200			
4.50	121	320	125	175	300		
5.00	127	360	100	150	275		
5.50	133	402	100	150	225		
6.00	140	446		125	200	300	
6.50	147	492		100	200	275	
7.00	155	541		100	175	250	

Design Table of Wingwall with TL-5 Barrier

L (m)	Vf at A-A (kN/m)	Mf at A-A (kN.m/m)	Sr (mm)				
			15M	20M	25M	30M	35M
2.00	178	147	275				
2.50	173	217	175	275			
3.00	170	281	125	200			
3.50	169	341	100	175	275		
4.00	169	399	100	150	250		
4.50	171	454		125	200	300	
5.00	173	509		100	175	250	
5.50	177	564		100	150	225	
6.00	181	620			150	200	300
6.50	186	677			125	175	275
7.00	192	735			125	175	250

Legend:

- L** Design length of wingwall
- Vf** The factored shear force due to lateral pressure from earth and traffic loads at the fixed end (U.L.S.) per unit height of the wall
- Mf** The factored moment due to lateral pressure from earth and traffic loads at the fixed end (U.L.S.) per unit height of the wall
- S** Spacing of Principal Reinforcement (mm)
- TL-4, TL-5** Traffic barrier test level
- P_t** Transverse traffic loads
- B_L** Barrier Length for P_{t force}
- $\alpha_{E}^{ULS} = 1.25$ $\alpha_L = 1.70$

NOTES: THIS TABLE TO BE READ IN CONJUNCTION WITH DRAWINGS SS105-11 & SS105-12

SECTION 5 - ABUTMENTS, WINGWALLS AND RETAINING WALLS

DESIGN AID 5-6

WINGWALL DESIGN TABLE (T = 600 mm)

T = 600mm

$f_c = 30 \text{ MPa}$
 Concrete Cover = 70 mm
 $f_y = 500 \text{ MPa}$

$P_t = 100 \text{ kN for TL-4 Barrier}$
 $210 \text{ kN for TL-5 Barrier}$
 $B_L = 1.05 \text{ m. for TL-4 Barrier}$
 $2.40 \text{ m. for TL-5 Barrier}$

Equivalent Fluid Pressure, $W = \gamma k = 7 \text{ kPa}$

SHEAR AND MOMENT AT A-A

$$M_{A-A} = \left\{ \frac{WL^2}{24} (H^2 + 2Hh + 3h^2) + \frac{WL^2}{6} S(H + 2h) + 5.1L^2 \right\} \alpha_E + P_t \left(L - \frac{B_L}{2} \right) \alpha_L$$

$$P_{A-A} = \left\{ \frac{WL}{6} (H^2 + Hh + h^2) + \frac{WL}{2} S(H + h) + 10.2L \right\} \alpha_E + P_t \alpha_L$$

Design Table of Wingwall with TL-4 Barrier

L (m)	Vf at A-A (kN/m)	Mf at A-A (kN.m/m)	Sr (mm)				
			15M	20M	25M	30M	35M
2.00	104	133					
2.50	105	171	250				
3.00	108	207	200				
3.50	112	244	175	275			
4.00	116	282	150	225			
4.50	121	320	125	200			
5.00	127	360	125	175	300		
5.50	133	402	100	150	275		
6.00	140	446	100	150	225		
6.50	147	492		125	200	300	
7.00	155	541		100	200	275	

Design Table of Wingwall with TL-5 Barrier

L (m)	Vf at A-A (kN/m)	Mf at A-A (kN.m/m)	Sr (mm)				
			15M	20M	25M	30M	35M
2.00	178	147	300				
2.50	173	217	200	300			
3.00	170	281	150	225			
3.50	169	341	125	175			
4.00	169	399	100	150	275		
4.50	171	454		125	225		
5.00	173	509		125	200	300	
5.50	177	564		100	175	250	
6.00	181	620		100	175	225	
6.50	186	677			150	200	300
7.00	192	735			125	200	275

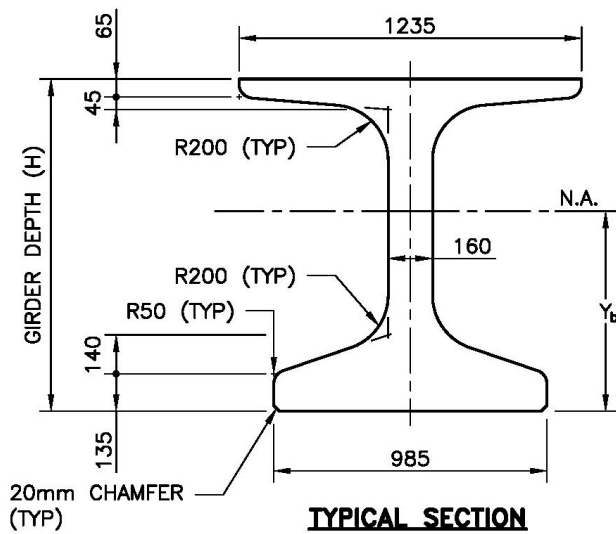
NOTES: THIS TABLE TO BE READ IN CONJUNCTION WITH DRAWINGS SS105-11 & SS105-12

SECTION 5 - ABUTMENTS, WINGWALLS AND RETAINING WALLS

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SECTION 7 - PRESTRESSED CONCRETE

DESIGN AID 7-1
PRESTRESSED NU GIRDERS



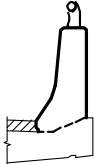
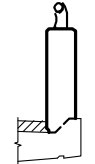
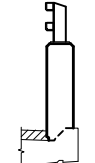
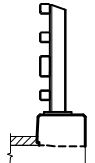
SECTION PROPERTIES

GIRDER DEPTH H (mm)	AREA (mm ²)	Y_b (mm)	MOMENT OF INERTIA ABOUT N.A. (mm ⁴)
900	427747	410.2	4.651 E+10
1200	475747	542.4	9.565 E+10
1400	507747	632.4	1.400 E+11
1600	539747	723.6	1.945 E+11
1800	571747	815.7	2.596 E+11
1900	587747	862.2	2.964 E+11
2000	603747	908.8	3.362 E+11
2400	667747	1096.8	5.260 E+11

SECTION 10 – BARRIERS AND RAILING SYSTEMS

DESIGN AID 10-1
PERMANENT BARRIERS DEAD LOAD

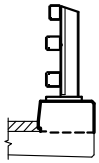
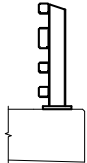
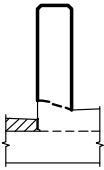
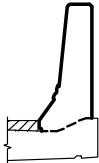
Note: Barrier Dead Load is based on barrier/curb concrete above the dashed line as marked

Barrier Type	Related Structural Standard Drawing	Barrier Shape	DL of Concrete Wall/curb (kN/m)	DL of Steel (kN/m)	Total Weight of Barrier (kN/m)
Barrier Wall With Railing, TL-4	SS110-21, SS110-54,58,68,69,91		6.1	0.3	6.4
Parapet Wall With Railing, TL-4	SS110-21, SS110-56,57,59,65,75,90,97,98,104,105		5.2	0.3	5.4
Parapet Wall for Combination Traffic/Bicycle Rail, TL-4	SS110-85, SS110-82,83,84		5.2	0.3	5.5
Four Tube Combination Traffic/Bicycle Railing, TL-4	SS110-34,36		3.4	0.9	4.3

SECTION 10 – BARRIERS AND RAILING SYSTEMS

DESIGN AID 10-1
PERMANENT BARRIERS DEAD LOAD

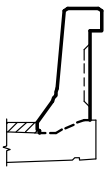
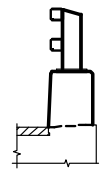

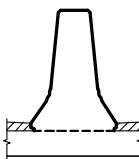
Note: Barrier Dead Load is based on barrier/curb concrete above the dashed line as marked

Barrier Type	Related Structural Standard Drawing	Barrier Shape	DL of Concrete Wall/curb (kN/m)	DL of Steel (kN/m)	Total Weight of Barrier (kN/m)
Three Tube Railing on Curb, TL-4	SS110-37,39		3.4	0.7	4.0
Four Tube Railing on Sidewalk, TL-4	SS110-46,49		N/A	0.9	0.9
Multi Use Path (MUP) Separation Barrier (Parapet), TL-4	SS110-110		5.5	N/A	5.5
Barrier Wall Without Railing, TL-5	SS110-92,94,109		7.9	N/A	7.9

SECTION 10 – BARRIERS AND RAILING SYSTEMS

DESIGN AID 10-1
PERMANENT BARRIERS DEAD LOAD

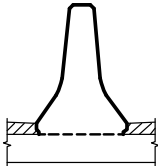
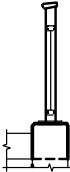
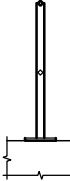
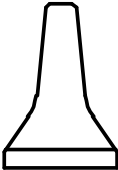
Note: Barrier Dead Load is based on barrier/curb concrete above the dashed line as marked

Barrier Type	Related Structural Standard Drawing	Barrier Shape	DL of Concrete Wall/curb (kN/m)	DL of Steel (kN/m)	Total Weight of Barrier (kN/m)
Barrier Wall With Architectural Finish, TL-5	SS110-70,71,72, 73 and 74,93,99		8.7	N/A	8.7
Parapet Wall for Two Tube Railing, TL-5	S110-95,96		7.0	0.6	7.6
Multi-Use Path (MUP), Separation Barrier, TL-5	SS110-111		7.8	N/A	7.8
Reinforced Concrete Median Barrier Wall on Structures - Type I	SS110-62		12.2	N/A	12.2

SECTION 10 – BARRIERS AND RAILING SYSTEMS

DESIGN AID 10-1
PERMANENT BARRIERS DEAD LOAD

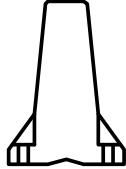
Note: Barrier Dead Load is based on barrier/curb concrete above the dashed line as marked

Barrier Type	Related Structural Standard Drawing	Barrier Shape	DL of Concrete Wall/curb (kN/m)	DL of Steel (kN/m)	Total Weight of Barrier (kN/m)
Reinforced Concrete Median Barrier Wall on Structures - Type II	SS110-63		6.9	N/A	6.9
Multi Use Path (MUP) Bicycle Barrier	SS0110.0022		3.5	0.8	4.2
Inspector Guard Details	SS110-33		N/A	0.4	0.4
Guide Rail System, Concrete Barrier, Type J Connection, 4 m Length Installation, Temporary	OPSD 911.150		6.9	N/A	6.9

SECTION 10 – BARRIERS AND RAILING SYSTEMS

DESIGN AID 10-1
PERMANENT BARRIERS DEAD LOAD

Note: Barrier Dead Load is based on barrier/curb concrete above the dashed line as marked

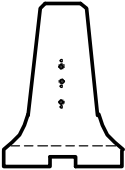
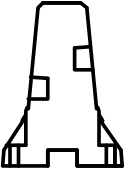
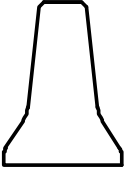
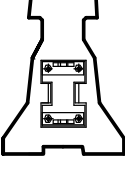
Barrier Type	Related Structural Standard Drawing	Barrier Shape	DL of Concrete Wall/curb (kN/m)	DL of Steel (kN/m)	Total Weight of Barrier (kN/m)
Guide Rail System, Concrete Barrier, Cast-In-Place, Type M, Installation	OPSD 911.16		6.8	N/A	6.8

SECTION 10 – BARRIERS AND RAILING SYSTEMS

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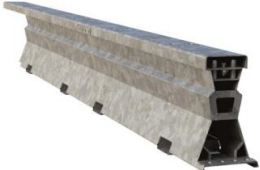
SECTION 10 – BARRIERS AND RAILING SYSTEMS

DESIGN AID 10-2
TEMPORARY BARRIERS DEAD LOAD

Barrier Type	Related Structural Standard Drawing	Barrier Shape/Photo	DL of Barrier (kN/m)
Guide Rail System, Concrete Barrier Type T Connection, 4.0m Length Installation - Temporary	OPSD 0911.1800		6.8
Guide Rail System, Concrete Barrier, Type X Connection, 4.0 m Length, Installation, Temporary	MTOD 911.194		6.8
Guide Rail System, Concrete Barrier, Type Z, 12.0 m Length, Installation, Temporary	MTOD 911.201		5.5
Guide Rail System, Concrete Barrier, QuickChange Moveable Barrier, Installation, Temporary	OPSD 911.610		6.7

SECTION 10 – BARRIERS AND RAILING SYSTEMS

DESIGN AID 10-2
TEMPORARY BARRIERS DEAD LOAD

Barrier Type	Related Structural Standard Drawing	Barrier Shape/Photo	DL of Barrier (kN/m)
BarrierGuard 800	RDM 5.1.4.1		0.9
ZoneGuard	RDM 5.1.4.2	<p>Figure 5-14: BarrierGuard 800 Temporary Steel Barrier</p>  <p>Figure 5-15: ZoneGuard Temporary Steel Barrier Installation Photo credit: Hill & Smith</p>	0.9
Defender	RDM 5.1.4.3	 <p>Figure 5-16: Defender Barrier Photo Courtesy of Safe Barriers</p>	0.8
Safezone	RDM 5.1.4.4	 <p>Figure 5-17: Safezone Barrier Photo courtesy of Laura Metaal</p>	0.8

DESIGN AID 10-2

SECTION 10 – BARRIERS AND RAILING SYSTEMS

TEMPORARY BARRIERS DEAD LOAD

<p>HV2</p>	<p>RDM 5.1.4.5</p>	 <p>Figure 5-18: HV2 Barrier Photo Courtesy of Safe Roads</p>	<p>3.5</p>
<p>Highway Guard</p>	<p>RDM 5.1.4.6</p>	 <p>Figure 5-19: Highway Guard Barrier Photo Courtesy of HighwayCare</p>	<p>0.9</p>

SECTION 12 - REINFORCING STEEL

DESIGN AID 12-1
REINFORCING STEEL BARS

PROPERTIES OF REINFORCING STEEL BARS

BAR SIZE	MASS (kg/m)	NOMINAL DIMENSIONS		
		DIAMETER (mm)	CROSS SECTIONAL AREA (mm ²)	PERIMETER (mm)
10M	0.785	11.3	100	35.5
15M	1.570	16.0	200	50.1
20M	2.355	19.5	300	61.3
25M	3.925	25.2	500	79.2
30M	5.495	29.9	700	93.9
35M	7.850	35.7	1000	112.2
45M	11.775	43.7	1500	137.3
55M	19.625	56.4	2500	177.2

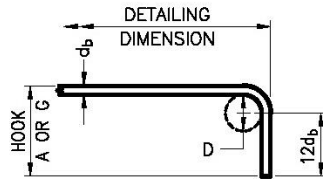
STEEL AREA PER METRE WIDTH (mm²)

SPACING (mm)	BAR SIZE							
	10M	15M	20M	25M	30M	35M	45M	55M
80	1250	2500	3750	6250	8750	12500	—	—
100	1000	2000	3000	5000	7000	10000	15000	—
120	833	1667	2500	4167	5833	8333	12500	20833
125	800	1600	2400	4000	5600	8000	12000	20000
140	714	1429	2143	3571	5000	7143	10714	17857
150	667	1333	2000	3333	4667	6667	10000	16667
160	625	1250	1875	3125	4375	6250	9375	15625
175	571	1143	1714	2857	4000	5714	8571	14286
180	556	1111	1667	2778	3889	5556	8333	13889
200	500	1000	1500	2500	3500	5000	7500	12500
220	455	909	1364	2273	3182	4545	6818	11364
225	444	889	1333	2222	3111	4444	6667	11111
240	417	833	1250	2083	2917	4167	6250	10417
250	400	800	1200	2000	2800	4000	6000	10000
260	385	769	1154	1923	2692	3846	5769	9615
275	364	727	1091	1818	2545	3636	5455	9091
280	357	714	1071	1786	2500	3571	5357	8929
300	333	667	1000	1667	2333	3333	5000	8333
350	286	571	857	1429	2000	2857	4286	7143
400	250	500	750	1250	1750	2500	3750	6250
450	222	444	667	1111	1556	2222	3333	5556
500	200	400	600	1000	1400	2000	3000	5000

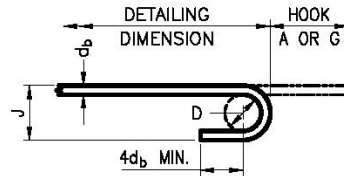
SECTION 12 - REINFORCING STEEL

DESIGN AID 12-2

HOOK DIMENSIONS FOR REINFORCING STEEL BARS



STANDARD 90° HOOK



STANDARD 180° HOOK

MINIMUM BENDING PIN DIAMETER, D, mm

BAR SIZE	STEEL GRADE	
	400R ⁽²⁾	400W
10M	70	60
15M	100	90
20M	120	100
25M	150	150
30M	250	200
35M	300	250
45M	450 ⁽¹⁾	400
55M	600 ⁽¹⁾	550

(1) Special fabrication is required for bends exceeding 90° for bars of these sizes and grade.

(2) For stainless steel, with $F_y = 420$, use the same D as for 400R.

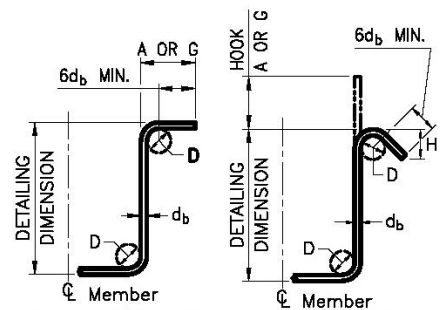
STANDARD HOOK DIMENSIONS

BAR SIZE	90° HOOKS		180° HOOKS			
	A OR G (mm)		A OR G (mm)		J (mm)	
	400R	400W	400R	400W	400R	400W
10M	180	180	140	130	90	80
15M	260	250	180	170	130	120
20M	310	300	220	200	160	140
25M	400	400	280	280	200	200
30M	510	490	400	350	310	260
35M	610	590	480	430	370	320
45M	790	770	680	630	540	490
55M	1030	1010	900	850	710	660

NOTE: All Hook Dimensions are according to the CHBDC-2000.

MINIMUM STIRRUP AND TIE HOOK DIMENSIONS

BAR SIZE	BAR DIAM. d_b (mm)	PIN DIAM. D (mm)	90°		135°
			A OR G (mm)	A OR G (mm)	H (approx.) (mm)
10M	11.3	45	100	100	70
15M	16.0	65	140	140	100
20M	19.5	80	180	175	115
25M	25.2	100	230		



MIN. 90° HOOK MIN. 135° HOOK

SECTION 16 - MISCELLANEOUS

DESIGN AID 16-5
IMPERIAL - S.I. CONVERSION FACTORS

SUBJECT	CONVERSION FACTORS	SUBJECT	CONVERSION FACTORS
<u>LINEAR</u>	1 in. = 25.4 mm 1 ft. = 304.8 mm = 0.3048 m 1 mile = 1.609 km	<u>AREA</u>	1 in. ² = 645.16 mm ² 1 ft. ² = 0.0929 m ²
<u>VOLUME</u>	1 in. ³ = 16.3871 cm ³ = 16387.1 mm ³ 1 ft. ³ = 0.0283 m ³ 1 yd. ³ = 0.7646 m ³	<u>MASS</u>	1 lb. = 0.4536 kg 1 kip = 453.6 kg 1 ton = 907.2 kg
<u>MASS</u> <u>DENSITY</u>	1 lb./ft. ³ = 16.018 kg/m ³ 1 lb./in. ³ = 27.680 kg/cm ³	<u>FORCE</u>	1 lb. = 4.44822 N 1 kip = 4448.22 N 1 N = 0.10197 kg 1 kg = 9.8067 N 1 kg = 2.2046 lb.
<u>SPEED</u>	1 mi./h = 1.609 km/h 1 ft./s = 0.305 m/s		
<u>LOADS</u> (U.D.L.)	1 kip/in. = 175.1 N/mm 1 kip/ft. = 14.5939 kN/m 1 lb./ft. = 14.5939 N/m	<u>BENDING</u> <u>MOMENT</u> OR <u>TORSION</u>	1 kip·in. = 112.985 N·m 1 kip·ft. = 1.35582 kN·m
<u>PRESSURE</u> OR <u>STRESS</u> OR <u>MODULUS</u> OF <u>ELASTICITY</u>	1 psi = 6.89476 kPa 1 ksi = 6.89476 MPa 1 psf = 47.88 Pa 1 ksf = 47.88 kPa	<u>MOMENT</u> OF <u>INERTIA</u>	1 in. ⁴ = 416231.43 mm ⁴
		<u>SECTION</u> <u>MODULUS</u>	1 in. ³ = 16387.064 mm ³
		<u>TEMPERATURE</u>	°C = 5/9 (°F - 32)
SELECTED SI PREFIXES			<u>THERMAL</u> <u>EXPANSION</u>
<u>Multiplying</u> <u>Factor</u>	<u>Prefix</u>	<u>Symbol</u>	in./in. °F = 5.556 x 10 ⁻¹ m/m °C
10 ⁹	giga	G	Thermal Coefficient of Linear Expansion for Concrete: 6.667 x 10 ⁻⁶ /°F = 12 x 10 ⁻⁶ /°C
10 ⁶	mega	M	
10 ³	kilo	k	NOTE: 1 Pa = 1 N/m ² 1 kip = 1000 lb. 1 ton = 2000 lb. 1 metric tonne = 1000 kg
10 ²	hecto	h	
10 ⁻¹	deci	d	
10 ⁻²	centi	c	
10 ⁻³	milli	m	

SECTION 16 - MISCELLANEOUS

DESIGN AID 16-10
 PROPERTIES OF A PARABOLIC CURVE

