# STRUCTURAL MANUAL




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

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## STRUCTURAL MANUAL ENQUIRIES

To all users of the: **STRUCTURAL MANUAL**

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# RECORD OF REVISIONS

This Structural Manual dated December 2025 includes Revision #61 and supersedes the previous edition. Minor changes were made from the previous edition including clarifications, corrections and visual improvement of figures throughout the manual. Major changes are summarised as follows:

## DIVISION 1 - EXCEPTIONS TO THE CHBDC

Section 1:

- Few Definitions and clauses are changed based on MTO's Policies
- Exceptions to the Hydrology Clause generally defer to MTO's Highway Drainage Design Standards; annual exceedance probabilities are based on highway classification rather than structure importance category

Section 3:

- CL-625-ONT Truck for Ontario Bridges has been added in CL-W Truck Clause

Section 4:

- Slight adjustment to calculation of Seismic Performance Category (SPC).

Section 5:

- Clause 5.7.1.5.2 a) has been modified to allow rational analysis while calculating factored transverse bending moment

Section 10:

- Additional grades of Steel Piles have been added in Clause 10.4.14
- Durability requirements for deck drains & railings

Low Volume:

- Minor change in TL1 barrier warrants.

Section 14:

- CL-625-ONT has been added

Section 16:

- Exceptions has been made in Clauses for development length & splices of FRP bars, FRP Ties & spirals

## DIVISION 2 - PROCEDURES

Section 2:

- Construction loading design considerations for slab-on-girder bridges have been added.
- Section about Bridge Hydrology and Hydraulics has been added
- Requirements of structural drawings contents has been added

Section 5:

- Updates on Integral Abutment Policy have been included

## RECORD OF REVISIONS

Section 6:

- Requirements for Pier Column Reinforcing Details have been added

Section 7:

- Durability requirements for Precast girder's concrete has been updated
- Note added to describe assumed construction loads for girder design

Section 8:

- Added information Hot Dip Galvanizing
- Note added to describe assumed construction loads for girder design

Section 9:

- A new Design Aid for deck cantilever design has been introduced

Section 12:

- Use of Premium Reinforcing steel policy has been updated to address durability requirements.
- A new Section has been added on the topic of composite construction with GFRP reinforcement
- Updates on GFRP reinforcement design

Section 16:

Structural grounding requirements have been updated

Section 17:

- Table 1 revised to be consistent with EQ probabilities of occurrence from the EQ Canada webtool.

**DIVISION 3 - DESIGN AIDS**

- Added design aid DA 9-1 & DA 9-2 for Deck Cantilever reinforcement design

**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.

## RECORD OF REVISIONS

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## 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

| | |
|---|---|
| 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 5.3) |
| MTOD | Ministry of Transportation of Ontario Drawing |

## 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 |
| RTAC | Roads and Transportation Association of Canada |
| RSS | Retained Soil System (also known as MSE, see Section 5.3) |
| 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 S6-25

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# SECTION 1 - EXCEPTIONS TO THE CANADIAN HIGHWAY BRIDGE DESIGN CODE (CHBDC), CSA S6-25

## 1 EXCEPTIONS TO THE CHBDC, CSA S6-25

### 1.1 Scope

These exceptions implement provisions that CSA S6-25, 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 and Abbreviations**

**1.3.1 Definitions**

The definition for **Capacity** is deleted.

The definition for **Checker** is added as follows:

**Checker** — a member or licensee of the engineering association who carries out the design check.

# SECTION 1 - EXCEPTIONS TO THE CANADIAN HIGHWAY BRIDGE DESIGN CODE (CHBDC), CSA S6-25

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

**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. Refer to Figure 1

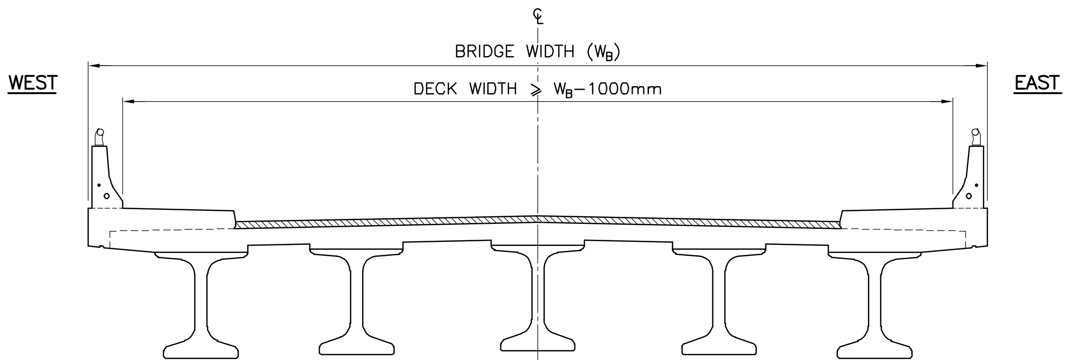



Figure 1 Deck Width

The “note” in the definition of **Engineer** is deleted and replaced with the following:

Note: This includes “designer”, “design engineer”, “engineer of record”, “checker”, and “design checker”.

The definition for **Regulatory Authority** is deleted and replaced with the following:

The Ministry of Transportation of Ontario and it’s Minister as represented by their Chief Engineer, who has jurisdiction and control over design and construction requirements.

## 1.4 General Requirements

### 1.4.4 Construction

#### 1.4.4.6 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.

# SECTION 1 - EXCEPTIONS TO THE CANADIAN HIGHWAY BRIDGE DESIGN CODE (CHBDC), CSA S6-25

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 Transportation Associate of Canada's *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 Transportation Association of Canada's *Geometric Design Guide for Canadian Roads*." are deleted and replaced by "the MTO's *Roadside Design Manual* or other standard approved by the Owner."

## 1.7 Bridge Deck Drainage

### 1.7.3 Drainage Systems

#### 1.7.3.1 General

The first paragraph is deleted and replaced with the following:

The maximum allowable spread distance onto the travel lanes shall be in accordance with the MTO *Highway Drainage Design Standard WC-4 Bridge Deck Drainage*.

## 1.9 Hydraulic Design

The following text is added to this clause:

Throughout clause 1.9 and all of its subclauses phrases such as:

- "using methods specified by the owner",
- "using methods approved by the owner",
- "unless otherwise specified by the owner",
- "unless otherwise approved by the owner",
- "shall comply with the requirements of the owner",
- "shall comply with the recommendations of the owner",

- “when approved by the owner”,
- “in accordance with the owner”,
- or any other similar phrase similar to those above,

shall be interpreted to give deference to, or direct the use of, the MTO’s *Highway Drainage Design Standards*. Where a requirement or constraint is defined or described by the *Highway Drainage Design Standards*, that shall be deemed as the “specified”, or “approved” or “recommendation” or similar of the owner. Where the requirement or constraint is not defined by the *Highway Drainage Design Standards*, then the designer shall seek direction from the owner.

### 1.9.3 Hydrology

#### 1.9.3.1 General

The second paragraph is deleted and replaced with the following:

For temporary structures, the design flood discharge shall be in accordance with the MTO *Highway Drainage Design Standard TW-1 Temporary Flow Passage Systems and Temporary Drainage Facilities*.

#### 1.9.3.3 General

The second paragraph is deleted and replaced with the following:

For future climate conditions, when design flood discharge is governed by rainfall-runoff response, the design flood discharge shall be determined using methods specified by the owner, or when approved, by applying a climate change temperature scaling approach to historic rainfall records as per CSA PLUS 4013.

### 1.9.4 Hydraulic Design

The second and third paragraph are deleted and replaced with the following:

Acceptability criteria for the performance of structures in withstanding a design flood shall be in accordance with Clause 1.9.5 Performance-Based Design Method. A lifeline structure may on approval by the owner be designed in accordance with Clause 1.9.6 Service Level Design and the difficulty in providing a design in accordance with Clause 1.9.5 shall be demonstrated to the owner.

### 1.9.5 Performance-Based Design Method

#### 1.9.5.1 Design High-Water Elevation

Table 1.3 “Annual Exceedance Probability” is deleted and replaced with the following:

| Hazard | Design criteria | Annual exceedance probability |
|---|---|---|
| Minimum structure sizing | 1.9.5.2 | MTO *Highway Drainage Design Standards WC-1* Table WC1-1 “Design Flows for Bridges and |

| | | |
|---|---|---|
| | | Culverts" column "Design Flow for Structure Sizing, Flood Depth, Clearance, Freeboard" |
| Backwater | 1.9.5.3 | MTO *Highway Drainage Design Standards WC-1* Table WC1-1 "Design Flows for Bridges and Culverts" column "Backwater Assessment Range of Flows" |
| Approach grade elevation | 1.9.5.4 | MTO *Highway Drainage Design Standards WC-1* Table WC1-1 "Design Flows for Bridges and Culverts" column "Design Flow for Structure Sizing, Flood Depth, Clearance, Freeboard" |
| Relief flow | 1.9.5.5 | As described in the MTO *Highway Drainage Design Standards* Definitions Section for "Regulatory Flow (or Regulatory Flood)". |
| Scour | 1.9.5.6 and 1.9.5.7 | MTO *Highway Drainage Design Standards WC-1* Table WC1-1 "Design Flows for Bridges and Culverts" column "Scour Analysis and Check Flow" |
| Channel erosion and piping | 1.9.5.8 | MTO *Highway Drainage Design Standards WC-1* Table WC1-1 "Design Flows for Bridges and Culverts" column "Design Flow for Structure Sizing, Flood Depth, Clearance, Freeboard" |

### 1.9.5.2 Minimum Structure Size

#### 1.9.5.2.2 Minimum Clearance

The clause is deleted and replaced with the following:

Clearance shall be in accordance with the MTO's *Highway Drainage Design Standards*.

Vertical clearance for a structure on a navigable waterway shall be measured from the highest water level at which usual navigation is likely to occur. This level, together with the vertical and horizontal clearances, shall be determined in accordance with the Government of Canada's Navigable Waters Protection Act.

### 1.9.5.4 Approach Grade Elevation

#### 1.9.5.4.3 Approach grade elevation freeboard

The clause is deleted and replaced with the following:

Freeboard shall be in accordance with the MTO *Highway Drainage Design Standard, WC-2 Freeboard and Clearance at Bridge Crossings* and *WC-7 Culvert Crossings*.

### 1.9.5.5 Relief Flow

#### 1.9.5.5.1 General

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

Relief Flow shall be in accordance with the MTO's *Highway Drainage Design Standards*.

**1.9.5.7 Protection Against Scour**

**1.9.5.7.2 Shallow Foundations**

**1.9.5.7.2.1 Depth of Footings**

List item a) ii) is deleted and replaced with the following:

a depth below the original bed not less than 1.7 times the estimated total depth of scour; and

List item c is deleted and replaced with the following:

c) Temporary structures: for temporary piers and abutments constructed of gabions or timber cribs, the depths specified in Items a) i) and a) iii) shall be reduced by half, and the factor specified in Item a) ii) shall be reduced from 1.7 to 1.3.

**1.9.6 Service Level Design**

**1.9.6.1 General**

The following shall be added to the start of the paragraph:

Service level design shall only be used if approved by the owner.

**1.9.6.2 Minimum Service Level**

Table 1.4 "Minimum Service Level" is deleted and replaced with the following:

| **Lifeline** | |
|---|---|
| **Service** | **Damage** |
| Immediate | Minimal |

**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**

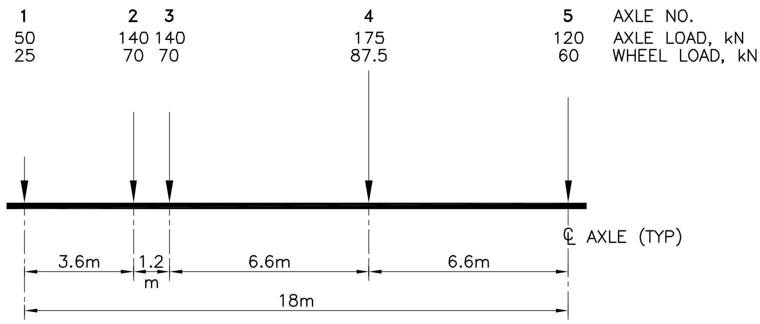
The last three paragraphs are deleted and replaced with the following:

## SECTION 1 - EXCEPTIONS TO THE CANADIAN HIGHWAY BRIDGE DESIGN CODE (CHBDC), CSA S6-25

A loading exceeding CL625 may be specified by the Owner.

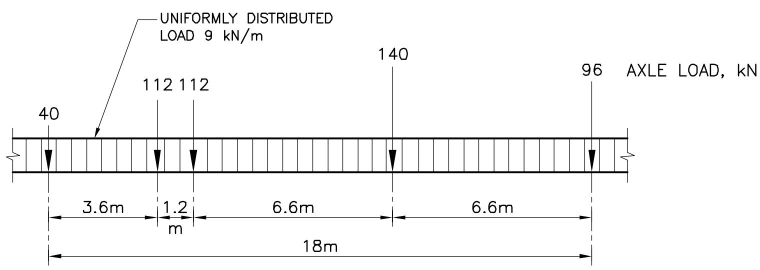
### 3.8.3.1.2 CL-W truck

The CL-W and CL-625 Truck shall be replaced with CL-625-ONT in all the clauses of the Code The loading for the CL-625 truck shown in Figure 3.2 is deleted and replaced with the following configuration of CL-625-ONT:




### 3.8.3.1.3 CL-W lane load

The CL-W lane load shall be replaced with CL-625-ONT lane load in all the clauses of the Code The lane load condition shown in Figure 3.3 is deleted and replaced with the following:




### 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.

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

### 4.18.1 Seismic evaluation of existing bridges

#### 4.18.1.3 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 4.1. 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.

5.7.1.5 Load effects in deck slab overhang due to traffic barrier loads

## SECTION 1 - EXCEPTIONS TO THE CANADIAN HIGHWAY BRIDGE DESIGN CODE (CHBDC), CSA S6-25

5.7.1.5.2 Factored transverse moments and tensile forces

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

a) The applied factored transverse bending moment (kNm per m along the slab in the direction of travel) and associated transverse tensile force (kN/m) at the traffic face of the barrier, resulting from the transverse vehicle collision load on the concrete bridge barrier specified in Clause 3.8.8.1 and applied in accordance with Figure 12.3, shall be obtained from Table 5.21 or from rational analysis. For $L_b$ greater than the upper limit, the upper limit shall be used in the equations.

**CHBDC SECTION 6 FOUNDATIONS AND GEOTECHNICAL SYSTEMS**

**6.9 Geotechnical Resistance**

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

The geotechnical resistance factor for static tests of compression and tension limit states in deep foundations may be increased by up to 0.05 for typical and high degree of understanding, subject to approval by the Owner.

**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.

**7.9 Reinforced Concrete Pipe, Boxes, and Three-Sided Buried Structures**

**7.9.11 Strength Design**

**7.9.11.2 Design for Shear**

**7.9.11.2.2 Box Structures without Stirrups or Ties**
**7.9.11.2.2.1 Shear strength for box structures**

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**

**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 and ASTM 955M are only permitted when approved for use by the owner. **8.4.2.1.7 Galvanized reinforcing steel**

This clause is amended by the addition of the following:

Galvanized reinforcing steel is only permitted when approved for use by the owner .

**8.4.4 Anchorages, mechanical connectors, and ducts**
**8.4.4.8 Ducts**
**8.4.4.8.4 Plastic Sheaths**

Clause 8.4.4.8.4 is amended by addition of the following:
Plastic sheaths may be manufactured from virgin, unfilled, non-coloured polypropylene meeting the requirements of ASTM D4101 with a cell classification range of PP0340B44541 to PP0340B6788

# SECTION 1 - EXCEPTIONS TO THE CANADIAN HIGHWAY BRIDGE DESIGN CODE (CHBDC), CSA S6-25

## 8.7 Prestressing

### 8.7.1 Stress limitations for tendons

The prestressing tendon stress limit in table 8.4 for low relaxation strands in pretensioning, immediately prior to transfer is amended to 0.74 $f_{pu}$.

## 8.8 Flexure and Axial Loads

### 8.8.4 Flexural Components

#### 8.8.4.6 Prestressed Concrete Stress Limitations

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

iii) tension in components with reinforcing bars in the tension zone: 0.6$f_{cri}$.

The contents of Clause (b)(iii) 2) are deleted and replaced with the following:

2) for components with all other Concrete Exposure Classes: 0.75$f_{cr}$.

## 8.11 Durability

### 8.11.2 Mitigation Measures

#### 8.11.2.2 Concrete Quality

##### 8.11.2.2.1 General

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.

#### 8.11.2.4 Concrete Cover

##### 8.11.2.4.1 Concrete Cover

The fourth 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.

**A8.2.10.2.2 Proof Loading**

**A8.2.10.2.2.3 Proof Load Testing Requirements**

This clause is amended by the addition of the following:

Alternative proof load testing requirements may be permitted when approved for use by the owner.

**CHBDC SECTION 9 – WOOD STRUCTURES**

**9.6 Flexure**

**9.6.3 Lateral Stability**

The formula for $C_k$ is deleted and replaced with the following:

$$C_k = \sqrt{\frac{0.97 E K_{SE} K_T}{F_b}}$$

**9.11 Sawn Wood**

**9.11.2 Specified Strengths and Moduli of Elasticity**

**Table 9.17 Specified Strengths and Moduli of Elasticity for Structural Joists and Planks, MPa**

The properties of species combination Hem-Fir of Table 9.17 are deleted and replaced with the following:

| Grade | $f_b$ | $f_v$ | $f_c$ | $f_{cp}$ | $f_t$ | E | $E_{05}$ |
|---|---|---|---|---|---|---|---|
| SS | 16.1 | 1.6 | 16.9 | 4.6 | 8.4 | 11000 | 8000 |
| No.1 / No.2 | 12.0 | 1.6 | 14.9 | 4.6 | 5.0 | 10000 | 7000 |

**9.17 Durability**

**9.17.1 General**

The second paragraph is deleted and replaced with the following:

Preservative treatment shall be specified to at least the use category specified in the CSA O80 Series of Standards.

**9.17.12 Protective Treatment of Hardware and Metalwork**

This clause is amended by the addition of the following:

Stainless steel hardware and metalwork or other protective treatment may be used when compatible with the preservative treatment specified and where otherwise specified by the MTO standards.

**9.22 Wood-Concrete Composite Decks**

**9.22.3 Concrete Slab**

**9.22.3.3 Reinforcement**

This clause is amended by the addition of the following:

The minimum reinforcement in the concrete slab may alternatively be fibre reinforced polymer reinforcing bars. The reference to “steel” may be taken to also mean FRP.

**CHBDC SECTION 10 - STEEL STRUCTURES**
**10.4 Materials**

**10.4.14 Steel piles**
Clause is delated and replaced with the following:
Steel piles shall conform to CSA G40.21, 300W(WT), 350W(WT) or 450W, ASTM A252/A252M, A572 or ASTM A913 Grade 450. The deviation from straightness for steel piles shall meet the requirements of CSA W59 for welded steel columns

**10.6 Durability**
**10.6.8 Other components**
**10.6.8.2 Deck Drains**

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

Steel components of deck drains shall be hot-dip galvanized, stainless steel or coated with a duplex system.

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

Steel railing systems shall be hot-dip galvanized, stainless steel or coated with a duplex system. The parts of steel railing posts in contact with concrete, shall also be coated with an approved paint system.

**10.12 Composite box girders**

**10.12.6 Diaphragms cross-frames, and lateral bracing**

**10.12.6.1 Diaphragms and cross-frames within girders**

The first paragraph of this clause is amended by the addition of the following:
Alternate restraint system may be permitted only when approved for use by the owner.

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

**12.4.3.4.2 Crash test requirements for traffic barriers**

The second paragraph is deleted and replaced with the following:

The crash test requirements for traffic barriers for Test Levels 1, 2, 3, 4, and 5 shall be the crash test requirements specified in the NCHRP Report 350 or the MASH.

# SECTION 1 - EXCEPTIONS TO THE CANADIAN HIGHWAY BRIDGE DESIGN CODE (CHBDC), CSA S6-25

## 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<br>- stirrups and ties | 300 | 350 | 400 | 300 |
| After 1978<br>- remainder | 300 | 350 | 400 | 350 |

### 14.9.1 Normal Traffic

Figure 14.1 is deleted and replaced with the following:

## SECTION 1 - EXCEPTIONS TO THE CANADIAN HIGHWAY BRIDGE DESIGN CODE (CHBDC), CSA S6-25

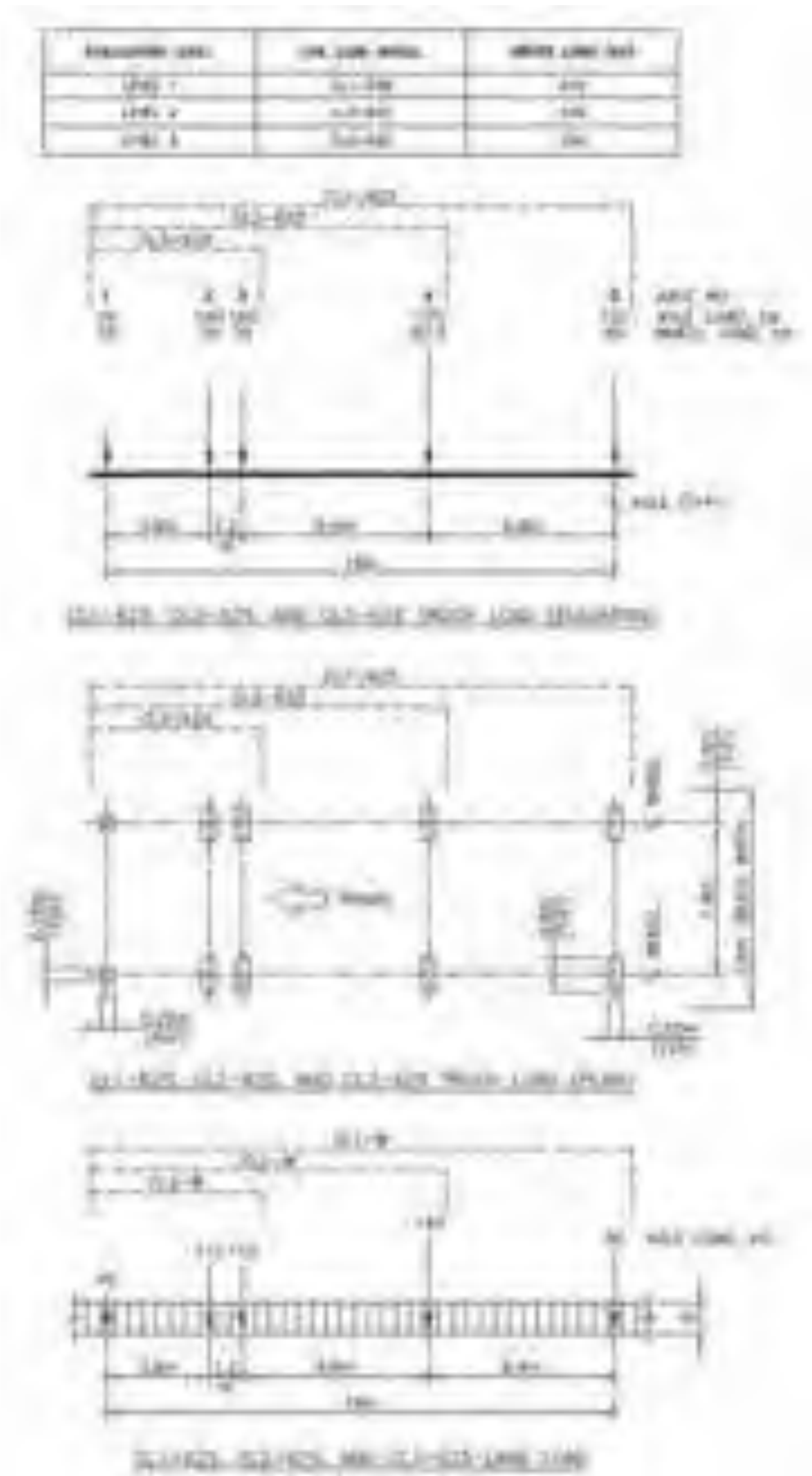


**Figure 14.1– Evaluation Level 1 Loads with CL1-625-ONT-truck**

# SECTION 1 - EXCEPTIONS TO THE CANADIAN HIGHWAY BRIDGE DESIGN CODE (CHBDC), CSA S6-25

**Notes:**
1. The values of the uniformly distributed load, q, for each highway class are as follows:
a) Class A: 9 kN/m
b) Class B: 8 kN/m
c) Class C or D: 7 kN/m
Highway Class is as per Section 1.

2. For **Elevation Level 2** in Ontario, the 4-axle CL2-625 Ontario truck load or the CL2-625 lane load shown in Figure 14.1 shall be used instead of CL2-W truck load and CL2-W lane load, respectively.
3. For **Elevation Level 3** in Ontario, 3-axle CL3-625 Ontario truck load or the CL3-625 lane load shown in Figure 14.1 shall be used instead of CL3-W truck load and CL3-W lane load, respectively

**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.

**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.

# SECTION 1 - EXCEPTIONS TO THE CANADIAN HIGHWAY BRIDGE DESIGN CODE (CHBDC), CSA S6-25

## CHBDC SECTION 15 REHABILITATION AND REPAIR

15.3.2 Limit States

This clause is amended with the addition of the following:

The contents of Clause 5.7.1.5.2 shall not apply to rehabilitated members.

## CHBDC SECTION 16 FIBRE-REINFORCED STRUCTURES

**16.3 Abbreviations and Symbols**

**16.3.2 Symbols**

The following definition of $\psi_s$ is added to the clause:

$\psi_s$ = curvature at a section when the moment is $\mathrm{M_S}$, mm-1 (see Clause 16.8.2.1).

$M_s$ = moment corresponding to the service load (see Clause 16.8.2.1) N•mm

**16.4 Durability**

**16.4.4 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 8.7, reduced by 10 mm, however, the minimum clear cover after consideration of tolerance shall not be less than 30 mm.

**16.5 Fibre-Reinforced Polymers**

**16.5.6 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:

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

When calculating $M_{ult}$, $\phi_{frp}$ = 0.8 and $\phi_c$ = 1.0 shall be used.

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

**16.8.4.1 General**

The equation for $\ell_d$ shall be replaced with the following:

$$\ell_d = 0.45\frac{k_\ell k_4}{\left[d_{cs} + K_{tr}\frac{E_{FRP}}{E_s}\right]}\left[\frac{0.8f_{FRPu}}{f_{cr}}\right]A_{FRP}$$

Bulleted item a) shall be replaced with the following:

a) the variable $k_4$ is the bar surface profile factor and may be taken as less than 1.00, but not less than 0.90, if the reduction is justified by experimental data and if it is approved by the owner. Otherwise, $k_4$ shall be taken as 1.00 for sand-coated bars or 1.05 for ribbed surfaces or spiral patterned surfaces;

**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, $\lambda_d$, for FRP bent bars in tension shall be calculated using the equation given in Clause 16.8.4.1.

However, the development length, $\lambda_d$, may be reduced by accounting for the tensile stress taken by the bent portion of the bar, $f_{FRBbendu}$, for bars that are bent 90 degrees or more. The development length may be reduced to, $\lambda_{dr}$, as shown, but neither $\lambda_d$ or $\lambda_{dr}$ shall be less than $12d_b$ or 230 mm.

$$\lambda_{dr} = \lambda_d\left(1 - \frac{f_{FRPbendu}}{f_{FRP}}\right)$$

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

**16.8.4.5 Splice length for FRP bars under compression load**

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

The length of lap for compression GFRP reinforcing bars shall not be less than 300 mm or $1.3\ell_d$, where $\ell_d$ is calculated in accordance with Clause 16.8.4.1, where $0.8f_{FRPu}$ is eplaced with $0.2f_{FRPu}$.

**16.8.9 Compression Components**

**16.8.9.4 Transverse Reinforcement**

**16.8.9.4.2 FRP Ties**

**16.8.9.4.2.3 FRP Ties for Circular Sections**

Item (c) is deleted.

Figure 16.8 c) is deleted.

**16.8.9.4.4 FRP Transverse Reinforcement Located in Potential Plastic Hinge Regions**

**16.8.9.4.4.3 FRP Ties and Hoops in Circular Sections**

The last sentence in the clause is deleted.

**16.8.9.4.4.4 FRP Spirals in Circular Sections**

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

The use of FRP spirals in plastic hinge regions is not permitted.

**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.

# SECTION 1 - EXCEPTIONS TO THE CANADIAN HIGHWAY BRIDGE DESIGN CODE (CHBDC), CSA S6-25

## 16.15 Composite Construction with GFRP Reinforcement

Sections consisting of a concrete deck slab connected to a structural steel girder or precast concrete girder, by means of shear reinforcement in Section 8 or shear connectors Sections 10, may have the deck reinforced with GFRP.

In positive moment regions, the compressive contribution of GFRP reinforcement may be ignored.

In negative moment regions, the tensile strain in longitudinal GFRP bars in a composite section, and corresponding stress, shall be determined based on a strain compatibility analysis of the section.

## Annex A16.4 Rehabilitation of Steel Bridges

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

This annex shall be used only if approved by the owner.

# SECTION 2 - APPENDIX A
# GUIDELINES FOR THE DESIGN OF BRIDGES ON LOW VOLUME ROADS

## Table of Contents

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

# 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. 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, are 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 is not a high probability risk.

## 2.3 Definitions

**CHBDC** means CSA-S6-25, Canadian Highway Bridge Design Code.

**TL-0 (formerly LVPL1)** means a barrier 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) barrier as described in the 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.

# SECTION 2 - APPENDIX A
# GUIDELINES FOR THE DESIGN OF BRIDGES ON LOW VOLUME ROADS

## 2.4 Provisions, Guidelines, and Commentaries

The CHBDC shall 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

1.4.2.4 Design Life

a) The design life of the bridge shall be 75 years unless reduced at the request of the Owner (Cl. 1.4.2.4).

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.

1.4.2.6 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 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

# SECTION 2 - APPENDIX A
# GUIDELINES FOR THE DESIGN OF BRIDGES ON LOW VOLUME ROADS

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 (Geometric Design Guide for Canadian Roads). 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.

**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.

## 1.6 Secondary and Auxiliary Components

# SECTION 2 - APPENDIX A
# GUIDELINES FOR THE DESIGN OF BRIDGES ON LOW VOLUME ROADS

1.6.3 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.

1.7 Bridge Deck Drainage

1.7.3 Drainage Systems

1.7.3.1 General

a) The maximum allowable spread distance onto the travel lanes of highway bridges in Ontario shall be in accordance with the MTO Highway Drainage Design Standard, WC-4 Bridge Deck Drainage.
b) Deck drains are only required as given by Clause 1.7.3.1. 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 Hydrologic and Hydraulic Design

a) Hydrologic and Hydraulic design standards shall be as specified in the Highway Drainage Design Standards, March 2024.

1.9.5.7 Protection Against Scour

1.9.5.7.5 Inerodible Inverts and Revetments

1.9.5.7.5.3 Special Protection Against Streambed Degradation

a) For closed culverts in scour resistant soils, a concrete cut-off wall is not required (Cl. 1.9.5.7.5.3).

1.9.5.8 Substructure and Embankment Erosion and Piping Control

1.9.5.8.1 Slope Protection

a) Slope protection, where required, needs only to be extended 150 mm above HWL (Cl. 1.9.5.8.1).

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. 1.9.5.8.1) (Drainage Management Manual).

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. | | | | | | | |

## 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.

## SECTION 2 - APPENDIX A
## GUIDELINES FOR THE DESIGN OF BRIDGES ON LOW VOLUME ROADS

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

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 O86-24 (Cl. 9.11) (CSA-086-24).

9.17 Durability

a) Preservatives do not need to meet the wood component use category requirements of OPSS 1601 but shall meet the use category requirements of CSA O80 Series.

Commentary: The use categories attributed to OPSS 1601 are for increased preservative retention levels intended for improved durability of permanent bridges (75+ year service life). While a lower use category may be used for a low volume bridge, it may not result in substantial treatment cost savings.

9.17.13 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.

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) (Faller, Rosson and Sicking).

## SECTION 2 - APPENDIX A
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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. 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 12.6 as long as the vehicle barrier interaction is not adversely affected.

**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 performance limits above are given in Figure 2.4-1 to Figure 2.4-3. 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.

## SECTION 2 - APPENDIX A
## GUIDELINES FOR THE DESIGN OF BRIDGES ON LOW VOLUME ROADS

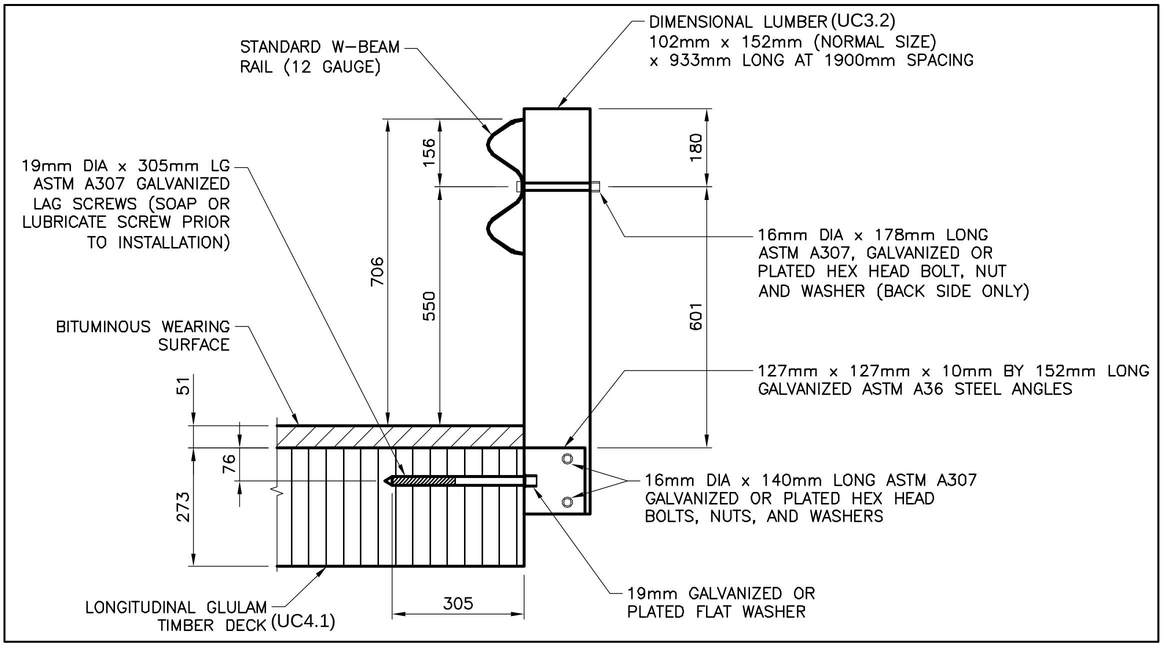


**Figure 2.4-1 – NCHRP-350 TL-1 Barrier**

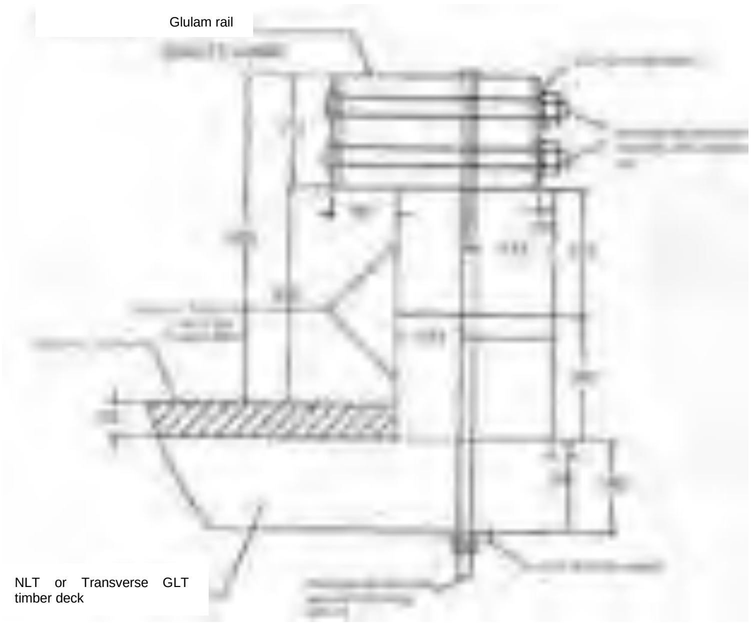


**Figure 2.4-2 – MASH TL-1 Barrier**

# SECTION 2 - APPENDIX A
# GUIDELINES FOR THE DESIGN OF BRIDGES ON LOW VOLUME ROADS

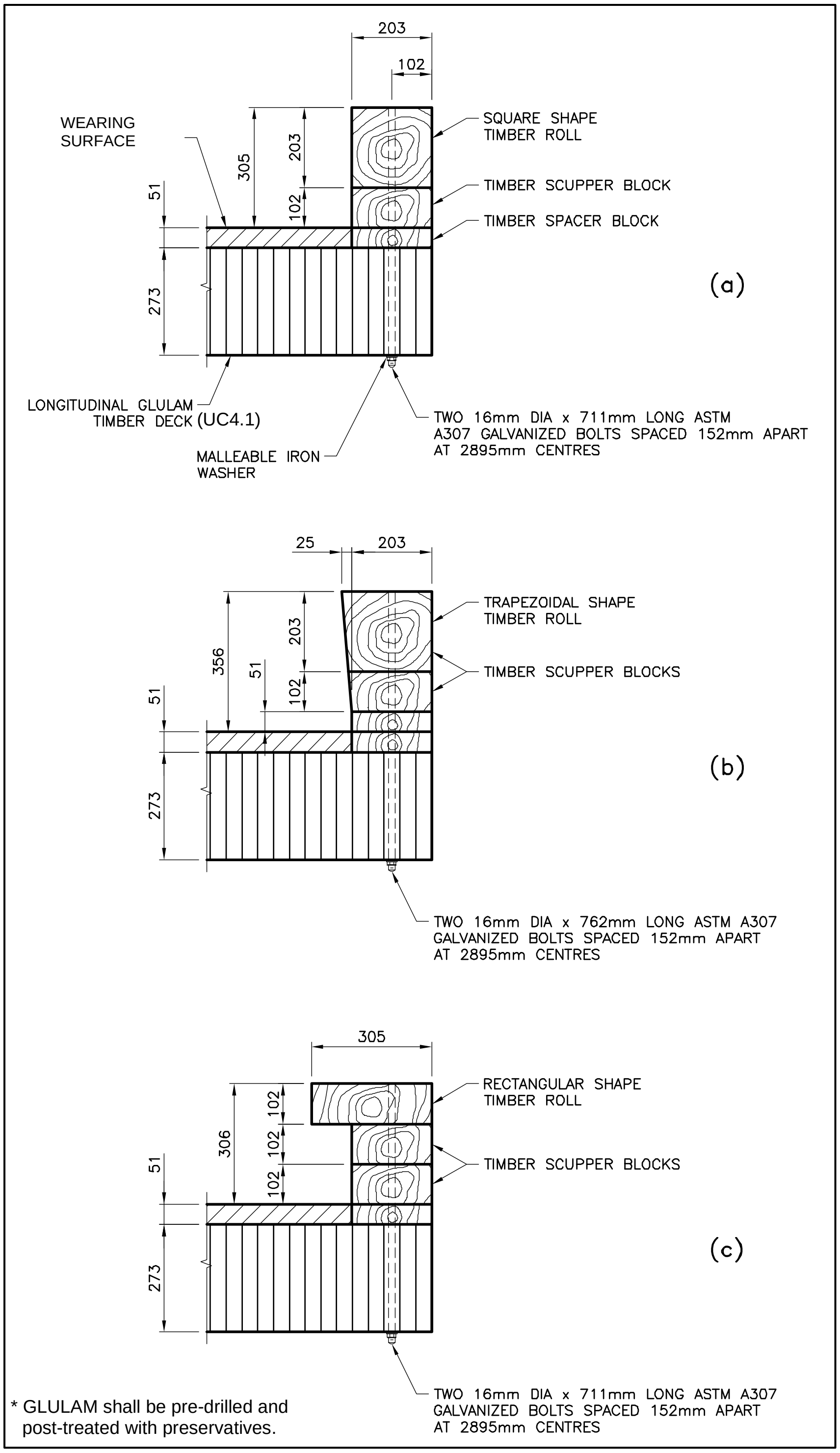


**Figure 2.4-3 – TL-0 Barrier Systems**

## SECTION 2 - APPENDIX A
## GUIDELINES FOR THE DESIGN OF BRIDGES ON LOW VOLUME ROADS

### 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

Alberta Transportation and Infrastructure. "Design Bulletin #1, Navigable Waters Protection Act Approvals for Bridge Projects." 2004.

CSA Group. Canadian Highway Bridge Design Code. 13th. CSA Group, 2025.

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# DIVISION 2 – PROCEDURES

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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.

## SECTION 1 - INTRODUCTION

### 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 2.6.14. 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.

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:

- 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

## 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.

### 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.

### 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 Slab-on-Girder Bridges

#### 2.1.6.1 Guidance on Construction Loading for the Design of Slab-on-Girder Superstructures During the Non-Composite Stage of Construction

The choice of deck form system is made by the contractor, but by necessity a load must be assumed during design. In general, the size (weight) of the system relates to the girder spacing but other factors influence the weight of the system. Girder systems without lateral bracing near the deck can be formed with simple systems of formply and dimensional lumber suspended from beam hanger supports. Structural steel girder systems with top flange lateral bracing require a deeper form support system that hangs below the bracing that may include engineered wooden joist systems or spacers and the system may be designed to carry loads around the brace obstacle. Or, contractors may also choose to work with proprietary support systems of engineered wood, steel, and aluminum construction.

The design shall indicate assumed loading on the deck details drawing as per the deck details notes.

<u>Consider a False Deck</u>
The designer shall assume that the contractor will opt to use a false deck system on all slab-on-girder structures. Contractors use the false deck for more than just accessing the deck formwork. They may also use it as a protection system to prevent material falling on the roadway, waterway, or workers below. A false deck may also be used as part of the contractor’s temporary lateral bracing system. As such, a false deck may be used even on shallow girder structures.

A false deck is typically supported on the bottom flange between steel I-girders or NU girders. A false deck may also be present between steel trapezoidal box superstructures where it would be hung from the top flange. False decks are seldom found on the outside face of exterior girders and need not be considered at that location.

A designer may indicate on the drawings that a false deck is not permitted, but only if it is justified by project constraints.

Entire vs Local Construction Live Loading

Designers sometimes apply construction live loads intended for local component design to the design of entire girder system behaviour in the non-composite stages of construction, which may be unnecessarily conservative.

The design of local components such as formwork and falsework systems are required to use relatively high construction live loads that consider loading from combined sources of labour, material in transit, and equipment as well as their variation in magnitude and coincidence that are superimposed on variable or fixed material loads. For example, a deck form hanger with a relatively small tributary area must be capable of supporting the weight of a clustering of workers standing in a heaping pile of concrete leaving the discharge chute.

In specific consideration of the bridge deck pour, all material and equipment not being cast into the deck is absent and the live loading can be represented by the bridge deck finishing machine, the concreting crew, and small hand operated pieces of placing equipment. It is also reasonable to assume that a uniformly applied construction live load and the placed, wet deck concrete material dead load are not coincident; however, the bridge deck finishing machine and work bridge live loads would be coincident.

CHBDC Treatment of Construction Live Loading

In general, the CHBDC leaves the determination of construction live loading to the designer. However, some guidance on construction loading may be found in other industry standards such as ASCE 37-14 “Design Loads on Structures During Construction”. It is important to note that the content of this ASCE document does not classify construction loading into dead and live loading in the same manner as the CHBDC. Instead, the loading is classified as working surface loads, or as material loads that are variable or fixed, etc.

As such, a variable material load represents the loading from materials not yet placed, such as bundled rebar that is set upon the deck form system or a pile of concrete under the discharge chute and which may be treated as a live load by the CHBDC. A fixed material load represents the loading from materials that are placed in final, such as rebar placed and tied to its proper location and spacing or concrete that has been screeded to final elevation and which may be treated as a dead load by the CHBDC.

#### 2.1.6.1.1 Dead Load for Deck Form System and False Deck

A dead load of 0.80 kPa should be assumed for the deck form system and applied to the design of the girder system during the non-composite stage of construction – typically for the stages between girder installation and the deck pour. For the purposes of applying the deck form system load, the designer should assume the formed area covers the gross plan area of the deck and that it extends a distance of 0.8 m beyond the deck fascia (on both sides) to account for the typical walkway at the tip of the cantilever formwork.

A load of 0.35 kPa should be used for the false deck system. The designer should assume the false deck covers the gross plan area between the girder webs supporting the false deck.

The dead load factor shall be as per the appropriate CHBDC load combination and may be assumed as wood material.

The specified dead load above represents a conservative loading that is suitable for the vast majority of typical deck systems used by contractors in slab-on-girder superstructure construction.

For atypical slab-on-girder superstructure system (e.g. girder spacing greater than 4.0 m, formwork for intermediate diaphragms, etc.), the designer shall investigate, determine, and consider larger dead loads.

A smaller specified load may be assumed where justified, such as very close girder spacings.

Cantilever Deck Form Load

A dead load of 1.75 kPa should be assumed for the deck form system of the cantilever portion of the deck forms *when designing the exterior girder*. This is because the deck cantilever forms are more substantial than the forms between girders for the interior deck slab. This load should replace the deck from system load from the previous section within the footprint of the cantilever forms; the two loads should not be superimposed on top of one another.

#### 2.1.6.1.2 Live Load for Deck Form Systems, False Deck, Screeding Operations, and Cantilever Overhang

The construction live loading described in this section shall be used for the design of girder systems during the non-composite stages of construction - for checking structural resistance and stability. The loading shall not be applied to the design of the formwork and falsework, which shall be designed by the contractor as per the loadings described in OPSS 919.

The load factor shall be the construction live load factor as per the appropriate CHBDC load combination.

Deck Form System

No construction live load acting on the deck form system need be considered for the purposes of the design of the non-composite girder system. This is because the unfactored and factored dead load from placed, wet concrete for a standard 225 mm deck will be larger than typical unfactored or factored specified construction live loads. An analysis of the sequencing and progression of the deck pour including the screeding operation is sufficient for the design of typical slab-on-girder bridges.

False Deck

No construction live load acting on the false deck system need be considered for the purposes of the design of the non-composite girder system. This is because, in general, the worst condition for the girder system will still be the deck pour on the deck form system when it is reasonable to assume there will be no live loading of the false deck. Further, the ability to load the false deck is limited by the installation of the deck form system as construction progresses considering the clearance between the working surface of the false deck and the soffit of the deck form system as well as other encumbrances such as K-frames and lateral bracing.

No live loading or storage of materials or equipment are permitted on the false deck during the deck pour and a note to that effect shall be included on the deck details drawing.

Cantilever Overhang Formwork

*When designing the exterior girder*, in consideration of the combined longitudinal and torsional forces acting on the exterior girders, a construction live load of 2.40 kPa shall be assumed to act on the walkway of the cantilever overhang over an area of 6.0 m by 0.8 m during the deck pour.

This load represents the concentration of workers that may be present locally on the walkway of the cantilever overhang for hand placement of concrete at the barrier wall deck joint. This load shall be assumed to act simultaneously with the worst longitudinal girder loading from the progressive pour analysis of the placed, wet deck load and the bridge deck finishing machine. This walkway load and a biased finishing machine load shall be centered on the section under consideration.

Bridge Deck Finishing Machine and Work Bridges

The bridge deck finishing machine and work bridge loads are live loads and should be considered by the designer and these should be treated as moving loads in conjunction with the progressive deck pour. Specific consideration should be given to the design of exterior girders – or the girders loaded by the screed rails for staged construction – and in the design of the non-composite girder system during the deck pour.

Bridge deck finishing machine and work bridge live loads are provided in the table below. The bridge deck finishing machine load is provided as a total load and as a heavy rail load. The total load may be divided between each rail for overall structural system considerations. Whereas the heavy rail load should be used for exterior girder design as it represents a situation that biases over one rail the weight of machine components such as the power unit, carriage, operator, and other accessories. The work bridge load is provided as a total load which may be divided over both rails per work bridge. It is appropriate to use the load for the listed span between screed rails that is just larger than the span in consideration. Loads may be linearly interpolated; however loading for the 10 m span should be used for any spans less than 10 m. The designer shall use rational analysis to determine loads for spans greater than 40 m and shall confirm the availability of equipment for such spans.

**Table 2.1-1 – Bridge Deck Finishing Machine And Work Bridge Live Loads**

| Span between Rails | Finishing Machine | | Work Bridge |
|---|---|---|---|
| | Total Weight | Heavy Rail | Total Weight |
| m | kN | kN | kN |
| 10 | 57 | 42 | 11 |
| 15 | 68 | 48 | 15 |
| 20 | 79 | 55 | 19 |
| 25 | 89 | 62 | 23 |
| 30 | 100 | 68 | 27 |
| 40 | 115 | 76 | 34 |

The specified loads above represent a conservative loading that is suitable for the vast majority of typical bridge deck finishing machines and work bridges used by contractors. The work bridge loads include the weight of the work bridge as well as the weight of materials and workers.

Additional attention shall be paid to exterior girder design if the screed rails are not located above the girder centrelines as required by OPSS 919.

Other Construction Live Load
When needed for the design of any local components of the girder system (e.g. an intermediate floor beam or stringer), a minimum construction live load of 2.40 kPa may be considered within the tributary area of the component acting on the deck form system or false deck system. This load may be assumed for operations such as the placement of rebar or concrete placement. Other loads, such as variable and fixed material loads should be considered as well.

For the design of the girder system during the non-composite stage of construction, no construction live loading is given beyond the guidance found elsewhere in this section. When needed, the designer shall use rational analysis to determine construction live loads.

#### 2.1.6.2 Live Load Forces for Future staging and Rehabilitation

For new slab-on-girder bridge designs, to ensure flexibility for future staging, rehabilitation, or changes in functional use, the live load effects applied to girder design at ULS and SLS shall not be less than those calculated using the simplified method of analysis in Clause 5.6 of the CHBDC & using the maximum number of design lanes that can be accommodated based on the definition of Deck Width in Division 1 Section 1 of this Manual, unless approved by the Head of the Structural Section. If the bridge does not qualify for the simplified methods, the design live load effect must still meet or exceed the values that would apply if the criteria were satisfied.

## 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 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.

**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 ≥ 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 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.

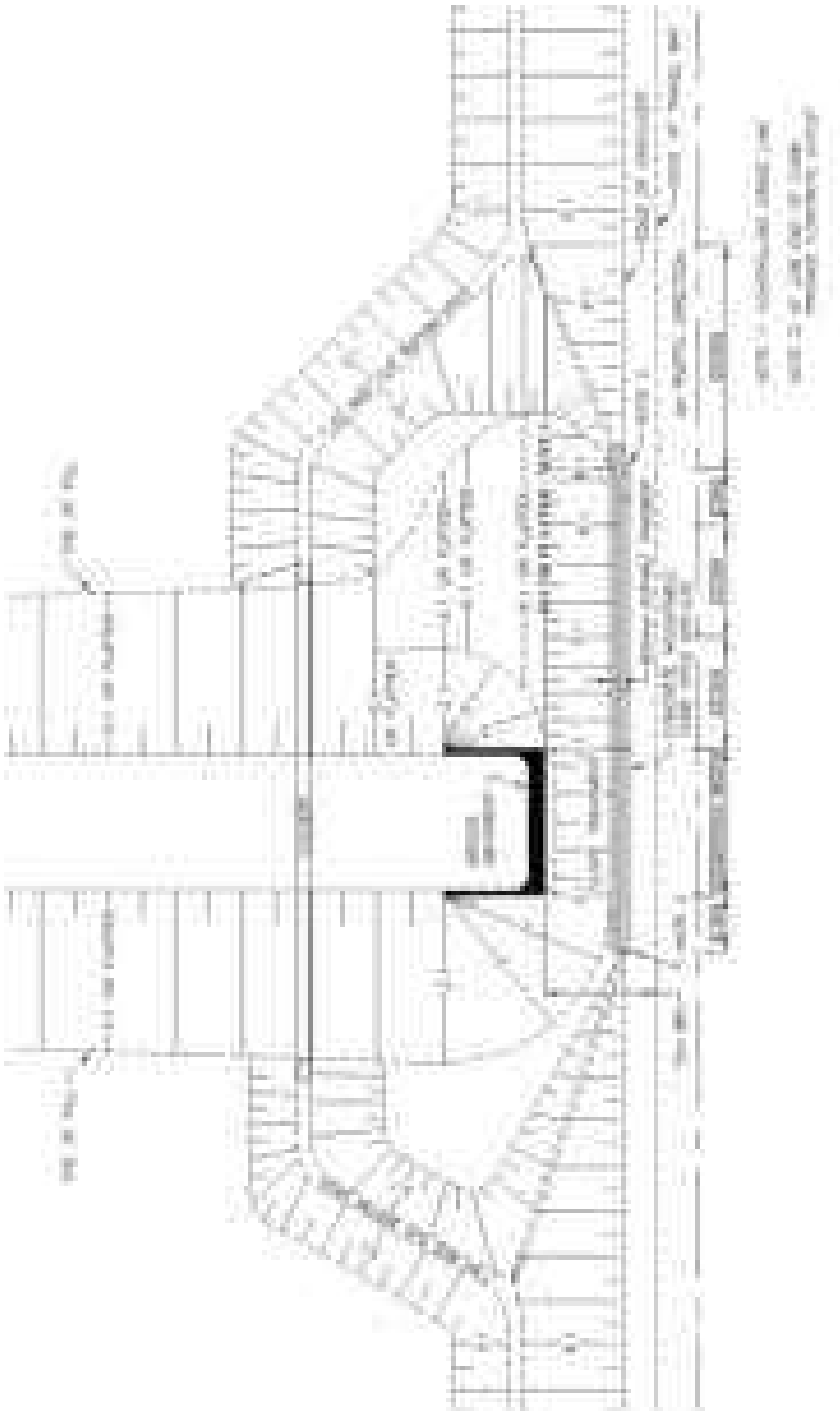


Figure 2.2-1 – Treatment of Slopes in Front of Abutments

### 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.

## 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:

- 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, 2$^{nd}$ edition" 2020 INTERIM REVISIONS, AASHTO and the "Construction Handbook for Bridge Temporary Works, 2$^{nd}$ 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.

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, 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 quality assurance testing for most concrete includes testing of Rapid Chloride Permeability (RCP), and air void system (AVS) parameters asthe basis for acceptance to ensure durability of the concrete. For some applications, other durability tests are also specified, such as salt scaling resistance.. Specifications currently require maximum RCP values of either 1000 or 2500 Coulombs, depending on the type of concrete (HPC or Normal). As such, concrete strength should be specified only for structural requirements since the durability properties are tested and specified in the concrete specifications.

Higher strengths of concrete come with increased cost, 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:

**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 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 OPSS1350), 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).

### 2.4.3 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. An NSSP is required for the use of bulk-bagged concrete or portable concrete mix plant. Designer shall consult with Head, Concrete Section, Engineering Materials Office (EMO), MTO, prior to including this NSSP in a contract.

### 2.4.4 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): Uncoated 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 resulting 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, when 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;

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.

### 2.4.5 Durability

Durability requirements for new structures, rehabilitation and replacement work shall meet the provisions of Section 2 of the CHBDC except, where modified by the contents of this manual.

### 2.4.6 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 8.7, and the CHBDC Exceptions.

The concrete cover, and placement tolerances for several typical conditions are provided in Section 2.6.7 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 2.6.7 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 need not be hooked over the outer-most reinforcing bars. They 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.

### 2.4.7 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:

- 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.

### 2.4.8 Corrosion Protection Policy for Concrete Bridge Components

**Definitions**

**Uncoated Carbon Steel** means uncoated deformed reinforcing bars made of carbon steel as per CSA G30.18.

**Corrosion Resistant Materials** means Premium Reinforcement.

**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 a 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 520, 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 uncoated carbon steel bars 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 **Section 12** 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 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 exposure classes A4, A5, D1, D2, D3, SSST5, and S2 of Table 2.1 in the CHBDC, 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-applied 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 detailed in the deck slab.

**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.

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 (previously referred to as 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. 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) b. shall be increased by 0.8 m.

### 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 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.

### 2.5.3 Design Guidance for Seismic Importance Categories

The seismic and hydraulic importance category shall be shown in the structural planning report.

For hydraulic design, the only importance category considered is lifeline. Lifeline structures may be eligible for design under CHBDC clause 1.9.6 Service Level Design. Otherwise, all bridges are required to meet CHBDC clause 1.9.5 Performance-Based Design Method, including lifeline bridges by default.

The importance category 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 $m^2$; 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 $m^2$ 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.

**Figure 2.5-1 – Canada's National Highway System – Ontario**

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

**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 | | |
| 3.0 | Hwy 401 | U.S. Border (Ambassador Bridge) | **11.0** | 11.0 | | |

| Ontario | | | | Core | Feeder | Northern and Remote |
|---|---|---|---|---|---|---|
| Route | From | To | **Length (km)** | km | km | km |
| 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 | | |
| Derry 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 | Derry 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** | **-** |

### 2.5.4 Bridge Hydrology and Hydraulic Commentary

The 2025 edition of the CHBDC made significant changes to its hydrologic and hydraulic design clauses and this was done at the same time as the 2025 update to the TAC *Guide to Bridge Hydraulics*. Taken together, there is now a greater amount of national guidance on the analysis and design aspects of water crossings.

While this is beneficial for other owner authorities, the MTO already had in place a robust set of policies and practices with respect to the analysis, modelling, and design of water crossings. As such, a broad set of exceptions to the hydrologic and hydraulic design clauses of the CHBDC (as described in Division 1 of this manual) either defer to, or direct, the use of the MTO's *Highway Drainage Design Standards*.

In particular, the MTO already had an established practice for defining the annual exceedance probabilities of water crossings for specific aspects of design on the basis of highway classification and this practice shall continue. The 2025 CHBDC sets those probabilities on the basis of the importance category of the structure – which is nonetheless defined by the owner.

The changes to the CHBDC also describe two methods of design: performance-based design, and service level design. The default practice is to meet the performance-based design requirements for all structures regardless of importance category; however, the MTO may permit the use of service level design for lifeline structures on a case-by-case basis.

Otherwise and in general, there is a lot of agreement between the MTO's *Highway Drainage Design Standards* and the other parts of the hydrologic and hydraulic design clauses of the CHBDC.

## 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 2.6.8.

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

### 2.6.3 Content of Structural Drawings

Drawings shall adequately describe the structural components unambiguously and shall be to scale. Drawings should follow industry standard practices. For most components, a minimum of three views are required to describe the work. For skewed bridges, both obtuse and acute corners of all components shall be shown. Details should be oriented the same direction as the section they are cut from.

The use of isometric or 3D views are encouraged for details with complex geometry that are difficult to fully show with a plan view and sections.

Settings, layering, linetypes, text size and font, and standard symbology for use on MTO projects shall be according to the AutoCAD Standards Guide Ontario Provincial Standards (IESCAD) available from https://www.xfer.mto.gov.on.ca/#iescad.

General drawing principles and techniques should follow CAN/CSA-B78.1, B78.2, and ASME Y14.100. Reinforcing Steel Detailing shall follow the RSIC Manual of Standard Practice and ACI 315R-18 Guide to Presenting Reinforcing Steel Design Details. Weld symbols shall follow AWS A2.4 Welding Symbol Chart.

### 2.6.4 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.

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

PROTECTION SCHEME DESIGN WATER LEVEL ________

### 2.6.6 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 of 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.

### 2.6.7 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.

**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 |
| 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, DUPLEX 2205, OR DUPLEX 2304 AND HAVE A MINIMUM YIELD STRENGTH OF 520 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.
(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 2.6.8(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 2.6.8(a).3).

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

Same as Notes for Slab-on-Girder Bridge (Section 2.6.8(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. (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 2.6.8(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 |

<u>ROUND VOIDED DECKS</u>

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

<u>RECTANGULAR VOIDED DECKS</u>

| | | |
|---|---|---|
| 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 2.6.8(a).3).

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

Same as Notes for Slab-on-Girder Bridge (Section 2.6.8(a).4).

5) CONSTRUCTION NOTES

Notes as applicable, see list under Section 2.6.8(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 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/m$^2$ 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 7.2.10**.

**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 should 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.

### 2.6.8 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 between 0.6 and 0.7 mm. Text needs 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 2.6.11) 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.

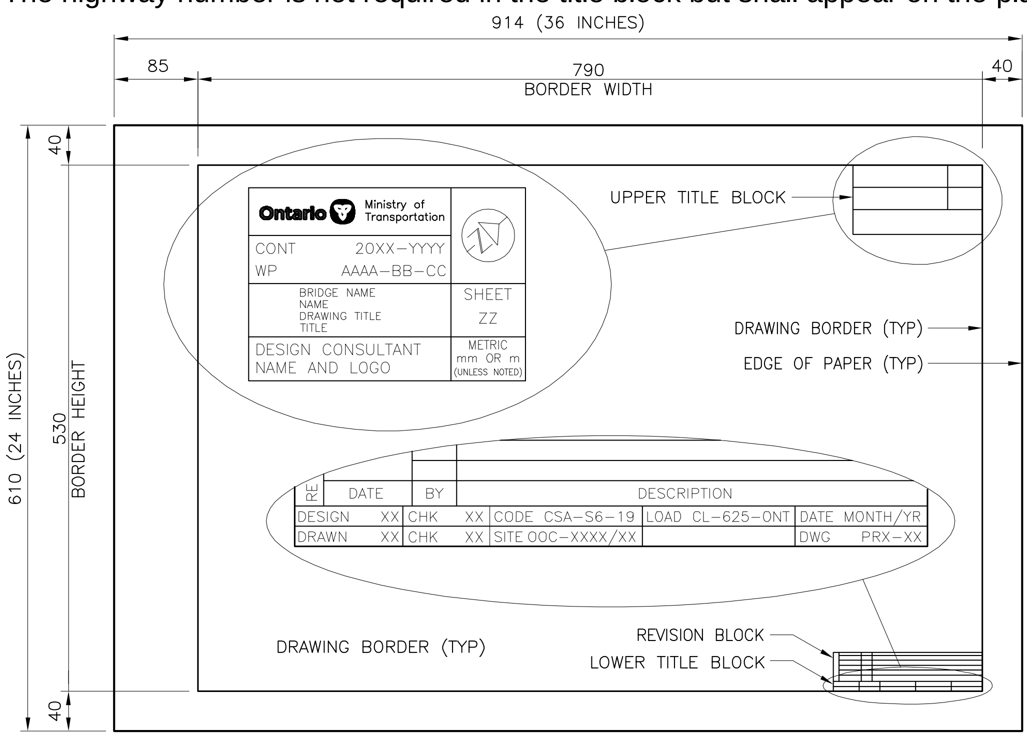



**Figure 2.6-1 – Master Drawing Border**

## 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 2.6.16 for more details.

**Lower Title Block**

All information shall be filled as indicated below and illustrated on Figure 2.6-1.

| | |
|---|---|
| DESIGN | Initials of actual 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 actual 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 2.6.12. |
| DWG | Add drawing number as per Section 2.6.1. |

### 2.6.9 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. When there is insufficient space to accommodate all the information, a second general arrangement drawing should be added for notes, profile, and if necessary, and, if necessary, typical sections. Every effort should be made to keep the plan and elevation together on the first general arrangement drawing.

**Key Plan**

A key plan is not required on the general arrangement if it is to appear on the soils drawing 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 45º 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.
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.

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 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 2.6.5) 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.

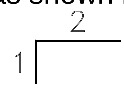
Dimensions shall be given showing overall cross-section width, distances between centre lines, curb or median widths, barrier wall face to outer deck edge width, slab thickness, pier shaft width or column diameter and the lane and shoulder widths even if there is no physical demarcation between them.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




**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 2.6.8). These include Construction Notes.

**Durability Design**

As required by Clause 2.5.1 of the CHBDC, two tables shall be added to the general arrangement drawing with the format given in Annex A2.1 of the CHBDC.

**Isometric Details**

Isometric details should/shall? be provided in situations where it is difficult to convey complex geometry with multiple sections. The detail shall be drawn with hidden lines so the reader can look through the object. Shading may be provided to differentiate surfaces on different planes, but shall not obscure the view through the object.

**Bench Mark**

In the lower left corner of the drawing give the description and elevation of two benchmarks in small text.

Benchmark details shall be obtained from the geomatics section in consultation with the project manager and highway section. A conversation should be had to identify suitable benchmarks; they should be local to the site, but not so close that they are likely to be disturbed by the work.

Confirm that the benchmark information conforms to the project horizontal and vertical datum.

Example:

DHO Benchmark 819980159
EL. 253.819
Top of Brass Cap
Sta. 12+647.565m LT 21.295m
TWP of Colquhoun

Or

0.019mx2.0m RIB
EL. 224.196
Top of RIB
Sta. 10+400.430m RT 23.100m
West Gwillimbury

**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 2.6.14.

**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 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.

**Working Points Table and Project Horizontal and Vertical Datum**

A Working Points table shall be provided above the profile in the lower right region of the sheet and it shall be a tabulation of the northings and eastings of the working points. To avoid clutter and duplication – this table shall only include the northings and eastings; the station and elevation of the working points shall be shown on the plan as described in the plan section. The table shall be in small text.

The Project Horizontal and Vertical Datum shall be included directly below the working point table. The Project Horizontal and Vertical Datum shall be obtained from the geomatics section in consultation with the project manager and highway section. It should match with the datum used on the highway drawings and typically found on sheet H&V Sheet.

The Project Horizontal and Vertical Datum shall report the i) horizontal datum and adjustment epoch, project system, & zone, and ii) the vertical datum and adjustment reference.

Note: The horizontal and vertical datum is also reported on the borehole sheets and the designer should ensure that reference is also coordinated with the project datum.

Example:

| Working Point | Northing | Easting |
|---|---:|---:|
| WP 1 | 4 981 771.968 | 416 677.710 |
| WP 2 | 4 981 778.841 | 416 724.710 |
| WP 3 | 4 981 785.713 | 416 771.710 |
| Horizontal Datum: NAD83(CSRS) V6 (2010), 3 deg MTM, Zone 8<br>Vertical Datum: CGVD28 (78) | | |

The following provides additional information on the 4 key items of the Project Horizontal and Vertical Datum. Additional information may be found in the *MTO Engineering Surveys Manual.*

| Item | Description | Example |
|---|---|---|
| Horizontal | | |
| Projection System | MTO assignments rely on the 3-degree wide Modified Transverse Mercator projection. | 3 deg MTM |
| Zone | MTM zones are fixed geographically by longitude and latitude as per sections 33-35 of O. Reg 216/10 and illustrated in Section 5 of the MTO Engineering Surveys Manual. | Zone 11 |
| Horizontal datum & adjustment epoch | Reference system best suited for the project that may depend on legacy data. See GO-ITS 541TES for details. | NAD83(CSRS) v.6 (2010), or NAD883(ORIG), or other |
| Vertical | | |
| Vertical datum & adjustment reference | Reference system best suited for the project that may depend on legacy data.<br>See GO-ITS 544TES for details. | CGVD28 (78), or<br>CGVD2013, or<br>other |

**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 appear on final drawings only and shall always be signed and dated when they are shown.

**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 before issuing the preliminary version. The note shall not appear on the final drawing.

### 2.6.10 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 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).

### 2.6.11 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. When there is a specific need for construction operations to proceed in a specific order, the sequence shall be described in a manner that is contractually enforceable. For structural rehabilitation contracts, construction staging and sequencing shall be detailed to correspond to the load cases considered in design where deviations from the specified staging or sequence results in adverse conditions. Operations that may be completed at any time during the work shall be defined as such on the drawings.
For new bridges, a construction sequence shall not be provided unless required by Clause 1.4.4.3 of CHBDC.

Occasionally, there may be a need to show a construction sequence for only a component of the work.

### 2.6.12 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 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).

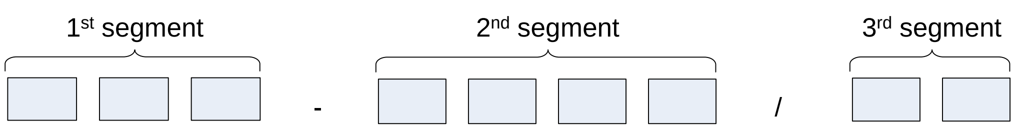
5) Ferry Docks, used to convey passengers and motor vehicles encompassing docks, piers, wharfs, ramp and other accessories.

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 "/":




**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 |

**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 |

**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.
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:

<table>
<tr><td>1</td><td rowspan="4">For:<br>- bridge carrying<br>- culvert under<br>- tunnel carrying</td><td>NB or EB lanes in twin structures or<br>NB or EB collector lanes of core/collector system</td></tr>
<tr><td>2</td><td>SB or WB lanes in twin structures or<br>SB or WB collector lanes of core/collector system</td></tr>
<tr><td>3</td><td>NB or EB core lanes of core/collector system</td></tr>
<tr><td>4</td><td>SB or WB core lanes of core/collector system</td></tr>
<tr><td>5, 6 etc.</td><td colspan="2">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.</td></tr>
</table>

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.<br><br>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.<br><br>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.

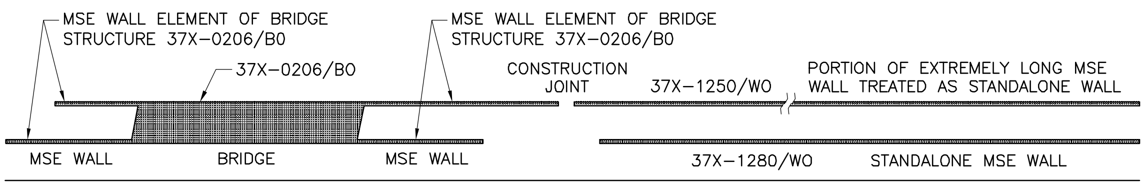



**Figure 2.6-2 – Structure Identification of MSE Walls**

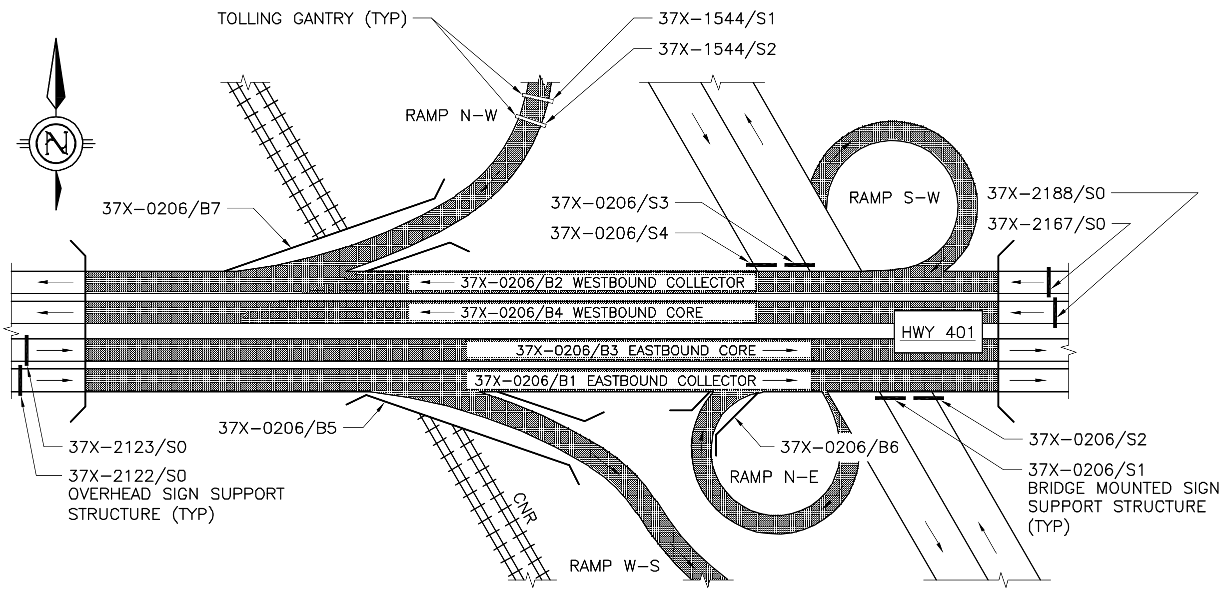



**Figure 2.6-3 – Site Numbers for Bridges and Sign Supports**

### 2.6.13 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.

   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 |
| CNR O/H @ East of Rossport (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.

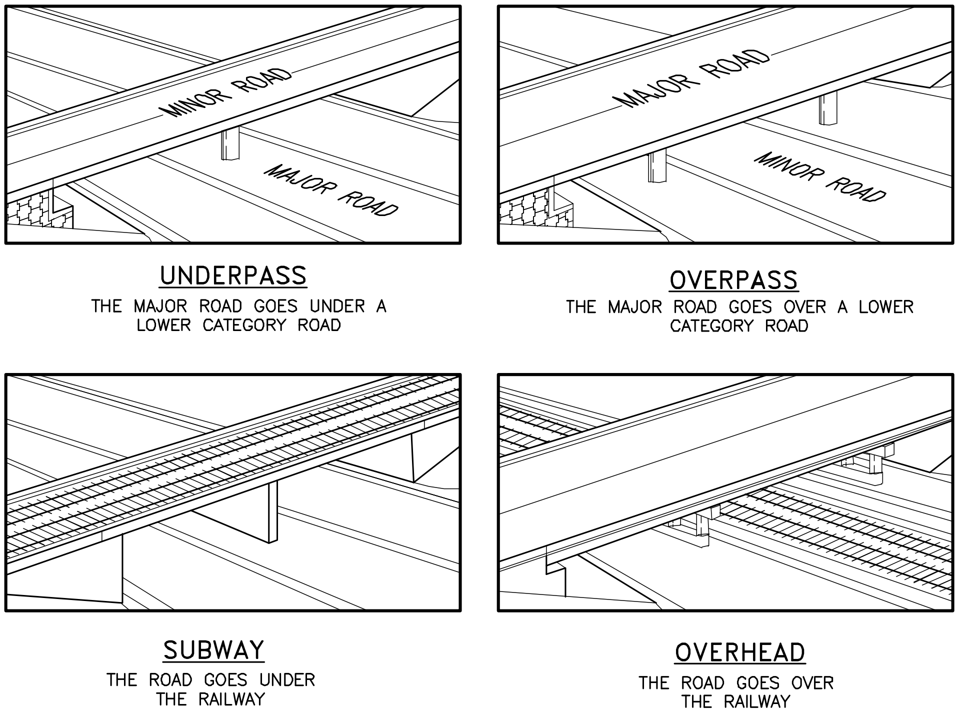



**Figure 2.6-4 – Underpass, Overpass, Subway and Overhead Bridges**

### 2.6.14 Standard Drawings

Structural Standard Drawings (SSDs) shall be modified to match the geometry and conditions of the specific structure and shall not portray standard new bridge geometry that is not applicable. In bridge rehabilitation drawings, they shall be modified to include existing reinforcing steel, based on the information available from existing drawings, as-builts and field data. Particular attention shall be paid to depict existing geometry on expansion joint and barrier 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 Details I" or II, (etc.). Initials are required as per Section 1.5.2 of this manual.

### 2.6.15 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.

OPSD and MTODs shall not be reproduced within the structural contract drawings unless they are modified.

### 2.6.16 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.

### 2.6.17 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 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.

# 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;

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, 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-reach 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 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 steel thickness per exposed face of steel piles in integral abutments as required by the CHBDC Clause 2.5.4.14 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.

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, $\left(\frac{b}{t} < \frac{250}{\sqrt{F_y}}\right)$, need not be satisfied for them to be treated as Class 3 sections.

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 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.)
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:

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

$e_f = 0.75\ for\ drop\ hammers$

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:

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

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

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

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

$g = 9.8\ [m/s^2]$

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

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

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

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

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

$e = Coefficient\ of\ restitution$
$e = 0.25\ for\ timber\ pile\ using\ cushion$

$e = 0.32 \; for \; steel \; pile \; using \; cushion$
$e = 0.55 \; for \; steel \; pile \; without \; cushion$

**A) Design Stage**

The requirement for pile design is as follows:

**Factored geotechnical resistance at ULS > Design load at ULS**
**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 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.

## 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 additional to permanent casings, to reduce the friction of soil when the permanent casing is installed.

#### 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.. A wet pour requires the use of a 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 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.

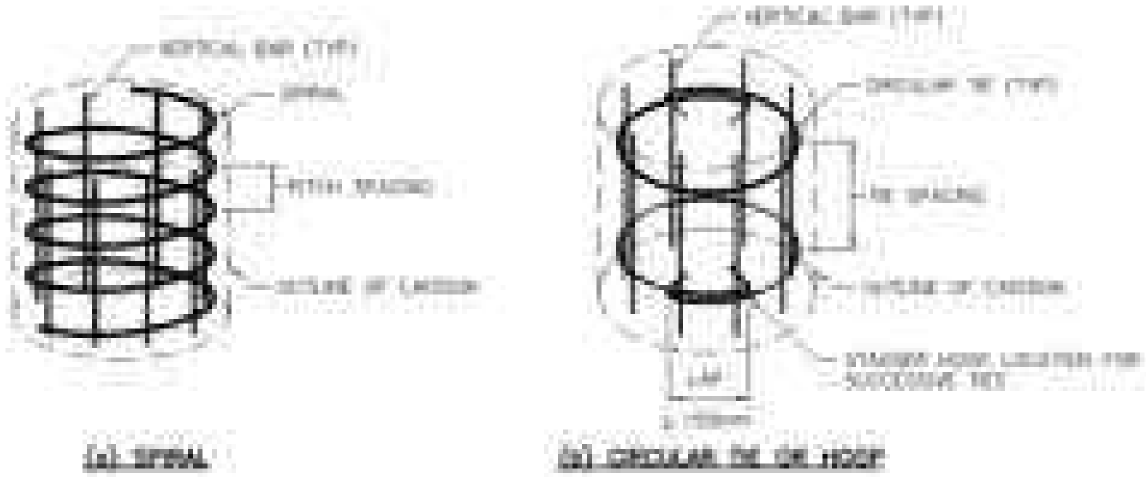


**Figure 3.3-1 – Transverse Reinforcing Details for Caisson Pile**

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 NORMINAL 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). WELDED CASING SHALL 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 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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# 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 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 in 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.

### 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

Bridges shall have integral abutments as required by Section 2.1.2 The following policy supersedes guidance in Structural Office Report # SO-96-01, "Integral Abutment Bridges".

Integral abutment bridges are single or multi-span bridges that accommodate expansion and contraction by flexing of the abutments and foundations, in lieu of 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. Approach slabs are tied to the abutments and move with the deck and abutments.
The feasibility of an Integral Abutment arrangement influenced by the following factors:

a) Type of superstructure: Slab-on-steel girders, slab-on-precast concrete girders, and cast-in-place reinforced concrete slab, and cast-in-place post-tensioned concrete bridges are suitable for integral abutment construction. In all concrete bridges, unidirectional movements due to elastic shortening, creep and shrinkage shall be accounted for in design and may necessitate that post-tensioning be completed prior to making the integral connection between the deck end and the abutments.

b) Length and skew of the structure: Integral abutment should be considered for structures with maximum thermal movement 40 mm at each one and the skew angle less than 30 degrees. The structure length and skew limitations can be exceeded subject to approval by the Structural Section.

c) Abutments: The height of the abutment should be minimized and shall be no greater than 6.0 m. In simply supported bridges, abutment height should be no greater than 20% of the span for girder bridges and no greater than 30% of the span for cast-in-place reinforced concrete bridges. Both abutments should have the same height and width to avoid any unbalanced force.

d) Wingwalls: The height of the abutment and the length of the wingwalls should be minimized to reduce the resistance to the longitudinal movement. The length of the wingwall shall be no greater than 7.0 m. The wingwalls shall be oriented perpendicular to the highway.

e) Multi-span structures: Span lengths and bearing articulation at intermediate supports should be designed to ensure such that equal movement would occur at each end of the structure. The piers supported on flexible foundation should be made integral with the deck.

f) Sub-soil conditions: The Integral Abutment arrangement requires flexible foundation support under the abutments. Integral abutments are less suitable to sites where piles are less than 5.0 m long. Where soil conditions immediately below the abutment are very stiff or irregular, holes may be pre-augured to a depth of 3 or 4 m, and backfilled with a consistent, soft material. A corrugated steel pipe, with diameter at least 400 mm larger than the pile, may be installed around the pile to mixing of the native soil and softer material within the hole.

g) Type of foundation: Integral abutments may be supported on steel piles, concrete filled steel tube piles (CFSTs), concrete caissons, or on columns supported on spread footings (e.g., spill-through abutments). Integral abutments may also be supported directly on spread footings in bridges with length of less than 50 m by accommodating deck movements through flexing of the abutment walls and rotation at the footing, where these systems can provide the flexibility needed to accommodate the movements from the superstructure.

Construction of integral abutment requires specific considerations and sequence to avoid any unexpected results. The design drawings shall specify following construction requirements:

a) The abutments including wingwalls shall be constructed first to the bearing seat elevation.

b) For girder bridges, the girders shall be placed on the abutments with simply supported condition that allows rotation and deflection of the girders due to self-weight and dead weight of the deck.

c) The deck and abutment above bearing seat elevation shall be placed integrally with the deck placement over the girders.

d) The deck end and abutment above the bearing seat level shall be placed in a sequence so that the structure becomes integral with low residual stresses.

e) The stability and the integrity of the structure shall be maintained at all stages of construction.

f) Backfill shall not be placed behind the abutments until the deck has reached 75% of its specified strength.

g) Backfill shall be placed simultaneously behind both abutments, keeping the height of the backfill approximately the same. At no time shall the difference in heights of backfill be greater than 500 mm.

### 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 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 between the deck end and ballast wall. They 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. The distance between the ballast wall and end of girders shall be no less than 600 mm to provide access to the expansion joint, the girder ends, and the bearings for inspection and maintenance (including replacement). Where a modular expansion joint is used, the distance between the ballast wall and end of superstructure shall be sufficient to permit inspection and maintenance of the modular joint components as required by Section 13, and in no case less than 800 mm. Corbels shall be detailed at the inside face of the ballast wall as necessary to provide this minimum distance. A corbel may be detailed at the back face of the ballast wall if needed. The deck shall extend beyond the girder ends, as necessary.

For post-tensioned bridges, the gap requirement between ballast wall and superstructure end shall be achieved by 90 days after casting the deck.

In conventional abutments, the abutment seat shall be detailed to provide at least 1000 mm vertically to the girders to permit access to the bearings for inspection and maintenance.

## 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 of 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 possibly 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:

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 $m^3$ 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 reducing admixture may be specified for massive 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.
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. MTO’s Bearing Replacement and Jacking Guidelines (BRO-062; ISBN 978-1-4868-2356-7) should be referenced for further details.

#### 5.2.2.1 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.

#### 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 abutments, see Figure 5.2-1.

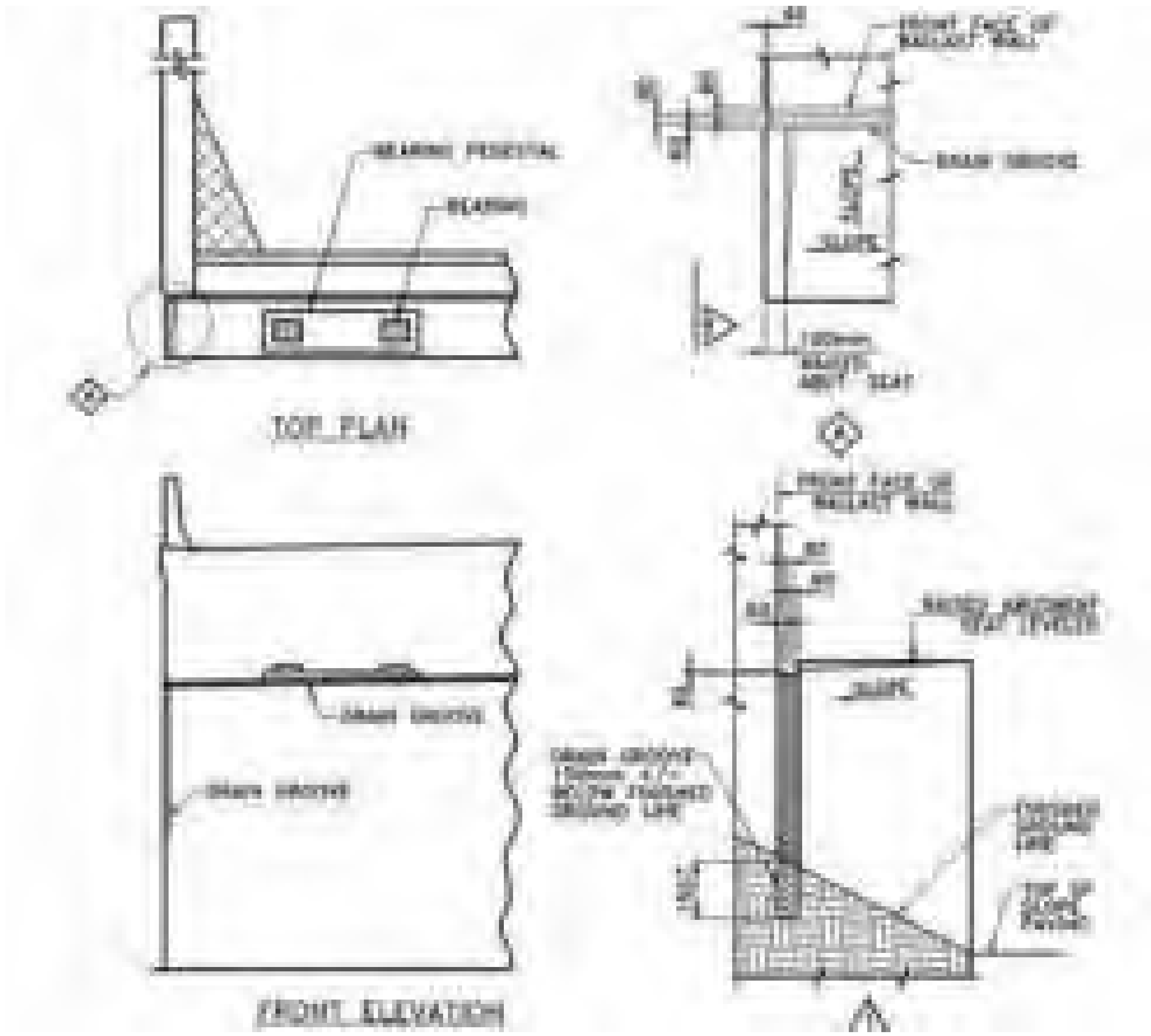


**Figure 5.2-1 – Standard Detail for Rust Stain Control at Abutment**

### 5.2.3 Ballast Walls

#### 5.2.3.1 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.

#### 5.2.3.2 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.

### 5.2.4 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 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.

#### 5.2.4.1 Wingwalls Length

The length of a wingwall, Lw, 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,
$\boldsymbol{L_w} = (E_T - E_L)S + 500mm$

Length of wingwall from the front face of the abutment for skew bridges,
$\boldsymbol{L_w} = \frac{(E_T - E_L)S}{COS\ \theta} + 500mm$ at the abutment acute corner
$\boldsymbol{L_w} = (E_T - E_L)S + 500mm$ at the abutment obtuse corner

Design Length of Wingwalls
L = Lw – D – Length of the cleat along the wingwall

Here,

Lw, L, $E_T$, $E_L$, D and $\theta$ are illustrated in Figure 5.2-2

S = Longitudinal slope of approach embankment along the wingwall as shown in Figure 5.2-2

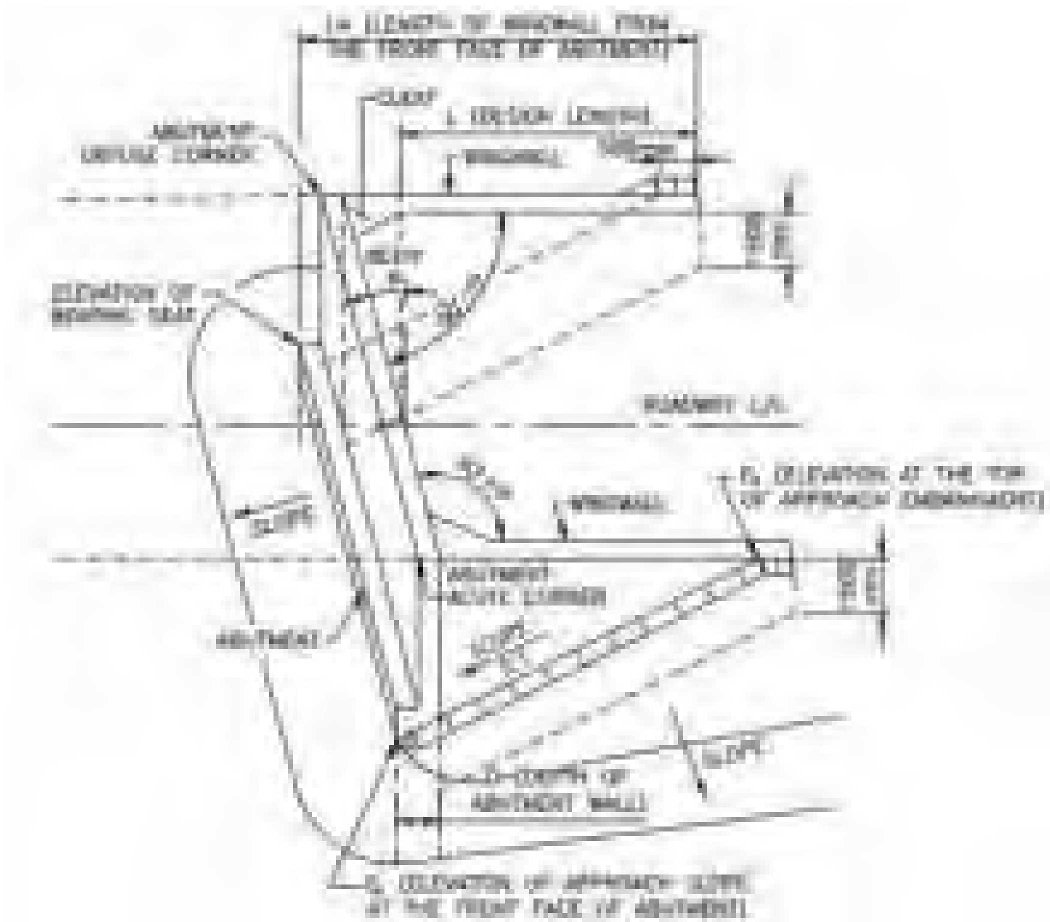

**Figure 5.2-2 – Geometry of Wingwalls Aligned Parallel to Roadway**

### 5.2.4.2 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.

### 5.2.4.3 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, Ka $\gamma$ = 7.0 kPa in the lateral earth pressure calculation. Figure 5.2-3 illustrates design loads on a wingwall.

#### 5.2.4.4 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.

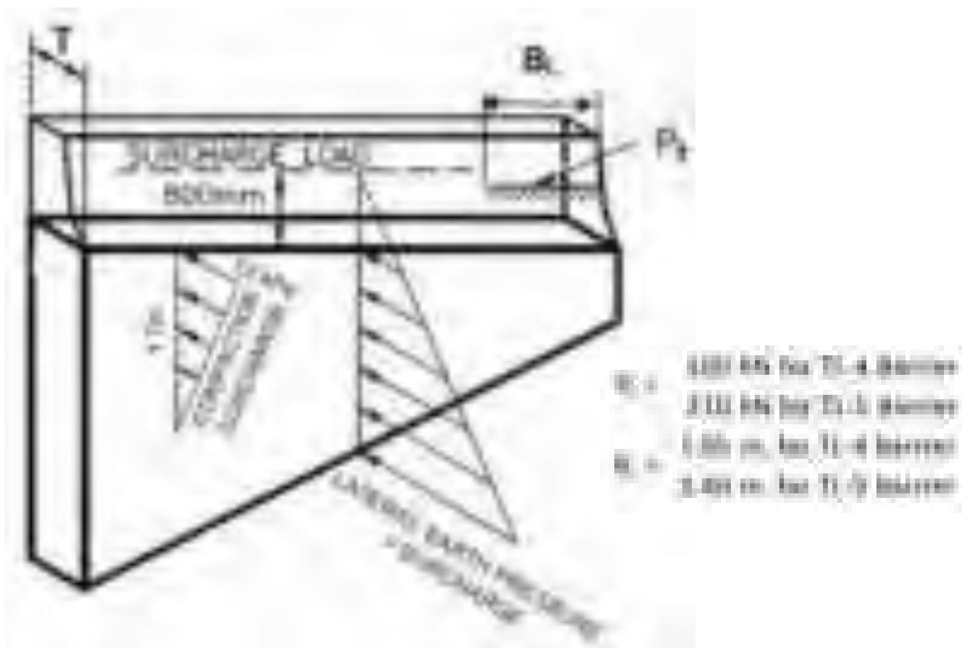


**Figure 5.2-3 – Design Loads Applied on Wingwalls**

#### 5.2.4.5 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.

#### 5.2.4.6 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).

#### 5.2.4.7 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.

## 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 (RRS) 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 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.

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 approach slab should be extended so that the sleeper slab drainage is beyond the end of the wingwalls. The approach slab reinforcement may need to be increased.

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

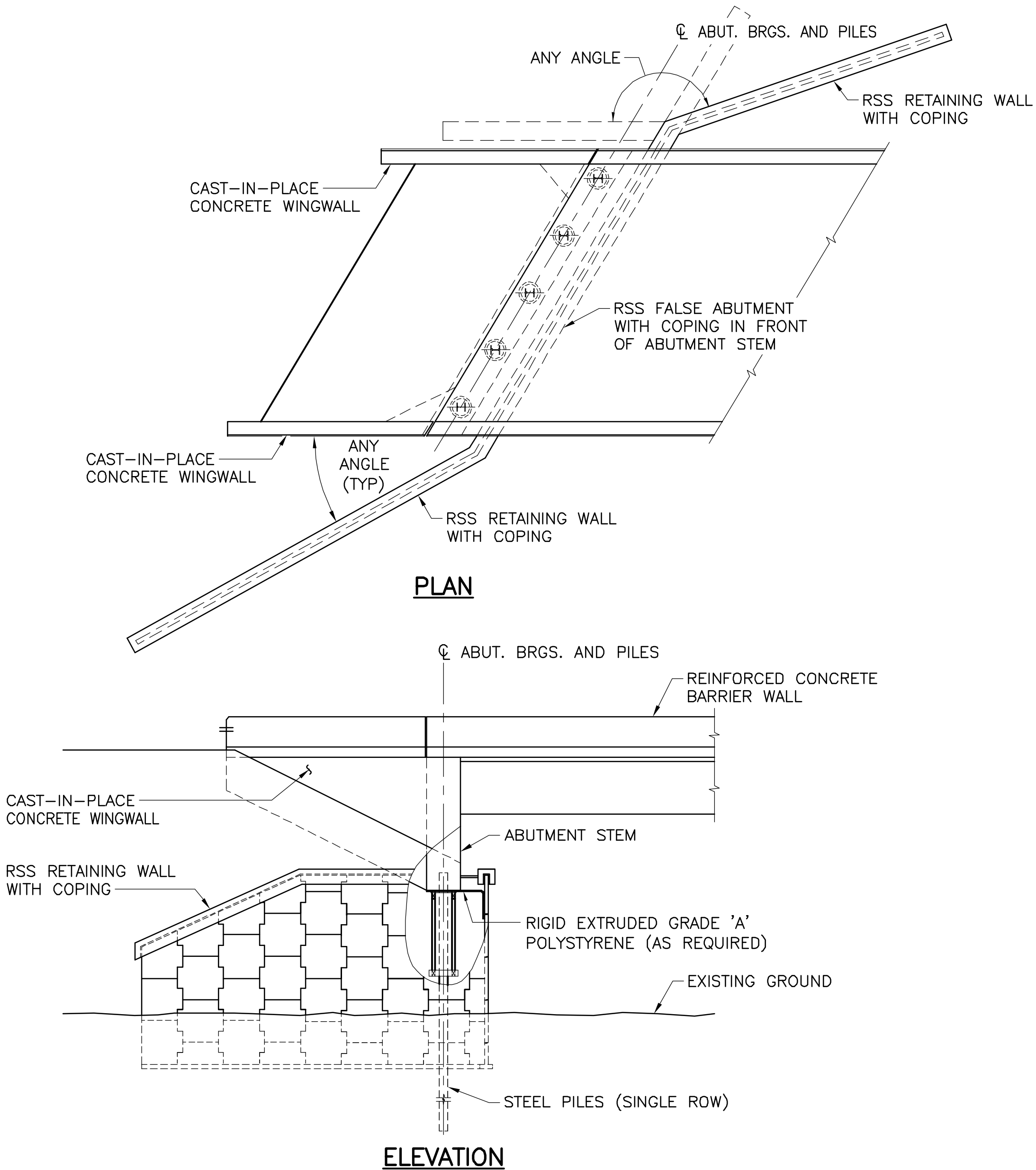


**Figure 5.5-1 – MSE (RSS) False Abutment (In Front of Integral Abutment Stem) with Flared MSE Walls**

(From Figure 4.5 RSS Design Guidelines, 2008)

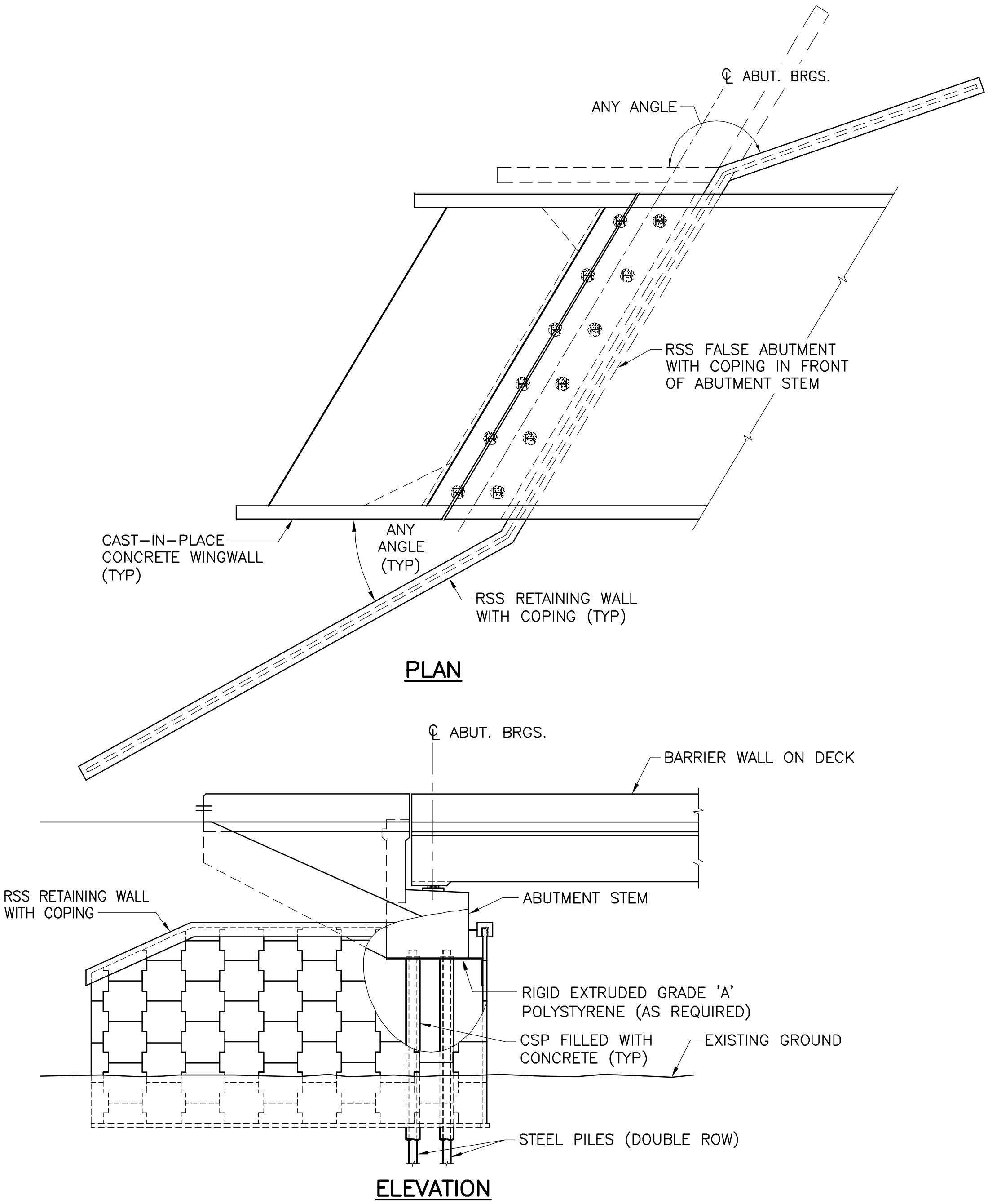


**Figure 5.5-2 – Rigid Piled Bridge with MSE (RSS) False Abutment and Flared MSE Walls**
(From Figure 4.10 RSS Design Guidelines, 2008)

# SECTION 6 - PIERS

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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.

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 6.1-1 and Figure 6.1-2)

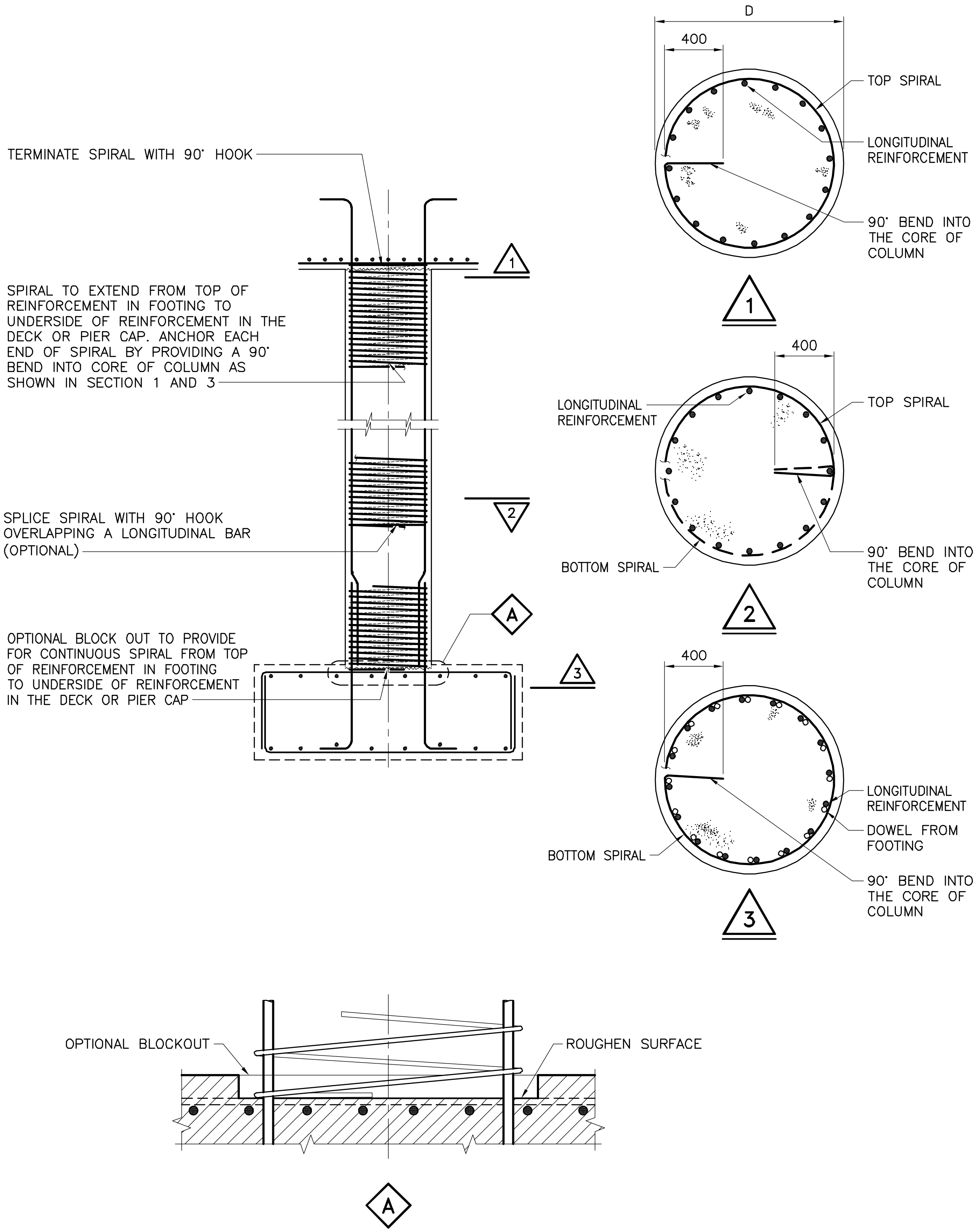


**Figure 6.1-1 – Pier Column Reinforcing Details – Seismic Performance Category 1**

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.

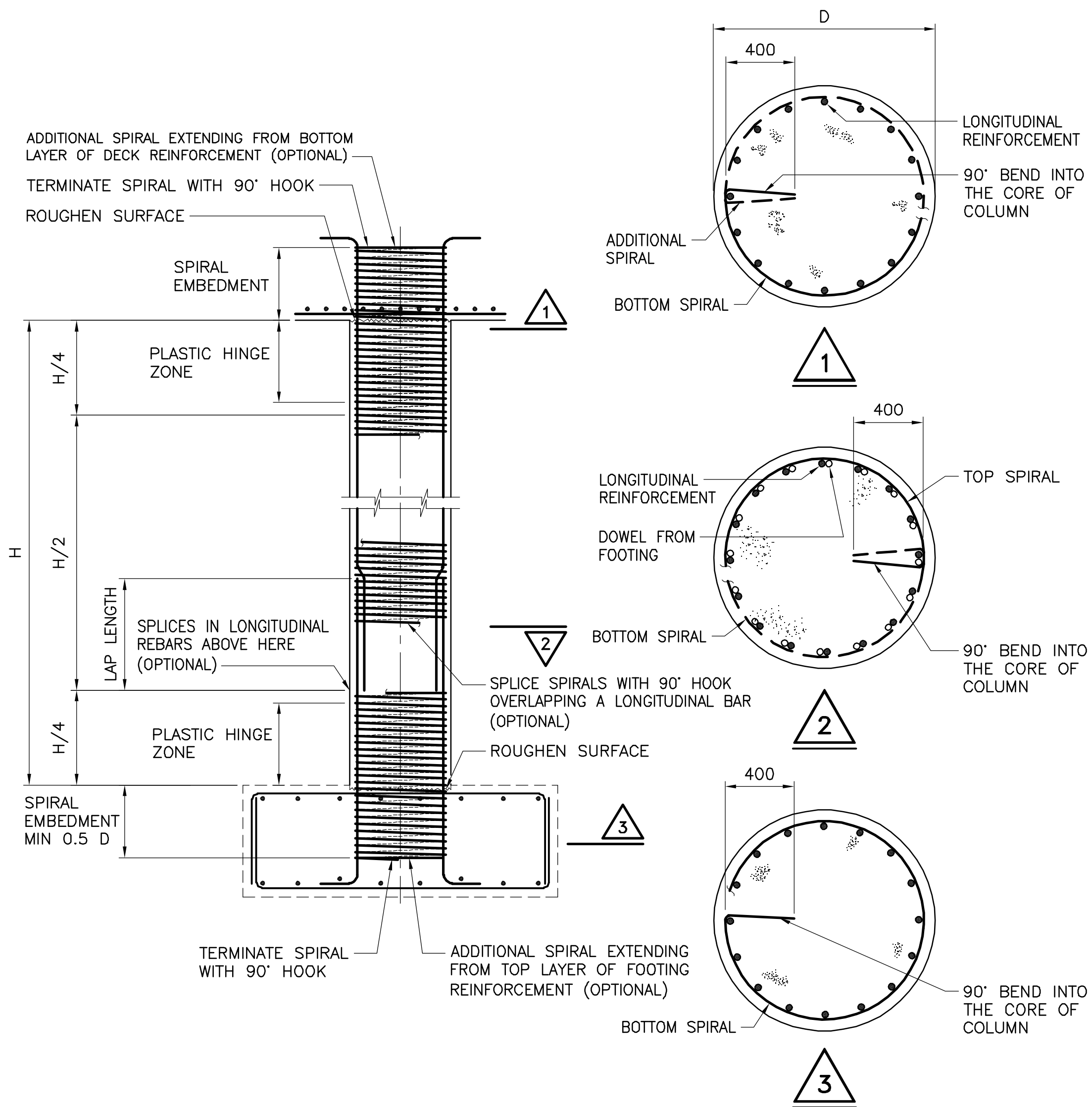


**Figure 6.1-2 – Pier Column Reinforcing Details – Seismic Performance Category 2 & 3**

### 6.1.3.1 Pier Column Reinforcing Details in Earthquake Zones

The use of GFRP spirals in the plastic hinge region of columns is not permitted due to the lack of detailing for proper spiral anchorage.

Discrete GFRP seismic hoops may be used in the plastic hinge regions.

Hybrid systems of mixed steel and GFRP reinforcement in the column are not permitted.

### 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.

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 loading and damage to 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.

## 6.2 Miscellaneous Details

### 6.2.1 Pier Nosing for River Piers

Steel nosing should not be provided unless approved by the Head of Structural Section for angular piers in rivers with heavy ice floes.

### 6.2.2 Pier Bearing Seats

If the bridge deck over a pier is not continuous, the requirements of Section 6.2.3 apply.

The requirements of Section 5.2.2 concerning provisions for jacking also apply 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. The bearing seats 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.

## SECTION 6 - PIERS

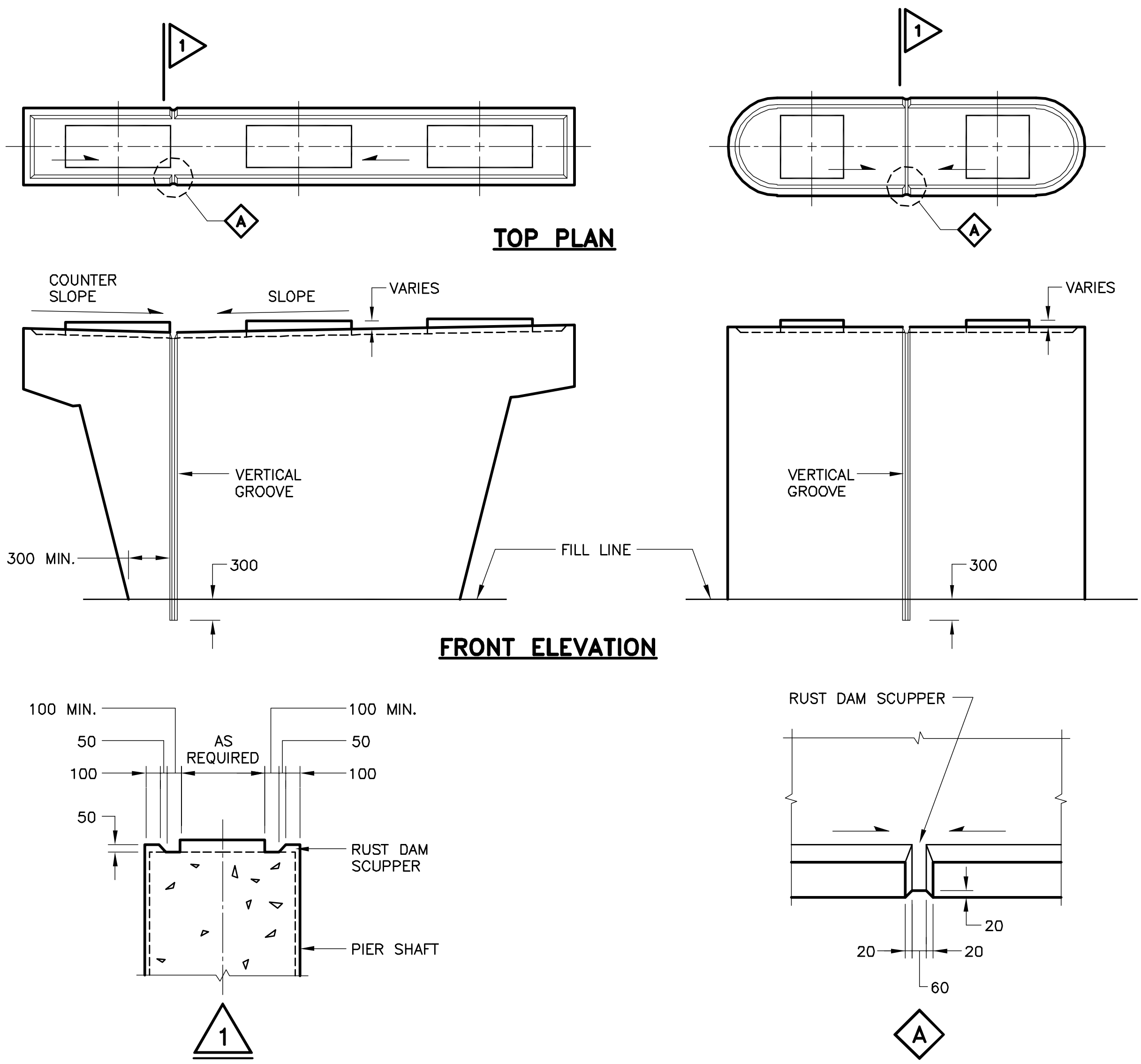


**Figure 6.2-1 – Rust Stain Control – Pier Shafts**

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.

## SECTION 6 - PIERS

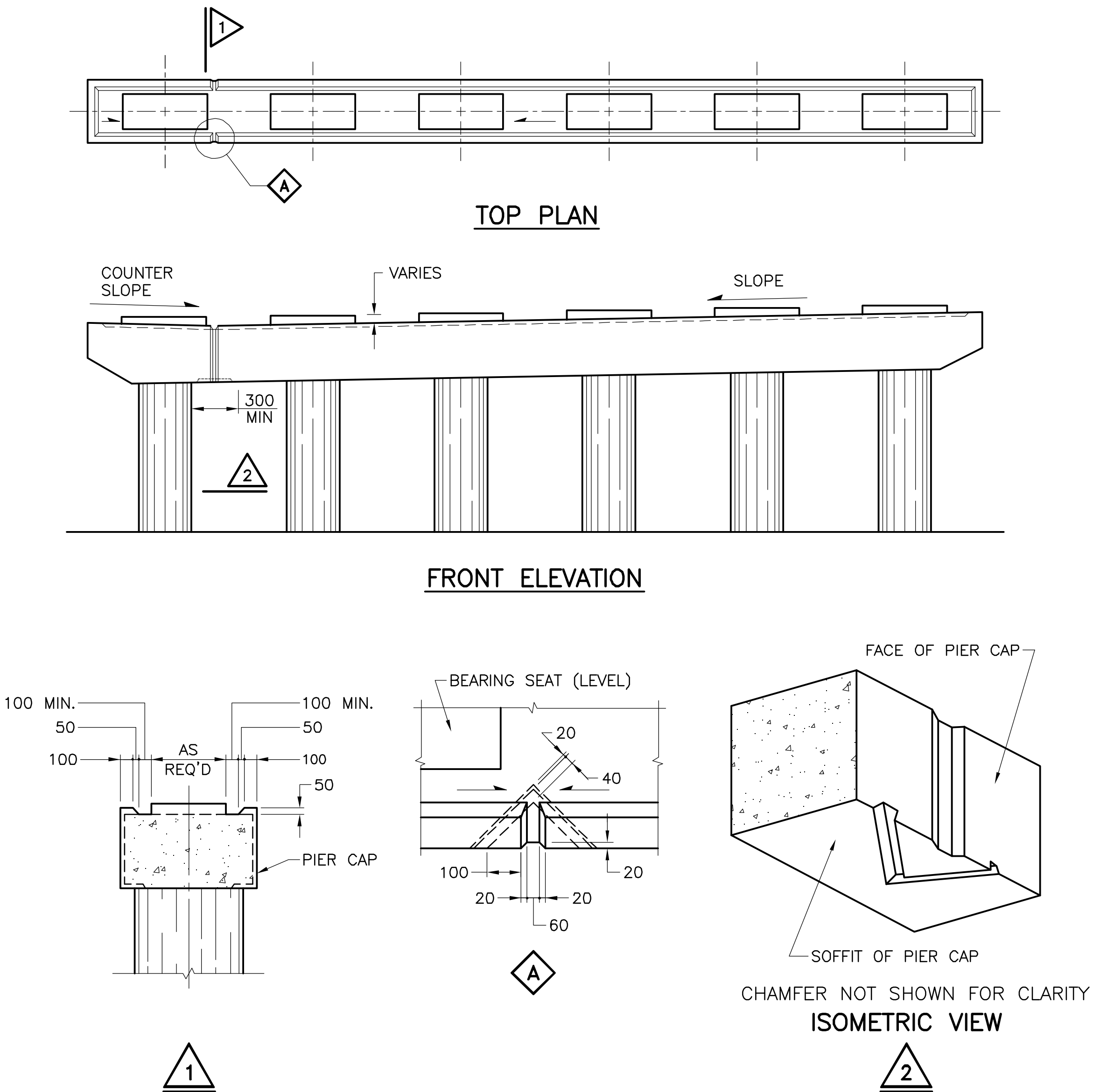


**Figure 6.2-2 – Rust Stain Control – Pier Caps on Columns**

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# SECTION 7 - PRESTRESSED CONCRETE

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# 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 where 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 girders 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.

## 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) The 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

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.

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 has been severely restricted as the cement industry in Canada moves exclusively towards GUL and HEL Portland-limestone cements to reduce environmental impacts. Therefore, it is no longer possible to achieve 45 MPa transfer strength with a single day with HEL because the limestone does not contribute to accelerated strength gain.

e) Precast, pre-stressed girders may be 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.

   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.

f) In calculating stresses, the transformed area of the strands may be used to compute section properties.

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. 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.

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 Durability

Based on Ministry experience and quality assurance testing, typical precast girders with compressive concrete strength of 50 MPa have RCP values averaging 1500 Coulombs at 28 days. When cover requirements are met, temperature limits respected, and other specification requirements are complied with, precast concrete girders will provide adequate durability for most exposures and situations.

Precast concrete girders in exposure classes A1 to A4, specified through OPSS 909, meet the requirements of CSA S6:25.

Precast concrete girders in exposure class A5 (within the vertical bounds of the splash and spray zone above highways) shall be specified to contain silica fume to meet the requirements of CSA S6:25. The corresponding tender items for Prestressed Concrete Girders Containing Silica Fume shall be used, and the following note shall be added to the precast girder drawing notes:

CONCRETE GIRDERS SHALL CONTAIN SILICA FUME.

### 7.2.4 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.

### 7.2.5 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 greater than 10 m shall be prestressed.

### 7.2.6 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 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.

### 7.2.7 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.

### 7.2.8 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 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.

### 7.2.9 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 containing the girder details, is a function of +G-D+C. At the high end, dimension ‘c’ 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.

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.

### 7.2.10 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. ASSUMED DEAD LOAD FROM DECK FORM SYSTEM: 0.80 kPa (or that assumed by designer if different than 0.80 kPa)
   (Choose one of the following as applicable)

ASSUMED DEAD LOAD FROM FALSE DECK SYSTEM: 0.35 kPa (or that assumed by designer if different than 0.35 kPa). NO LIVE LOADING OR STORAGE OF MATERIALS OR EQUIPMENT ARE PERMITTED ON THE FALSE DECK DURING THE DECK POUR.
Or
FALSE DECK SYSTEM IS NOT PERMITTED. (when approved)
ASSUMED BRIDGE DECK FINISHING MACHINE BIASED (HEAVY SIDE) LOAD ON ONE RAIL: XX kN.
ASSUMED WORK BRIDGE LOAD ON ONE RAIL: XX kN

## 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 20º, transverse prestressing cables and reinforcing steel should be square to the deck except over skewed supports.

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.

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 tendons 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.

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 mm$^2$.

Solid slab and circular voided post-tensioned structures do not require tendons larger than size designation 19/15 (containing 19, Size 15 strands). In the case of trapezoidal voided and box girder post-tensioned structures, large tendons are beneficial to reduce the width of the web and/or increase the eccentricity of the strand 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 bears 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 lowe 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.

**Table 7.3-1 Post-Tensioing Tendons and Duct Sizes**

| No. of 15 mm Strands | Plastic 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 area 25 x 76 mm and outer dimensions are 40 x 90 mm.

### 7.3.7 Post-Tensioning Tendon Duct and Web Details

Duct arrangement and corresponding reinforcement layout and web dimensions are shown in Figure 7.3-1. Dimensions are based on 27/15 tendons.

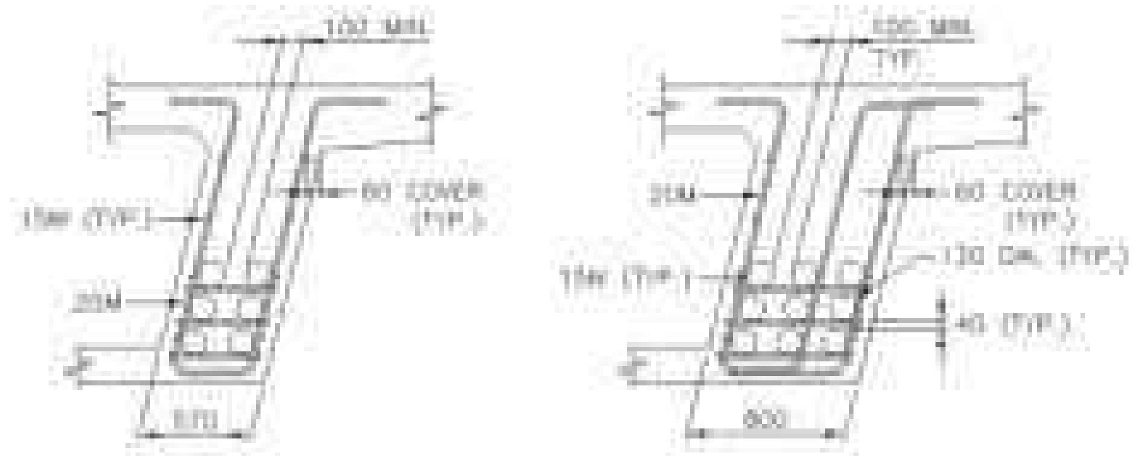

**Figure 7.3-1 – Duct Arrangement for Rectangular Voids**

### 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.

### 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.<br>± … 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 OF ASTM A416/A416M. MINIMUM BREAKING STRENGTH SHALL BE 260 kN PER STRAND.

12. SHEATHS FOR POST-TENSIONING DUCTS SHALL BE CORRUGATED PLASTIC.

**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 MPa, K=0.001, μ=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 MPa, K=0.001, μ=0.14

| 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 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.

2. STRESS ALL TRANSVERSE TENDONS AT PIERS AND ABUTMENTS, THEN IN SPANS, WHEN CONCRETE IN DECK HAS REACHED A STRENGTH OF 30 MPa.

3. STRESS ALL LONGITUDINAL TENDONS AFTER CONCRETE IN TRANSVERSE ANCHORAGE RECESSES HAS REACHED A STRENGTH OF 30 MPa.

4. GROUT ALL TRANSVERSE AND LONGITUDINAL TENDON DUCTS.

5. PLACE 35 MPa CONCRETE AT RECESSES FOR ALL LONGITUDINAL AND TRANSVERSE TENDONS.

6. 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).

# SECTION 8 - STRUCTURAL STEEL

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# 8 STRUCTURAL STEEL

## 8.1 Design

### 8.1.1 General

The Service Life Design procedure within the CHBDC shall be followed to mitigate the risk of structural steel corrosion. CHBDC Commentary Clause C2.3.2.4 shall be applied to determine the limits of the deicing splash and spray zone adjacent to and above highway lanes. The planned ultimate lane configuration of the facility shall be used to determine the outside edge of the lanes.

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. To achieve the performance expected of uncoated ACR steel, prolonged exposure to wetness without opportunity to dry, and exposure to high levels of chlorides ($Cl^-$) need to be avoided. Over time, requirements for local coating of ACR in areas of wetness or chlorides have been added. By observation and testing, some sites have proven to be inappropriate for ACR and the structural steel has been subsequently coated.

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. 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 which slow the drying process and accelerate corrosion; and,
- Debris that traps moisture.

The goal of design and detailing is to prevent the conditions above from occurring, and to provide additional protection to the structural steel if they cannot be avoided.
Locations with past performance deficiencies of ACR steel (a stable patina has not developed, and the corrosion product flakes off) shall be classified as exposure class A5 with corrosivity category C4, and replacement with uncoated ACR steel is not recommended. For these locations, enhanced corrosion protection (i.e., zinc-rich paint coating system, galvanized, 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.

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. Exterior longitudinal girders shall not have less capacity than interior girders.

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) For primary members and diaphragms, plate girders shall be specified rather than specifying rolled W sections which are difficult to procure in Ontario. There is no advantage to specifying welded wide flange (WWF) and welded reduced flange (WRF) designations, and instead the girders should be designed with an optimal plate configuration for the demands.

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. At connections which are coated on one side only and ACR on the other side, Type 3 bolts shall be used. 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.

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 structural steel shall be checked for structural adequacy assuming a loss of steel thickness per CSA S6:25 Table 2.5, as required by Clause 10.6.4.2.

n) Exterior girders shall be classified as exposure class D1 in the following scenarios and protected according to Clause 10.6.4 of the CHBDC.

   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). For durability, a larger gap between twin structures is preferred over narrow, longitudinal joints.

   2. Girder webs below open railings for bridges with steel box-girders as shown in Figure 8.1-1(b);

   3. Girder webs and bottom flanges below open railings for bridges with steel I-girders as shown in Figure 8.1-1(c).

## SECTION 8 - STRUCTURAL STEEL

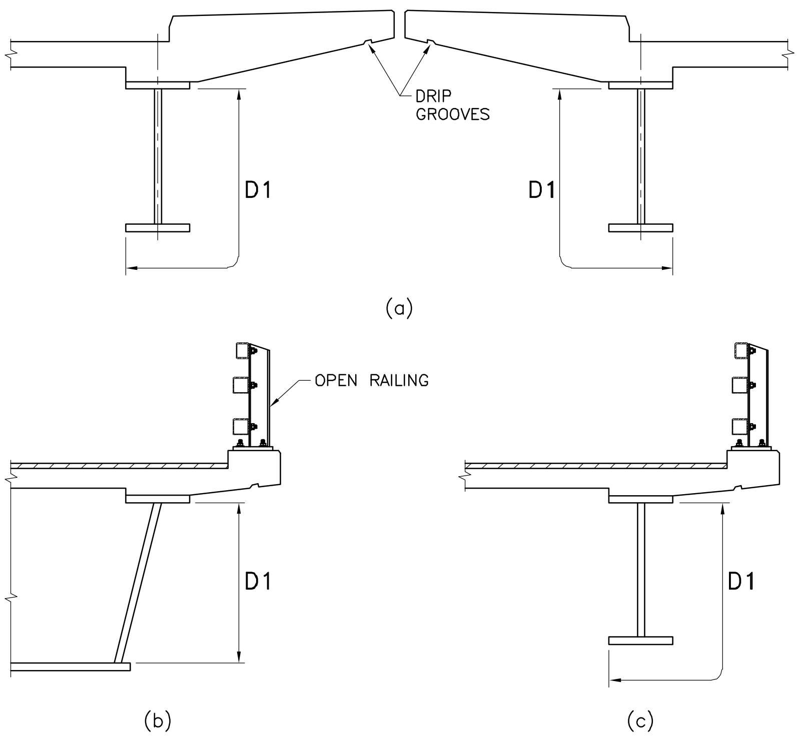



**Figure 8.1-1 – Exposure Class for Steel Girders Below Longitudinal Joint and Open Railings**

### 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 in exposure classes D1, D2 and D3. 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 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).

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). The coating is required to mitigate the risk of corrosion from increased condensation at the abutments, and to prevent staining of the concrete.

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 and are classified as corrosivity category C4. 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.

The coating class (i.e., Class A to D as per S6, Table 10.8) assumed in design shall be stated on the drawings. Wherever possible, Class A coating shall be assumed in design to provide more options for coating systems in construction.

### 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 coating 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.

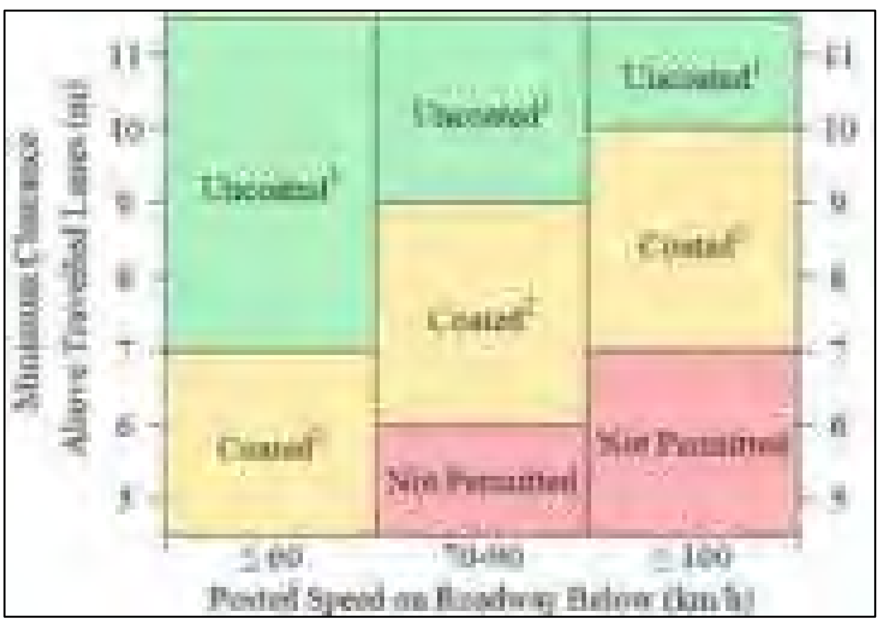



**Figure 8.1-2 – Use of ACR I-Girders Over Highways**

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 which are classified as corrosivity category C4 (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.

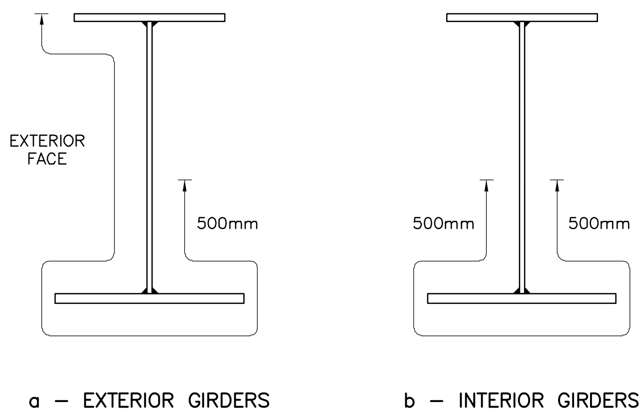



**Figure 8.1-3 – Paint Coating Minimum Limits**

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. I-girders are appropriate for most railway and water crossings.

In the last decade, 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 Hot Dip Galvanizing

Galvanizing plants in Ontario provide kettle sizes up to approximately 17 m in length, 2.4 m in width, and 3.6 m in depth. The largest kettle in Ontario is approximately 16.7 m x 2.4 m x 3.5m, and can accommodate a structural component up to 16.1 m x 2.3 m x 3.3 m. The longest kettle in Ontario is approximately 17 m x 1.8 m x 3.6 m which can accommodate a structural component up to 16.9 m x 1.6 m x 3.4 m. Larger components require progressive (double) dipping or dipping in the horizontal position, both of which are not advisable due to the potential for distortion from uneven thermal expansion and for aesthetic reasons, such as a visible overlap area of increased coating thickness.

For more specific details of the kettle sizes, clearance requirements, and locations of galvanizing plants across Ontario, contact the MTO Structures Office Manager.

Considerations should be made with respect to the material used in components to be galvanized. Plate thicknesses under 6 mm should be avoided. The use of thick and thin sections in the same assembly should be minimized. For girders, the flange to web thickness ratio should be less than 3:1. Non-symmetrical sections such as angle and channel are more susceptible to distortion.

During fabrication, steps to reduce induced stresses should be considered, such as the use of backstep welding and minimizing cold working. Adequate venting and drainage must be provided to avoid air pockets, ensure complete coverage, and reduce trapped cleaning solutions.

For further information, refer to *ASTM A384/A384M Standard Practice for Safeguarding Against Warpage and Distortion During Hot-Dip Galvanizing of Steel Assemblies* and the American Galvanizers Association Design Guide: *The Design of Products to be Hot-Dip Galvanized After Fabrication* and *Hot-Dip Galvanized Steel Bridges: A Practical Design Guide*.

Metallizing with a seal coat from MTO's DSM list should be specified for components which are too large to be hot dip galvanized.

### 8.1.6 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, resulting in a corrosivity category C4 or C5. 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.

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, and should look closely at nearby bridges with similar exposure conditions to appropriately assess the corrosivity category of the bridge under considerations. The structure's micro-climate may be assessed by referencing the National Steel Bridge Alliance (NSBA) Uncoated Weathering Steel Reference Guide, 2022.

### 8.1.7 Use of ACR Steel Girders Over Water Crossings

Uncoated ACR steel girders shall not be used over water crossings in problematic wet environments defined in Clause 10.6.4.3 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.

### 8.1.8 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.

### 8.1.9 Standardised Nomenclature for Structural Steel Shapes

The designations for various structural sections are standardised as listed below.

**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<br>(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.<br>(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.

### 8.1.10 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.

### 8.1.11 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.

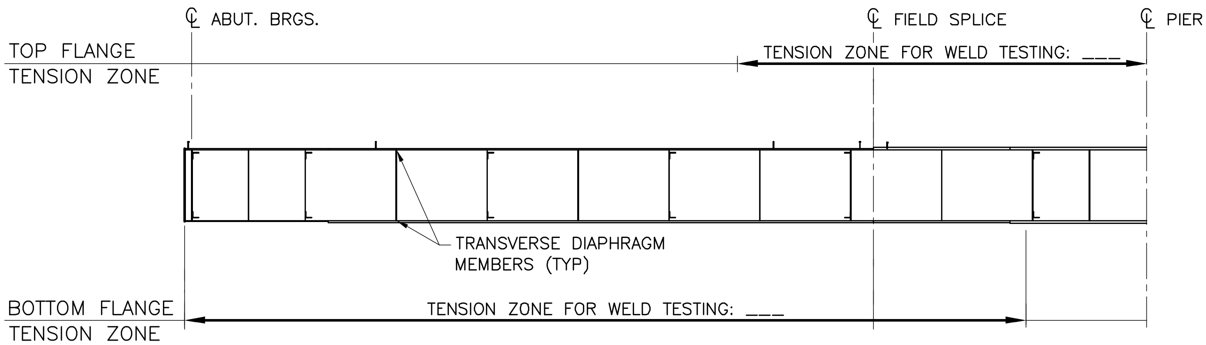



**Figure 8.1-4 – Drawing Showing Tension Zones for Weld Testing**

### 8.1.12 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.

### 8.1.13 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).

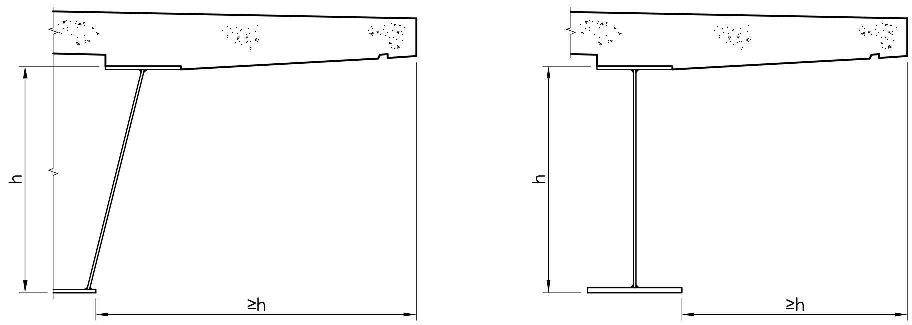



**Figure 8.1-5 – Minimum Cantilever Overhang**

### 8.1.14 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.

### 8.1.15 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.

### 8.1.16 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.

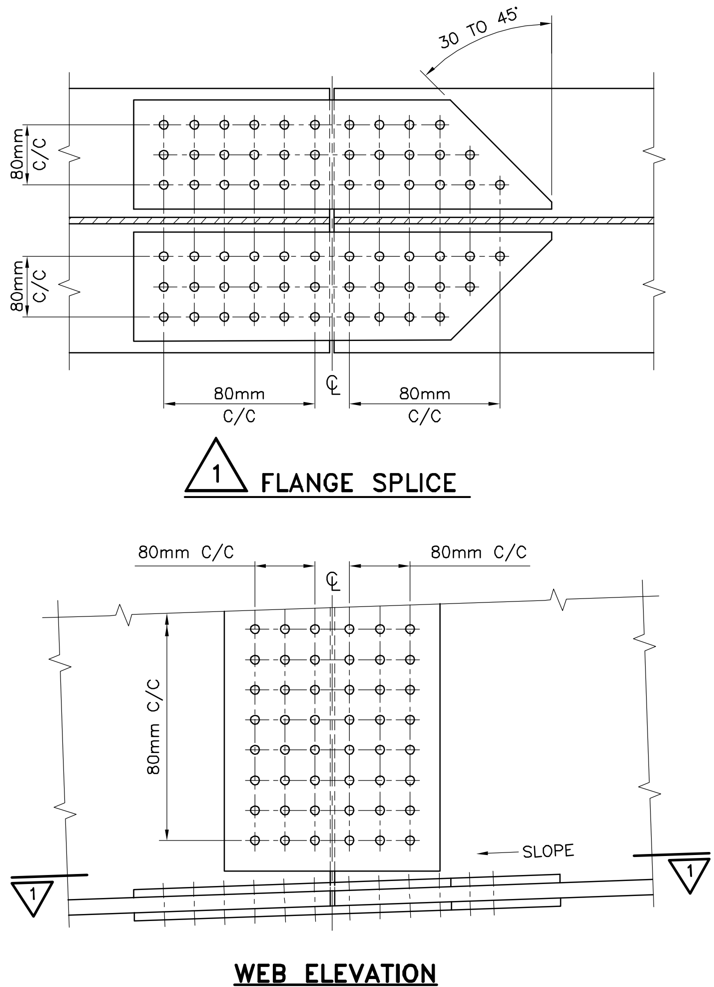



**Figure 8.1-6 – Possible Bolt Layout in a Field Splice**

## 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 ordinates are given at the 10$^{th}$ 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).

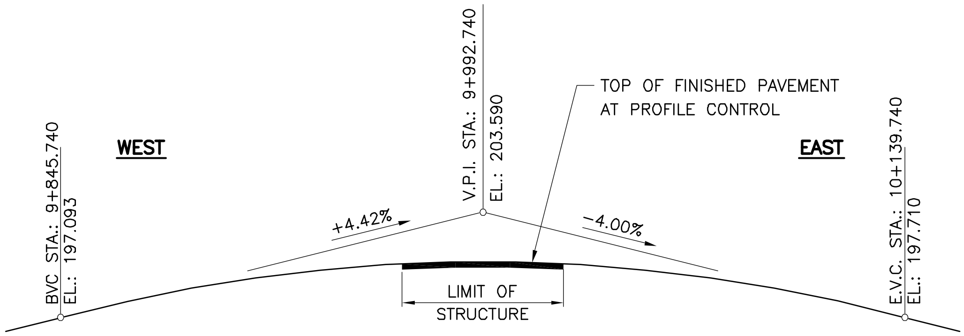
### 8.2.2 Elevations

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

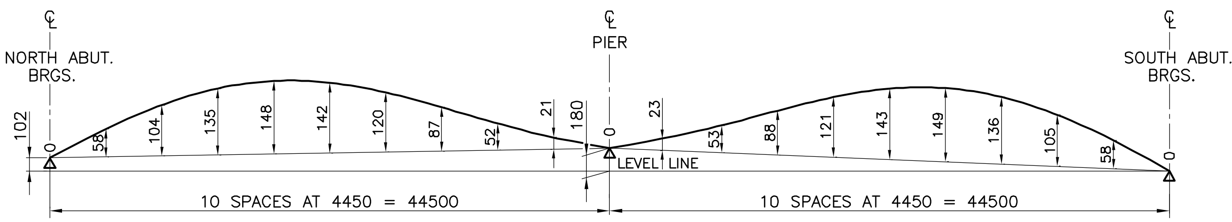
The sketches in Figure 8.2-1 illustrate the various diagrams required for steel box girders, to be shown on the structural steel contract drawings.




PROFILE OF ROAD

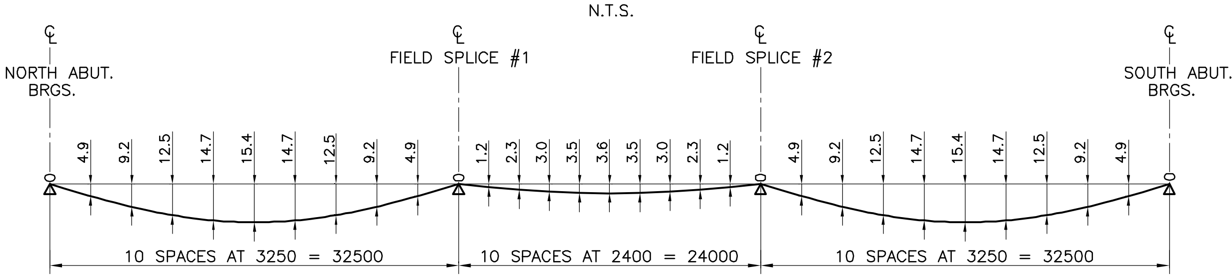
N.T.S.

(FROM GENERAL ARRANGEMENT DRAWING)




RELAXED CAMBER DIAGRAM

N.T.S.




DEFLECTIONS FOR GIRDER SEGMENTS

N.T.S.

| GIRDER ERECTION ELEVATIONS | | | | | | | |
|---|---|---|---|---|---|---|---|
| GIRDER | ℄ W. ABUT. BRGS. | ℄ WEST SPAN | ℄ W. FIELD SPLICE | ℄ PIER | ℄ E. FIELD SPLICE | ℄ EAST SPAN | ℄ E. ABUT. BRGS. |
| 1 | | | | | | | |
| 2 | | | | | | | |
| 3 | | | | | | | |
| 4 | | | | | | | |

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.

Water ponds and debris accumulate on horizontal surfaces and in corners formed by horizontal and vertical plates (re-entrant corners), fostering excessive corrosion.

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).

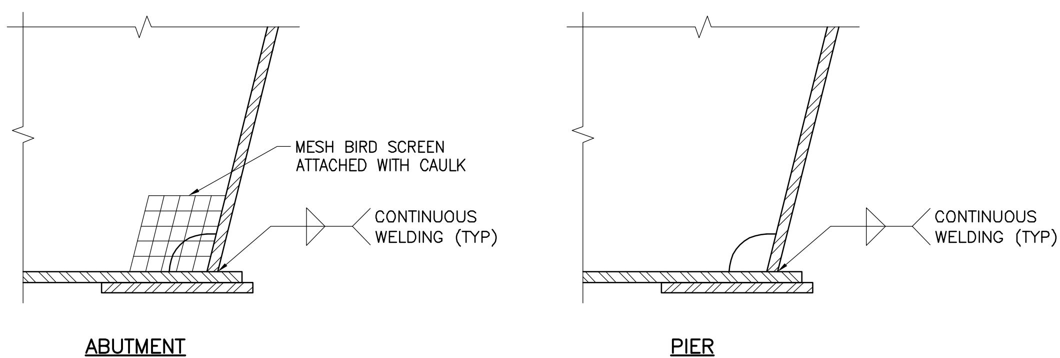


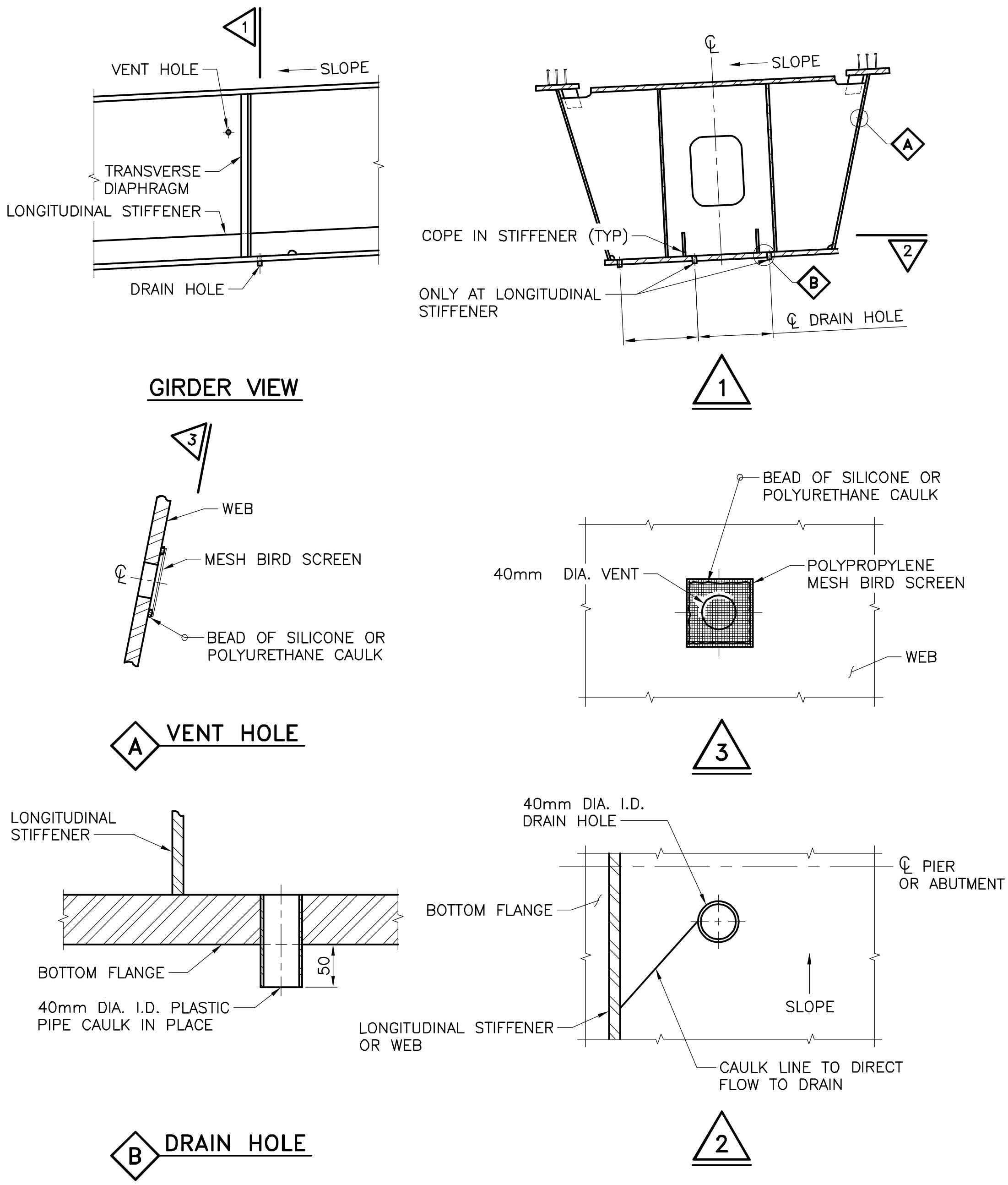

**Figure 8.3-1 – Bottom Flange Welding Details**

### 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.




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.

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) Simple span steel box girders with walk through cross bracing shall have two access openings per box girder, one located near each end;

b) Continuous steel box girders with walk through cross bracing shall have pier diaphragms with openings and shall have two access openings per box girder, one located near each end. For exceptionally long girders, the need for additional intermediate access 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 they 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 designed along with transverse and longitudinal stiffeners around the opening, as required.

### 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.

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.

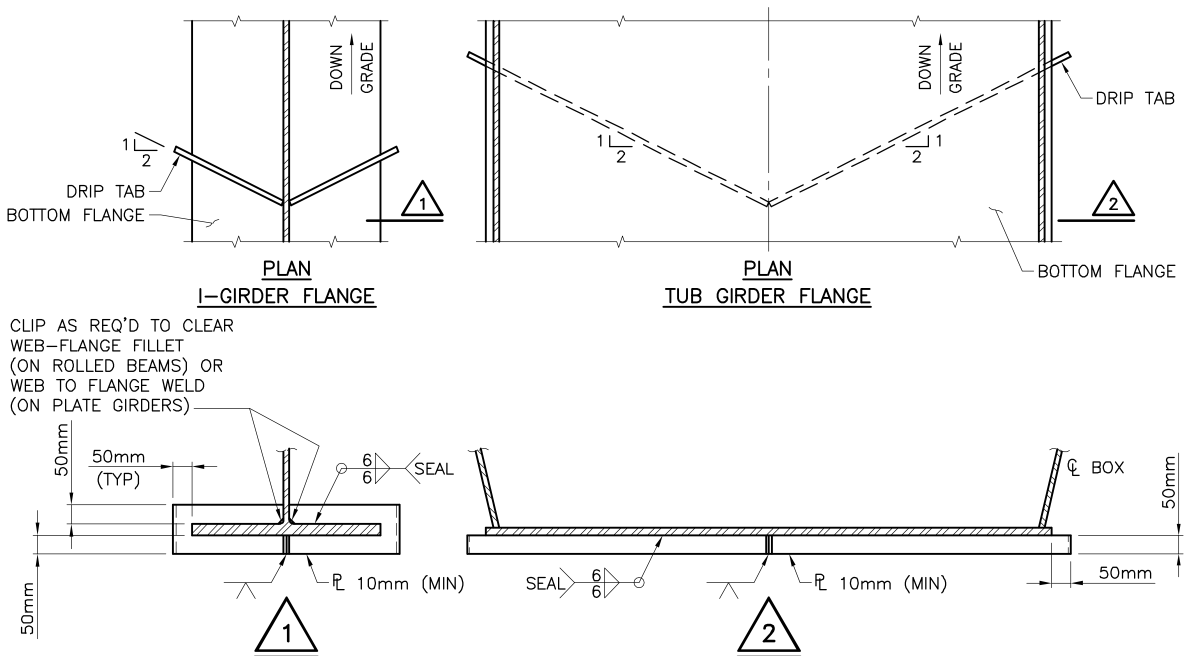



Figure 8.3-3 – Drip Tab Details for I-Girders and Box Girders

### 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.

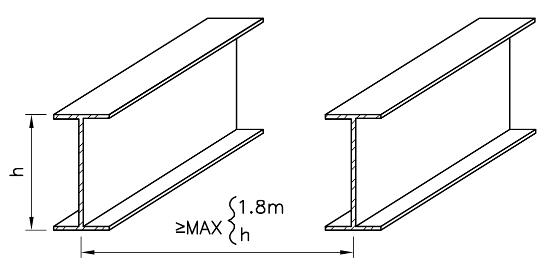



Figure 8.3-4 – Girder Spacing

## 8.4 Stiffeners and Connection Plates

### 8.4.1 Coping of Stiffeners and Gusset Plates

Copes on transverse stiffeners shall 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).

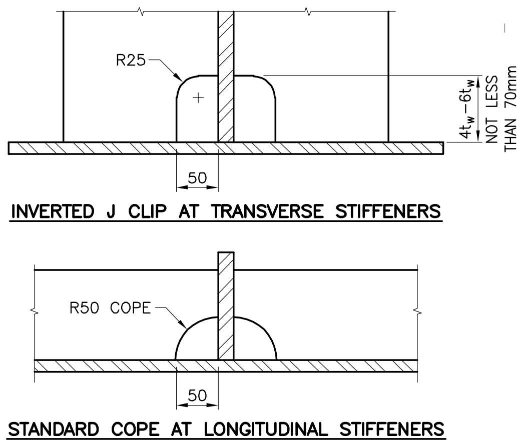



**Figure 8.4-1 – Cope Details**

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.

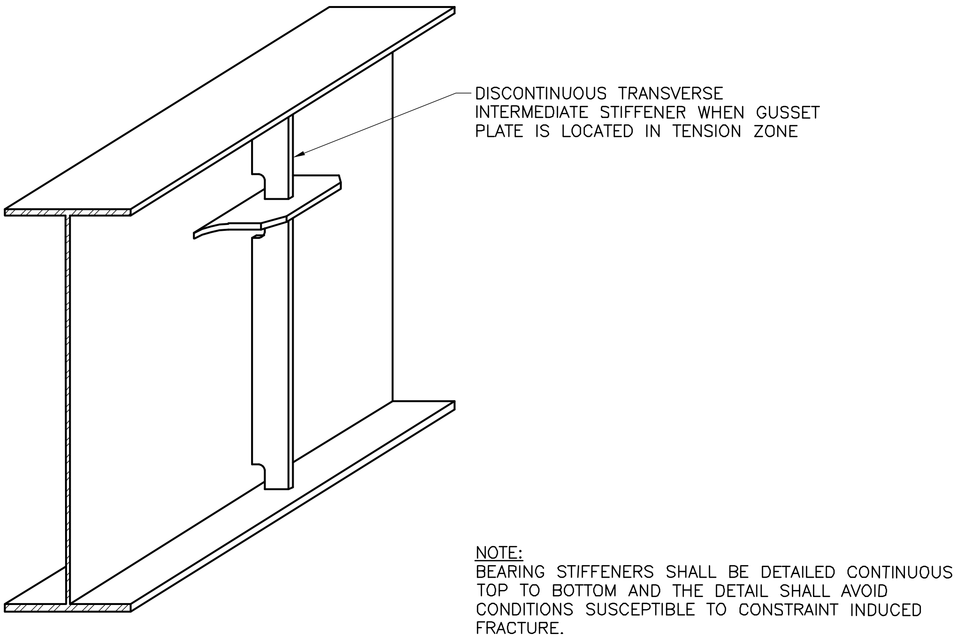



**Figure 8.4-2 – Coping of Stiffeners and Location of Gusset Plates**

### 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.

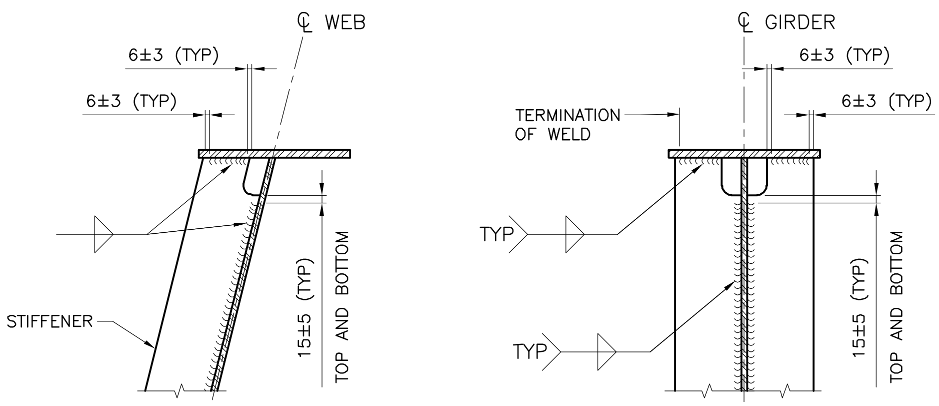



**Figure 8.4-3 – Weld Termination Details**

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 fop 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.

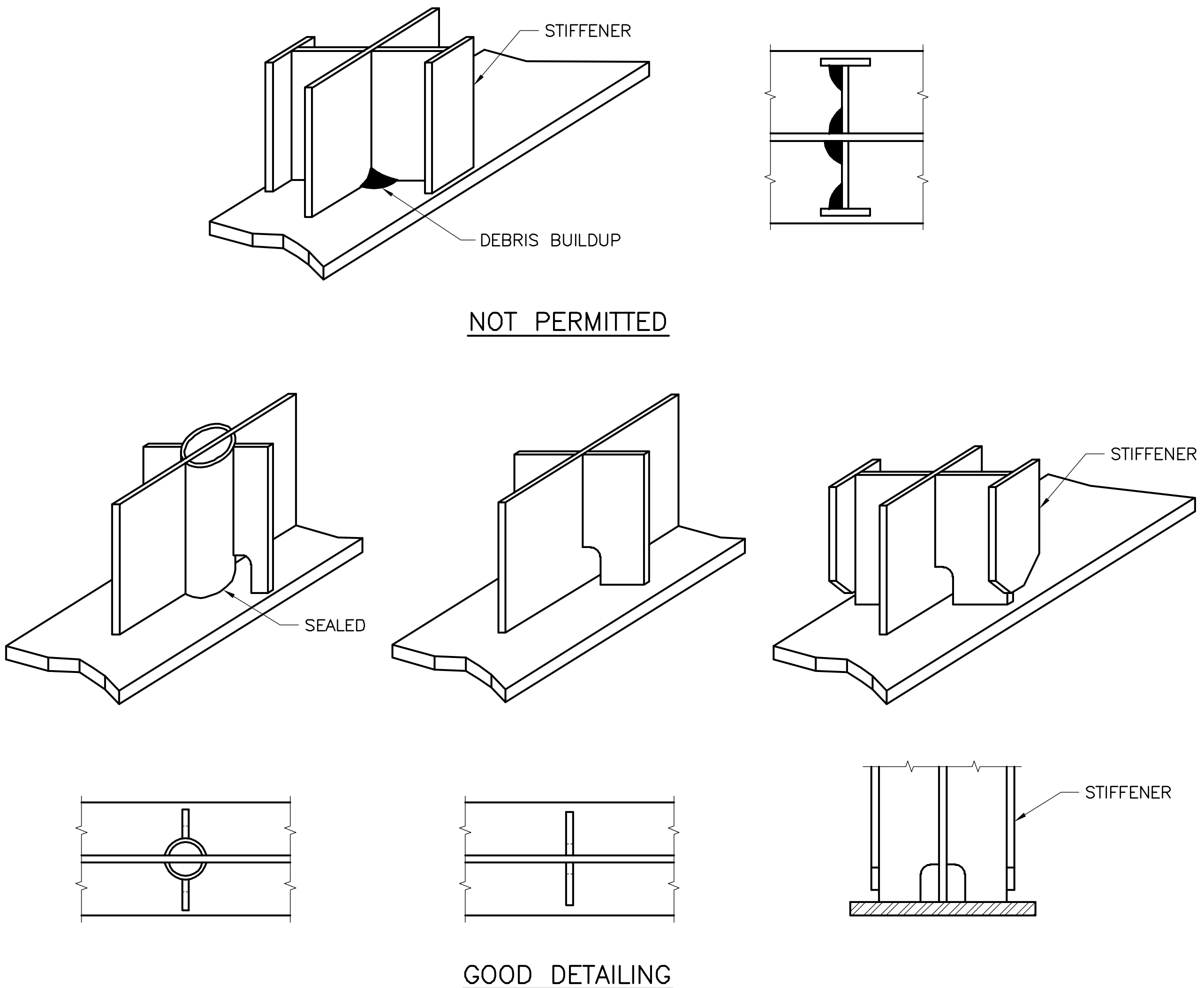



**Figure 8.4-4 – Choice of Vertical Stiffeners**

### 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”.

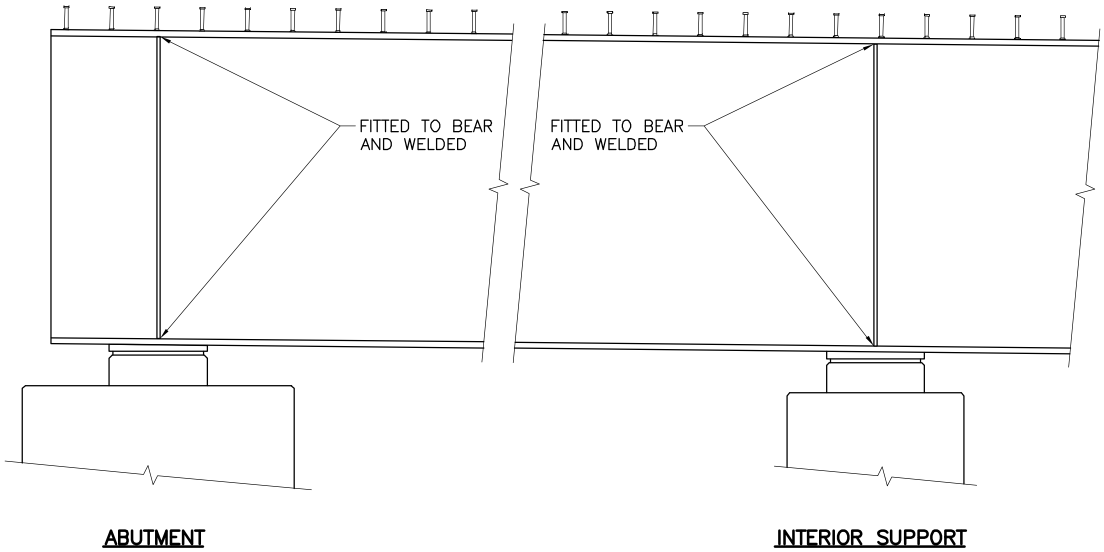



**Figure 8.4-5 – Bearing Stiffener to Flange Connection**

### 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.

### 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.

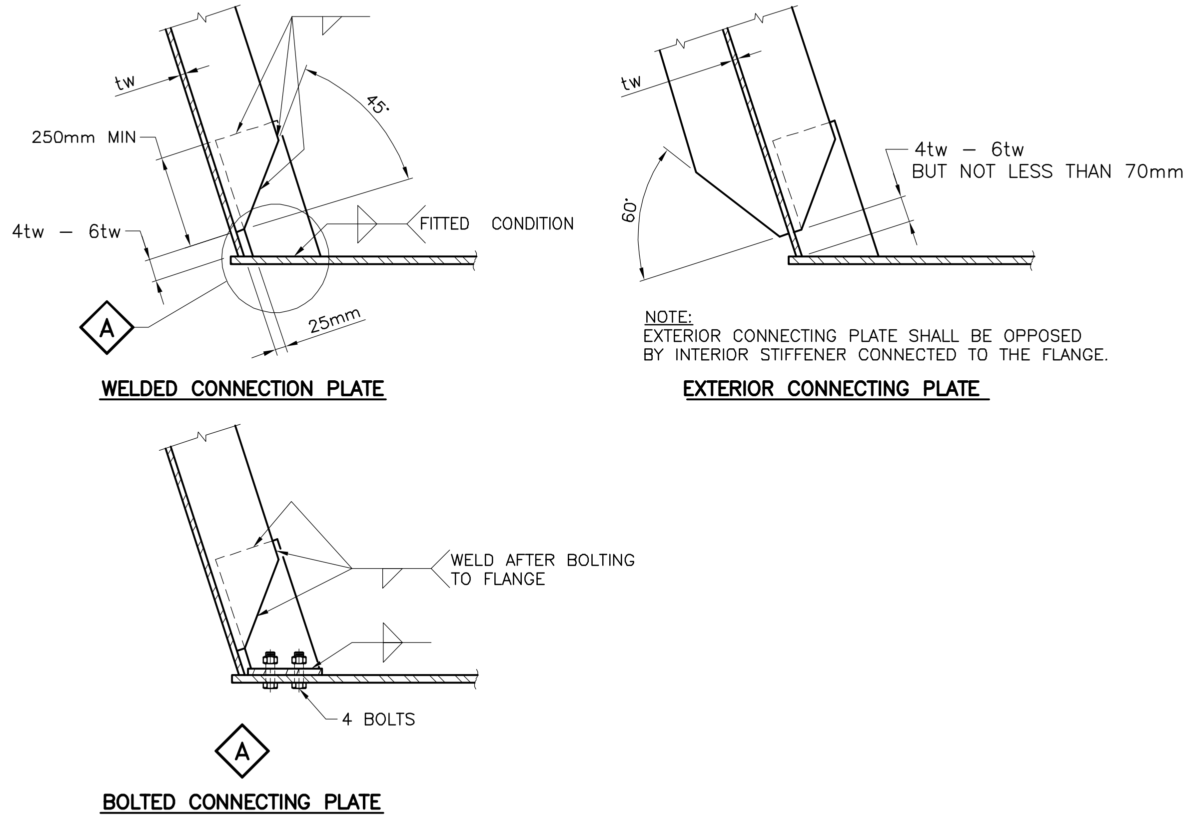



**Figure 8.4-6 – Box Girder Web Stiffeners**

### 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.

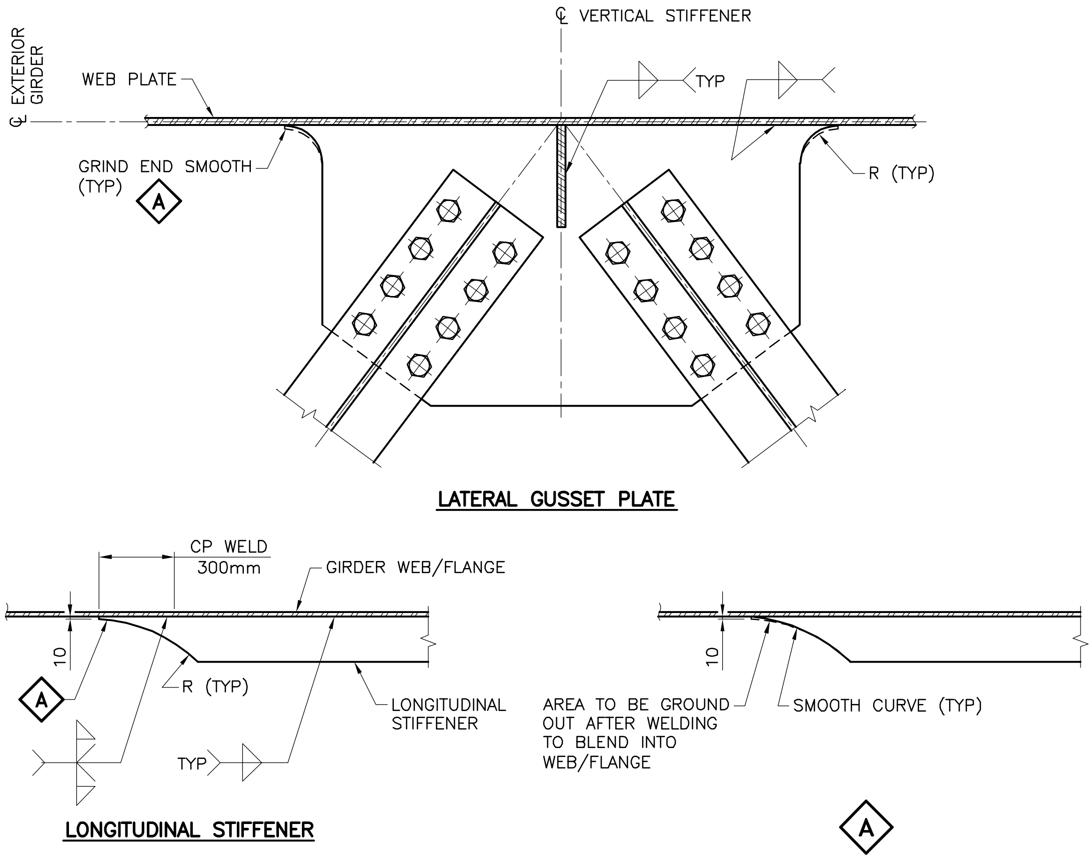



**Figure 8.4-7 – Transition Radius for Gusset Plates and Longitudinal Stiffeners**

### 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).

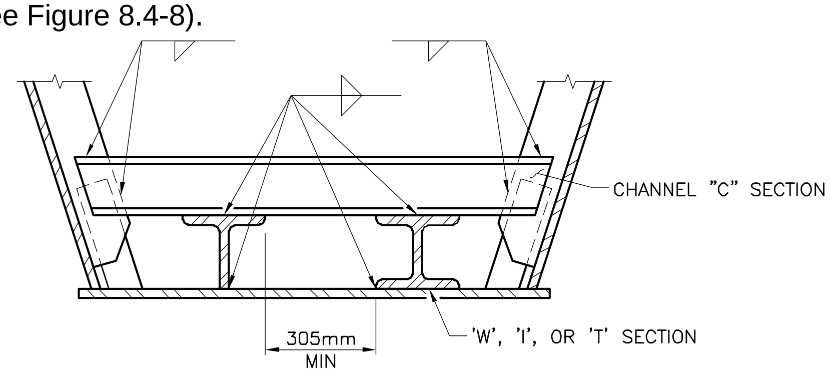



**Figure 8.4-8 – Bottom Flange Stiffener Details**

### 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. Except for bridges over freeways, the preferred "K" brace system for use between girders shall consist of angles shop welded to one side of gusset plates which are field bolted to the girder stiffeners. This 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.
. Oversized holes in the gusset plates are permitted when required.

The size and length of welds connecting angles of K-frames to gusset plates shall be detailed on the contract drawings.

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.

### 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.

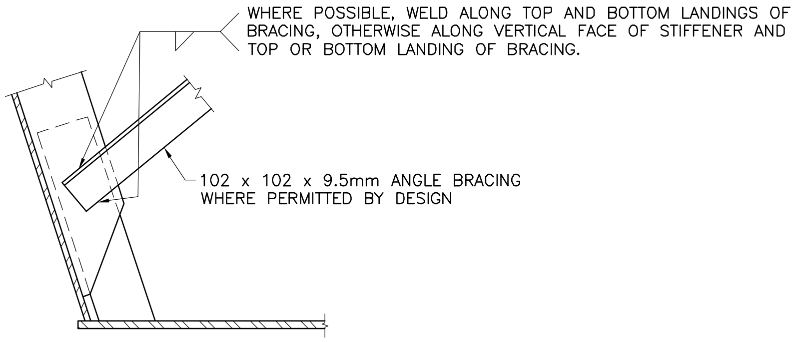



**Figure 8.5-1 – Box Girder Bracing**

### 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.

The details as shown in Figure 8.5-2 were developed to meet design and fabrication needs.

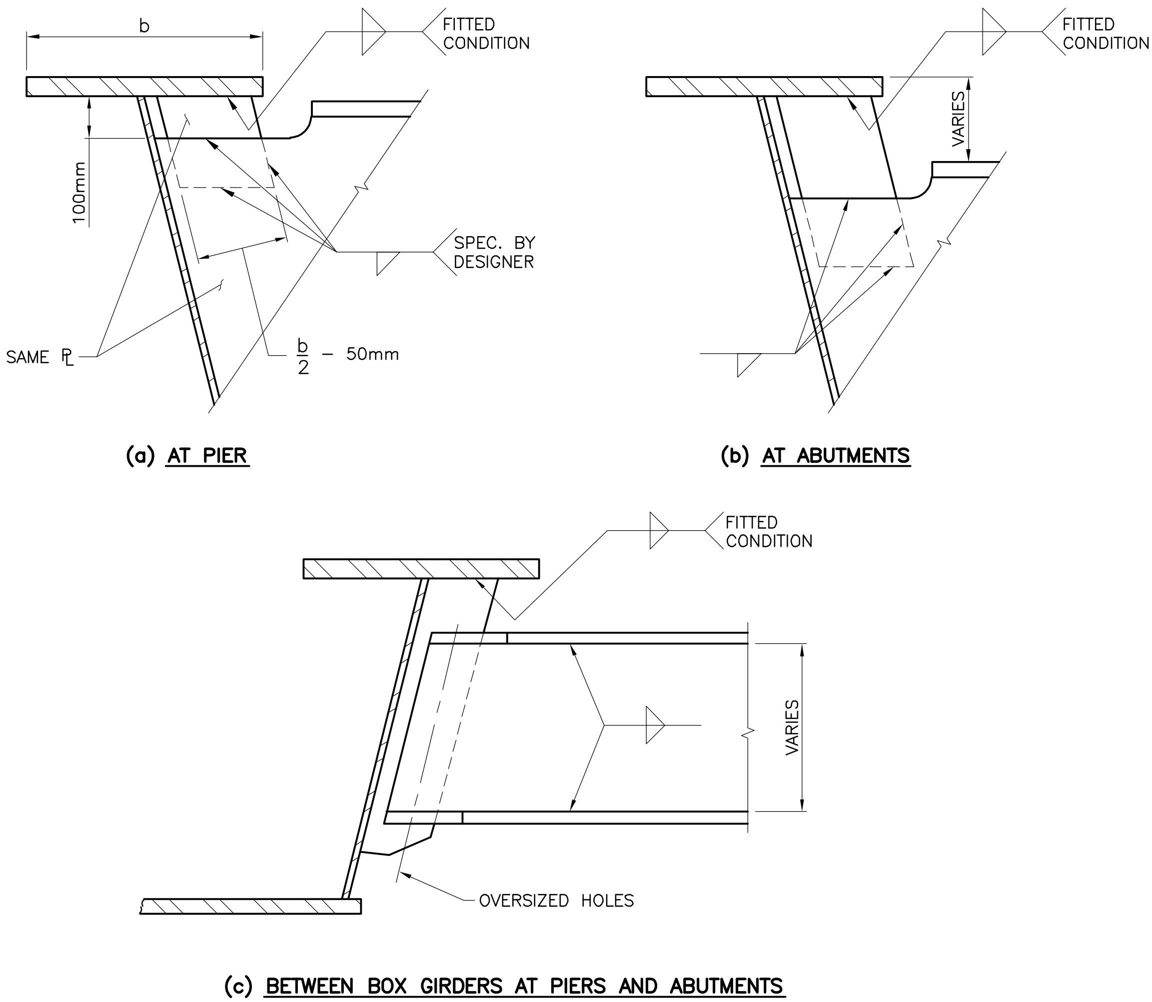



**Figure 8.5-2 – Box Girder Diaphragms**

## 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.

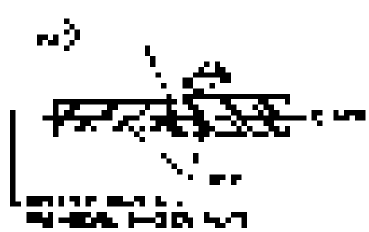


**Figure 8.6-1 – Web Splice Detail**

### 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 8.6-3.

**Figure 8.6-3 – Flange Width Transition Plan**

### 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).

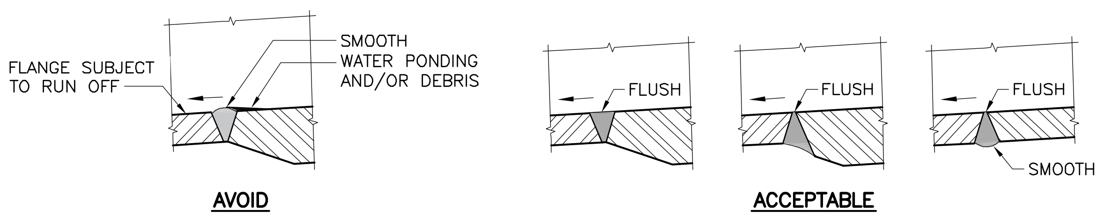



Figure 8.6-4 – Grinding Flush of Welds Which Otherwise Form Water Traps

### 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.

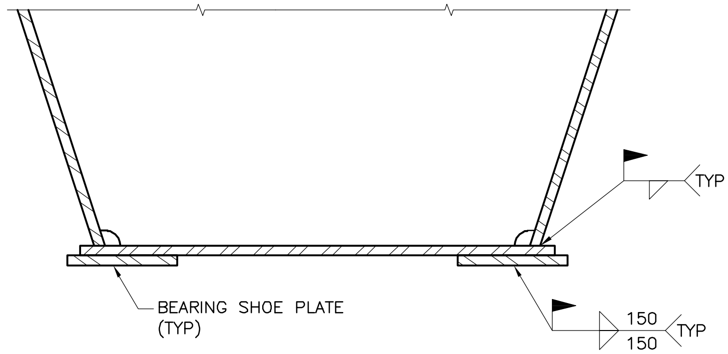



Figure 8.6-5 – Bearings at Piers and Abutments

## 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.

   (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.

   (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. PRIMER FOR FAYING SURFACES ARE ASSUMED TO BE CLASS…

    The class of the coating assumed in design of the connections shall be stated on the drawings as follows:

    **Class A/B/C/D** as defined in S6 Table 10.8

    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 Aerospace Material Specification Standard 595A Colours (for ACR steel girder bridges); or,

    - **16307 grey** According to Aerospace Material Specification Standard 595A 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 …

    (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 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.
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. ASSUMED DEAD LOAD FROM DECK FORM SYSTEM: 0.80 kPa (or that assumed by designer if different than 0.80 kPa)
   (Choose one of the following as applicable)
   ASSUMED DEAD LOAD FROM FALSE DECK SYSTEM: 0.35 kPa (or that assumed by designer if different than 0.35 kPa). NO LIVE LOADING OR STORAGE OF MATERIALS OR EQUIPMENT ARE PERMITTED ON THE FALSE DECK DURING THE DECK POUR.
   Or
   FALSE DECK SYSTEM IS NOT PERMITTED. (when approved)
   ASSUMED BRIDGE DECK FINISHING MACHINE BIASED (HEAVY SIDE) LOAD ON ONE RAIL: XX kN.
   ASSUMED WORK BRIDGE LOAD ON ONE RAIL: XX kN

## 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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