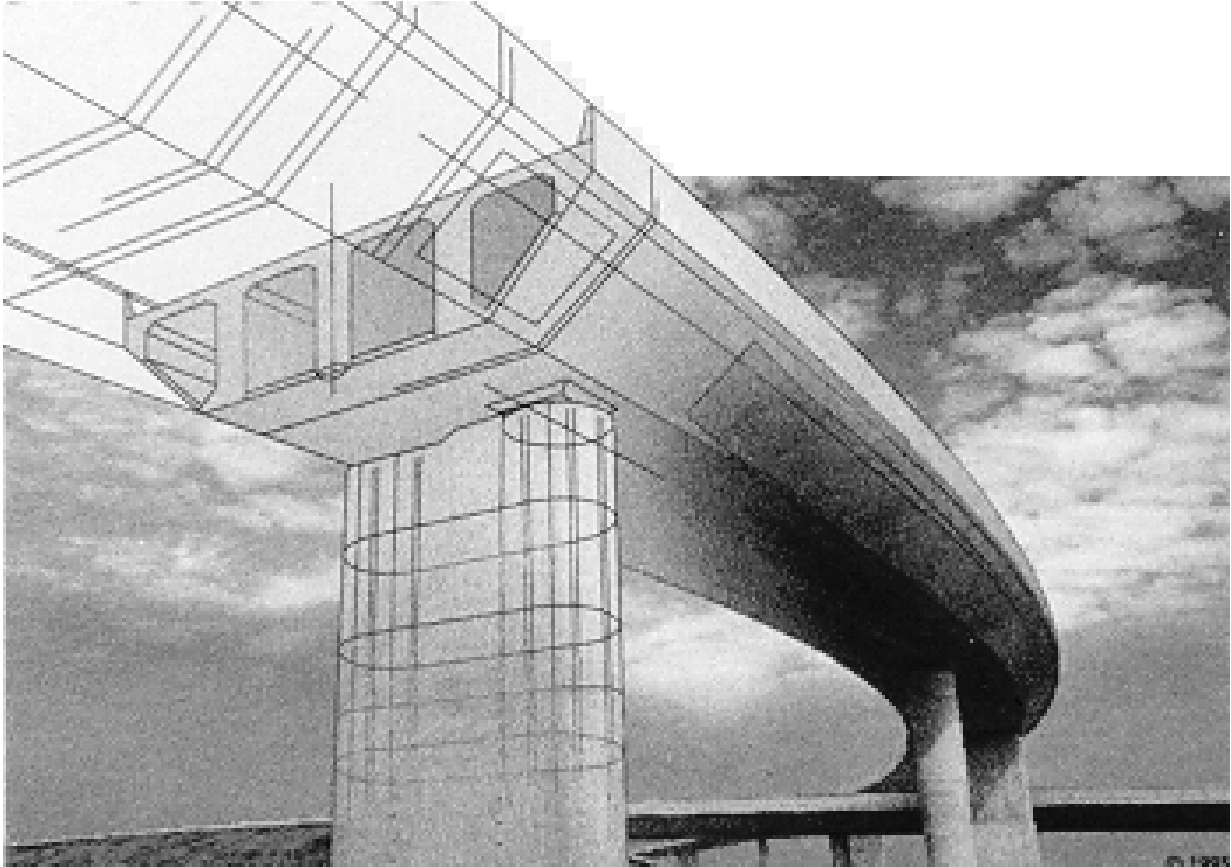


STRUCTURAL MANUAL



Ministry of Transportation
Policy, Planning & Standards Division
Engineering Standards Branch
Bridge Office
December, 2003

Acknowledgement

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The HWY 401/403/410 interchange bridge was designed in the Structural Office.

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STRUCTURAL MANUAL

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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 Bridge Office when appropriate.

The current manual is divided into the following four divisions:

- Division 1 - Exceptions to the Canadian Highway Bridge Design Code
 - Division 2 - Procedures
 - Division 3 - Design Aids
 - Division 4 - Structural Standard Drawings - Section 1
Structural Standard Drawings - Section 2

Previous manuals only had the divisions 2 to 4. A Table of Content or a List precedes each division as appropriate.

Although the Ministry of Transportation 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 affect on the safety and movement of people and goods. Any deviation from this manual should be based on operational experience and objective analysis. The Ministry of Transportation does not accept responsibility in any connection with the variations.

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

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Notation

AADT	Annual Average Daily Traffic
AASHTO	American Association of State Highways and Transportation Officials
ACR	Atmospheric Corrosion Resistant
AREA	American Railway Engineering Association
ASTM	American Society for Testing and Materials
AWPA	American Wood Preservers Association
BBR	Canadian BBR Inc.
BDIMS	Bridge Document Imaging Management System
CDED	Contract Design, Estimating and Documentation
CHBDC	Canadian Highway Bridge Design Code
CPCI	Canadian Prestressed Concrete Institute
CPS	Contract Preparation System
CSA	Canadian Standards Association International
DA	Design Aids
DD	Design Details
DSM	Designated Sources for Materials
DSM	Designated Sources Material
DYWIDAG	DYWIDAG Systems International Canada Ltd.
ERT	Effective Rubber Thickness
HPC	High Performance Concrete
HWL	High Water Level
kN	Kilonewton
m	metre
mm	millimetre
MPa	Megapascals
NTS	Not to Scale
NWPA	Navigable Waterways Protection Act

OHBD	Ontario Highway Bridge Design Code
OPSD	Ontario Provincial Standard Drawings
OPSS	Ontario Provincial Standard Specifications
PI	Point of Intersection
RSS	Retained Soil Systems
SC	Spiral to Curve
SLS	Serviceability Limit States
SS	Structural Standard
TC	Tangent to Curve
TFE	Polytetrafluoroethylene Polymer
TM	Temporary Modular
TMB	Temporary Modular Bridge
ULS	Ultimate Limit State
VSL	VSL Canada Ltd.
WP	Work Project/Working Point

STRUCTURAL MANUAL

**DIVISION 1 - EXCEPTIONS TO THE
CANADIAN HIGHWAY BRIDGE DESIGN CODE**

June, 2002

**EXCEPTIONS TO THE CANADIAN HIGHWAY BRIDGE DESIGN CODE
CAN/CSA-S6-00 FOR ONTARIO
JUNE 2002**

1.0 Scope

These exceptions implement provisions that the Canadian Highway Bridge Design Code CAN/CSA-S6-00 (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 in order 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 to be found in Appendix A. In the event of any inconsistency or conflict in its contents and the exceptions given in Section 4.0 below, the appendix will take precedence and govern.

These provisions are subject to change.

2.0 Authority

Amendment to Ontario Regulation 104/97 made under the "Public Transportation and Highway Improvement Act."

3.0 Implementation

Immediately by all owners upon the date the Amendments to the Public Transportation and Improvement Act (PTHIA) and Highway Traffic Act for the CHBDC implementation are filed with the Registrar of Regulations.

4.0 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.

Section 1 - General

1.2.1 General Administrative Definitions

The following definitions shall apply:

Engineering Association: means Professional Engineers Ontario

Regulatory Authority: means the Minister of Transportation of Ontario or his nominee

1.5.2.5 Single Load Path Structures

This clause is amended by the addition of the following:

For bridges, a single load path structure shall not be used unless Approved.

1.5.4.5

Plans

The third paragraph is deleted and replaced with the following:

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

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

Specifications for construction and rehabilitation shall be in accordance with the *Ontario Provincial Standards for Roads and Municipal Services* or other Approved standard. In the event of any inconsistency or conflict in the contents of these standards and the Code, the *Ontario Provincial Standards for Roads and Municipal Services* or other Approved standard will take precedence and govern.

For the Ministry of Transportation the following order of precedence will govern:

- a) Ministry approved standards
- b) Ontario Provincial Standards for Roads and Municipal Services
- c) Canadian Highway Bridge Design Code

1.6

Geometry

In this subsection all references to "...the Regulatory Authority, or in their absence, with the TAC *Geometric Design Guide for Canadian Roads*." are deleted and replaced by "...the *Geometric Design Standards For Ontario Highways* or an Approved standard."

1.7

Barriers

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

1.10

Hydraulic Design

In this subsection all references to "...the Regulatory Authority, or in their absence, with the TAC *Guide to Bridge Hydraulics*." are deleted and replaced by "...the *Drainage Management Manual* or an Approved standard." Similarly all other references to the TAC *Guide to Bridge Hydraulics* are deleted and replaced by "...the *Drainage Management Manual* or an Approved standard."

1.10.1.2 Normal Design Flood

The first paragraph is deleted and replaced with the following:

The normal design flood in Ontario shall have a return period in accordance with the Ministry of Transportation drainage directive *PHY Directive B100*.

1.10.1.3 Check Flood

The first paragraph is deleted and replaced with the following:

The check flood in Ontario shall have a return period of 100 years.

Section 3 - Loads

3.8.8.1 Traffic Barriers

This clause is amended by the addition of the following:

No DLA need be applied.

Section 4 - Seismic Design

4.4.2 Importance Categories

The Ministry of Transportation, Ontario, shall approve the classification of lifeline bridges.

Section 6 - Foundations

6.4.5 Consultation

The first sentence is deleted and replaced with the following:

Consultation between the Structural Engineer and the Geotechnical Engineer shall take place during planning, design, and during construction when unanticipated subsurface conditions are encountered or for any foundation-related construction problem requiring changes to the foundation design.

6.8.10.2 Splices

The last sentence of the second paragraph is deleted and replaced with the following:

Wood piles shall not be spliced.

Section 7 - Buried Structures

7.1 Scope

This subsection is amended by the addition of the following:

The provisions of this Section apply only to 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 Municipal Services*. In the event of any inconsistency or conflict in the contents of these standards and the Code, the *Ontario Provincial Standards for Roads and Municipal Services* will take precedence and govern.

7.5.5 Seismic Requirements

The provisions of these clauses are deleted and replaced by the requirements of Section 4, unless Approved.

7.8.8.2.2 Box Sections and Segmental Structures without Stirrups or Ties

The third paragraph and its contents (a) and (b) is deleted and replaced with the following:

Unless Approved, the shear strength shall be determined in accordance with Section 8.

Section 8 - Concrete Structures

8.8.4.5 Maximum Reinforcement

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

The amount of reinforcement provided shall be such that the factored flexural resistance, M_r , is developed with c/d not exceeding 0.5. This requirement may be waived provided it is established that the consequences of reinforcement not yielding are acceptable.

8.11.2.1 Concrete Quality

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

The minimum concrete strength, maximum water/cement ratio, and the minimum air content requirements for structural concrete shall be as specified in the *Structural Manual* and *Ontario Provincial Standards for Roads and Municipal Services* or other Approved standard for the appropriate combination of deterioration mechanisms and environmental exposures.

8.11.2.2 Concrete Cover and Tolerances

The minimum concrete cover and tolerance of 50 ± 10 for reinforcing steel in cast-in-place concrete shown in Table 8.11.2.2(a)(5) for "Soffits of Slabs < 300 mm thick" is deleted and replaced with 40 ± 10 .

Section 10 - Steel Structures

10.6.3 Superstructure Components

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

The minimum corrosion protection shall be as given by the Ministry of Transportation, Ontario, corrosion protection policy, or other Approved standard, for the appropriate environmental exposure conditions and superstructure components.

10.6.3.1 Structural Steel

The first paragraph is deleted and replaced with the following:

All structural steel, including contact surfaces of bolted joints, diaphragms and bracing but excluding surfaces in contact with concrete, shall be coated with an Approved coating system for a minimum distance of 3000 mm from expansion and fixed joints.

10.6.3.2 Cables, Ropes and Strands

The first paragraph is deleted and replaced with the following:

An Approved method of corrosion protection shall be used for all wires in the cables of suspension bridges, stay cables of cable-stayed bridges, suspension bridge and arch bridge hangers and other ropes or strands.

10.6.4 Other components

The second paragraph is deleted.

10.10.9.1 Intermediate Cross-Frames or Diaphragms

This clause is amended by the addition of the following:

Where girders support deck slabs proportioned in accordance with the empirical design method of Clause 8.18.4 the lateral spacing of intermediate cross-frames or diaphragms shall meet the requirements of Clause 8.18.5.

10.11.7.3.1 Composite Sections

The last sentence is deleted and replaced with the following:

The factored moment resistance shall be then reduced by the factor specified in Clause 10.10.4.3 if longitudinal stiffeners are not provided when $2d_c/w > 1900/\sqrt{F_y}$.

10.12.6.2 Diaphragms and Cross Frames between Girders

This clause is amended by the addition of the following:

Where girders support deck slabs proportioned in accordance with the empirical design method of Clause 8.18.4 the lateral spacing of intermediate cross-frames or diaphragms shall meet the requirements of Clause 8.18.5.

10.13.5 Diaphragms, Cross-Frames, and Lateral Bracing

This clause is amended by the addition of the following:

Where girders support deck slabs proportioned in accordance with the empirical design method of Clause 8.18.4 the lateral spacing of intermediate cross-frames or diaphragms shall meet the requirements of Clause 8.18.5.

Section 11 - Joints and Bearings

11.5.1.1 Functional Requirements

This clause is amended by the addition of the following:

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

11.5.1.2 Design Loads

The third paragraph is deleted and replaced with the following:

A horizontal load of 60 kN per metre length of the joint shall be applied at the roadway surface, in combination with forces that result from movement of the joint, to produce maximum force effects except for modular joint systems. For modular joint systems the horizontal load shall be as Approved.

11.5.3.2.4 Bolts

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

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

11.6.1 General

The sixth paragraph is deleted and replaced with the following:

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

11.6.6.4 Deformation and Rotation

The second sentence of the second paragraph is deleted and replaced with the following:

Where rotation occurs, the bearing shall be proportioned so that, at SLS, there is no uplift at the edge of the bearing and the edge compressive deformation does not exceed $0.14h_e$.

Section 12 - Barriers and Highway Accessory Supports

12.5.2.1.1 Alternative Performance Levels

The first sentence is deleted and replaced with the following:

Performance levels other than Performance Levels 1, 2, and 3 shall be Approved by the Regulatory Authority and shall be defined by specifying their crash test requirements.

Section 14 - Evaluation

14.11 Target Reliability Index

This subsection 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 14.11(a), shall be reduced by 0.25. This value shall not be less than 2.5.

14.13.2 Resistance Adjustment Factors

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 14.13.1, shall be multiplied by a resistance adjustment factor, U , where $U = 1.0$ for all cases. Table 14.13.2 shall not be used.

Section 16 - Fibre-Reinforced Structures

16.7 FRC Deck Slabs

This subsection is amended by the addition of the following:

- (o) The deck slab shall be provided with an orthogonal assembly of reinforcement near the bottom of the slab with a ρ in each direction in the assembly being at least 0.003 as detailed in 8.18.4.2 (a).

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APPENDIX A

Guidelines for the Design of Bridges on Low Volume Roads

A. Introduction

Present design codes, standards and policies have typically been developed for bridges with high traffic volumes. It has become apparent, however, that for bridges with low traffic volumes these requirements have become too stringent. In order to achieve economies by the relaxing of requirements, and without compromising safety, a task force with Regional and Head Office participation was set up to develop guidelines for this purpose. As a result, this section has been formulated.

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

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

These design guidelines apply for bridges on roads with an average annual daily traffic (AADT), in both directions, of 400 or less.

B. Definitions

Recreational Road	A road used for the access of parks, scenic and historic sites, or seasonal cottages.
Resource Access Road	A road used for mining, forestry and energy development.
Local Road	A road primarily used for land access.
Collector Road	A road providing for land access and traffic circulation.
Arterial Road	A road primarily used for high volumes of through traffic.
AADT	Average annual daily traffic
LVPL1	A performance level at which a traffic barrier is to perform to reduce the consequences of a vehicle leaving the roadway. A LVPL1 railing must be able to successfully redirect a pick-up truck with a speed of 25 km/hr and an impact angle of 15°. The level of performance is less than that for LVPL2 and is intended for very low volume and low speed traffic.
LVPL2	A performance level at which a traffic barrier is to perform to reduce the consequences of a vehicle leaving the roadway. A LVPL2 railing must satisfy Test Level 1 of NCHRP Report 350 (successfully redirect a pick-up truck with a speed of 50 km/hr and an impact angle of 25°). The level of performance is better than that for LVPL1 but less than that for PL1 from CHBDC and is intended for low volume traffic at moderate speeds.
CHBDC	Canadian Highway Bridge Design Code, CAN/CSA-S6-00.

C. CHBDC

The 2000 CHBDC should be used for all bridges with the following exemptions allowed for bridges on low volume roads. The relevant CHBDC clause numbers are given next to each provision.

1. GENERAL PROVISIONS

a) General

- i) The 10 year growth need not be considered for AADT unless a significant change in the road use is foreseen (Cl. 1.6.1).
- ii) 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. 1.5.2.5).
- iii) The approach slabs may be omitted. Approach slabs may be beneficial for high abutments to reduce the effects of live load surcharge (Cl. 1.8.2).
- iv) The design life of the bridge shall be 75 years unless reduced at the request of the owner (Cl. 1.5.2.3).
- v) Reuse of existing bridges may be considered at the owner's discretion. Material condition and physical properties should be determined prior to their use.
- vi) Deck drains are only required as given by Clause 1.9.2.3.1. Shorter bridges and bridges with catch basins at the ends of the structure often do not require deck drains.

b) Geometry

- i) 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. 1.6.1).
- ii) The use of single lane bridges is acceptable for bridges on low volume roads. (Cl. 1.6.2) The minimum bridge, lane, and shoulder widths shall be as given in Table 1 (Ref. 2,3). Adequate warning should be given to traffic in the case of narrow or one lane bridges.

Table 1: Minimum Lane, Shoulder and Bridge Widths

AADT	OPERATING SPEED	MIN. LANE WIDTH (m)	MIN. SHOULDER WIDTH (m)	MIN. BRIDGE WIDTH (m) ^{1, 2, 6}
0 - 50	< 50 km/hr	3.0	0.5	4.0
	50 - 70	3.0	0.5	4.0
	> 70	3.0	1.0	5.0
50 - 200	< 50	3.0	0.5	4.0
	50 - 70	3.0	1.0	5.0
	> 70	3.0	1.0	5.0
200 - 400	< 50	3.0	0.5	4.0
	50 - 70	3.5	1.0	5.5 ³
	> 70 ⁴	2 @ 3.0	0.5	7.0
	> 70 ⁵	2 @ 3.25	1.0	8.5
1 - Width measured between the inside face of the barriers, guiderails, or curbs. 2 - Farm or other special vehicles may require larger widths. 3 - Still a single lane bridge, but adequate for two vehicles to pass at reduced speed. 4 - For bridges >50 m in length and with good roadway visibility. 5 - For bridges < 50 m in length or bridges with poor roadway visibility. 6 - A bridge width that matches the existing approach roadway may be used.				

c) Hydrology

- i) The return period for the design flood shall be taken from Table 2 (Cl. 1.10.1.2, Directive B100) (Ref. 4).
- ii) Minimum vertical soffit clearances shall be as given in Table 2 (Cl. 1.10.7.1) (Ref. 1).
- iii) The check flood need not be considered (Cl. 1.10.1.3).
- iv) Scour and erosion protection is required only for susceptible structures. The stone sizes for scour and erosion protection may be taken from Table 3 instead of RTAC recommendations (Cl. 1.10.6) (Ref. 5).
- v) The freeboard requirements may be waived (Cl. 1.10.8.2).
- vi) Slope protection, where required, needs only to be extended 150mm above HWL (Cl. 1.10.9.1).
- vii) Low water crossings, which accommodate the normal flow but overtop during flooding, should be considered as an alternative, but not for collector or arterial roads. The return period should be determined by the Owner to determine an acceptable length of time the structure is impassable. Approval is required from other authorities (MNR, MOE, etc.) (Ref. 6).
- viii) For closed culverts in scour resistant soils, a concrete cut-off wall is not required (Cl. 1.10.5.6).

Table 2: Return Period and Soffit Clearance for Bridges

ROAD FUNCTION	VULNERABILITY ¹	RETURN PERIOD (yrs.) ³ FOR DESIGN FLOOD		SOFFIT CLEARANCE ^{2,3} (m)
		Bridges <6.0m Long	Bridges >6.0m Long	
RECREATION	HIGH	5	10	0.3
	LOW	2	2	0
RESOURCE ACCESS	HIGH	5	10	0.3
	LOW	5	10	0
LOCAL	HIGH	10	25	0.3
	LOW	10	25	0
COLLECTOR AND ARTERIAL	HIGH	25	50	1.0
	LOW	25	50	0.3
1- High Vulnerability is defined as a bridge that can be washed away or significantly damaged during flooding. High vulnerability bridges include log bridges, bridges with timber beams and deck, and light trusses. Low Vulnerability is defined as a bridge that will not experience significant damage during flooding. Low vulnerability bridges include rigid frames, integral abutment bridges, and slab or voided slab bridges. 2- Navigation clearances and hydraulic effects must also be considered. 3- The soffit elevation should be based on the above table or from an existing opening that has proven to perform satisfactorily in the past.				

Table 3: 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 ¹ (mm)	100	200	300	400	500	800	1000
1- 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.							

2. LOADS

a) Loads

- i) Lane load need not be considered (Cl. 3.8.3.2).
- ii) Modification factors for multi-lane loading need not be considered. The bridge can be designed using one truck positioned in the location to cause the maximum effect (Cl. 3.8.4.2).
- iii) The bridge can be designed to a load lower than the CHBDC truck provided that the bridge will be properly posted. The live load surcharge can be proportionately decreased with the decrease in truck load (Cl. 6.9.5). Emergency and maintenance vehicle weights should be considered when determining the appropriate design loading (Cl. 3.8.4).
- iv) The full DLA and load factors should be used (Cl. 3.8.4.5) (Cl. 3.5.1).

b) Limit States

- i) The deflection limit may be reduced to L/360 and need not be checked for temporary modular bridges (Cl. 3.4.4).
- ii) Unless the bridge is subjected to significant truck traffic, the Fatigue Limit State shall consider 500,000 cycles of loading. (Cl. 3.4.3) (Cl. 10.17.2.3) (Ref. 7).

3. ANALYSIS

- a) Section 5 of CHBDC may be used for analysis.

4. JOINTS AND BEARINGS

- a) Joints should be eliminated where possible by using integral or semi-integral abutment designs.
- b) Ministry approved joints are only required on paved bridges over 20m span.
- c) Lower cost sealing systems can be used for small movements.

5. BARRIERS

- a) A lower performance level is acceptable for some low volume roads (Cl. 12.5.2.1.3)(Ref. 8).
- b) A LVPL1 and LVPL2 railing can be used for bridges meeting the criteria of Table 4. If the criteria of Table 4 are not met then CHBDC must be followed.
- c) The height of barriers shall be as crash tested. Where significant pedestrian or bicycle traffic is expected, a barrier height consistent with the requirements for PL1 of the CHBDC should be used (Cl. 12.5.2.2).
- d) Railings that have been successfully crash tested to the above performance limits are given in Figures 1 and 2.

Table 4: Barrier Selection criteria

	AADT	Height above water	Operating Speed	Bridge Width
LVPL2	≤ 400	≤ 5.0 m	≤ 50 km/hr	no limit
			≤ 80 km/hr	≤ 5.0 m
LVPL1	≤ 100	≤ 2.5 m	≤ 25 km/hr	no limit
			≤ 40 km/hr	≤ 5.0 m

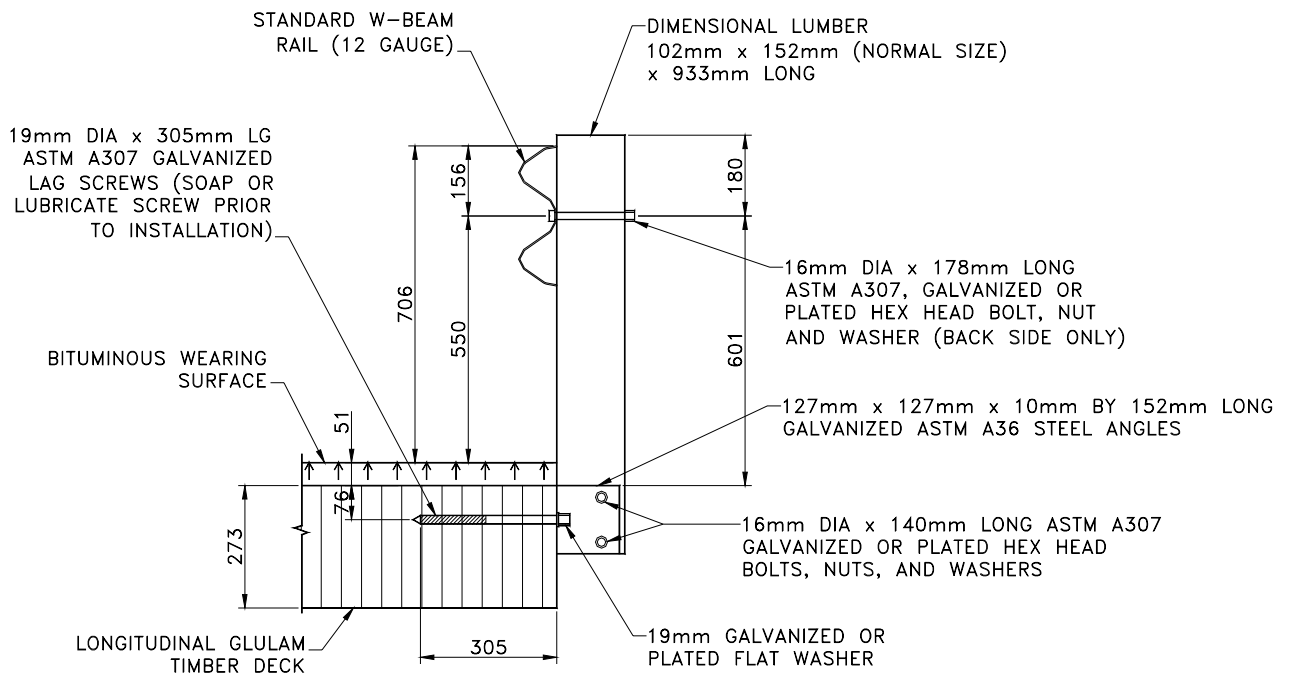
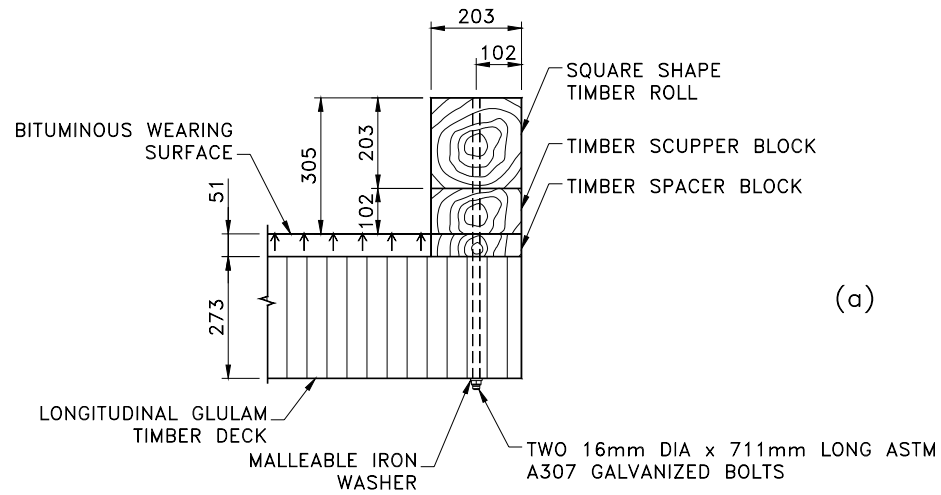
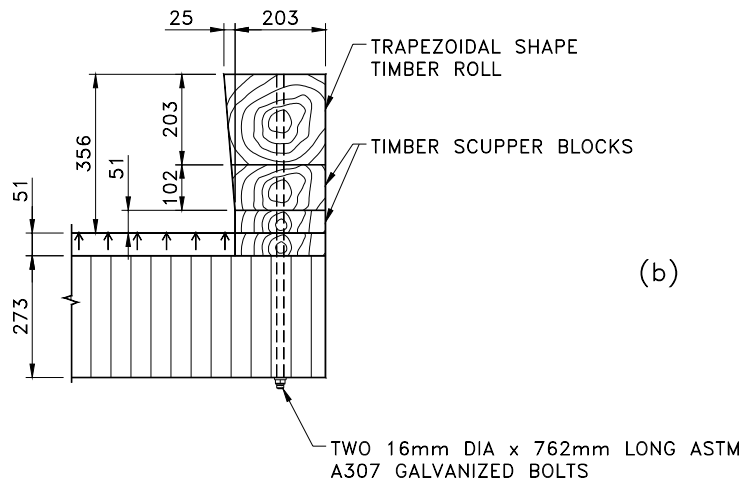


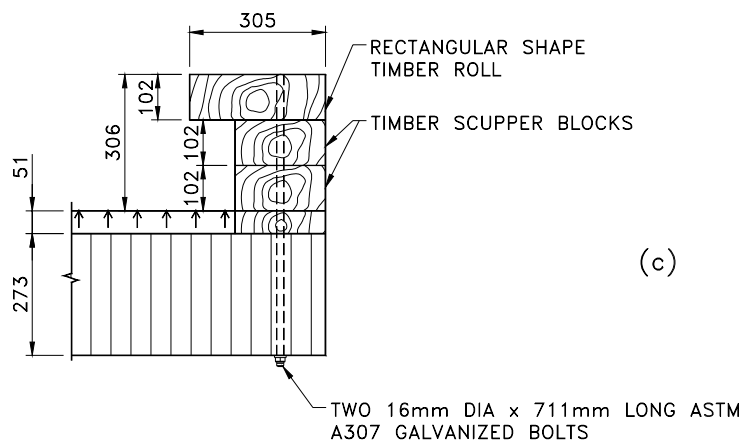
Figure 1: Approved LVPL2 barrier



(a)



(b)



(c)

Figure 2: Approved LVPL1 barrier systems

6. FOUNDATIONS

- a) Bridges designed to a lower live load than the CHBDC truck may have the 600mm live load surcharge reduced proportionately (Cl. 6.9.5).
- b) A limited foundation investigation or use of soils information from a nearby site may be adequate for temporary structures and structures with spans less than 10m (Cl. 6-5.1).
- c) Gabion baskets may be used for retaining the approach road instead of wingwalls.

7. SOIL-STEEL STRUCTURES

Section 7 of the CHBDC should be used for soil-steel structures.

8. CONCRETE STRUCTURES

a) Material

- i) The minimum strength of concrete in decks shall be 30 MPa as specified in the CHBDC.
- ii) Slabs thinner than 225 mm are allowed but a minimum of 175 mm should be met (Cl. 8.18.2). The reduced slab thickness can be used provided that the constructability can be ensured (Ref. 9).

b) Durability

- i) The 10mm allowance for wear on exposed concrete decks may be omitted (Cl. 8.18.3).
- ii) For bridges that will not receive winter salting:
 - Uncoated reinforcement may be used (Cl. 8.11.2.3).
 - Cracking at the Serviceability Limit State (SLS) need not be considered (Cl. 8-12).
 - Waterproofing is not required (Cl. 8.11.2.7) (Cl. 2.4.3).

9. WOOD STRUCTURES

- a) Hardwoods may be used and strengths and other properties shall be taken from CSA O86-94 (Ref. 10).
- b) Transverse nail laminated decks shall have a nail length that pass through two laminates and 10mm into the third laminate (Cl. 9.21.2.1.1).
- c) Longitudinally nail laminated decks can be used and the wheel load shall be assumed to be distributed over a width of 1.7m. (Cl. 9.21.3).
- d) Water borne preservatives may be used for stress laminated timber decks with a minimum dimension of less than 50mm (Cl. 9.17.1).
- e) Allowable stresses can be increased by 25% for gluelam beams (Ref. 10).

10. STEEL STRUCTURES

- a) Section 10 of the CHBDC should be used for steel structures.
- b) For the various stress categories, the allowable stress range for steel members at 500,000 cycles shall be taken from Table 5 (Cl. 10.17.2.3) (Ref. 11).

Table 5: Allowable stress range for 500,000 cycles

Category	a	b	b-1	c	d	e	f	w	y
Allowable Stress Range (MPa)	255	200	159	145	110	90	63	83	105

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STRUCTURAL MANUAL

DIVISION 2 - PROCEDURES

December 2003

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1 INTRODUCTION

1.1.1 General

The procedures in this division are to be followed where practical, when preparing structural contract documents for the Ministry. The preparation of contract documents and 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 marked by the date and revision number at the bottom right corner of the page. Any non-technical changes in page appearance will be indicated by the date only.

When additions or revisions are necessary, they will be made available through Ronen Publishing House Inc., as detailed in Section 1.1.5.

1.1.2 Definitions

For the purpose of this manual the following definitions apply:

Engineer: means a member or licensee of the Professional Engineers Ontario, who carries out the design or checking of a rehabilitation, design or evaluation of a bridge or structure.

Ministry: means Ministry of Transportation, Ontario.

MTO: means Ministry of Transportation, Ontario.

1.1.3 Standard Drawing Format

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

- a. Standard drawings for attachment on or insertion into contract drawings in the manner specified in 2.5.9.
- b. Standard drawings for use as contract drawings.

New standard drawings being issued in category (a) are generally in 216 x 279 mm format. For the purpose of this Introduction, standards in this category are referred to as "small size" standards.

Standard drawings in category (b) are issued in the structural contract drawing format. They contain two title blocks that must be completed by the user. For the purpose of this Introduction, standards in this category are referred to as "full size" standards.

In this Manual, reduced scale copies of the full size standard drawings have been assembled in a separate category following the small size standards.

1.1.4 Numbering System

Design Aids bear the prefix DA and Standard Drawings the prefix SS. Procedures have no prefix. Except for full-size standard drawings (see later), the one or two digit number after the prefix, if there is a prefix, is the Section number. All Divisions use the same subject-oriented Section numbering system. E.g., SS 13-3 would represent the Section

13 of the Procedures Division, 'Bearing Assemblies and Expansion Joints' - Sheet 3. The same principle applies to Design Aids.

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

In order to distinguish between the full size standard drawings and the small size drawings, the full size standard section numbers have been increased by 100.

E.g., SS 116-10

The first digit (1) of the first number group indicates that it is a full size drawing. The next two digits (16) show that the drawing belongs to Section 16 (Miscellaneous).

1.1.5 Distribution

Copies of the Manual and revisions may be obtained from:

Ronen Publishing House Inc.
505 Consumers Road, Suite 910
Toronto, Ontario
M2J 4V8
Tel.: 1-800-856-2196
Fax: 1-800-870-7239
Web site: www.ronenhouse.com

Holders of copies thus obtained will be registered for notifications of future revisions.

1.1.6 Availability of Standard Drawings

Transparent copies of standard drawings for use in the production of contract documents are not available. Electronic CAD files containing standard drawings in AutoCAD may, however, be obtained from Ronen Publishing House Inc. at the address given in Section 1.1.5. The latest revision date will be that available through Ronen Publishing House Inc.

1.1.7 Use of Standard Drawings

Most standard drawings bearing an "SS" prefix require job-specific information to be added by the user. The job-specific information should be added at the time the standards are included in or attached to the drawings.

If details and notes as shown on SS standards are not applicable, they must be deleted, crossed through, or labelled "Not Applicable".

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.

1.1.8 Modifications to Structural Standard Drawings

Standard drawings shall be reviewed to determine what information, if any, needs to be added to them.

Where information in tables and dimensions are added to a standard drawing 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 standard drawing are not applicable and are deleted, crossed through, or labelled "Not Applicable", the drawing shall be identified as "Modified" and bear the seal, date and signature of an Engineer. This Engineer accepts full responsibility for the modifications to the drawing only.

Where changes are made on a standard drawing that affect the original structural design, the drawing shall be identified as "Modified" and bear the seals, dates and signatures of two Engineers. These Engineers accept full responsibility for the design that results from the changes.

1.1.9 DD Standards

In the past, a number of small size structural standards (SS) drawings which did not require the addition of job specific data, were converted into DD standards by the Highway Design Office. These were contained in Book 3 - Structural, of the Highway Engineering Standard Drawings Series and had numbers from DD 3000. Support for this manual has been discontinued by the Highway Design and Bridge Offices as all standard drawings in this manual have been replaced by either Structural Standard Drawings (SS), Ontario Provincial Standard Drawings (OPSD) or deleted.

1.1.10 Ontario Provincial Standard Drawings (OPSD's)

Ontario Provincial Standard Drawings (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 the owner's requirements for quality and the contractor's need for efficiency are satisfied.

The Ontario Provincial Standard Drawings are provincial publications and are not automatically used by the Ministry. Those that the Ministry has implemented may be found in Chapter H of the Ministry's Contract Design, Estimating and Documentation (CDED) Manual, and the Contract Preparation System (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 Contract Preparation System (CPS) News File of the CPS Main Menu.

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

Modifications to OPSD's shall follow the policy and procedures of the Ontario Provincial Standards Administration.

1.1.11 Bridge Office Policy Memos

Whenever an immediate change to manuals, codes, procedures, policies, or an innovation is required, the Bridge Office (formerly Structural Office) periodically 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, the publication/memo with the latest date takes precedence.

Current policy memos may be obtained from the Manager, Bridge Office.

2 GENERAL DESIGN AND DRAUGHTING

2.1.1 Design Specifications

All structures with spans greater than 3 m shall be designed in accordance with the Canadian Highway Bridge Design Code (CHBDC). CHBDC clauses identified in this revision refer to the edition of the Code published in December 2000. Structures supporting railway loading must be designed to criteria provided by the railway authority.

2.1.2 Design and Check Calculations

For all Ministry bridges designed in house, the design and check calculations, are to be filed with the Design Section, Bridge Office or Regional Structural Section as appropriate. This shall be done by the Design Engineer immediately after the Checker's P.Eng. stamp has been placed on the drawings.

For Consultant designs, a copy of calculations of the final new and rehabilitation bridge designs including any changes as a result of design check should be submitted to the Ministry project manager, within six (6) weeks of the executive review. The design calculations should be submitted on letter or legal size paper. The use of any commercial computer software, its name and version should be clearly identified and a hard copy of the input data and a sketch of computer model should also be included with the design notes. If an in-house spread sheet type program is used then the hard copies of both the input and output files should be submitted. The submission should clearly identify the designer and checker and should be dated. The project manager would be responsible for adding the submission to the structure file copy for any future rehabilitation needs.

2.1.3 Design for Earthquake Effects

When analysing bridges for earthquake effects, the Importance factor should be taken as 1.0, unless otherwise indicated in the Structural Planning Report.

2.1.4 Critical Design Details

During the design process for new and rehabilitated structures, the designer should identify all critical details and ensure that they can be easily inspected after construction.

2.1.5 Steel Truss and Arch Design

Normally in the design and evaluation of steel trusses and arch structures connections are assumed pinned. However the effects of connection rigidity or seized pins should also be considered during analysis to determine whether resulting bending stresses due to fatigue are acceptable.

2.2.1 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 must 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 a supplementary measure is required 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 and water out of excavations.

The type of retaining structure or protection system is selected by the Contractor except as modified below. Contractors will design their schemes according to their proposed staging, available materials, expertise, and enterprise. The result should be a scheme that is structurally adequate and economic to both the Owner and Contractor. The design should 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 is not to 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 design should remain with the Ministry. For these situations the Structural Design Report must justify the need for the detailed design.

In order for the Ministry to still 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 is shared with the contractor
- The change proposal should not compromise any essential design criteria, safety or preliminary engineering commitments
- The change proposal cannot be the basis for a contract claim

The above criteria will be included in the Ministry's documents either as a special provision or part of the General Conditions of Contract.

2. When an anticipated protection system is not detailed, adequate information must be given on the contract drawings to enable the contractor to design and construct a safe and economic 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 should meet the requirements of the Ontario Provincial Standard Specification OPSS 539 "Construction Specification for 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 proposal will be submitted to the Contract Administrator as required by OPSS 539. Review of these cases will be at the discretion of the Regions.
5. The Ministry is responsible for the safety of the travelling public. All schemes proposed by the Contractor should be reviewed where the safety of the public is involved.
6. The review of protection systems shall be the responsibility of the Regions. Where complexity or work load prevent a review within the four-week period before construction, the Bridge Office may be consulted at the discretion of the Regions.

For assistance in the design or review of protection systems the designer may consult the "Guide Design Specifications for Bridge Temporary Works" and the "Construction Handbook for Bridge Temporary Works," both published by AASHTO in 1995. Where conflict occurs in these publications with OPSS 539 or the recommendations of the Pavement and Foundation Section, the requirements of the latter should take precedence.

2.2.2 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 shown on the contract drawings. The cut-off and tip elevations shall be shown, 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, but nevertheless remain the responsibility of the contractor.

2.2.3 Fill Stages at Structures

Where fill is to be brought up to footing level, piles (if any) driven, footings cast, and fill completed, 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 pile may be used, the maximum particle size for tube piles should be specified.

2.2.4 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 Design Section and the Regional Structural Section and shall be specified on the General Arrangement Drawing. The extent of sub-excavation is not shown on structural drawings. A note saying "For limits of sub-excavation see grading drawings" is required, close to where the sub-excavation is indicated.

2.3.1 Concrete Strengths

Concrete must be specified by metric class. Classes listed below are for normal concrete, and not high performance concrete. For high performance concrete class requirements see the Corrosion Protection Guidelines for Concrete Bridge Components in Section 2.8.2. Accepted minimum normal concrete classes with their normal applications are as follows:

Class	Applications
15 MPa 20 MPa 25 MPa	)) Not used for structural purposes)
30 MPa	Used in decks, curbs, sidewalks, approach slabs, barrier walls, cast-in-place concrete culverts & for sub-structures.
35 MPa	For prestressed post-tensioned concrete, columns where strength requirements dictate, and cast in place segmental structures.
40 MPa	Precast prestressed beams and precast segmental units. (See also 7.2.1)

For special applications, where size or weight must be minimised, higher strengths may be authorised.

Concrete strengths required at transfer of prestress are not classes. They may be specified between 20 MPa and the class strength for post-tensioned concrete, and between 25 MPa and the class strength for pre-tensioned concrete. The normal minimum however is 30 MPa and above this, the value should be as low as strength requirements permit, rounded to the nearest MPa.

Tremie concrete is produced in accordance with the requirements of OPSS 1350 and no strength class should be shown on the drawings.

2.3.2 Availability of Concrete with Strength in Excess of 30 MPa

There are areas in the Province where, for cast-in-place concrete, strengths in excess of 30 MPa are difficult to obtain.

When high strength cast-in-place concrete is proposed for a structure and information as to its availability is not given in the Structural Planning Report or in related correspondence, the Head, Regional Structural Section, should be asked to confirm, in writing, the availability of concrete meeting the proposed requirements before the final design is begun. The required concrete strength must be shown on the preliminary version of the General Arrangement Drawing.

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

The nominal cover and tolerance specified on drawings shall be as per Table 2.4.1. The information in this Table is derived from CHBDC Table 8.11.2.2, but is not complete in that only the most common member types and exposure conditions are listed. For member types, surfaces and exposure conditions not listed in Table 2.4.1, the designer shall refer to Chapter 8 of the CHBDC.

Table 2.4.1 – Concrete Cover Requirements

Type of Member or Exposure Condition	Reinforcing Bars (principal, ties, stirrups)	Pre- Tensioning Strands	Post- Tensioning Ducts
(A) Concrete <u>not</u> submerged or exposed to earth or de-icing chemicals			
a) Slabs ⁽²⁾			
(i) Cast-in-place, Top	60 ± 20	---	80 ⁽¹⁾ ± 15
(ii) Top of bott. slab for rectangular voided deck	40 ± 10	---	60 ⁽¹⁾ ± 10
(iii) Cast-in-place, Soffit			
- slab < 300 mm thick	40 ± 10	---	60 ⁽¹⁾ ± 10
- slab ≥ 300 mm thick	50 ± 10	---	70 ⁽¹⁾ ± 10
(iv) Precast, Top	50 ± 10	70 ± 5	70 ± 10
(v) Top of bott. slab for rectangular voided deck	40 ± 10	55 ± 5	60 ⁽¹⁾ ± 10
(vi) Precast, Soffit, all slab thicknesses	40 ± 10	55 ± 5	60 ⁽¹⁾ ± 10
b) Buried Structures			
(i) Cast-in-place concrete	60 ± 20	---	80 ⁽¹⁾ ± 15
(ii) Precast concrete	40 ± 10	55 ± 5	60 ⁽¹⁾ ± 10
c) Precast Girders	30 + 10 or - 5	45 ± 5	50 ⁽¹⁾ ± 10
(B) Concrete cast-in-place against and permanently exposed to earth			
(i) Footings	100 ± 25	---	---
(ii) Caissons	100 ± 25	---	120 ± 15
(C) Concrete submerged or exposed to earth			
a) Footings, Piers, Abutments, Retaining Walls			
(i) Cast-in-place concrete	70 ± 20	---	90 ⁽¹⁾ ± 15
(ii) Precast concrete	55 ± 10	75 ± 5	80 ⁽¹⁾ ± 10
b) Buried Structures with greater than 600 mm of fill*			
(i) Cast-in-place concrete	60 ± 20	---	80 ⁽¹⁾ ± 15
(ii) Precast concrete	40 ± 10	55 ± 5	60 ⁽¹⁾ ± 10
(D) Concrete exposed to de-icing chemicals due to spray or run-off			
a) Slabs ⁽²⁾ , Medians, Curbs and Sidewalks			
(i) Cast-in-place, Top	70 ± 20	---	---
- longitudinal ducts	---	---	130 ⁽¹⁾ ± 15
- transverse ducts (d _d ≤ 60 mm)	---	---	90 ⁽¹⁾ ± 15
- transverse ducts (d _d > 60 mm)	---	---	130 ⁽¹⁾ ± 15
- top of bott. slab for rect. voided deck	40 ± 10	---	60 ⁽¹⁾ ± 10
(ii) Cast-in-place, Soffit			
- slab < 300 mm thick	40 ⁽³⁾ ± 10	---	70 ⁽¹⁾ ± 10
- slab ≥ 300 mm thick	60 ± 10	---	80 ⁽¹⁾ ± 10
(iii) Precast, Top	55 ± 10	70 ± 5	---
- longitudinal ducts	---	---	120 ⁽¹⁾ ± 10
- transverse ducts (d _d ≤ 60 mm)	---	---	80 ⁽¹⁾ ± 10
- transverse ducts (d _d > 60 mm)	---	---	120 ⁽¹⁾ ± 10
- top of bott. slab for rect. voided deck	40 ± 10	55 ± 5	60 ⁽¹⁾ ± 10
(iv) Precast, Soffit			
- slab < 300 mm thick	45 ± 10	60 ± 5	65 ⁽¹⁾ ± 10
- slab ≥ 300 mm thick	50 ± 10	65 ± 5	70 ⁽¹⁾ ± 10
b) Buried Structures			
(i) Cast-in-place concrete	70 ± 20	---	90 ⁽¹⁾ ± 15
(ii) Precast concrete	50 ± 10	65 ± 5	70 ⁽¹⁾ ± 10
c) Precast Girders	35 + 10 or - 5	50 ± 5	55 ⁽¹⁾ ± 10
(E) Components, surfaces and environmental exposure other than those specifically mentioned in (A) to (D)	See CHBDC Table 8.11.2.2		

Notes: dimensions are in mm

d_d = outside diameter of ducts

* Buried structures with < 600 mm of fill shall have a distribution slab

(1) Greater of 0.5 d_d or indicated cover.

(2) For concrete decks without waterproofing and paving, increase the concrete covers by 10mm to allow for wear of the surface concrete.

(3) MTO policy

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 must 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. Stirrups must not be hooked over the top-most reinforcing bars. They should be hooked over bars in the lower layer of the top mat.
3. The appropriate standard drawing showing the method of supporting the reinforcing steel must be referenced as follows:

OPSD-3922.000 for thick slabs, OPSD-3923.000 for thin slabs (i.e., less than 300 mm thick).
4. If fanned bars are used, a location must be found for them such that there is no encroachment on cover and that location must be clearly shown in plan and section.

2.5.1 Site and Drawing Numbers

Site Numbers

Every structure designed for the Ministry and municipalities in Ontario is identified by an alphanumeric designation. This identifier, also called a "site number," indicates the structure's location and type. To obtain a structure identifier or "site number" a written request along with a map showing the location of the structure must be submitted to the Administrative & Records Clerk, Bridge Office, St. Catharines. This office will then issue a site number by incrementing the last site number issued to a structure in the county or municipal district where the structure is located.

For structures designed for and by municipalities the Bridge Office assigns numbers as detailed above.

For structures designed by and for the Ministry, the Bridge Office allocates blocks of numbers to the Regional Structural Sections, to be assigned by them. In this case, the Regions keep maps showing the location of the sites to which they have assigned numbers.

This identifier comprises three elements XX(L), XXXX and (/MX) which together are used in the following format:

XX(L)-XXXX(/MX)

where "X" represents a number, "L" or "M" a letter and () is optional depending on conditions as detailed below.

XX(L) is the county number except in some northern areas of the province where municipal district numbers are used. In the latter case municipal districts 38, 39, 41, and 48 have alphabetic characters (L), N (North), S (South), E (East), W (West), and C (Central) added after their number because of the large area they cover.

Examples:	48C Thunder Bay 39E Cochrane	37 York 3 Carleton

XXXX is the actual site number used to identify the location of a structure within a given county or municipal district. It is unique within these boundaries. Where two or more structures occur parallel to each other, they may have the same site number but distinguished as shown below. For structures in series, i.e. along the highway they should be given separate numbers. Leading zeroes are not required in the site number.

Examples: 48C-87
3-220
37-1175

(/MX) where (M) represents the type of structure, and (X) whether there is more than one structure having the same site number. This is illustrated below for various structure types.

Bridges

M is left blank for all bridges.

The single digit number "X" after the type of structure is used when there is more than one structure having the same site number. This may be used to identify twin structures or bridges at the same location such as a core/collector system. For this situation the following values for "X" shall apply:

X = 1	(a) Structures carrying Northbound, or the Eastbound lanes on twin structures.
	(b) Structures carrying Northbound or Eastbound <u>collector</u> lanes of a core/collector system.
X = 2	(a) Structures carrying Southbound, or the Westbound lanes on twin structures.
	(b) Structures carrying Southbound or Westbound <u>collector</u> lanes of a core/collector system.
X = 3	Structures carrying Northbound or Eastbound <u>core</u> lanes of a core/collector system
X = 4	Structures carrying Southbound or Westbound <u>core</u> lanes of a core/collector system

Ramps should be assigned unique site numbers

Example: 37-1234/2 Twin bridge carrying Southbound lanes

Culverts

Site numbers are only assigned to culverts with spans greater than 3.0m. For these cases M=C. Where there is more than one culvert at a site they should be assigned unique site numbers.

Example 48C-1234/C

Temporary Modular Bridges

For this case M=B. Previously Temporary Modular Bridges (TMB) were assigned the letter "B" as the M designation. Although B represented Bailey structures, it should represent all TMB's, even permanent ones.

Example 34-222/B

Overhead Sign Supports

These are not assigned site numbers. They may be identified by a number allocated by the district Bridge & Sign Installation Crew.

Retaining Walls

These are not assigned site numbers

Rehabilitations

See "Drawing Numbers " below.

Once a number is assigned to a structure, it will not be changed even after the structure has been modified, replaced by another or when the direction of travel has changed.

Drawing Numbers

Final drawings for all new structures are numbered consecutively with numbers only.

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 should be numbered PA1, PA2, etc.

Rehabilitations or modifications should 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.

Examples:	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 above rehabilitation, final drawing, twelfth sheet

2.5.2 Sequence and Titles of Structure Drawings

For all types of bridges:

- General Arrangement
- Borehole Locations and Soil Strata
- Roadway (Track) Protection
- Foundation Layout
- Footing Reinforcement
- Foundation Layout and Footing Reinforcement
- North (West) Abutment
- North (West) Wingwall
- South (East) Abutment
- South (East) Wingwall
- Abutments
- Wingwalls (and Retaining Walls)
- Retained Soil System
- Piers
- Bearings
- Piers and Bearings

For bridges with post-tensioned decks:

- Deck Details
- Longitudinal Tendons I, II, etc.
- Transverse Tendons I, II, etc.

For bridges with precast, prestressed concrete girders:

- Prestressed Girders
- Prestressed Girders and Bearings
- Deck Details

For bridges with steel girders:

- Structural Steel I, II, etc.
- Deck Details

For all types of bridges:

- Deck Reinforcement
- Joint Anchorage and Armouring
- Barrier Wall with Railing
- Barrier Wall w/o Railing
- Railing for Barrier Wall
- 6000mm Approach Slab
- Details of Concrete Slope Paving
- As Constructed Elevations and Dimensions.
- Pile Driving - Steam and Diesel Hammers
- Standard Details I, II, etc.
- Electrical Embedded Work
- Quantities - Structures I, II, etc.

2.5.3 Existing Bridges - Access to Contract Drawings

Requests for prints only (not originals) of the contract drawings of existing bridge structures should be submitted through the Regional Structural Section to the Administrative & Records Clerk of the Bridge Office.

Through the Bridge Document Imaging Management System (BDIMS), from compact discs the Records Clerk will provide a set of half-size prints as requested, or if the Region or the requesting Consultant has capabilities to transfer electronic files, he may download them and print out the drawings. For structures constructed prior to 1978, the originals will still be on Microfiche which the Region as well as Head Office has. In that case, the Region should make any prints they require from their own resources.

2.5.4 Water Level Designations

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

- a. A water level must be shown.
- b. The water level shown must 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. "429.000 (Dec/84)."
- c. If more than one elevation is known, the Section Head shall be consulted.
- d. In addition to the above, an estimated 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 Head, Regional Structural Section, shall be responsible for supplying all pertinent water levels in the Structural Planning Report.

2.5.5 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 is obliged to pay for unnecessary over-excavation and additional concrete.

The word "minimum" should not be used in such cases. The required depth of footing and the required recess into rock should be shown. (See 4.3.1). The word "minimum" should not be used to specify the strength of concrete, as it results in a conflict with the strength requirements in OPSS 1350.08. Concrete must be specified by metric class (see 2.3.1).

2.5.6 Notes and Labels Forming Part of Views & Details

Notes placed on views or details, describing components or requirements should be precise, unambiguous, contain no unnecessary words, and be used only when necessary.

A few which give trouble are as follows:

- a. "Barrier wall and rail". There is no need for the word "Standard". The note must 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 should not be repeated on the Cross Section.

- b. "Slope paving (typ.)". There is no need for other description and the note should appear only once, on the elevation.
- c. "Top of concrete end dams to suit pavement profile". This note should always appear on the abutment drawing.
- d. "Top of cleat to be cast 35 mm below approach slab ledge". Reference to the styrofoam or neoprene is unnecessary since this is on the approach slab drawing. Reference to the approach slab is undesirable, since even if a slab is detailed, it may be subsequently deleted.
- e. Notes calling for the removal of formwork or expanded polystyrene should never be used. The specifications require all formwork be removed and if expanded polystyrene is not to be left in place, it should not be shown at all.
- f. Notes concerning construction joint grooves and sealing must not duplicate or conflict with OPSD-4670.000 if this standard is included in the drawings.

2.5.7 General Notes

The following are standard notes of the type shown below the title block on the General Arrangement 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.

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. CLASS OF CONCRETE

(Example, see 2.3.1)

30 MPa

CLASS OF CONCRETE 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 CAPS	70 ± 10	(For PIER CAPS in splash zone with epoxy coated steel)
REMAINDER	70 ± 20	UNLESS OTHERWISE NOTED

3. REINFORCING STEEL

REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED.

BAR MARKS WITH PREFIX 'C' DENOTE COATED BARS.

STAINLESS REINFORCING STEEL SHALL BE TYPE 316LN OR DUPLEX 2205 AND HAVE A MINIMUM YIELD STRENGTH OF 420 MPa.

BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS

UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES SHALL BE CLASS B.

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 DRAWINGS SS12-1 AND SS12-2, UNLESS INDICATED OTHERWISE. (The designer shall include SS12-1 and SS12-2 on the contract drawings).

4. RETAINED SOIL SYSTEM (where applicable)

RETAINED SOIL SYSTEM WALLS SHALL HAVE THE FOLLOWING ATTRIBUTES:

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 2.5.7(d) below)

(b) GENERAL ARRANGEMENT DRAWINGS - CONCRETE RIGID FRAMES

1. CLASS OF CONCRETE

30 MPa

2. CLEAR COVER TO REINFORCING STEEL

FOOTINGS 100 ± 25

DECK TOP 70 ± 20

BOTTOM 50 ± 10

REMAINDER 70 ± 20 UNLESS OTHERWISE NOTED

3. REINFORCING STEEL

REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED.

BAR MARKS WITH PREFIX 'C' DENOTE COATED BARS.

STAINLESS REINFORCING STEEL SHALL BE TYPE 316LN OR DUPLEX 2205 AND HAVE A MINIMUM YIELD STRENGTH OF 420 MPa.

BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS

UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES SHALL BE CLASS B.

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 DRAWINGS SS12-1 AND SS12-2, UNLESS INDICATED OTHERWISE. (The designer shall include SS12-1 and SS12-2 on the contract drawings).

4. RETAINED SOIL SYSTEM (where applicable)

RETAINED SOIL SYSTEM WALLS SHALL HAVE THE FOLLOWING ATTRIBUTES:

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

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

(See 11.1.1 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 11.1.1 for note to be used for special site access conditions)

(Other notes as applicable, see list under 2.5.7(d) below).

(c) GENERAL ARRANGEMENT DRAWINGS - CAST-IN-PLACE POST-TENSIONED CONCRETE BRIDGES

1. CLASS OF CONCRETE

DECK & SIDEWALKS	35 Mpa
REMAINDER	30 MPa

2. CLEAR COVER TO REINFORCING STEEL

FOOTINGS	100 ± 25

ROUND VOIDED DECKS

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

RECTANGULAR VOIDED DECKS

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

3. REINFORCING STEEL

REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED.

BAR MARKS WITH PREFIX 'C' DENOTE COATED BARS.

STAINLESS REINFORCING STEEL SHALL BE TYPE 316LN OR DUPLEX 2205 AND HAVE A MINIMUM YIELD STRENGTH OF 420 MPa.

BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS

UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES SHALL BE CLASS B.

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 DRAWINGS SS12-1 AND SS12-2, UNLESS INDICATED OTHERWISE. (The designer shall include SS12-1 and SS12-2 on the contract drawings).

4. RETAINED SOIL SYSTEM (where applicable)

RETAINED SOIL SYSTEM WALLS SHALL HAVE THE FOLLOWING ATTRIBUTES:

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 2.5.7(d) below)

(d) **CONSTRUCTION NOTES (to be used where applicable)**

"THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT. (In addition to the preceding note, the following shall be shown on the drawing where top of bearing elevations are shown: Top of bearing elevations shall be denoted with an asterisk(*), and add note,) " * SEE CONSTRUCTION NOTES ON DRAWING 1 ".

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 2.2.3, 50 mm is for tube piles).

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

NO CHAINS, HOOKS OR PEAVIES SHALL BE USED IN HANDLING TREATED WOOD. SURFACES EXPOSED BY FIELD CUTTING OR DRILLING SHALL BE TREATED BY SOAKING WITH AN APPROVED PRESERVATIVE EQUAL IN TOXICITY TO THE ORIGINAL PRESERVATIVE. THREE APPLICATIONS ARE REQUIRED, EACH TO AIR DRIED SURFACES. (Use only for treated wood).

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 CSPI SPEC 501-78M. 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 MUST BE AT LEAST (give depth).

(f) **STRUCTURAL STEEL NOTES**

(See 8.8.1)

(g) **PRE-TENSIONED GIRDER DRAWINGS**

(See 7.2.7)

(h) **POST-TENSIONED DECK NOTES**

(see 7.3.12)

(i) **PRESERVATIVE TREATED WOOD STRUCTURE DRAWINGS**

(See 15.2.1)

(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 may be used. Temperatures should be shown thus: 23°C, 0°C, - 10°C, etc.

2.5.8 General Arrangement Drawings - Content and Format

The following instructions are not exhaustive and are intended only to provide guidance in the preparation and checking of General Arrangement drawings. The General Arrangement drawing should appear within the Master Drawing Border for which a reduced scale hardcopy is shown in Figure 2.5.8(1). Line thicknesses and text sizes given are specified to ensure legibility after drawing reduction. A "**thin**" line should 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 need be used in only three sizes: **large** 4 to 4.5 mm high, **medium** 3 to 3.5 mm, and **small** 2.5 mm. Vertical, uppercase, Roman Simplex text font is preferred. Large text within the title block should be done, using 3.5 mm text height.

Title Block

The name of the structure and the W.P. No. 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 2.5.8(2).

The Contract No. and the sheet number should be left blank since it will be completed by others at the time of tendering.

The District number should be given above the Contract number only on the General Arrangement drawing. The standard north point should be shown on the General Arrangement drawing and on others, where required by the contents of the drawing.

The drawing title shall be as given in 2.5.2.

The highway number is not required in the title block but must appear somewhere on the Plan.

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 be given also.

Initial Block

All blocks must be filled as indicated below and illustrated on Figure 2.5.8(1).

DESIGN	Initials of <u>actual</u> designer with responsibility for the design. For the General Arrangement drawing the initials should indicate general responsibility for the preliminary design, and not that complete design computations have been carried out.
CHK (DESIGN)	Initials of <u>actual</u> checker with responsibility for checking the design.
CODE	Name of design code used in design. For current MTO projects this is the Canadian Highway Bridge Design Code and shall be designated as CHBDC-00. For railway structures this may be "A.R.E.A.."
LOAD	Loading used in design specified in the above applicable code. For new MTO highway structures this will be CL-625-ONT. Railway loading may be according to A.R.E.A. 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 Regional Structural Sections, stockpiled, or sent 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 2.5.1.
DWG	Add drawing number as per 2.5.1.

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 should 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 must be clearly indicated, and the plan must 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 must 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 should be considered.

The key plan needs show only major roads, railroads, rivers, lakes and towns. The roads should be shown in **thick** single lines, railroads thus ++++++, lakes and rivers in **thin** "shaky" lines. Double parallel lines should be avoided unless, for example, a river is of great width. The title of "Key Plan" should 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 should 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, should 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 should be considered to obtain the largest plan possible within the space limits of the master drawing border.

The Plan should 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 standard north arrow should be shown in the space provided in the title block. A north arrow, rotated up to 45° from true north may also be shown on the Plan and marked "North for Construction", but this should 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 should 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 should be discussed with Regional Planning and Design to find out if the chainage direction was correctly determined and to ensure consistency between the road and bridge drawings.

Existing contours should 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 should 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 should be **thin**, except that 5 m interval contours may be of **medium** thickness.

On the Plan, the following are required:

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.

All centre lines and control lines, pier centre lines and abutment bearing centre lines in **thin** centre lines.

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 should be shown in lines of **medium** thickness. Railings should not be shown.

Hidden work such as pier or column shafts, abutment faces and grading details under the structure should be shown in **thin** dashed lines. Footing outlines should be shown to indicate proximity to pavements, tracks, creeks, etc., but need not be shown for abutments that are remote from such features. Services likely to affect or be affected by construction should 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 must 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 should be used.

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

A Plan title must 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 should 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 should be shown and labelled WP #1, WP #2, etc; names should be written along all centre or control lines and the skew angle should be given, all in **small** text. In **small** text also, give stations, slopes, at least one top of pavement elevation at some well defined central point, and control line geometry, e.g. degree of curvature, azimuth, etc. If coordinates are available or obtainable, they should be shown for TC's, SC's, PI's, etc., on the General Arrangement drawing, and for WP's, on the Foundation Layout. In any case, they should be shown together with stations. These coordinates may be tabulated separately to avoid congestion on the Plan.

On the approach slab the following note should appear "6 000 mm approach slab with 80 mm asphalt". Rigid frame bridges and Integral Abutment bridges should be detailed with 90 mm asphalt since they have no expansion joint dams, therefore the top of the approach slab and the top of the deck can be kept flush. This note should be shown with a dimension line showing the 6 000 mm dimension. The "6 000 mm" should appear above the dimension line and the remainder of the note below it.

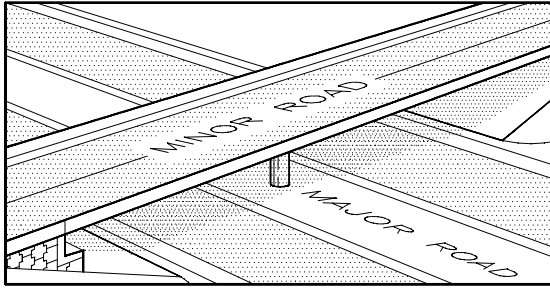
Elevation

The Elevation should 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, should be at the near bridge face and skew effects do not appear.

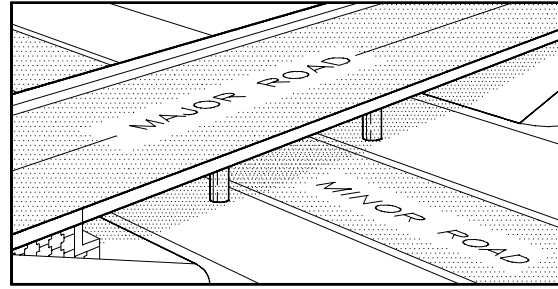
The outline appearing on the elevation, in lines of **medium** thickness, should include barriers, an indication of railings, coping, soffit, and parts of piers, abutment faces and wingwalls that are above ground, final ground line, and preliminary grading lines (See 2.2.3.) Hidden detail lines, thin and dashed, should show footings, piles, and other parts of piers and abutments that are buried.

A **thin** long-dashed line is used to 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 the W.P. symbol . Print in **small** text footing top elevations, water levels, (see 2.5.4) an approximate lower pavement level, bearing types ("exp", "fixed" etc.), 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 should 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.

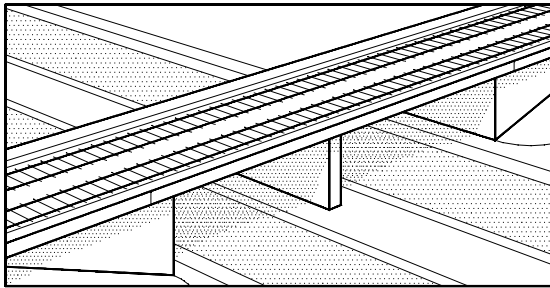
A triangular numbered symbol (optional Elevation title) followed by the scale should show, by reference to the corresponding symbol on the plan, where and in what direction the view is taken.



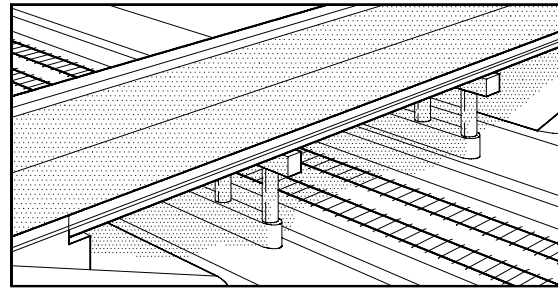
UNDERPASS
THE MAJOR ROAD GOES UNDER
A LOWER CATEGORY ROAD



OVERPASS
THE MAJOR ROAD GOES OVER
A LOWER CATEGORY ROAD



SUBWAY
THE ROAD GOES UNDER
THE RAILWAY



OVERHEAD
THE ROAD GOES OVER
THE RAILWAY

Figure 2.5.8(2) UNDERPASS, OVERPASS, SUBWAY AND OVERHEAD BRIDGES

Cross-Section

The Cross-Section should be located to the right of the Plan but may be placed elsewhere if the Plan extends across the sheet. The scale should be 1:100 if possible. One Cross-Section should be sufficient. If the Cross-Section varies in some way, this can generally 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 must 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 should be given.

The Cross-Section should be shown in **medium** lines; 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 in **thin** lines should be shown and labelled with **small** text.

Dimensions must 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 should be covered by a note stating the number, type and spacing of the beams.

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

"ASPHALT AND WATERPROOFING SYSTEM, 90 mm TOTAL".

No title should be used, but a triangular symbol, numbered to agree with the one showing the section plane on the Plan. This should appear beneath the Cross-Section, followed by the scale in **small** text. The direction of the Cross-Section should 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 should be added to show crossfall, wearing surface, number, size and type of ducts, etc. The section should be taken looking in the direction of increasing chainage.

Crossfall and superelevation 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 2:1. The use of a symbol as shown in the following example is preferable:

$$1 \sqrt{2}$$

Bench Mark

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

General Notes

In a column about 150 mm wide below the title block and under the title "General Notes" in **large** text, give the notes in **small** text as specified elsewhere. (2.5.7). These include Construction Notes.

List of Drawings

Below, or if necessary, to the left of the General Notes, list vertically the number and title of all drawings forming part of the structural contract documents (see 2.5.2). This may include embedded work drawings. For reference to DD and OPSD drawings, see 2.5.10.

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 should be given especially if the compass directions could be misinterpreted.

Miscellaneous

When road, rail or water traffic is permitted under the structure during construction, construction clearances must 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-4602.000 can be referenced. A separate final clearance diagram is always required for a railway opening and this should show construction clearances if these are less than final clearances.

Non-standard backfill requirements must be shown as specified in 5.5.1.

Functions of skew angles should not be given.

Hatching must 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 must appear to the left of the revision block (bottom right-side corner). P.Eng. stamps must always be signed and dated.

Preliminary Version of the General Arrangement

The preliminary version should 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 should be "P1" rather than 1. The title must 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 obligatory.

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 should be clearly noted, and some assurance that it is feasible should be obtained by telephone before issuing the preliminary version. The note must not appear on the final drawing.

2.5.9 Small Size Standard Drawings

For inclusion with contract drawings, small size drawings bearing an SS number must be added to a full-size standard drawing sheet as per Section 1.1.3.

The title block of the full size drawing must be completed. The title should be "Standard Details I" or II, (etc.). Initials are required as per Section 1.1.7.

2.5.10 Referencing Standards on Structural Drawings

OPS drawings (OPSD) 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 may be any ambiguity as to a standard's application, reference to the standard drawing is also to be made in close proximity to the affected detail.

2.5.11 Drafting Scales

For Structural drawings, the following scales are acceptable:

1:20*
1:25*
1:30
1:40
1:50*
1:75*
1:100*
1:125*

Those marked with an asterisk are widely used in other highway design work and are preferred. In the above and what follows, it should be understood that 1:20 legitimises 1:2, 1:200, etc.

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

1:60
1:150

2.5.12 Mylar Originals

Sepias or darkened mylars shall not be used to form part of any contract package because all too often the quality of the reproduction is very poor, causing contractors to make errors.

2.6.1 Procedure for Distributing and Filing Drawings and Calculations

This procedure is described in Chapter 5 of the Structural Contract Handbook.

2.6.2 Revisions to Original Drawings and File Copies

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

(a) **For all Structures Not Requiring Railway or Navigable Waters Approval**

1. REVISIONS TO FILE COPIES BEFORE TENDER PRINTS ARE PREPARED (generally before a contract number is added to the originals).

Revisions shall be entered in red pencil on the file copy. All revisions must be identified by letter code in a hexagon on the file copy. Letter 'A' for the first revision, 'B' for the second, etc. The same symbol is to be used in the revision column. The date, initials of the drafter making the changes and a short description of the nature of the revision should also be given.

2. REVISIONS TO ORIGINALS BEFORE TENDER PRINTS ARE PREPARED

Revisions must be transferred onto the originals immediately. No identification is needed on the revised details. To indicate that a revision has taken place, the last letter code used on the file copy is marked on the right hand lower corner of the margin; no entry in the revision column is required.

3. REVISIONS TO FILE COPIES AFTER TENDER PRINTS ARE PREPARED (generally after a contract number is added to originals).

Revisions at this stage may cause contractual complications. Changes that could have an effect on bid prices must be discussed with the Contract Preparation and Control Section as early as possible and with the Region. At the bidding stage, a Contract Addendum may be necessary. After award, a price change may have to be negotiated.

The revisions shall be entered in red pencil on the file copy. All revisions 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.

4. REVISIONS TO ORIGINALS AFTER TENDER PRINTS ARE PREPARED

All information described in 3 above shall be transferred onto the originals as soon as possible, but originals must not be revised during the period between advertising and award without the approval of the Contract Preparation and Control Section.

When originals are being revised, the Construction Services Engineer must ensure that the Regional Construction staff is informed. He should forward 8 full-size prints of all revised drawings to the Regional Manager of Construction and notify the Contract Preparation and Control Section of the need to issue prints of revised drawings to Contract book holders.

(b) **For All Railway Overheads and Subways, and Structures Subject to the Navigable Waterways Protection Act**

1. BEFORE THE FIRST SUBMISSION TO THE APPROPRIATE AUTHORITY FOR APPROVAL.

The procedures for the revisions outlined in Section (a), paragraphs 1 and 2 apply.

2. AFTER THE FIRST SUBMISSION TO THE APPROPRIATE AUTHORITY FOR APPROVAL.

The procedures for revisions outlined in Section (a), paragraphs 3 and 4 apply.

(c) **The Following Procedures Apply to All Revisions**

1. Originals shall be revised only on instruction from the Section Head responsible, who shall make certain that all required approvals have been obtained from the appropriate authorities.
2. A Bridge Office Section Head must sign to release originals from Structural Records.
3. Each revision must be approved by the designer by initialling in the margin to the right of the revision column of the file copy.
4. If an original has to be redrawn, the previous original and file copy shall be marked prominently "Superseded" across the drawing, dated, initialled and returned to Structural Records.
5. The revision slips (in duplicate), which may be hand written or typed, originals and file copies must be returned to Structural Records at the same time.

2.7.1 Bridge Deck Cross-Sections

The CHBDC, CHBDC Commentary and Section D.7 of the Geometric Design Standards for Ontario Highways shall be used to determine deck cross-sections.

The Geometric Design Standards for Ontario Highways uses functional classification in establishing side clearances. To assist in deciding the required classification Figure 2.7.1 may be used.

On long bridges (greater than 50 m long), particularly on long-span structures where the cost per square metre is greater than the cost on short-span structures, structure widths less than desirable may be acceptable; however, economy alone should not be the governing factor in deciding structure or clearance widths. The analysis of traffic characteristics (design speed, traffic volume, % trucks etc.), safety features, operating and emergency contingencies, and cost benefit ratios should be considered before the desirable structure width is compromised.

In deciding the bridge deck cross-section, and determining the dimensions, location, and design of the structure as a whole, the designer should aim to provide a bridge on which driver reaction and vehicle placement will be essentially the same as elsewhere on the highway. The resulting width should be of proportionate value to its usefulness and safety.

2.7.2 Horizontal Clear Zone Width Requirements at Bridge Substructures

This subsection gives direction in setting the ministry's clear zone width requirements to bridge abutments and piers.

For the purpose of this subsection the following definitions apply:

Auxiliary Lane	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.

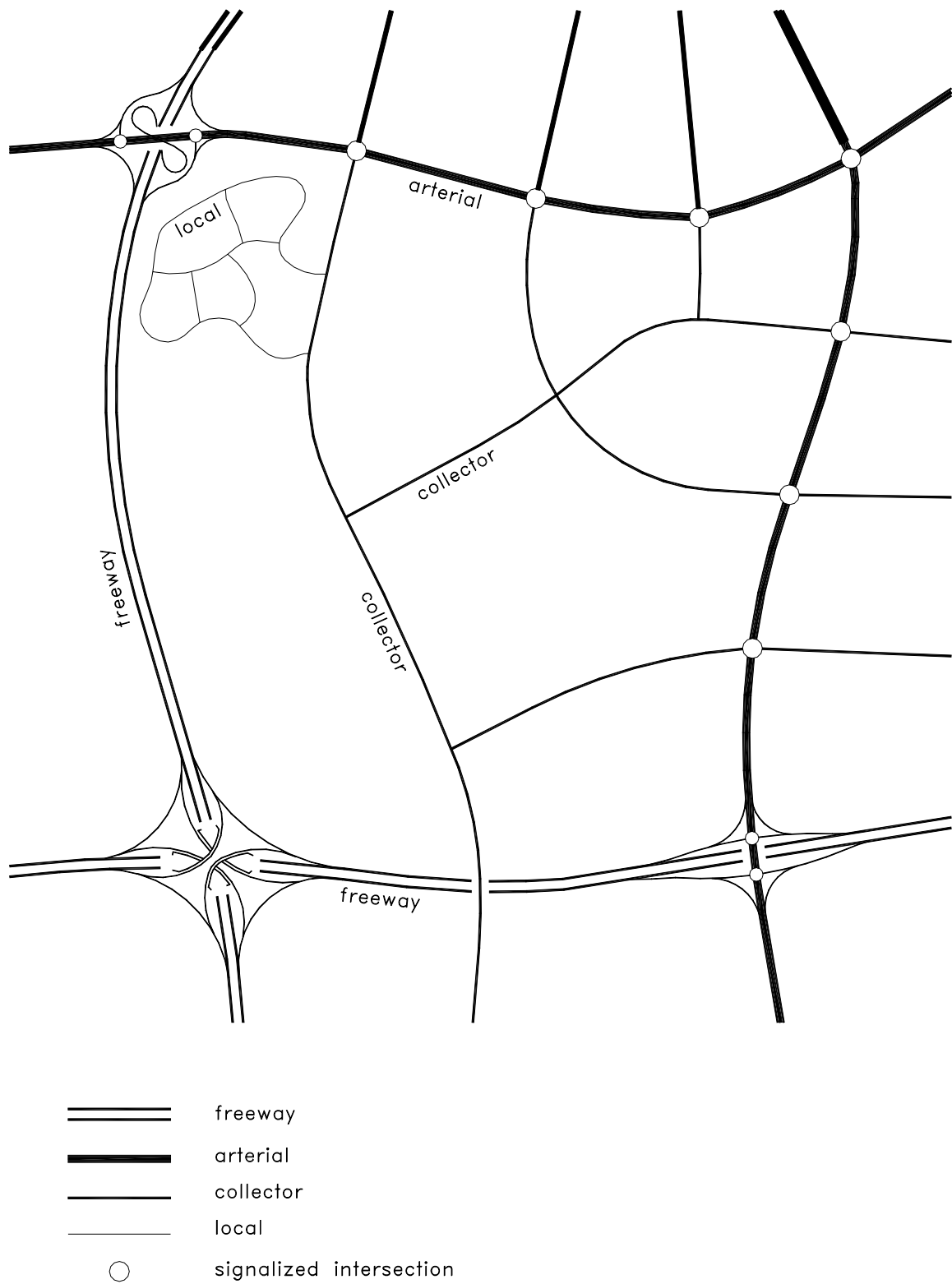


FIGURE 2.7.1 RELATIONSHIP OF ROAD CLASSIFICATIONS

Clear Zone	The total roadside border area, starting at the edge of the travelled way, available for safe use by errant vehicles. This area may consist of a shoulder, a recoverable slope, a non-recoverable slope, and/or a clear run-out area. The desired width is dependent upon traffic volumes and speed, and on the roadside geometry.
Clear Zone Width	The distance from the edge of the travelled way to the face of an unprotected hazard.
Freeway	A fully controlled access road limited to through traffic, with access through interchanges.
Gore area	Area between the through travelled way, edge of ramp and bullnose.
Travelled Way	That part of a roadway intended for vehicular use, excluding shoulders.
Through Travelled Way	That part of a roadway intended for vehicular use, excluding shoulders and auxiliary lanes.

In 1979 and 1983, the Ontario Highway Bridge Design Codes (OHBDC) required 10 m from the travelled way to the face of the abutment or pier. The 10 m clearance originally came from an AASHTO 1967 Traffic Committee Report "Highway Design and Operational Practice Related to Highway Safety" requirement which indicated that 80% of vehicles could recover within a distance of 30 feet (10 m) from the travelled way. In 1991, the OHBDC required horizontal clearances from the travelled way to a hazard to be established with consideration to the clear recovery zone determined in accordance with the Roads and Transportation Association of Canada (RTAC) "Manual for Geometric Design Standards for Canadian Roads." This manual also stated that a clearance of 10 m is desirable to eliminate the use of barriers and to give an open pleasing appearance. Current practice is now set by the CHBDC and that requires the regulatory authority to set standards for clearances. The "Roadside Safety Manual" is therefore the standard on which the Ministry bases its requirements.

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 curve correlation factor given in the Roadside Safety 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.

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 Safety Manual for the appropriate design speed for $AADT \geq 6000$ and radius of the through travelled way.

- (ii) width of gore area plus width of auxiliary lane or ramp plus the clear zone width given in (i) from the outside edge of the auxiliary lane or ramp.

For exceptions see below.

Exceptions

Where the above requirements 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. When this occurs reasons for this deviation from the above policy should be demonstrated and documented in the Structural Design Report. Mitigation measures can include an approved barrier system as specified in the Roadside Safety Manual. The barrier system must 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 must be the greater of the following requirements:

- (i) the minimum clearance to barriers as required by the Roadside Safety Manual.
- (ii) the shoulder widths on the approaches as required by the Geometric Design Standards for Ontario Highways.

Consideration should 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.

2.7.3 Treatment of Slopes in Front of Abutments

At overpass locations where the backslope parallel to the roadway in front of the abutment is within the clear zone width and at a slope not steeper than 2:1, 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 Fig. 2.7.3 this slope and the exposed side of the abutment is a hazard to errant vehicles. In this case the approach grading should be contoured to steer vehicles away from the abutment back on to the highway. An example of such grading is shown in Fig. 2.7.3.

Where possible, grading the earth fill at the approaches is a safer and more economical practice than placing a traffic barrier in front of the abutment. The introduction of a barrier replaces one hazard with another, which is not desirable. Using AASHTO's severity index system* this contouring treatment would be rated as a safer condition than using barriers.

(*AASHTO has developed a comprehensive severity index system, representative of a wide variety of typical roadside hazards, that can be translated into accident costs and an evaluation of the relative effectiveness of alternate safety treatments. This system assigns a severity index (SI) number from zero to ten, used to categorize accidents by the probability of their resulting in property damage, personal injury, or a fatality, or any combination of these possible outcomes. These indices may be found in the most recent version of their *Roadside Design Guide*.)

In situations where it is not practicable to contour the approaches to the abutment fill, for example at closed abutments, appropriate mitigation measures such as traffic barriers should be considered. However this should be looked at as a last resort when there are no other options.

2.7.4 Protection of Retained Soil Systems Abutments

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

- (i) Where RSS concrete facing panels are used they should be protected by an approved barrier system or a 2:1 upward slope for a distance of 4.0 m minimum measured horizontally from the edge of shoulder to the face of panels. Facing panels at or beyond the clear zone requirements need not to be protected.
- (ii) Where integral or semi integral abutment design is used the abutments should be protected as in (i) or designed for impact load.
- (iii) The approved barrier system should be mounted at the following minimum distances to the front face of the abutment wall:
 - (a) 1.5 m from the front face of a steel beam guiderail.
 - (b) 1 m from the toe of the back face of a concrete barrier.

2.7.5 Clearances for N.W.P.A. 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 will 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.

2.7.6 Vertical Clearance over Highways

Vertical clearances for structures are prescribed in the Ministry publication "Geometric Design Standards for Ontario Highways." 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.

The determination of a new structure's profile must also take into account required falsework clearances and falsework depth. Table 2.7.6 lists usual falsework dimensions for various lane arrangements.

Table 2.7.6

Req'd 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.

2.8.1 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.

2.8.2 Corrosion Protection Policy for Concrete Bridge Components

Definition of Terms (For the purpose of these Guidelines)

AADT (Average Annual Daily Traffic): means the total yearly traffic volume on a given road, in all lanes and both directions, divided by the number of days in the year.

Corrosion resistant materials: means stainless steel and high performance concrete.

Normal Concrete: means concrete of a minimum specified 28-day compressive strength up to 50 Mpa and which does not include silica fume.

High Performance Concrete (HPC): means concrete with a minimum specified 28-day compressive strength of at least 50 MPa, which must include silica fume and may include other supplementary cementing materials, and having a rapid chloride permeability at 28 days of 1000 Coulombs or less.

Stainless Steel Reinforcing: means stainless steel reinforcing bars conforming to ASTM A276 and ASTM A955M, minimum Grade 420, Type 316 LN or Type 2205 Duplex. Stainless steel requirements for MTO work are to be found in Special Provisions 905S04 and 114S03, and the "Guidelines for the Inspection and Acceptance of Stainless Steel Reinforcement" * provides information regarding quality issues associated with stainless steel.

* "Guidelines for the Inspection and Acceptance of Stainless Steel Reinforcement", Ontario Ministry of Transportation. (Available from Concrete Section, Materials Engineering and Research Office, 1201 Wilson Ave., Downsview, ON M3M 1J8, Telephone 416 235 3705.)

Background

Currently Ministry allows the use stainless steel and high performance concrete as corrosion resistant materials in the salt splash zones of structures. To provide guidance to designers for their use on MTO projects the following guidelines have been developed. These guidelines replace all previous policy documents and memos dealing with this subject.

Although it is recognized that both stainless steel and high performance concrete have higher strengths than conventional materials, these guidelines only address the durability aspects of these materials.

The ministry will continue to review the performance of existing high performance concrete and stainless steel installations to date, review the experience of other jurisdictions and monitor costs and issues associated with construction contracts.

Other materials are under consideration at this time in MTO research activities, including stainless steel clad reinforcing bars, and bars made of steel alloys that may offer improved corrosion resistance. These may offer advantages over the current approach to corrosion protection, but their long-term performance has not been proven and they remain under assessment at this time.

All this information will provide input to the ministry's corrosion protection strategy, so that the policy can be refined. However, until a sufficient level of comfort is reached, with performance,

availability and cost to consider broadening the use of premium corrosion protection materials, the guidelines described below will remain in place.

Recommendations

New Bridges

(I) Bridges Carrying Strategic Highways

Premium corrosion protection and durability enhancement will be specified for bridges **carrying** strategic highways.

For the purpose of applying these guidelines, strategic highways are defined as follows:

Highways **carrying** traffic volumes with an AADT $\geq 100,000$, and shown in bold in Fig.2.8.2. All structures between the highway limits shown are to be included. This map is based on highways carrying traffic volumes with an AADT $\geq 100,000$ for the year 2000 and will be updated when necessary.

For bridges **carrying** the above highways corrosion protection materials will be specified as follows:

- (i) Stainless steel will be used in areas of all bridge components where their surface is within the Splash Zone as defined in Table 1. On overpasses, overheads, and bridges that span roads, railways and waterways, where the application of de-icing chemicals is minimal, this requirement may be waived for piers and columns.
- (ii) For waterproofed decks and all other components not covered in Table 1 epoxy coated steel and black steel will be used as specified in the Structural Manual (Section 12.) If there is regional interest in using stainless steel in bridge decks, the Region would have to demonstrate that the additional costs of using stainless steel is justified.
- (iii) High performance concrete (HPC) may be specified for all bridge components except for the following:
 - Footings
 - Post-tensioned bridges unless approved by the Ministry
 - Prestressed precast concrete girders with 28 day design strength less than or equal to 50 Mpa
- (iv) All concrete decks shall be protected by an asphalt and waterproofing system.

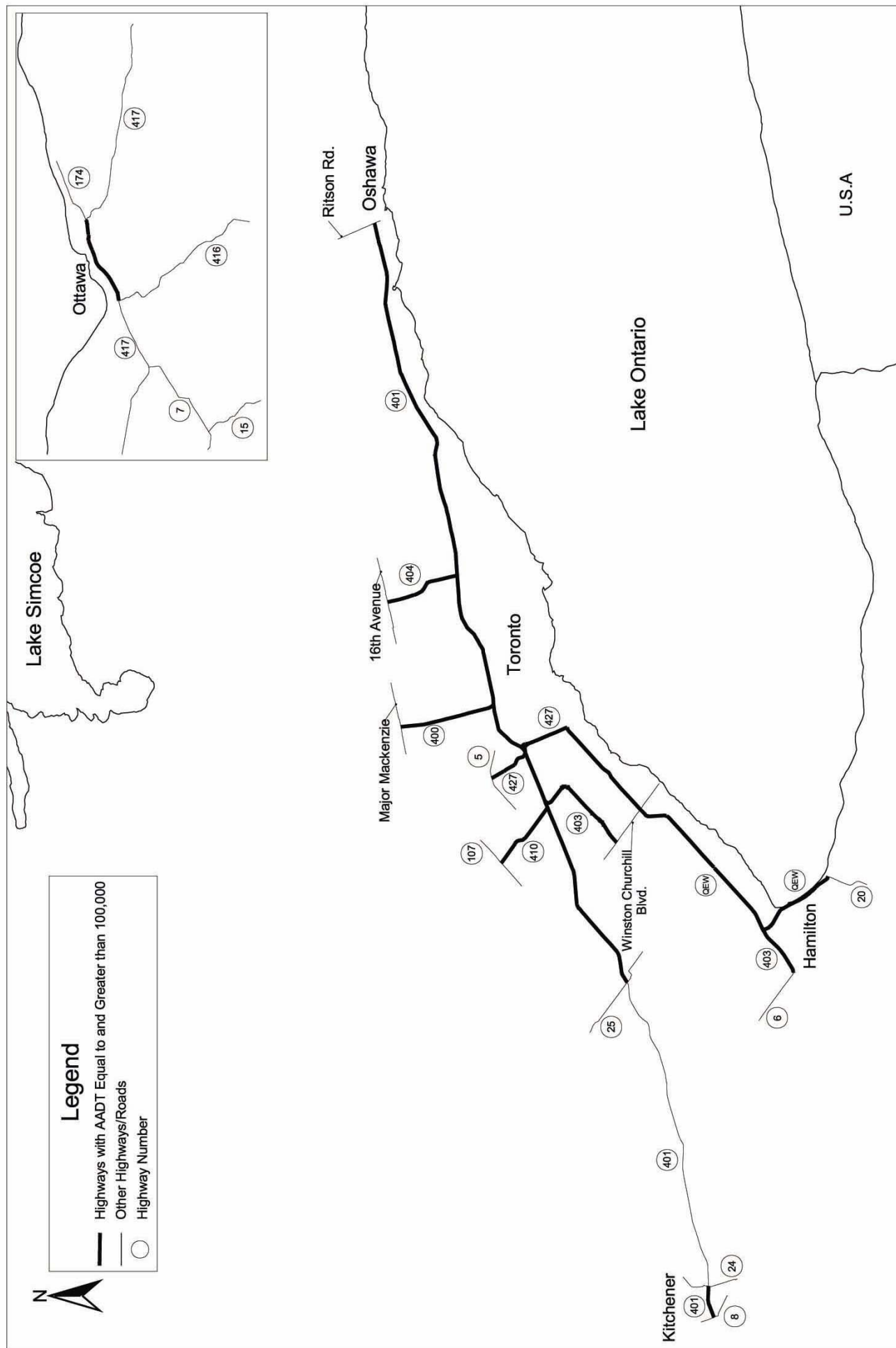


Figure 2.8.2: Location of Highways with AADT Equal to and Greater than 100,000
Based on Traffic Figures For the Year 2000

Table 1: Component Surfaces Within Splash Zone

• Top and side surfaces of barrier walls and parapet walls
• Top and side surfaces of sidewalks, medians and curbs
• Top surfaces of expansion joint end dams
• Surfaces of columns within 10 m horizontally and 5 m vertically of an existing or future roadway measured from the edge of the nearest travelled lane. Pier caps are exempt from this requirement except as detailed below.
• Side and end surfaces of pier shafts within 10 m horizontally and 5 m vertically of an existing or future roadway measured from the edge of the nearest travelled lane
• Bearing seats and sides of pier shafts and columns, including pier caps, below expansion joints within 5 m measured vertically from the bearing seat.
• Top and side surfaces of bases for pole and sign structures that are attached to bridges

(II) All Other Bridges

For these bridges the guidelines are as follows:

- (i) Stainless steel reinforcing will be used in areas of all bridge components where their surface is within the Splash Zone as defined in Table 1 except as follows:
 - The superstructure components of bridges carrying traffic volumes with an AADT of ≤ 2000
 - On overpasses, overheads, and bridges that span roads, railways and waterways, where the application of de-icing chemicals is minimal, this requirement may be waived for piers and columns
- (ii) For all components, including decks, where stainless steel is not used, black steel and epoxy-coated steel is to be used as specified in the Structural Manual (Section 12.)
- (iii) For these bridges normal concrete is to be used. Where considered appropriate by the Regions high performance concrete (HPC) may be specified for all bridge components except for the following:
 - Footings
 - Post-tensioned bridges unless approved by the Ministry
 - Prestressed precast concrete girders with 28 day design strength less than or equal to 50 MPa
- (iv) All concrete decks shall be protected by an asphalt and waterproofing system.

Bridges for Rehabilitation

In general, the use of premium corrosion protection treatments, as described above, shall be followed in bridge rehabilitation when the remaining life of the bridge is equal to and greater than 35 years. For less than 35 years, normal concrete, black and epoxy coated steel shall be used as specified in the Structural Manual.

Additional Guidelines for Rehabilitation

In addition to the requirements described above, the following policies should 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 should be used in patch repairs, depending on the size and dimensions of the repair area. High performance concrete is not to be used in patch repairs, regardless of the remaining life of the bridge or its exposure condition.

Overlays: Two low permeability overlay options are available, silica fume concrete and latex-modified concrete, as outlined in the Structure Rehabilitation Manual. There is no “high performance concrete” overlay option available in current MTO specifications and this should not be specified. The current strength specification for silica fume concrete overlay is 40Mpa compressive strength at 28 days. Conventional concrete remains an option for overlays.

Widening of bridge, infill sections, replacement of outer deck edges: Bridge widening, infill sections, or partial replacements should not use premium corrosion protection systems, where there is no value in extending the life of the new section beyond that of the existing bridge. If the widened portion of the bridge is to remain in place for a longer time than the original bridge, use of stainless steel or HPC systems should be considered as per policy for new bridges.

3 PILES

3.1.1 Design Criteria

Prior to the initiation of the Foundation Engineering investigation, the Structural Engineer shall provide the Foundation Engineer with a conceptual layout of the proposed structure foundations.

The subsequent Foundation Design Report produced by the Foundation Engineer may contain information and recommendations with respect to pile foundations. When pile foundations are selected, the final design shall be developed through consultation between the Structural Engineer and the Foundation Engineer.

In the Foundation Design Report the Foundation Engineer shall provide recommendations for:

- i) type of pile
- ii) axial resistances and horizontal reactions for a single pile with the provision that these values may require revision for the pile group effect once a proposed pile layout is known. They shall be given as follows:
 - factored axial and horizontal resistances at Ultimate Limit State where the resistances are based on the lesser of the structural resistance* or the geotechnical resistance of the piles

*MTO's current policy specifies that the maximum Factored Axial Resistance at ULS for any pile is the factored structural resistance, which for HP 310 x 110 is limited to 2000 kN, and for HP 310 x 79 to 1450 kN

(NB. Factored Geotechnical Resistance at ULS = Ultimate Geotechnical Resistance x Resistance Factor.)
 - axial and Horizontal Reactions at Serviceability Limit State** for identified settlements or displacements and corresponding subgrade reactions

**Pile reactions 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.

(NB. 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

Pile resistances given in the Foundation 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 unsupported length of the pile, if piles are driven and left partially exposed or immersed in water.

Piled foundations with piles less than 3 metres long are generally not recommended because they are neither economical nor structurally efficient in that they are not flexible enough to distribute the loads. In such cases, alternate foundation systems such as spread footings on compacted granular pad (engineered fill) or on mass concrete, or the use of caissons, should be investigated.

3.1.2 Pile Tip Treatment

The following recommendations on pile tip treatment are guidelines only and may be superseded by the Foundation 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-3301.000 or OPSD 3302.000 respectively for H-piles or tube piles are required.
- 2) For piles driven to end bearing on hard material, through material presenting hard driving or obstructions such as boulders, driving shoes such as the Titus 'H' Bearing Pile Point (Standard Model) or equivalent shall be specified.
- 3) 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 3304.000 shall be specified and set into bedrock as detailed in the contract

A Non-Standard Special Provision shall be incorporated into the Contract Documents stating the procedure required for setting rock points into bedrock. Essentially the technique required is to initially reduce hammer energy to 10% of maximum and strike the pile in sets of 20 blows until no further penetration is observed. Then the pile should be struck with an additional set of 20 blows and if no penetration is observed the energy of the hammer should be increased by 10% and the procedure then repeated incrementally to 100% of maximum hammer energy.

3.1.3 Other Considerations

- 1) 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.
- 2) Refer to Section 6 of the CHBDC for minimum edge distance, embedment and spacing.
- 3) Maximum batter for all piles is typically 1:3. The Foundation Engineer must approve exceptions to this.
- 4) Pile driving tip reinforcement and rock points are to be used for steel piles only when specified in the Foundation Design Report (See 3.3.1). See applicable OPSD's.
- 5) Accessibility for pile driving equipment, allowing for batter, and any infringement on required construction clearances during driving should be considered.
- 6) 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-augered to an elevation below the level of the utility.

3.2.1 Steel H Piles

Steel H-piles are adequately described by giving the standard steel section designation only, e.g. HP 310 x110.

Based on the successful performance of H-piles in the past, the width-thickness ratio requirement given in Section 10.9.2.1 of CHBDC, $(b/t < 200/\sqrt{F_y})$, does not need to be satisfied.

The most common sizes available in Ontario are:

HP 310 x 79

HP 310 x 110

Piles HP 310 x 79 are preferred for use as friction piles in moderate driving conditions, while HP 310 x 110 piles are preferred for end bearing piles or hard driving conditions.

Piles stiffer than HP 310 x 110 shall be considered for lengths greater than 30 m when driven to end bearing through soft soil. The availability of such sections, however, should be investigated.

The type of steel is specified in the standard specifications.

3.2.2 Steel Tube Piles

When steel tube piles are required, the pile cross section must be specified by giving the outside diameter and the wall thickness as part of the Pile Data.

The most common sizes are:

273 mm O.D. x 6.3 mm wall thickness

324 mm O.D. x 6.3 mm wall thickness (This is a preferred size.)

The type of steel is specified in the standard specifications.

3.2.3 Wood Piles

When wood piles are required, 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)
- (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 the CSA publication CAN3-056-M79 *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 are not allowed.

Refer to Section 6 of the CHBDC for restrictions concerning the use of untreated wood piles, which are less expensive than treated wood piles. Refer to 15.2.1 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.

3.2.4 Precast Concrete Piles

Proprietary precast concrete piles may be used with permission of the Bridge Office Manager and the approval of the Foundation Design Section.

3.3.1 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 Foundation Engineer, rounded up to the nearest 0.5 m.

Pile notes are required close to the pile data table as appropriate. See also 3.3.3.

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-3301.000 OR OPSD-3302.000 TYPE ... OR AS APPROVED *(specify if pile reinforcement is required, see also 3.1.2.)
- 5) PILES SHALL HAVE DRIVING SHOES AS APPROVED.
- 6) PILES SHALL BE FITTED WITH ROCK POINTS AS PER OPSD-3304.000. (specify if rock points are required, and if so, the type. See also 3.1.2.)
- 7) PILE SPLICES SHALL BE AS PER OPSD 3301.000 AND IN ACCORDANCE WITH SP 903S01.

* Titus "H" Bearing Pile Points or Bermingham Buffalo Points are alternate products that are acceptable.

3.3.2 Pile Driving Control

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 principle 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, 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 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:

$$R = \frac{n e_f W g H}{S + C/2} \quad e_f = 0.75 \text{ for drop hammers}$$

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

$$R = \frac{n e_f E}{S + C/2} \quad \begin{array}{l} e_f = 0.6 \text{ to } 0.8 \text{ for steam hammers} \\ e_f = 1.0 \text{ for diesel hammers} \end{array}$$

Diesel hammers are currently the most commonly used type.

where

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

H = height of free fall of mass [metres]

R = Ultimate Pile Resistance (pile capacity) by Hiley Formula
[kilonewtons]

S = measured penetration of pile per hammer blow
[millimetres]

C = measured rebound of pile per hammer blow
[millimetres]

E = maximum rated energy of hammer (from Contractor, or tables)
[Joules/blow]

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

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

P = mass of components receiving blow
(pile+cushion+anvil) [kilograms]

W = mass of components (ram) delivering blow (from
tables) [kilograms]

$g = 9.8$ [metres per second²]

(a) Design Stage

The requirement for Pile Design is as follows:

$$\boxed{\text{Factored Geotechnical Resistance at ULS} > \text{Design Load at ULS}}$$

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 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:

$$\boxed{\text{Ultimate Pile Resistance } R > \text{Ultimate Geotechnical Resistance}}$$

- 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 above equation:

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 analyser 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.

3.3.3 Pile Driving Notes

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

When reference is made to SS 103-11 (Hiley Formula for steam and diesel hammers), it shall be included in with the drawings, which is usually the case unless the pile is driven to the bedrock.

Foundation Reports are to indicate which note is applicable.

1. PILES TO BE DRIVEN IN ACCORDANCE WITH STANDARD SS 103-11 USING AN ULTIMATE GEOTECHNICAL RESISTANCE OF kN PER PILE.
2. PILES TO BE DRIVEN IN ACCORDANCE WITH STANDARD SS 103-11 USING AN ULTIMATE GEOTECHNICAL RESISTANCE OF..... kN PER PILE BUT MUST BE DRIVEN BELOW EL
3. PILES TO BE DRIVEN IN ACCORDANCE WITH STANDARD SS 103-11 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 STANDARD 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 U.L.S. and at S.L.S. should be given and identified as such, e.g. MAX. FACTORED LOADS: U.L.S. 1450 kN, S.L.S. 1055 kN.

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4 FOOTINGS

4.1.1 Scour Protection

The Hydrology Report must state what is required to provide adequate scour protection for footings.

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.

4.2.1 Working Slabs

Working slabs shall not be shown on structure drawings. This applies even when a working slab is recommended in the Foundation Design Report.

The Regional Construction Staff will decide, at the time of construction of the footings, if a working slab is necessary and will make the necessary arrangements with the Contractor.

In cases where the Foundation Design Report requires that the foundation material must be covered within a limited time after exposure to prevent deterioration of the foundation material, this requirement and time limit should be noted on the footing drawing and mass concrete (not to be labelled as a "working slab") should be detailed if, after consultation with the Regional Construction staff, it appears that the time available for footing construction is not sufficient. The note should give the time limit, e.g., "Mass Concrete to be placed within 3 hours of excavation".

4.3.1 Foundations in Rock

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

As far as the footing stability is concerned the footing will perform as intended, i.e. resistance against sliding will be provided by the approximately vertical rock surface or 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 class 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 satisfaction of the Engineer and the over-excavation is replaced with concrete of the same class as the footing concrete.

Since OPSS 902.07 currently states that over-excavation is to be replaced by "a material suitable for the particular application and on the approval of the Engineer", it is essential that the drawings state that over-excavation of rock be replaced by concrete.

Where it is necessary to key the footings into rock the footing drawing should contain the following note:

- i. Footing(s) shall be set mm into sound bedrock.
- ii. Rock surfaces in over-excavated areas shall be subject to approval by the Engineer.
- iii. Over-excavation shall be replaced with concrete of same class as footing concrete.

4.4.1 Minimum Frost Protection to Footings

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

OPSD 3400.010 – Contours of Frost Depths For Northern Ontario

OPSD 3400.011 – 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 do not require frost protection; but if the rock fill is on soil, the minimum frost protection depth applies to the soil.

5 ABUTMENTS, WINGWALLS AND RETAINING WALLS

5.1.1 Design of Abutments

Beneficial effects of axial compression in flexural reinforced concrete abutment components shall be taken into account in proportioning the reinforcement whenever it is economical and practical to do so.

5.1.2 Earth Pressure

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.

5.2.1 Abutment and Retaining Wall Drainage

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 subdrains 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. Miradrain) other than wall drains if possible.
- (d) Wall drains shall be shown on the drawings by reference to OPSD-3506.000 or, if this is not appropriate, as "75 mm dia. non metallic wall drains at 3 000 mm c/c, elevation to be determined by the Engineer".

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

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

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

5.2.2 New Jersey Barrier Profile on Walls

When proposed abutments or retaining walls are close to travelled lanes, consideration should be given to forming a New Jersey Barrier Profile on the wall face. Possible applications of this treatment should be identified at the preliminary planning stage and discussed with Regional

5.3.1 Abutment Bearing Seats

Bearings should be supported on concrete pedestals that are at least 50 mm above the adjacent bearing seat ledge. The concrete surface between the pedestals must slope towards the front face at 5% min. under a sealed joint and at 1 in 3 min. under an open joint. In the direction parallel to the front face, this surface should be horizontal for simplicity.

Corrosion resistant structural steel bridges require special treatment to prevent rust staining of piers and abutments. For a standard detail on piers, see Section 6.3.2. For a standard detail on abutments, see Figure 5.3.1.

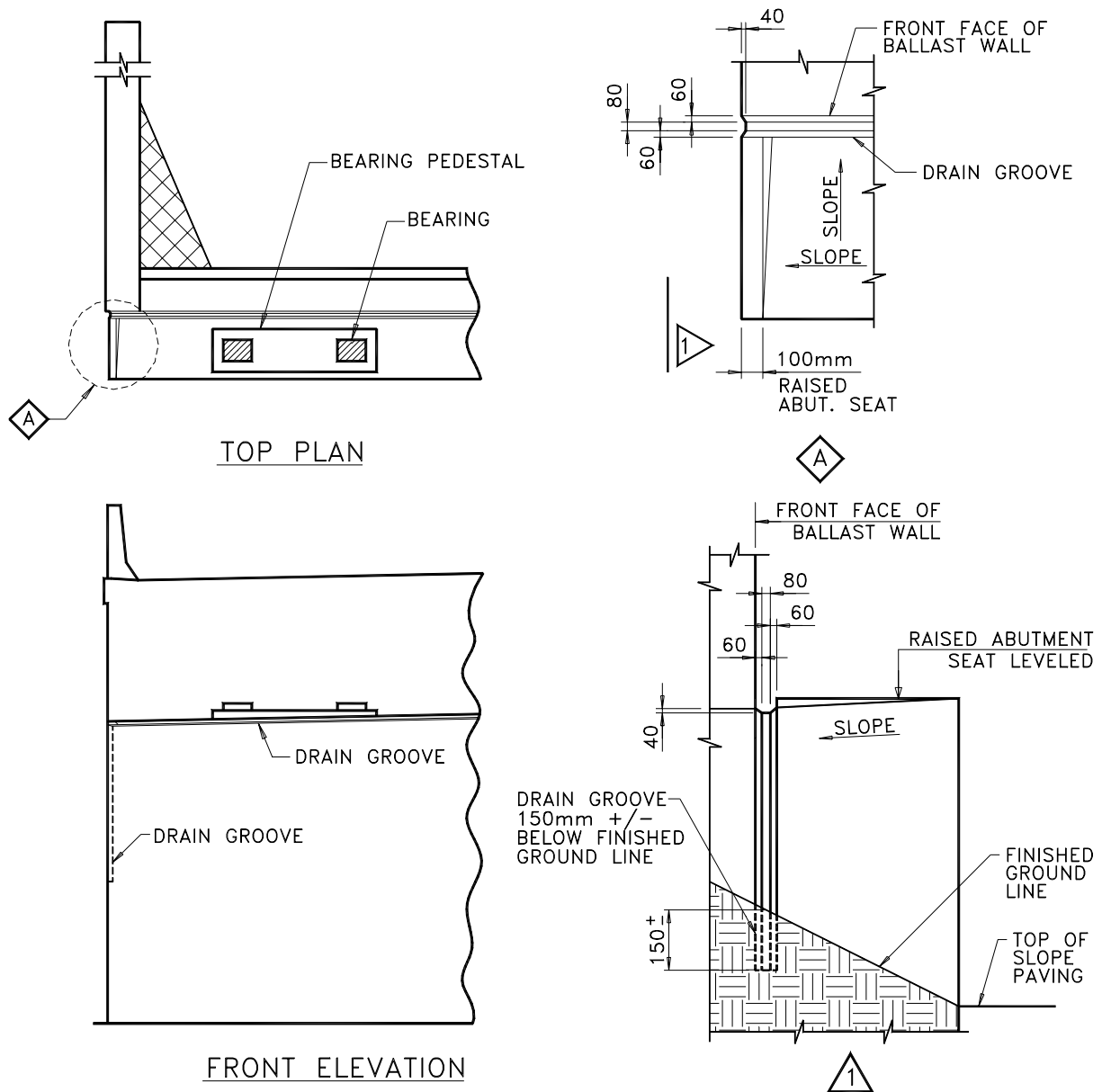


Figure 5.3.1 RUST STAIN CONTROL - ABUTMENT

All bridges, supported on bearings, having steel or concrete superstructures must have provision for jacking¹. It is feasible to jack from a 5% slope, using shims, but if the slope of the bearing seat ledge at intended jack locations is steeper, level areas must be detailed. The gap provided for jacks should be 150 mm min. in height.

The unfactored dead load reactions (for jacking purposes) and the permissible location for jacking points should be shown on the drawings. If it is anticipated that the bridge cannot be closed for jacking, live load reactions should be included.

5.3.2 Abutment-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 bearing seat. 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.

5.3.3 Dowels, Abutments to Approach Slabs

It is important that the 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 with dangerous consequences. In addition, dowels should be hooked so that they do not project beyond the back face of the abutment and obstruct backfill compaction.

Abutment drawings must show a vertical dimension from the approach slab seat to the dowel. The dowels should be hooked to a diameter such that providing the correct top cover will ensure correct vertical location; a vertical leg on the other end of the dowels should be of such length that the dowels cannot be installed upside down.

The dowels provided should be size 15M @ 150.

5.3.4 Ballast Wall Dimensions and Elevations

Ballast walls which are not more than 1 200 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 expansion joint armouring and the dam armouring at the top of the expansion joint. (See 13.1.1). The desirable width of the concrete dam at each side of the expansion joint is 500 mm to provide space for placing the concrete and for the bent "hairpin" rebar.

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 standard 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.

¹ See CHBDC Section 1.9.3.3

5.3.5 Abutment Stem Thickness

Requirements covering the gap between ballast walls and superstructures, and minimum ballast wall thicknesses, can lead to excessively thick abutment stems. For all except low, perched abutments, consideration should be given to corbelling the back face of the abutment in below the bearing seat ledge.

5.3.6 Staining of Concrete Abutments by Weathering Steel

Where aesthetics is important, the Regions may recommend a sealer to prevent rust staining of concrete abutments from exposed steel beams. This may occur during a prolonged delay, 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 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.

Northern Region has successfully used "Dekguard System Sealer" manufactured by Fosroc Construction Chemicals of Guelph, Ontario as a suitable sealer.

5.4.1 Wingwalls, Frost Cover

The minimum depth of frost cover for cantilever wingwalls in well drained soil and integrally connected to abutments shall be 1.5m. This is measured vertically from the ground surface at the end of the wingwall to the bottom of the wall. For self standing structures independent of the abutment eg. RSS wingwalls, the frost cover shall be as specified in Section 4.4.1 for footings.

5.5.1 Granular Backfill to Bridges

The Regional Offices compute approach grading quantities prior to receiving final structure drawings and assume backfill arrangements as shown on OPSD-3501.000. If granular backfill requirements differ from those shown on the Standards, they must be shown on the preliminary version of the General Arrangement.

Whichever of the above Standards is appropriate must always be referenced.

There is little benefit to be derived from granular backfill that cannot be drained, and it is not generally necessary to drain the back of a wall below the elevation of level soil or water in front of the wall.

5.6.1 Integral Abutment Bridges

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

Integral abutment bridges are well suited for the concrete slab-on-girder type of superstructures for total bridge span length less than 100 m, and skew less than 20 degrees. The height of the abutment and length of wingwall should not exceed 6 m and 7 m respectively, to minimise the effect of soil pressure and resistance. The abutment should be supported on relatively flexible piles such as H-piles. Where the load-bearing strata is near the surface or where the use of short piles less than 5 m in length or the use of caissons is planned, the site is not considered suitable for integral abutment bridges.

Short span bridges have been designed with integral abutments supported on narrow spread footings capable of providing a small amount of rotation. The behaviour of the structure and its durability is greatly influenced by the movement required and detail of the footing. It is therefore restricted to structures less than 25 m in length. Alternatively, further consideration may be given to details providing a semi-integral arrangement. "Semi-integral 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, movable bearings and horizontal joints at the superstructure and abutment interface. This arrangement should only be considered where an integral arrangement cannot be used due to unfavourable foundation conditions.

The structure length and skew limitations can be exceeded where approved. Reference should be made to the Structural Office Report # SO-96-01, "Integral Abutment Bridges" and Bridge Office Report BO-99-03, "Semi-Integral Abutment Bridges".

5.7.1 Retained Soil Systems (RSS)

Retained Soil Systems 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.

Retained soil systems may be considered for use where conventional gravity, cantilever, or counterforted concrete retaining walls 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. RSS are categorised in terms of three attributes, Geometry, Performance, and Appearance.

- For geometry, the categories are: Vertical, Terraced, Sloped
- For performance, the categories are: High, Medium, Low
- For appearance, the categories are: High, Medium, Low

These requirements are more fully detailed in an MTO Special Provision.

The Systems approved for use are listed in the Designated Sources for Materials (DSM) Listing, DSM#9.70.55. All design, construction and fabrication drawings and specifications required to complete this work shall be obtained by the Contractor from the supplier of the proprietary RSS system listed on the DSM.

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6 PIERS

6.1.1 Pier Nosings for River Piers

Steel nosings should not be provided unless specifically called for in the Structural Design Report or in subsequent correspondence by the Head, Regional Structural Section.

6.1.2 Round Column Diameters

Most contractors rent their round column forms. Listed below are the basic series of metric sizes of round column forms generally available. Some smaller sizes are available. Special sizes may be considered only where a very large number of columns are involved.

750 mm

900 mm

1000 mm

1200 mm

1350 mm

1500 mm

1800 mm

2100 mm

2400 mm

The forms are available in modular lengths of 300, 600, 1200, 2400 and 3600 mm, except that those greater than 1200 mm are not available in 2400 mm lengths.

6.2.1 Pier Bearing Seats

If the bridge deck over a pier is not continuous, the requirements of 5.3.1 concerning the sloping of the bearing seat ledge apply also to the pier.

The requirements of 5.3.1 concerning provision for jacking apply also to piers.

6.2.2 Expansion Columns

Expansion columns shall be proportioned based on a sliding coefficient of friction of 0.03, as described in 13.3.1.

6.3.1 Column Reinforcing Details in Earthquake Zones

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

Reinforcing details on Figure 6.3.1(1) are to be used in Seismic Performance Zone 1. They provide sufficient flexibility for the Contractor and do not compromise the integrity of the column at the interface of the footing.

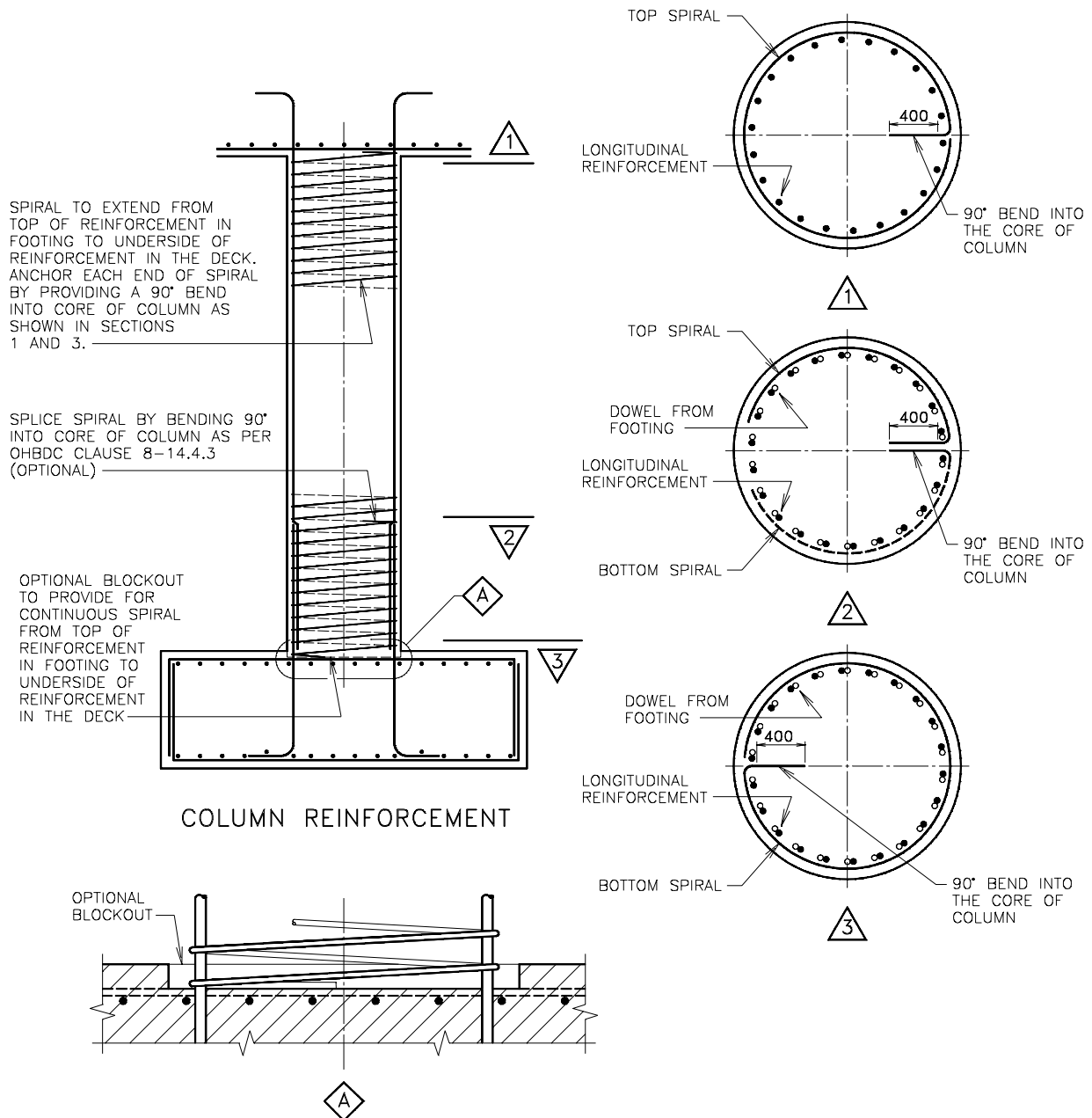


Figure 6.3.1(1) COLUMN REINFORCING DETAILS - SEISMIC ZONE 1

Figure 6.3.1(2) shows the reinforcing details to be used for Seismic Performance Zones 2, 3 and 4 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 (see Figure 6.3.1(2)).

It is intended that details on Figures 6.3.1(1) or 6.3.1(2) be shown on the contract drawings to avoid debate during construction.

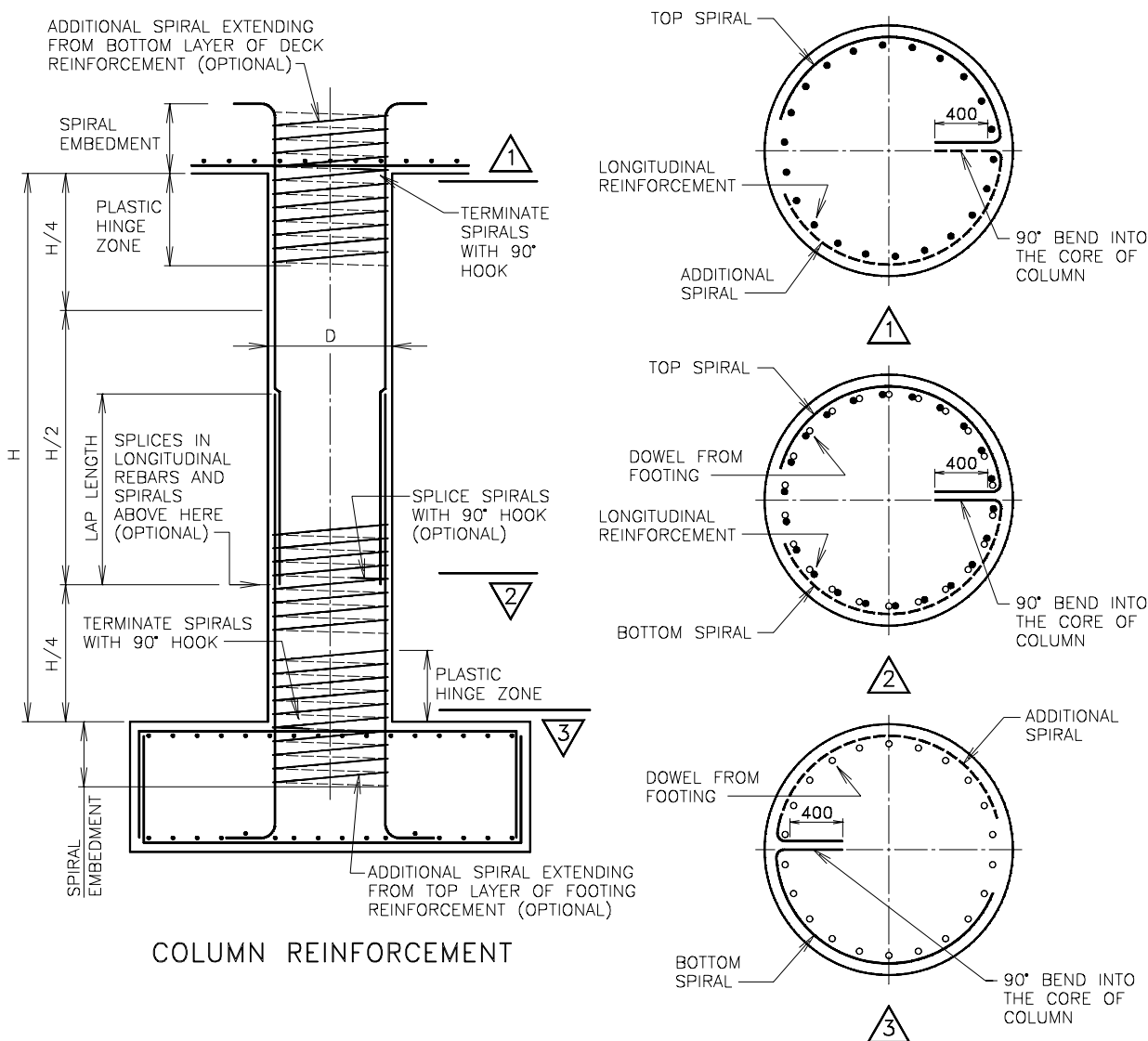


Figure 6.3.1(2) COLUMN REINFORCING DETAILS - SEISMIC ZONES 2, 3 AND 4

6.3.2 Rust Stain Control for Steel Girder Bridges

Piers are to be detailed so that bearing seats are 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 crossfall.

The vertical groove should clear any radius of shaft ends by at least 300 mm in pier shafts (see Figure 6.3.2(a)) and clear the column by at least 300 mm in pier caps on columns (see Figure 6.3.2(b)). 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.

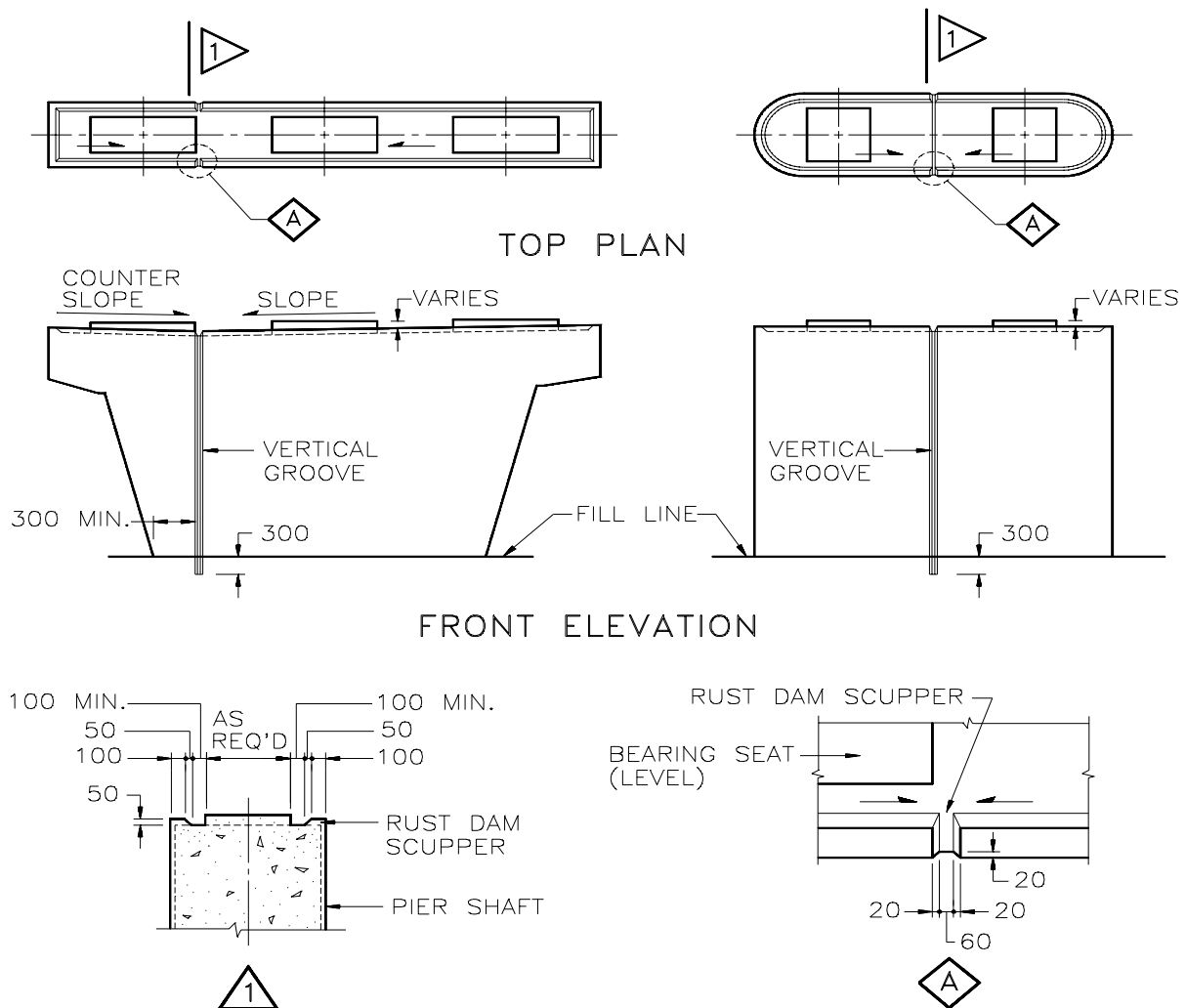


Figure 6.3.2(a) RUST STAIN CONTROL - PIER SHAFTS

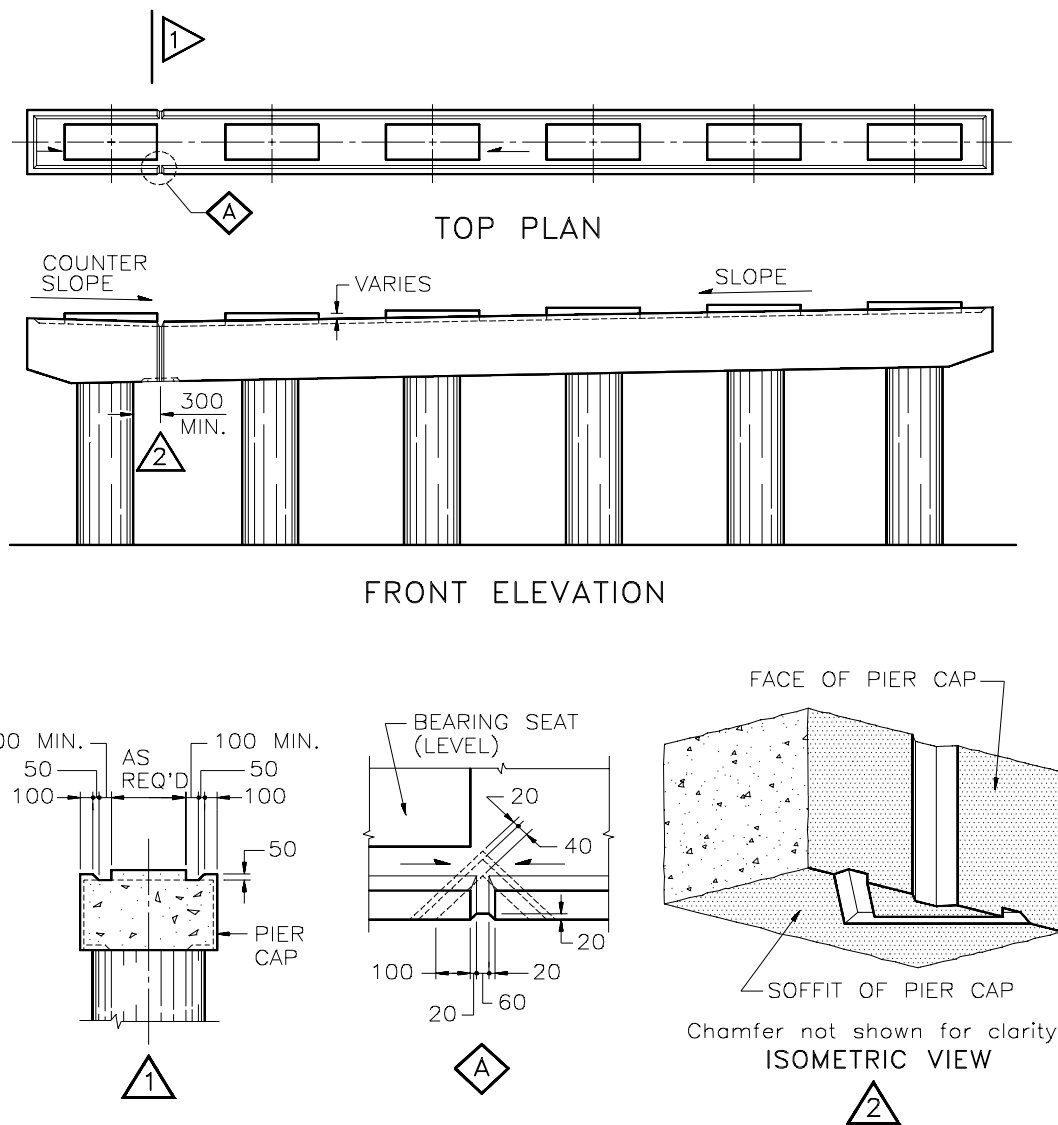


Figure 6.3.2(b) RUST STAIN CONTROL - PIER CAPS ON COLUMNS

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7 PRESTRESSED CONCRETE

7.1.1 General

This section on prestressed concrete describes the requirements for pretensioned girders (7.2.1 to 7.2.6) and post-tensioned decks (7.3.1 to 7.3.11).

7.2.1 Precast, Prestressed Girders

- (a) Girders should be pre-tensioned rather than post-tensioned unless transportation problems dictate post-tensioning (see 16.2.1).
- (b) The design of prestressed concrete members should normally be based on a 40-50 MPa class of concrete. Higher strength concrete may be used but only where there is a definite economic advantage and availability is not a problem.

If the 28-day compressive strength required is 50 MPa or less, normal concrete should be specified in increments of 5 MPa (eg. 40 MPa, 45 MPa.)

Where design requirements dictate the use of 28-day compressive strengths greater than 50 MPa, high performance concrete should be specified. Concrete strengths greater than 70 MPa should not be used.

Normal concrete for precast concrete girders means concrete with a specified 28-day compressive strength of 50 MPa or less and not containing silica fume. The lowest appropriate strengths should be specified, as there are increasing costs associated with increasing concrete strength. There are also special additional requirements for 50 MPa normal concrete, including additional protection for moisture loss during the fabrication process, monitoring and recording of temperatures to ensure compliance with CSA requirements and submission of records to MTO. Based on past ministry experience, girders up to and including 50 MPa compressive strength, of normal concrete, should provide adequate durability for most exposures and situations. This assumes cover requirements are met, temperature limits are observed, and other specification requirements are complied with.

High performance concrete (HPC) for precast concrete girders means concrete with a specified 28-day compressive strength greater than 50 MPa, which must include silica fume and may include other supplementary cementing materials, and having a rapid chloride permeability at 28-days of 1000 coulombs or less. There are special requirements that go along with the use of silica fume; HPC has additional requirements for fog misting during production, to ensure the durability benefits of the silica fume addition are achieved and to avoid potential problems with cracking of improperly cured silica fume concrete. There will also be additional physical testing requirements to verify that the concrete has attained the desired low permeability.

- (c) The transfer strength specified in the contract should be only as high as is actually required as it has a critical impact on the production schedule. The concrete strength at transfer shall preferably be such that the girder can be poured, cured and stripped on a 24-hr cycle. Too high a stress may mean that the girder will stay a day longer on the casting bed and could result in a loss of productivity and expected savings. When transfer strengths higher than 35 MPa are required, a 24-hr cycle may not be possible. In this case it is recommended that this issue be discussed with precasters, and any effect of a longer casting cycle be considered in an economic analysis.

It should be ensured that the lowest allowable concrete strength at transfer, as dictated by the design, is stipulated on the drawing. The actual concrete strength at transfer required, shall be specified to the nearest MPa, without rounding up to the next 5 or 10 MPa, using the term "Concrete Strength at Transfer... MPa".

- (d) Vertical bundling is permitted in accordance with the requirements of the CHBDC.
- (e) Deflected strands must be fanned out at the ends of the beams to obtain the most uniform distribution consistent with stress requirements. The designer should keep the centroid of the deflected strands as low as design parameters will allow, while attempting to spread the strands uniformly 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 at a slope of 1 to 6.

At hold down locations, use 25mm horizontal grids.

- (h) At the ends of the girders, the strands must be located on a 50 mm x 50 mm grid with the lowest horizontal line of the grid 60 mm above the bottom of the beam and vertical grid lines 25 mm either side of the beam centre line. For deflected strands, a 100 mm vertical spacing of the grid is recommended to avoid stress concentration and hence longitudinal cracking in girder webs.
- (i) Details of the positive moment connection over piers are given on OPSD-3701.000.
- (j) The transition dimension for the end block of CPCI 2300A girders is 1200mm.
- (k) The supply of diaphragm dowels is shared between the precaster and the contractor. The precaster will supply the threaded inserts and corresponding diaphragm dowels for the exterior girders. The other dowels should be part of the diaphragm steel supplied by the contractor.
- (l) All diaphragms shall be cast integrally with the deck slab pour, without construction joints.
- (m) The standard stirrups projection above the top girder must be verified and revised if necessary, to ensure that the projection is made long enough to account for girder hogging (camber) and the vertical curve of roadway profile.

7.2.2 Availability of Metric Girders

The available metric girder sizes that the ministry currently uses are:

CPCI 900, 1200, 1400, 1600, 1900, 2300

CPCI 1500 is also available but supplier's are limited and therefore is not encouraged for ministry projects unless special circumstances such as vertical clearance restrictions or costs justify its use. If required the standard drawing SS107-4 for CPCI 1600 girders may be appropriately modified.

7.2.3 Debonded Strands

As it is possible for moisture to penetrate around debonded strands from the ends of girders, debonded strands shall not be used at ends of girders at expansion joints.

7.2.4 Precast, Prestressed Plank Units (Solid and Voided Sections)

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 Section 8.21 gives requirements concerning the use of units placed side-by-side in contact.

7.2.5 Temporary Bracing

The temporary bracing referred to in 8.3.1 is required also for precast concrete girders.

7.2.6 Multi-beam Decks

Bridge decks constructed of side-by-side precast concrete box girders with an integral structural concrete slab, do not require shear keys between adjacent girders nor transverse stressing. Such units may however require some form of temporary tying to ensure their stability during construction.

Besides the requirement for stability during construction, adjacent girders must be prevented from deflecting differentially during the pouring of the concrete deck, which would cause cracking of partially set-up concrete.

Unrestrained adjacent exterior girders will deflect differentially if they are unequally loaded during the deck pour by a large cantilever slab, 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.

Therefore, to prevent deck cracking during the deck pour of such bridges, all girders should be detailed with sufficient anchored attachments at the top of the girders that would connect adjacent girders by field welding, eliminating differential deflection.

7.2.7 Pretensioned Girder Notes

Precast beams are generally detailed on standard drawings which have the notes preprinted. If standard drawings cannot be used, copy the appropriate notes from them.

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 SLAB SHALL BE RETARDED USING A TYPE "Rx" RETARDER FOR THE DURATION OF THE POUR.
4. CONCRETE IN BARRIER WALLS OR SIDEWALK SHALL NOT BE PLACED UNTIL ALL CONCRETE IN DECK SLAB HAS REACHED A STRENGTH OF 20 MPA.

5. FALSEWORK FOR THE CANTILEVER PORTIONS OF THE DECK SLAB (FOR A LENGTH OF 2.0 m) SHALL NOT BE REMOVED UNTIL THE CONCRETE AROUND THE INSTALLED EXPANSION JOINT HAS REACHED A STRENGTH OF 20 MPa (TYP. AT ALL FOUR CORNERS OF THE BRIDGE).
6. ALL DIAPHRAGMS SHALL BE CAST INTEGRAL WITH DECK SLAB, AND CONCRETE IN DIAPHRAGMS SHALL BE VIBRATED THOROUGHLY.

7.3.1 Post-tensioned Decks

- a. Post-tensioned superstructures which are solid or are 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 (d) below.
- b. Span/depth ratios shall not exceed 28.
- c. For skew angles in excess of 20° , transverse cables and reinforcement should be square to the deck except for capping beam prestress over skewed supports.
- d. Transverse moments over piers and abutments shall be resisted by transverse prestressing rather than reinforcing steel.
- e. 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. The minimum clear cover to round voids from the top of the deck shall be 180 mm, and from the bottom, 125 mm. The recommended clear covers are 200 mm and 150 mm respectively.
- g. There shall be no tension in the top surface of deck slabs in the direction of prestress except as permitted by the CHBDC.
- h. The final average longitudinal prestress after all losses shall not exceed 6.5 MPa at any cross-section in round voided slabs. The minimum average effective transverse prestress over the voids, after losses but without applied loads i.e. without D.L., L.L., etc., shall be 4.5 MPa.
- i. Post-tensioned decks shall be designed using 35 MPa concrete except in areas of the Province where 35 MPa concrete is not readily available. In these areas, reduced strengths shall be as specified in the Bridge Planning Report.
- j. Wide bridges, where the length of any span is less than 1.5 times the width of the deck, and bridges having a skew of more than 25° shall be designed using an approved analytical method such as STRAND for solid and voided slab bridges without integral legs, or BR 1230/BR 1240 for bridges requiring a 3-D FEM solution.
- k. The location of the outlets for cable duct vents should not be shown on the drawings.
- l. 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.
- m. Chamfer strips shall be detailed around the perimeter of the concrete that constitutes the bottom slab of rectangular voided post-tensioned decks.

- n. 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 in no case be less than 700 mm, and shall preferably be a minimum of 800 mm. This end distance for longitudinal cables may need to be even greater, depending on skew of bridge, size of tendon and size of bearing.
- o. Dead-end and live-end anchorages (local zone components such as anchorage plates, spirals, bond length, anchorage bulb dimensions and bulb spacings, etc) that are adequate for the tendon force are to be fully detailed and supplied by the post-tensioning supplier. The method of providing support for the dead-end anchorages is also the responsibility of the post-tensioning subcontractor.
- p. Whenever possible, the cantilever portion of cast-in-place, post-tensioned cross-sections shall be 2 100 mm measured horizontally. Where economically justifiable, this dimension may be reduced in 100 mm increments down to 1 600 mm.
- q. The spacing of stirrups should be such that, wherever possible, a stirrup or row of stirrups is located at each tendon ordinate location.
- r. Stockpiled projects which do not meet the above requirements are to be referred to the Bridge Office Manager by the appropriate Section Head in time to permit redesign.

7.3.2 Span to Depth Ratio for Circular Voided Decks

The following table provides recommended structure span/depth ratio for prestressed concrete circular voided decks.

Span Range (m)	Structure Depth (mm)	Void Size (mm)	Span/Depth Ratio
*28.5-30.5	1150	800	24.8-26.5
31-33	1250	900	24.8-26.4
33.5-35.5	1350	1000	24.8-26.3
36-38	1450	1100	24.8-26.2
38.5-40.5	1550	1200	24.8-26.1
**41-43	1650	1300	24.8-26.1

Notes:

(*) Solid section may be more appropriate in this range.

(**) Rectangular voided section may be more appropriate in this range.

This table is based on the recommended 150 mm cover to bottom of void and 200 mm cover to top of void. Where deck widening is expected the cover to the top of the void may be increased.

7.3.3 Round Void Forms in Decks

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:

Diameter mm	Diameter mm
300	800
400	900
450	1 000
500	1 200
600	1 400
700	1 500

Round voids larger than 1 200 mm may be used in special cases with Structural Office approval. 1 100 mm and 1 300 mm voids may be used if savings are significant.

The updating of stockpiled projects to comply with the above size requirements is not obligatory; but it should be considered, particularly if other substantial design changes are required.

Void forms and longitudinal reinforcing cables, etc., must be arranged so as to provide between all void forms at least one unobstructed vertical passage 100 mm or more in width as recommended in the CHBDC Commentary to Section 8.

Standard drawings showing void drainage, hold-down details and grout vents must be used or referenced. Stirrups should be shaped so that void forms can be lowered into place after the longitudinal cables have been placed.

7.3.4 Box Girder Voids and Access Locations

CHBDC Clause 1.9.3.1.5 requires access openings for the inspection and maintenance of concrete 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 1 200 mm. The minimum clearance for walking through intermediate cross bracing and diaphragms inside box girders shall be preferably 600 mm and not less than 500 mm.

The number of access openings and their locations shall be as follows:

- a) For cast-in-place concrete box girders with solid diaphragms: One opening per box cell.
- b) For precast concrete box girders and segmental 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.

This policy shall apply to all current and future designs, and wherever possible, to stockpiled designs.

7.3.5 Post-Tensioning Tendons, Ducts, Strands, Detailing & Friction Factors

When strand with a breaking strength of 260.6 kN is required, Grade 1860, Size Designation 15 or 0.60" diameter strand should be specified. The nominal area of size designation 15 strand is 140.0 mm². The main effect of revising stockpiled projects containing grade 1760, Size Designation 16 strand is the increase in elongation.

The majority of post-tensioned superstructures, whether solid slab or circular voided or trapezoidal voided, do not require tendons larger than size designation 1906. In some cases, if larger tendons are required, tendons up to size designation 2706 may be used.

Bright rigid ducts shall be detailed for the full length of all tendons in all post-tensioned concrete bridges. Rigid duct is defined for this purpose as being a steel duct having a wall thickness of not less than 0.60 mm. The radius of curvature for rigid ducts shall not be less than 9 m.

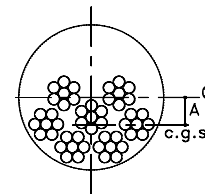
Low relaxation strand shall be utilised and specified on the drawings for all tendons. Stockpiled projects that call for stress-relieved strand or wire must be updated.

Tendons formed by merging strands from two or more ducts into one are not permitted.

7.3.6 Location of Post-Tensioning Tendons in Draped Ducts

After stressing, the post-tensioning tendons in curved ducts are pushed to the bottom of the duct in the negative moment areas, and to the top of the duct in the positive moment areas. The location of the tendon centre of gravity with respect to the centreline of the duct at the high and low points shall be considered in the design and may be assumed as follows:

<u>Duct Size</u>	<u>A</u>
75 mm or less	12 mm
over 75 mm to 100 mm	20 mm
over 100 mm	25 mm



7.3.7 Anchorage Slip for Strand Post-Tensioning Systems

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.

The anchorage slip specified on the plans for strand post-tensioning systems shall not be less than the following values:

For anchorages with up to 7 strands	8 mm
For anchorages with 8 to 12 strands	10 mm
For anchorages with more than 12 strands	12 mm

For Dywidag bars, the slip allowed for in design shall not be less than 1.5 mm.

7.3.8 Post-Tensioning Tendons and Steel Duct Sizes

Standard steel duct sizes for strand post-tensioning systems are shown on Table 7.3.8. Duct diameters given are nominal and actual diameters can vary by $\pm 3\text{mm}$.

Table 7.3.8

No. of Tendons		Duct I.D./O.D. (mm)
13 mm strand	15 mm strand	
up to 7	up to 5	55/60
-	7	65/70
12	9	70/75
15	-	80/85
19	12	85/90
20	-	90/95
-	15	95/100
27	19	105/110
31	-	110/115
-	22	115/120
37	25	120/125
-	27	125/130
42	31	130/135

7.3.9 Post-Tensioning Anchorage Recesses

Anchorage recesses should be detailed based on the largest available anchorage devices, unless special circumstances dictate otherwise.

7.3.10 Post-Tensioning Tendon Duct and Web Details

Duct arrangements and corresponding reinforcement layout and web dimensions are shown in Figure 7.3.10. Dimensions are based on 110mm O.D. sheaths.

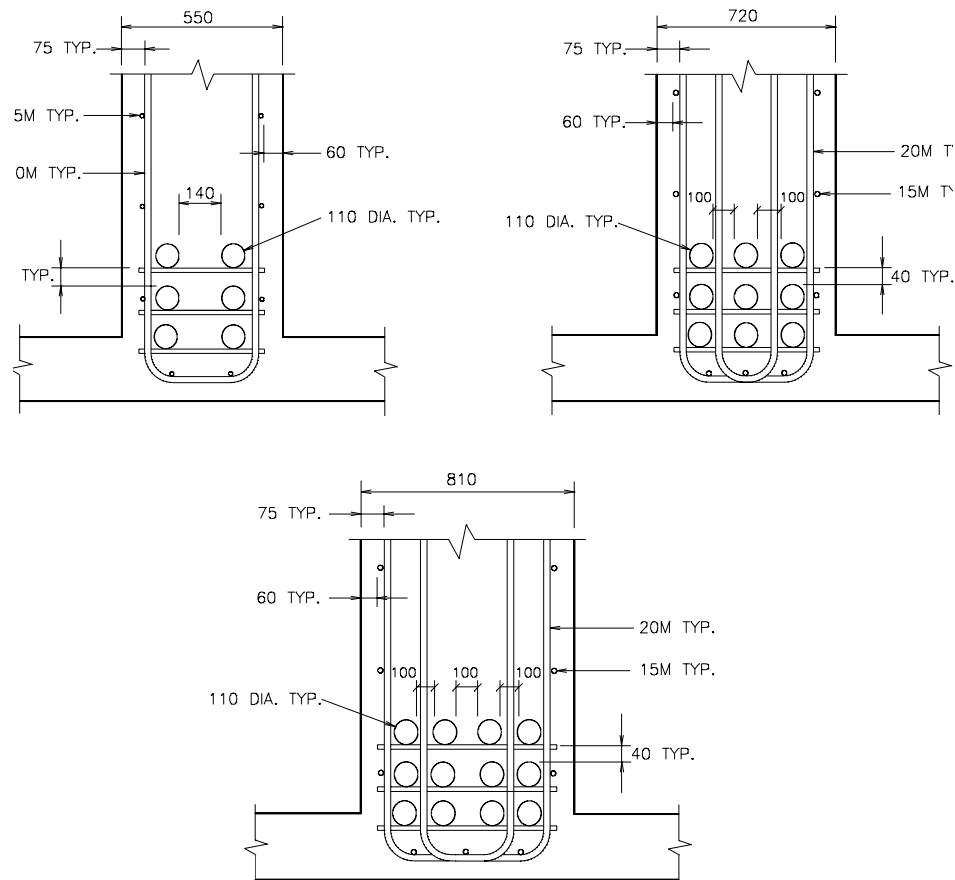


FIGURE 7.3.10(a) RECTANGULAR VOIDS

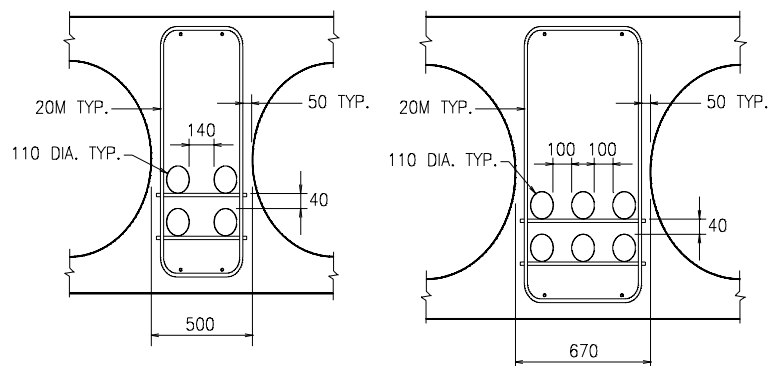


FIGURE 7.3.10(b) ROUND VOIDS

7.3.11 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 the tapered portion of the voided section must be shown on contract drawings (see Figure 7.3.11). 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.

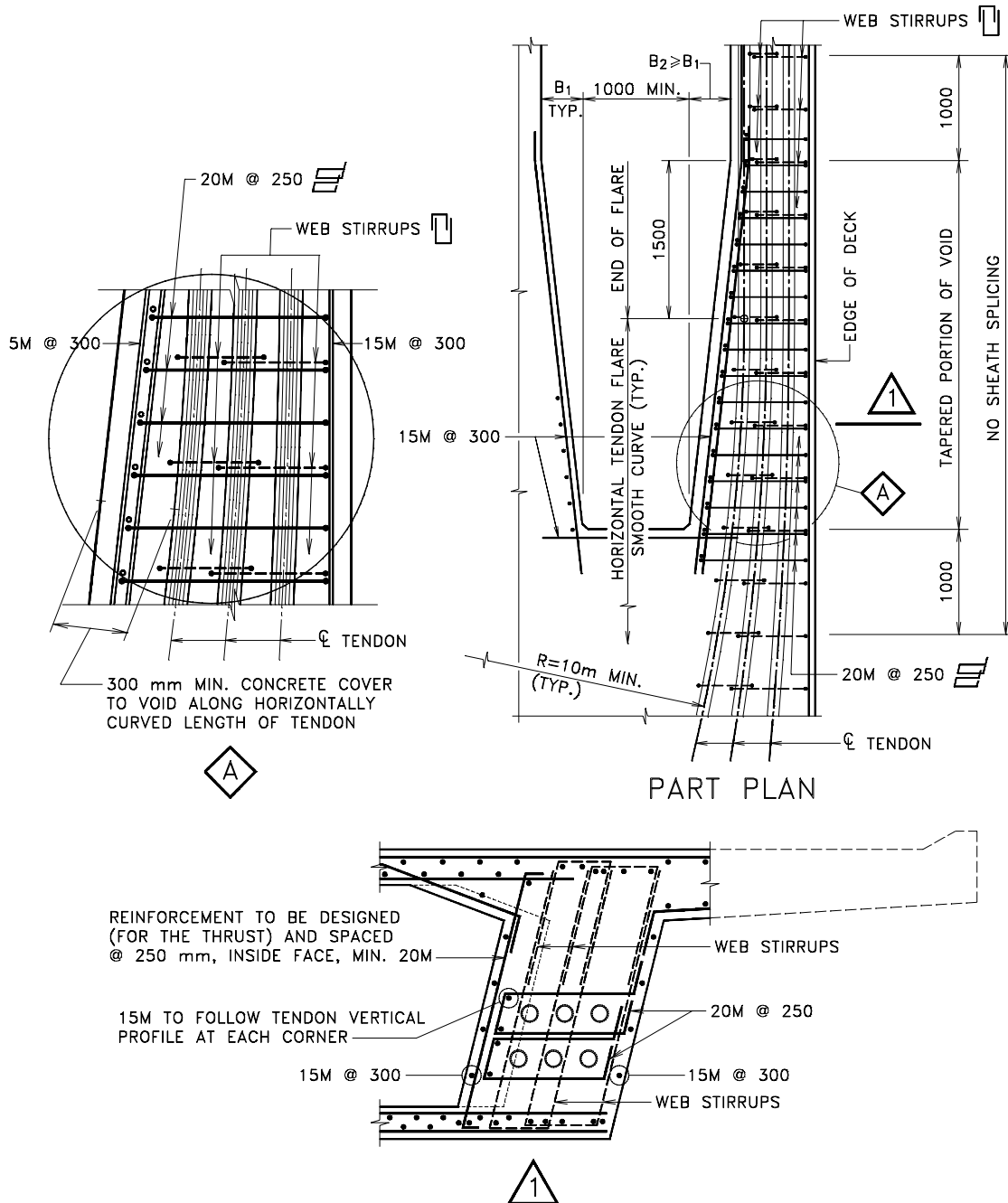


FIGURE 7.3.11 ADDITIONAL EXTERIOR WEB REINFORCING

7.3.12 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 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 NOTES**

1. ALL TENDON ANCHORAGES SHALL BE INTERNAL TYPE AS SHOWN ON THE DRAWINGS.
2. PRESTRESSING TENDONS SHALL BE B.B.R., V.S.L. OR DYWIDAG.
3. DUCT GROUTING VENTS SHALL BE PROVIDED AT HIGH POINTS AND AT BOTH ENDS OF ALL TENDONS.
4. MIN. CLEAR COVER TO TENDONS DUCTS:
TOP $\pm$ LONGIT.
..... $\pm$ TRANSV.
ELSEWHERE $\pm$
5. WELDING IS NOT PERMITTED WITHIN 3000 mm OF ANY TENDON OR TENDON DUCT .
6. ALL TENDONS SHALL BE STRESSED IN THE NUMERICAL ORDER SHOWN ON THE DRAWINGS.
7. CLASS OF CONCRETE: 35 MPa
8. CONCRETE STRENGTH BEFORE STRESSING MPa
9. ALL ANCHORAGE RECESSES SHALL BE ABRASIVE BLASTED AND COATED WITH AN APPROVED CEMENT PASTE PRIOR TO CASTING OF 35 MPa CONCRETE IN THE RECESSES AND TO BE COLOUR MATCHED WITH ADJACENT CONCRETE.
10. THE POST-TENSIONING SUPPLIER SHALL DETAIL AND SUPPLY LIVE-END SPIRAL REINFORCEMENT AND DEAD-END ANCHORAGES, INCLUDING THE METHOD OF SUPPORT AND ASSOCIATED REINFORCEMENT.

(b) **LONGITUDINAL TENDONS**

1. PRESTRESSING TENDONS SHALL BE TYPE WITH STRANDS.
2. PRESTRESSING STEEL SHALL BE LOW RELAXATION SEVEN WIRE STRAND, SIZE DESIGNATION, GRADE, AND SHALL MEET THE REQUIREMENTS OF CSA STANDARD G279-M1982.
3. MINIMUM BREAKING STRENGTH kN/ STRAND
4. SHEATHS FOR POST-TENSION DUCTS SHALL BE BRIGHT STEEL, RIGID, CORRUGATED TYPE, ... mm OD WITH A MINIMUM WALL THICKNESS OF 0.6mm OR EQUIVALENT APPROVED BY MTO.
5. REQUIRED ELONGATION AND THE TENDON 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 = ...MPa, K = ..., m = ...

STAGE	LOCATION	TENDON	TYPE	JACKED FROM	JACKING FORCE (kN)	ELONGATION (mm)	REQUIRED SLIP (mm)

6. 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.

(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).

(c) **TRANSVERSE TENDONS**

1. PRESTRESSING TENDONS SHALL BE TYPE AND WITH AND STRANDS RESPECTIVELY.
2. PRESTRESSING STEEL SHALL BE LOW RELAXATION SEVEN WIRE STRAND, SIZE DESIGNATION, GRADE, AND SHALL MEET THE REQUIREMENTS OF CSA STANDARD G279-M1982.
3. MINIMUM BREAKING STRENGTH kN/ STRAND
4. SHEATHS FOR POST-TENSION DUCTS SHALL BE BRIGHT STEEL, RIGID, CORRUGATED TYPE, ... mm OD WITH A MINIMUM WALL THICKNESS OF 0.6mm OR EQUIVALENT APPROVED BY MTO.
5. REQUIRED ELONGATION AND THE TENDON 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 = ...MPa, K = ..., m = ...

LOCATION	TENDON	TYPE	JACKED FROM	JACKING FORCE (kN)	ELONGATION (mm)	REQUIRED SLIP (mm)

6. 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.
7. FOR GENERAL NOTES, SEE DWG.

(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).

(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 METRES 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. PLACE CONCRETE AT TRANSVERSE ANCHORAGE RECESSES.
4. STRESS ALL LONGITUDINAL TENDONS AFTER CONCRETE IN TRANSVERSE ANCHORAGE RECESSES HAS REACHED A STRENGTH OF 30 MPa.

5. PLACE 35 MPa CONCRETE AT RECESSES FOR ALL LONGITUDINAL TENDONS.
6. GROUT ALL TRANSVERSE AND LONGITUDINAL TENDON DUCTS.
7. PLACE CONCRETE IN SIDEWALKS, MEDIAN, CURBS AND BARRIER WALLS.

(f) **SEQUENCE OF DECK CONSTRUCTION (on DECK DETAILS drawing, for trapezoidal-voided post-tensioned superstructure)**

1. PLACE CONCRETE IN BOTTOM SLAB (ACCESS HATCH IN POSITION).
2. PLACE REMAINING CONCRETE IN DECK EXCEPT AT (SIDEWALKS, MEDIAN, CURBS AND BARRIER WALLS) PRESTRESSING ANCHORAGE RECESSES AND EXPANSION JOINTS.
3. STRESS ALL TRANSVERSE TENDONS AT PIER(S) AND ABUTMENTS WHEN CONCRETE IN DECK HAS REACHED A STRENGTH OF 30 Mpa.
4. PLACE CONCRETE AT TRANSVERSE ANCHORAGE RECESSES.
5. STRESS ALL LONGITUDINAL TENDONS AFTER CONCRETE IN TRANSVERSE ANCHORAGE RECESSES HAS REACHED A STRENGTH OF 30 MPa.
6. PLACE 35 MPa CONCRETE AT RECESSES FOR ALL LONGITUDINAL TENDONS.
7. GROUT ALL TRANSVERSE AND LONGITUDINAL TENDON DUCTS.
8. PLACE CONCRETE IN SIDEWALKS, MEDIAN, CURBS AND BARRIER WALLS.

(The above notes apply to single-stage construction. If the deck is cast and stressed in multiple stages, the order of steps 6 and 7 should be reversed).

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8.1.1 Structural Steel

- (a) All structural steel used in highway structures shall be corrosion-resistant steel. Steel shall conform to CSA Standard CAN/CSA-G40.21-M92 Grade 350AT or Grade 350A. For less severe climatic conditions, Category 1 shall be specified for Fracture Critical and Primary Tension Members. Likewise, for structures located in more severe climatic zones, Category 2 shall be specified. Rolled sections shall conform to CSA Standard CAN/CSA-G40.21-M92 or ASTM Specification A588.

(NOTE: ASTM 588 may be substituted for G40.21 grade 350A steel, and when the Charpy impact energy requirements are verified by the submission of test documentation, ASTM A588 may be substituted for G40.21 grade 350AT steel.)

- (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 'AT' grade steel.
- (c) The contract drawings must show which components are to be Grade 350AT.
- (d) Steel trapezoidal box girders should be used in preference to I girders for crossings over highways because they are considered to be less likely to entrap salt and debris.
- (e) The availability of the required widths and thicknesses of plate should be confirmed before finalising the design. Available lengths are sometimes of importance in relation to the positioning of field splices.
- (f) Galvanized A490 bolts shall not be used.
- (g) A325M bolts, type 3, shall be used for all connections. Bolts shall be 22mm size, except that where large numbers are required, in restricted spaces, 25mm size may be used.

(NOTE: ASTM A490M TYPE 3 bolts may be substituted for ASTM A325M Type 3 bolts on Atmospheric Corrosion Resistant steel, but galvanized A490 Type 1 bolts on coated steel are not permitted because of delayed fracture due to hydrogen embrittlement.)

- (h) Huck Bolt fasteners are acceptable if proposed as an alternative to high strength bolts by the Contractor, but should not be shown as such on contract drawings.
- (i) Field welding is generally not permitted. Field splices are generally bolted connections.
- (j) Jacking points and loads must be shown.
- (k) Accurate determination of structural steel quantities is no longer required of the designer. An estimate of this quantity shall be provided to the Regional Planning and Design or Regional Structural Section for in-house checking and cost estimation purposes only.
- (l) Exterior girders, including girders adjacent to longitudinal median joints, which are not protected from splashing from the roadway above by a solid barrier, shall be checked for structural adequacy assuming a 2 mm loss of steel from all outer surfaces and the soffit. (See Fig. 8.1.1).

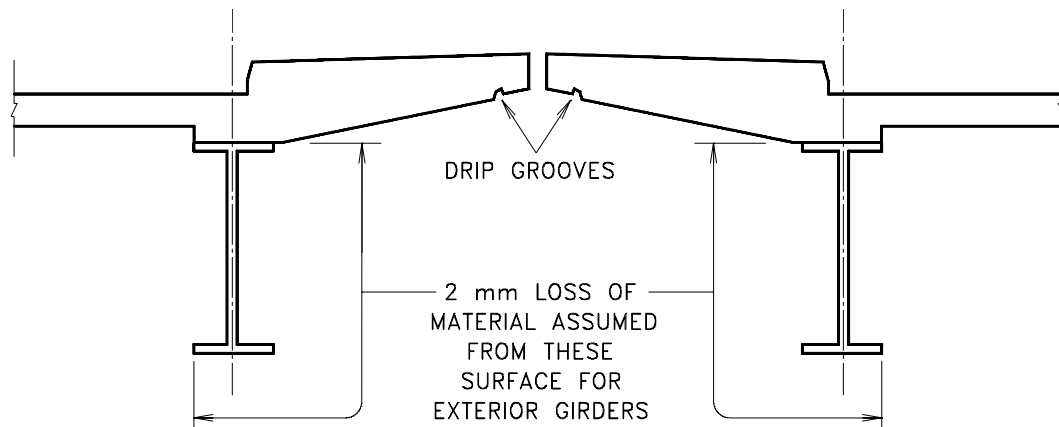


FIGURE 8.1.1 STEEL GIRDER AND SLAB DECK

8.1.2 Protection of Steel

An approved shop applied coating is required to protect structural steel at abutments and deck expansion joints from salt laden water run-off. The coatings should be applied as follows: except for integral and semi-integral abutment bridges, all structural steel surfaces including diaphragms and bracing, but excluding surfaces in contact with concrete and the contact surfaces of bolted joints shall be coated for a distance of 3000 mm from the ends of the girders.

For integral abutment bridges all structural steel surfaces 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 8.8.1).

For semi- integral bridges 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 finish coat shall be indicated on the drawings (see 8.8.1.)

Lapped surfaces of sign and light posts are susceptible to crevice corrosion. Contact surfaces at lap joints shall be coated.

8.2.1 Standardised Nomenclature for Structural Steel Shapes

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

<u>SHAPE DESIGNATION</u>	<u>STRUCTURAL SECTION</u>	<u>EXAMPLE</u>
C	Standard Channels	C230x30
HP	H Pile Sections	HP310x79
HSS	Hollow Structural Sections	HSS101.6x50.8x3.18
L	Angles (Give nominal leg x leg x thickness)	L200x100x13
M	Miscellaneous Shapes M250x34.1	
MC	Miscellaneous Channels	MC250x37.7

MT	Structural Tees. Cut from M Shapes	MT100x24.3
S	American Standard Beams	S460x104
ST	Structural Tees. Cut from S. Shapes	ST 150x37.3
T	Tees	T100x16.7
W	Wide Flange. Give nominal depth x mass per unit length	W410x67
WT	Structural Tees. Cut from W shapes	WT305x108.5
WWF	Welded Wide Flange	WWF700x141
WWT	Structural Tees. Cut from WWF Shapes	WWT250x325.5
Z	Zeds	Z130x17.3

Standard Designation for Steel Plate, e.g. 13 x 510 x 3050, Thickness x Width x Length. Give all dimensions in mm.

8.3.1 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 be provided throughout the cross section 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 is required, particularly when the deck is being cast. Tack welding the reinforcing steel to the studs may not in itself be sufficient. The stability of the girders must be ensured as an integral part of the falsework design.

8.3.2 Structural Steel Box Girders - Bottom Flanges

The weld joining the bottom flange and the web should be detailed so as to minimise any projection of the flange on which salt could collect. Pier and abutment diaphragms shall be coped to clear continuous welding of the web-to-flange joint. At abutments, the copes shall be sealed with weld (see Figure 8.3.2)

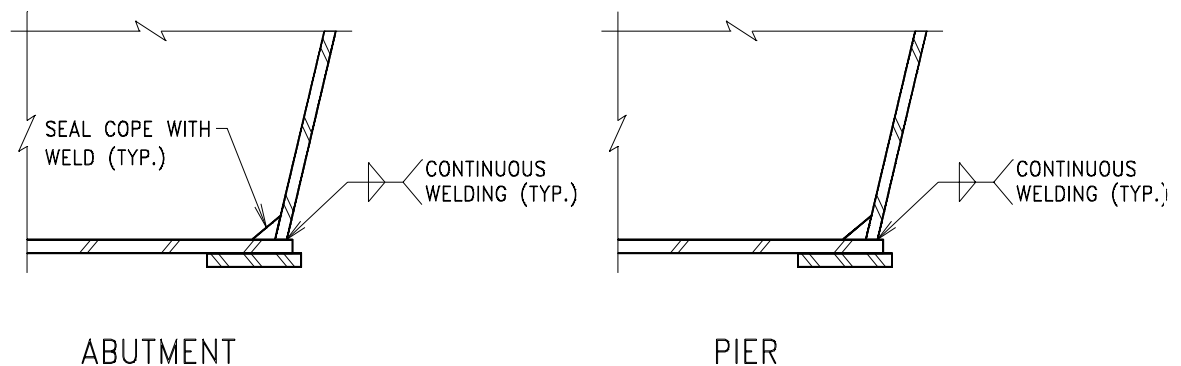


FIGURE 8.3.2 BOTTOM FLANGE WELDING DETAILS

8.3.3 Structural Steel Girders - Camber

Camber is defined as the deviation from a straight line of the vertical profile of a flexural member. 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 20th points of the span.

According to Section 10.7.4 of 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).

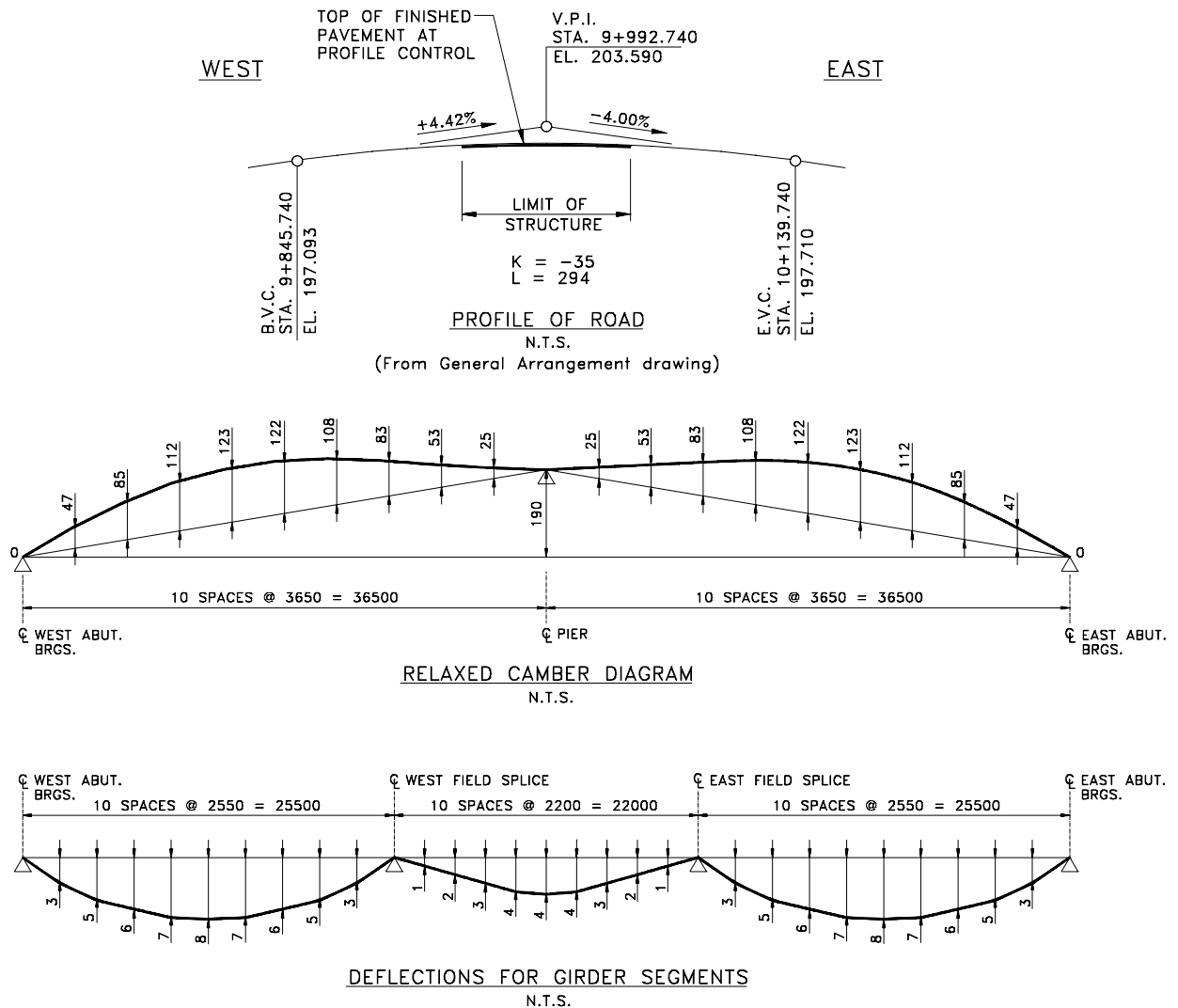
8.3.4 Structural Steel Girder Elevations

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 pouring 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.

8.3.5 Example: 2-span steel box girder

These sketches illustrate the various diagrams required for steel box girders, to be shown on The Structural Steel contract drawings.



GIRDER ERECTION ELEVATIONS						
GIRDER	CL W. ABUT. BRGS.	CL WEST SPAN	CL W. FIELD SPLICE	CL PIER	CL E. FIELD SPLICE	CL EAST SPAN
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 CL 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.

8.3.6 Box Girder Drainage & 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.

Adequate ventilation of the interior of box girders must be ensured to allow a draft.

8.3.7 Box Girder Voids and Access Locations

CHBDC Clause 1.9.3.1.5 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 1 200 mm. The minimum clearance for walking through intermediate cross bracing and diaphragms inside box girders shall be preferably 600 mm and not less than 500 mm.

The number of access openings and their locations shall be as follows:

- a) For simple span steel box girders with walk-through cross bracing: Two openings per box girder, one located near each end.
- b) For continuous steel box girders with walk-through cross bracing and pier diaphragms: Two openings per box girder, one located near each end. For exceptionally long girders, the need for additional intermediate openings should be determined by the Regional Structural Sections, 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.

This policy shall apply to all current and future designs, and wherever possible, to stockpiled designs.

8.4.1 Coping of Stiffeners and Gusset Plates

Copes on details such as stiffeners shall be 50 mm horizontally by the greater of 50 mm and 6 times the girder web thickness vertically. Copes on details such as gusset plates shall be the greater of 50 mm x 50 mm and 6 times the girder web thickness by 6 times the girder web thickness. See Figure 8.4.1.

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.

At end diaphragms, copes are not permitted. This generally dictates a drain at the diaphragm.

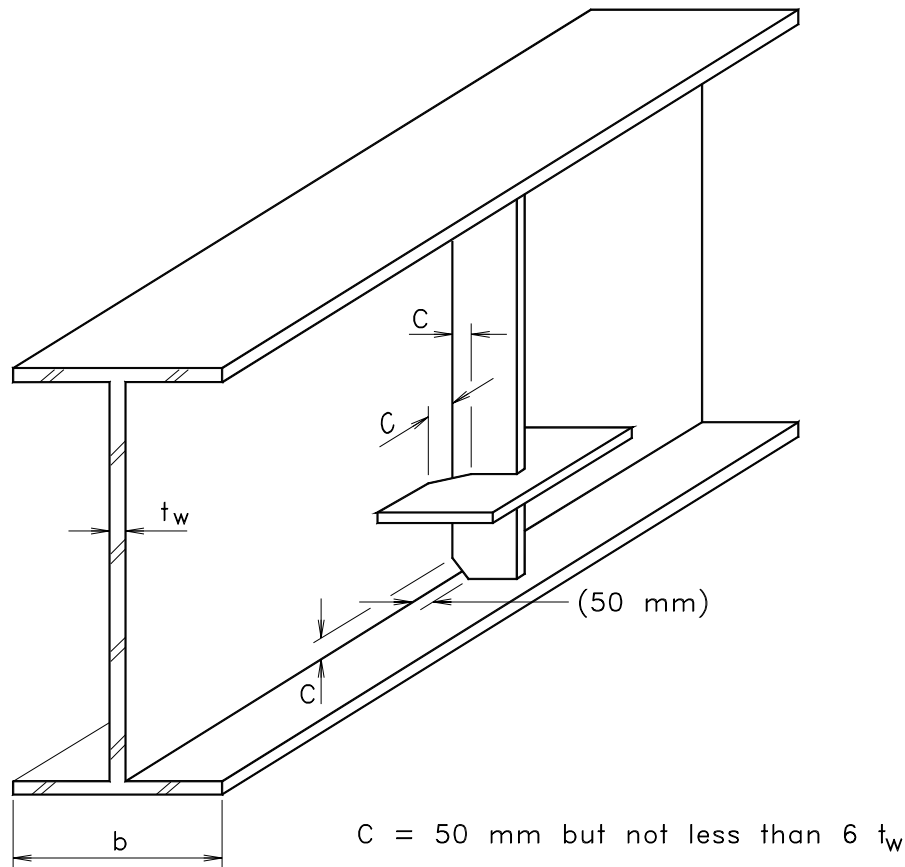


FIGURE 8.4.1 COPING OF STIFFENERS AND LOCATION OF GUSSET PLATES

8.4.2 Gusset Plates for Lateral Bracing

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 and "Bridge Fatigue Guide, Design and Details" by J. W. Fisher should be consulted when determining the location of bolt holes. See also Figure 8.4.1.

Two factors have been 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 improved fatigue performance which results when the gusset plate is moved away from the flange into a lower stress region.

Although this is the preferred detail, under certain circumstances a designer may wish to consider a radiused gusset plate or a bolted connection.

8.4.3 Frames for Intermediate Bracing

Frames should be used for intermediate bracing in lieu of angle sections shipped loose to the site. The frames for use between girders should be designed for shipping and erection as a single unit. 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, using rectangular gusset plates. Oversized holes in the gusset plates are permitted.

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

8.4.4 Box Girder Bracing

Unless design requirements dictate otherwise, 100 x 100 x 10 mm angles should be used as a standard angle size for box girder bracing. If additional interior bracing is required for handling of the girders, in excess of what the contract drawings call for, the fabricator shall show this on the shop drawings which shall be subject to approval by the Ministry. The designer should ensure that the interior bracing can be welded to the web stiffeners (see Figure 8.4.4); and if design permits in the case of X-bracing, the intersection of the two bracing elements need not be connected.

Because of minimum tonnage orders that can be placed with mills, standardisation of angle bracing will result in economy. The size chosen is believed to be adequate for the normal range of bridge spans built in Ontario. Additional economies can be achieved when connection details are such that fabrication can be performed more easily.

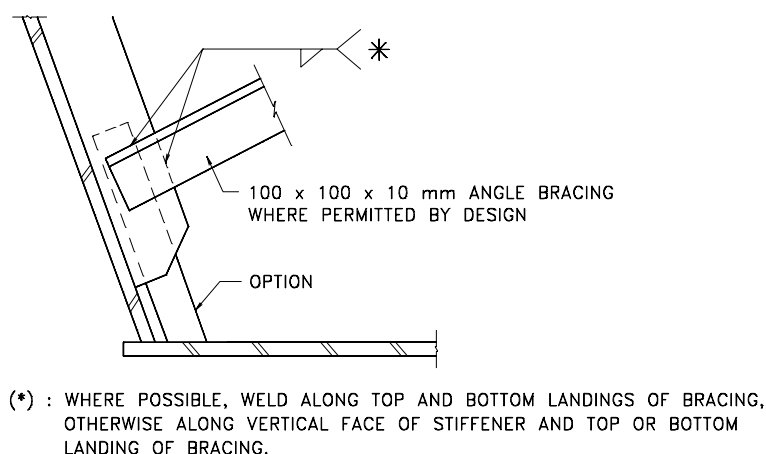


FIGURE 8.4.4 BOX GIRDER BRACING

8.4.5 Intermediate Diaphragms in Shallow Girders

Constant depth intermediate diaphragms, in lieu of frame bracing, are preferred in I-girders bridges up to approximately 1 200 mm in girder depth.

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

8.4.6 Box Girder Diaphragms at Piers and Abutments

Diaphragms at piers should be detailed so that the box girder and diaphragm flanges are not connected (see Figure 8.4.6-a). Two possible solutions are shown. Also, provisions for jacking within the width of the bottom flange should be provided for by design. Diaphragms at abutments are normally of a shallower depth to allow for deck details. In this case, the box girder flanges should be stabilised against rotation (see Figure 8.4.6-b). Diaphragms between box girders at piers and abutments should be of constant depth, and bolted to exterior box girder web stiffeners (see Figure 8.4.6-c). Oversized holes in diaphragms or stiffeners are permitted.

The details as shown in Figure 8.4.6, were developed to meet design and fabrication needs.

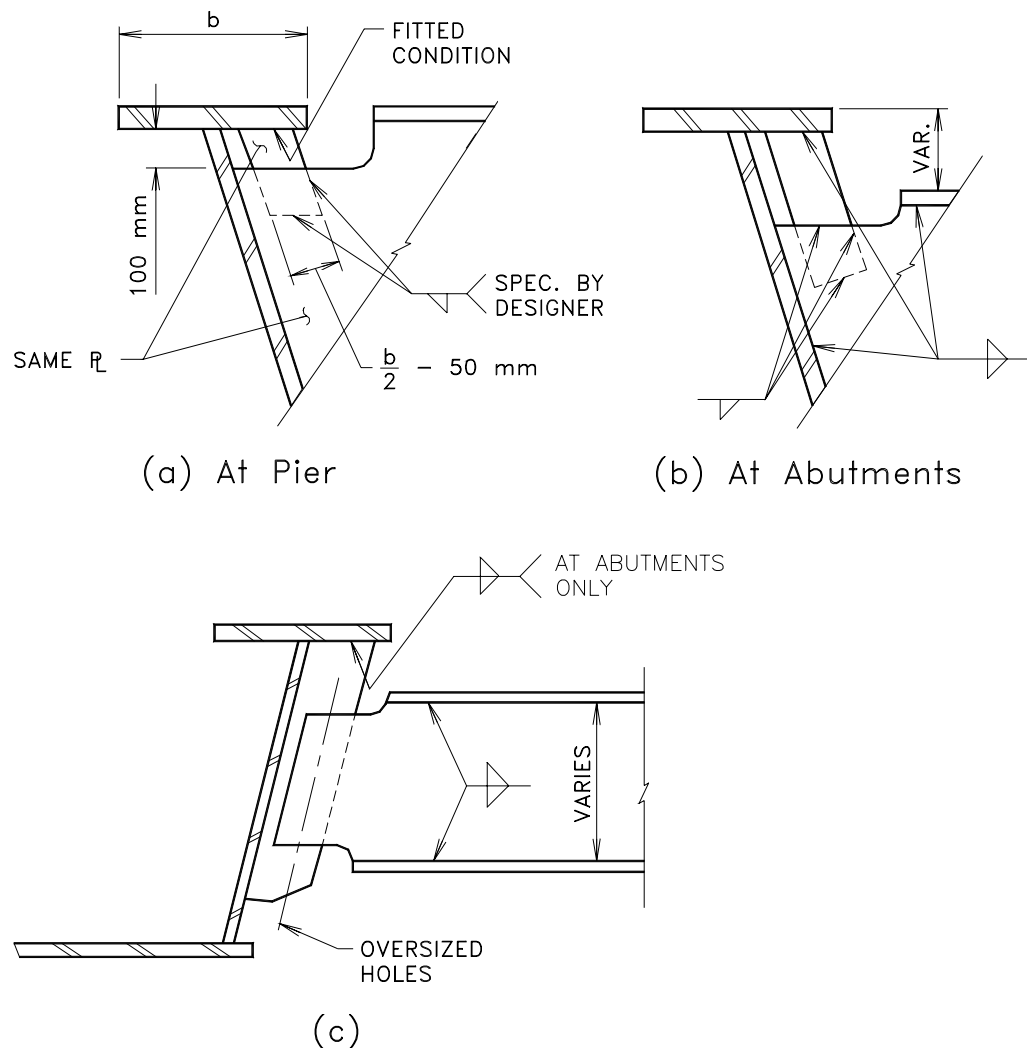


FIGURE 8.4.6 BOX GIRDER DIAPHRAGMS

8.5.1 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.

8.5.2 Transition of Flange Thicknesses at Butt Welds

Transition of flange thickness at butt welds should be made in accordance with CSA Standard W59-1984 with a slope through the transition zone not greater than 1 in 22.

A slope of 1 in 22 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.

8.5.3 Transitions of Box Girder Flange and Web Thicknesses

Flange thickness transitions should be made so that a constant depth web plate is maintained. Web thickness transitions should be made to maintain a flush outer box girder face.

Flange thickness transitions, made so that a constant web depth is maintained, result in economy. Web thickness transitions made so that a flush outer face is maintained, facilitate web splice details. In the event that 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.

8.6.1 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 RT or 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 overgrinding and repair.

In webs of girders, butt welds more than $\frac{1}{3}$ 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.

8.6.2 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 downhand welding.

For box girders, unless design requirements dictate otherwise, 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 so as to allow longitudinal downhand welds 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 8.6.2). Sufficient access should be provided to execute these transverse welds.

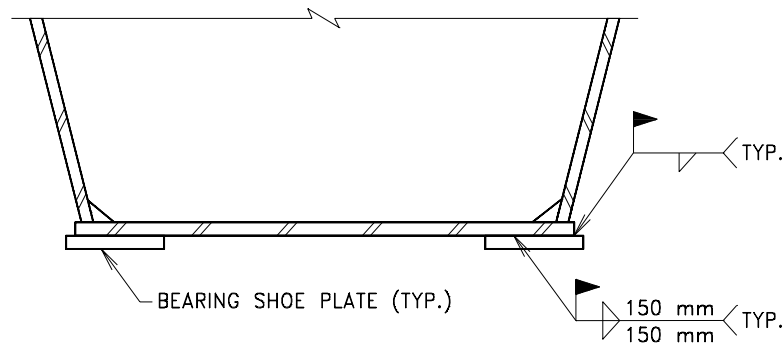


FIGURE 8.6.2 BEARINGS AT PIERS & ABUTMENTS

8.6.3 Bolt Grid

Bolt holes should be set on a 80 mm x 80 mm grid, or multiples of 80 mm, to facilitate the use of multi-spindle drills. See (Figure 8.6.3).

The use of a standard grid pattern simplifies fabrication and results in economies.

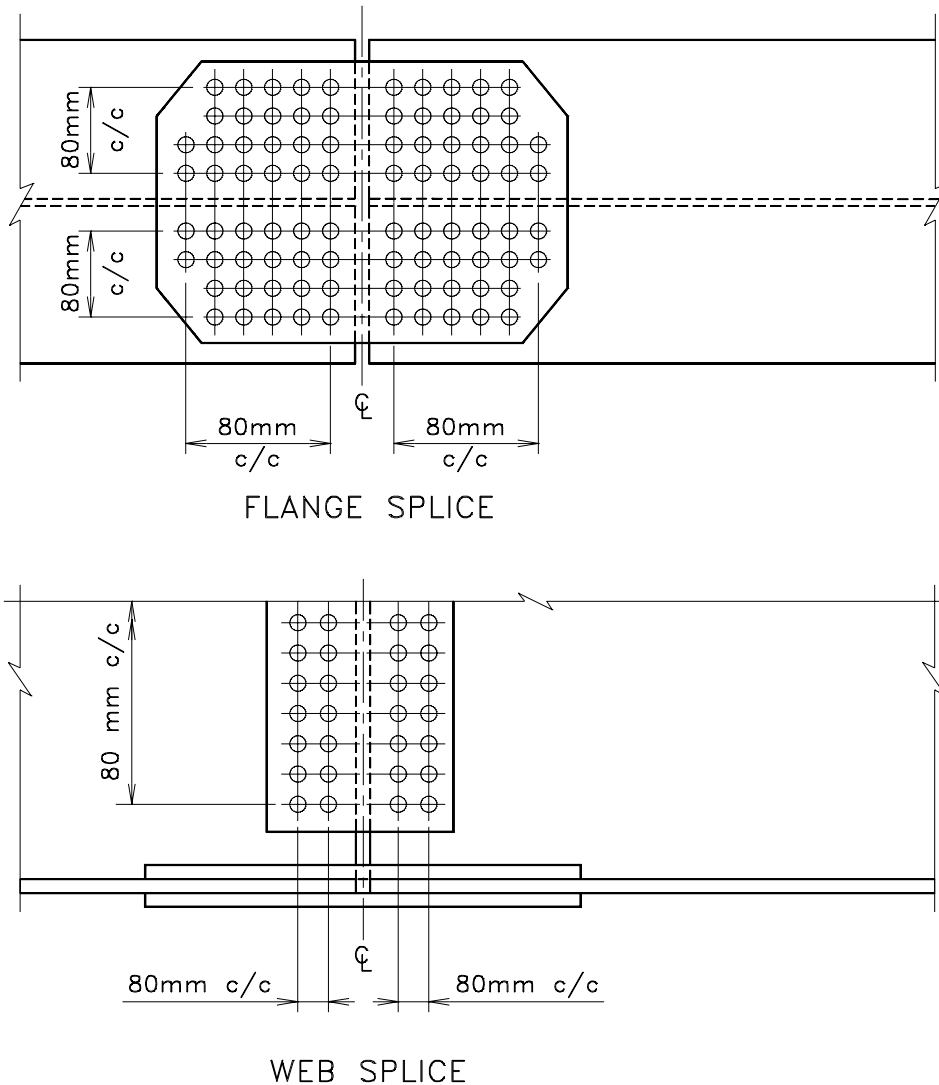


FIGURE 8.6.3 - POSSIBLE BOLT LOCATIONS IN A FIELD SPLICE

8.7.1 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 2.5.6(f)). 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.

8.7.2 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 8.7.2. 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.

CSA W59 describes the contact tolerance in the definitions of “fitted” and “fitted to bear”.

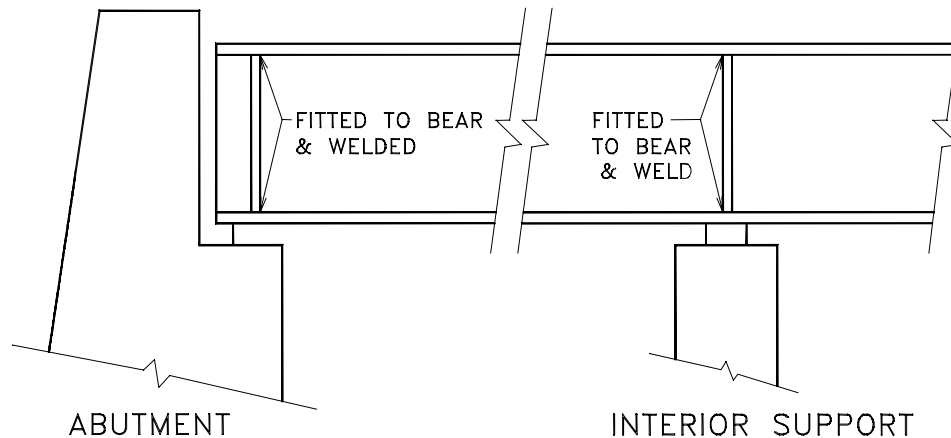


Figure 8.7.2 - BEARING STIFFENER TO FLANGE CONNECTION

8.7.3 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.

8.7.4 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 $0.0001 hF_y^{1.5}$ 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.

8.7.5 Intersecting Longitudinal and Transverse Stiffeners

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

longitudinal stiffener should be cut short of the transverse stiffener. However, in tension regions, where fatigue is a governing design criterion, and where longitudinal and transverse stiffeners intersect, the longitudinal stiffener may be made continuous and the transverse stiffener welded to it at the intersection.

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. 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.

8.7.6 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 8.7.6 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 8.7.6. 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 connection of bracing to the outer faces of box girders shall be as shown in Figure 8.7.6.

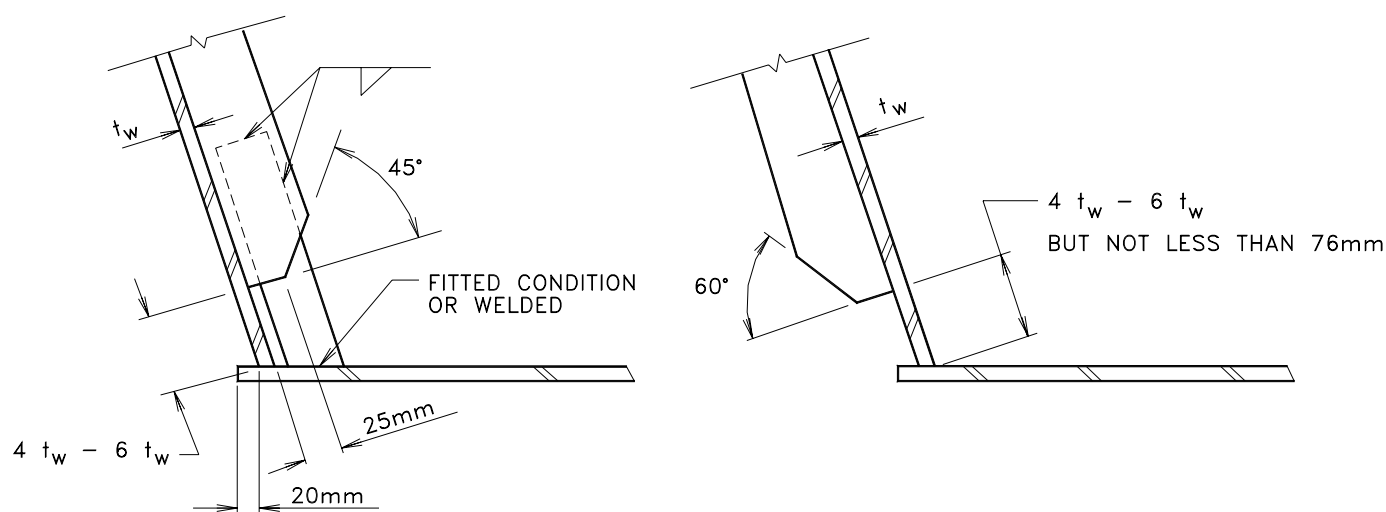


FIGURE 8.7.6 BOX GIRDER WEB STIFFENERS

8.7.7 Bottom Flange Stiffener Details

Wide flange "I" or "T" section longitudinal bottom flange stiffeners are preferred in lieu of plate sections. The sections should be spaced a minimum of 305 mm between flanges to allow the use of automatic welding equipment. Channel sections, welded to the top of the wide flange longitudinal stiffeners, and to the inner web stiffeners, are preferred as transverse bottom flange stiffeners (see Figure 8.7.7).

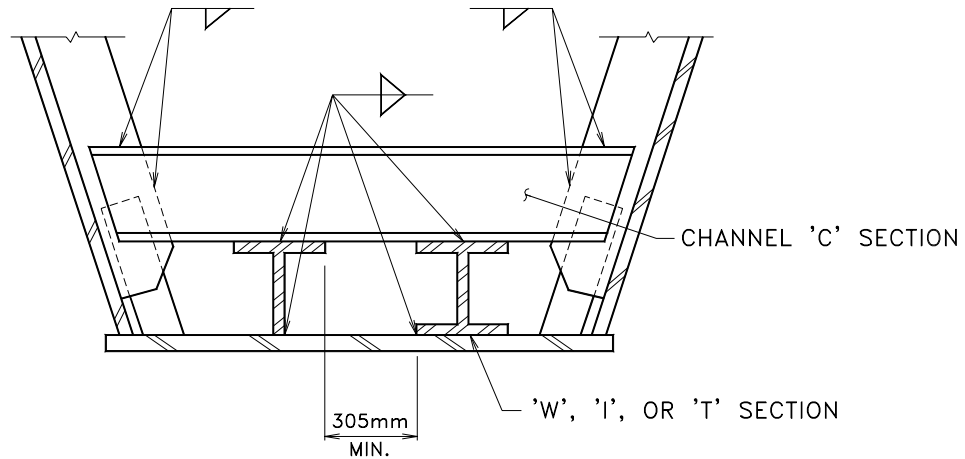


FIGURE 8.7.7 BOTTOM FLANGE STIFFENER DETAILS

8.8.1 Structural Steel Notes

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

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

(a) GENERAL NOTES

1. ALL STRUCTURAL STEEL SHALL CONFORM TO CSA STANDARD CAN/CSA-G40.21-M92. STEEL MARKED 'AT' SHALL BE GRADE 350AT CATEGORY (...), ALL OTHER STRUCTURAL STEEL SHALL BE GRADE 350A. (Category 1 or category 2, to be determined by designer and inserted, see 8.1.1). ROLLED SECTIONS SHALL CONFORM TO CSA STANDARD CAN/CSA-G40.21-M92 or ASTM SPECIFICATION A588.
2. MEMBERS OR COMPONENTS OF MEMBERS FOR WHICH 'AT' STEEL IS SPECIFIED ARE PRIMARY TENSION MEMBERS.
3. BOLTS ON ATMOSPHERIC CORROSION RESISTANT STEEL SHALL BE ASTM A325 TYPE 3, M22. BOLTS ON COATED STEEL SHALL BE GALVANIZED ASTM A325M TYPE 1, M22. BOLT THREADS SHALL BE EXCLUDED FROM THE SHEAR PLANES.
(NOTE: ASTM A490M TYPE 3 bolts may be substituted for ASTM A325M Type 3 bolts on Atmospheric Corrosion Resistant steel, but galvanized A490 Type 1 bolts on coated steel are not permitted because of delayed fracture due to hydrogen embrittlement.)
4. STUD SHEAR CONNECTORS SHALL BE 22 mm DIA., AND CONFORM TO ASTM STANDARD A108 AND CSA W59.
5. ALL LENGTHS SHOWN ARE IN THE HORIZONTAL PLANE AND MEASURED AT 20°C.
6. GIRDERS SHALL BE CAMBERED TO VALUES SHOWN IN THE RELAXED CAMBER DIAGRAM.

7. 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.

8. THE ENDS OF GIRDERS, AND BEARING STIFFENERS SHALL BE TRULY VERTICAL UNDER FULL DEAD LOAD.
9. ALL BUTT WELDS IN FLANGE AND WEB SHOP SPLICES SHALL BE FINISHED FLUSH OR SMOOTH AS INDICATED, BY GRINDING WHERE NECESSARY IN THE DIRECTION OF APPLIED STRESSES. IF RELOCATION OR ADDITIONAL SHOP SPLICES ARE REQUIRED, THEIR LOCATION SHALL BE APPROVED BY THE ENGINEER. (The weld finish to be indicated on the drawing using the appropriate standard symbol (See 8.6.1).
10. UNLESS OTHERWISE NOTED THE MINIMUM FILLET WELD SHALL BE AS FOLLOWS:

<u>MATERIAL THICKNESS OF THICKER PART JOINED</u> (mm)	<u>MINIMUM SIZE OF SINGLE PASS FILLET WELD</u> (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

11. UNLESS OTHERWISE NOTED, LONGITUDINAL WEB STIFFENERS SHALL BE CUT 25 mm SHORT OF THE TRANSVERSE WEB STIFFENERS.
12. BOLT HEADS IN FIELD SPLICES FOR BOX GIRDERS SHALL BE LOCATED ON THE EXTERIOR SURFACES.
13. ALL STRUCTURAL STEEL SURFACES, INCLUDING DIAPHRAGMS AND BRACING, BUT EXCLUDING SURFACES IN CONTACT WITH CONCRETE AND THE CONTACT SURFACES OF BOLTED JOINTS SHALL BE COATED FOR A DISTANCE OF 3000 mm FROM THE ENDS OF GIRDERS AT EXPANSION JOINTS. THE COLOUR OF THE TOPCOAT SHALL BE

(The colour of the topcoat or finishing coat to be inserted by the designer as follows:

504-217 brown (1-GP-12C) (for atmospheric corrosion resistant (ACR) steel girder bridges.)

or

501-107 grey (1-GP-12C) (for all other steel including any ACR steel used in the rehabilitation of carbon steel structures.))

14. Note: 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 13.)

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 600mm 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 13.)

15. THE 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. Note: 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.

(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 SLAB SHALL BE RETARDED USING A TYPE "R_x" RETARDER FOR THE DURATION OF THE POUR.
4. CONCRETE SHALL REMAIN PLASTIC IN POURING OF SEGMENTS WITH THE SAME SEQUENCE NUMBER.
5. MINIMUM CONCRETE STRENGTH OF PREVIOUS DECK POUR SHALL BE 20 MPa BEFORE PROCEEDING WITH THE POUR.
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).

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9 DECK AND CURBS

9.1.1 Bridge Deck Waterproofing

Bridge decks detailed with an asphalt wearing surface must also be shown as being waterproofed. The drawings should state only "Asphalt and Waterproofing System, 90 mm total." (see 2.5.7).

9.1.2 Future Wearing Surface

All structures detailed with a concrete wearing surface shall be designed for 90 mm future asphalt wearing surface. To allow for wear, the cover to the reinforcing steel from the top surface shall be increased by 10 mm.

The heights of curbs and barrier walls should not be increased to accommodate the future wearing surface.

9.2.1 Structure Deck Drainage

Structures having not more than two lanes draining to one side, built with normal crossfall to a more or less symmetrical vertical curve and not more than 120 m long, normally do not require deck drainage inlets.

Requirements for deck drainage inlets must always be established by Regional Planning and Design staff as part of the overall drainage design of the crossing. The size and number of drains shall be such that during a 10 year design storm, the swale or gutter flow will not encroach more than 1.50 m onto the traffic lane. Bridge deck drains shall be provided only where this criterion is exceeded.

For bridges on grades or sag curves, the roadway surface runoff shall be intercepted by catchbasins or other means located on the approaches to prevent flow into the expansion joints or onto the bridge deck.

When bridge deck drainage inlets are required, they shall be used with an air drop discharge. Water may not be discharged onto railway property, pavements, sidewalks, unprotected embankment slopes or waterways if environmental concerns prevail. When water is discharged onto other surfaces, where stability or appearance are a consideration, provision shall be made to prevent scour. The position and length of the discharge pipes shall be such that water falling at an angle of 45° to the vertical does not touch any part of the structure. Discharge pipes should project 400 mm below the bottom flange of adjacent girders to prevent splash. Consideration, however, must also be given to minimum vertical clearance requirements and aesthetics. Pipes need not be attached to adjacent girders if overall length is less than 2.5 m.

200 mm diameter deck drainage inlets do not collect a significant quantity of water and should be used only to prevent local ponding. This is sometimes necessary when flat grades are unavoidable or structures are subject to substantial permanent deflections that cannot be accurately predicted.

Catch basins are normally necessary just beyond the structure limits to intercept runoff from bridge decks. A continuous length of curb or gutter is provided to connect the bridge curb or barrier to the catch basin to prevent wash-outs around the ends of wing walls. Detailing of the wing wall or approach slab should be such as to permit straight vertical junctions with approach curbs. The final grading drawings should be reviewed in conjunction with the structure drawings to ensure that this or an equally acceptable arrangement has been adopted. This should be done by the Regional Structural Section.

9.2.2 Drip Grooves

Continuous drip grooves are required along the soffit on both sides of all concrete decks. They should be provided on each side of the joint between abutting twin bridges, even if the joint is sealed.

The dimensions are shown in Figure 9.2.2.

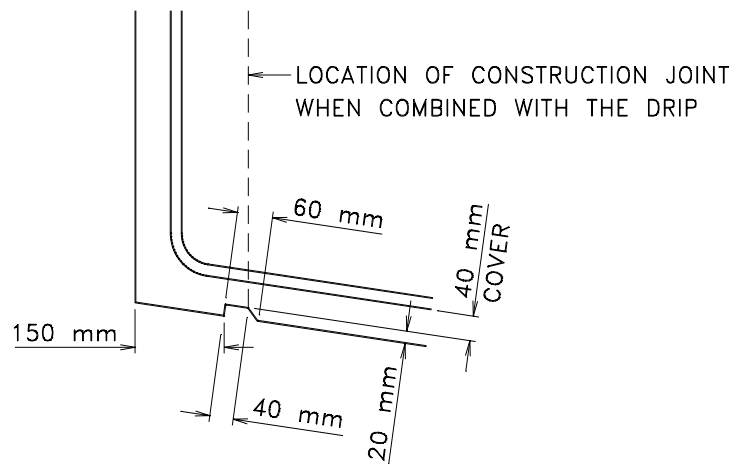


Figure 9.2.2 DRIP GROOVE

For practical and aesthetic reasons it may be desirable to combine the drip with construction joints that result from transverse prestress anchorage recesses, or decks with curbs or sidewalks. For this case the drip should not be located greater than 250 mm from the fascia (see OPSD-3904.040).

9.3.1 Concrete in Deck Slabs

In order to ensure adequate durability, the concrete class specified for decks shall be a minimum of 30 MPa.

9.3.2 Minimum Thickness of Deck Slabs

In order to permit placing four layers of reinforcing steel with the required cover and associated tolerances, the minimum thickness of deck slabs shall be 225 mm. Such slabs should be detailed so that the bars in the two layers of distribution (longitudinal) reinforcement are not vertically in line with one another. If possible, size 15 bars shall be used except for continuity steel over piers or cantilever steel, for which size 20 or 25 bars may be used.

9.3.3 Sequence of Deck Pours for Beam and Slab Bridges

No deck placing sequence should be specified, unless there are specific reasons, such as continuous multispan beams with short end spans that could lift off the abutment bearings when concrete is placed in the adjacent span.

When necessary, the deck placing sequence should be shown on the drawings. Each deck pour should be as large as practical, taking into account structure considerations and the availability of concrete. The following note concerning strength of the previous pour before allowing the next pour should be given on the deck drawing:

CONCRETE STRENGTH OF PREVIOUS DECK POUR SHALL BE AT LEAST 20 MPa BEFORE PROCEEDING WITH THE NEXT POUR.

CONCRETE IN THE DECK SLAB SHALL BE RETARDED USING A TYPE R_x RETARDER FOR THE DURATION OF THE POUR.

The deck placing sequence should be shown in numerical order with no duplication of the numerals.

Note: Simultaneous concrete placements should not be specified unless absolutely necessary, in which case the intent should be clarified on the deck slab drawing.

9.3.4 Reinforcing Steel below Barrier Walls

Thin slab cantilever decks tend to develop transverse cracks at barrier wall construction joint locations.

In order to control the cracks, the top and bottom longitudinal reinforcement in cantilever slabs under barrier walls, extending 500 mm from the face of the barrier wall or to the outside girder flange, whichever is less, shall not be less than 15M bars at 200 mm centres.

Thin slab cantilever decks with sidewalk do not require this additional crack-control reinforcement.

9.4.1 Screed Elevations on Bridge Decks, Check Elevations on Girders

a. Pre-cast Prestressed Concrete Girders and Steel Girders

Screed elevations shall be given at the centreline of all exterior girders, the break points in the deck, and on the deck at the faces of curbs and barrier walls. Screed elevations shall be given at intervals not exceeding 3 m.

It is important to ensure that the girders do not penetrate more than 25 mm into the nominal thickness of the deck slab. This may happen if an insufficient allowance has been made at mid-span for the upward prestress camber of precast girders and also if the deck has a sag vertical curve. As a general rule it is necessary to provide haunches raising the bottom of the deck above the top of precast beams, with heights at the supports as follows:

CPCI 900	40 mm
CPCI 1200	50 mm
CPCI 1400 or larger	75 mm
Spaced box beams	50 mm

Screed elevations should include an allowance for long term dead load deflection.

b. Post-Tensioned Concrete Decks

Screed elevations shall be given at break points in the deck, and on the deck at the face of curbs or barrier walls. The screed elevations shall be given at intervals not exceeding 3 m. No allowance is required normally for dead load deflections provided that prestress and dead load are more or less balanced.

9.4.2 As-Constructed Elevations

Steel pins used to record as-constructed elevations of bridges shall be installed when called for in the Structural Design Report. The decision to install steel pins shall be made by the Head of the Structural Section in consultation with the Design Section Head of the Bridge Office. Standard drawing SS116-40 shall be included in the contract when steel pins are used.

Steel pins shall only be required where long term settlement can be expected due to specific site conditions and on superstructures that are sensitive to long-term creep effect, such as bridges of segmental construction, or cast-in-place post-tensioned bridges of staged-construction.

9.5.1 Steel Forms

Steel forms are considered to have many disadvantages and are therefore not to be used. They are or may become unsightly. They may cause maintenance problems and at the very least impede inspection. It does not appear that there is any economic advantage to their use.

9.6.1 Deck Construction Joints

Construction joints across which force effects are assumed to be transmitted or which must resist leakage, require special treatment. This should be specified in an appropriate and current Special Provision.

It should be noted that waterstops in deck construction joints are no longer used.

9.6.2 Waterproofing of Deck Cantilever of Steel Box-Girder Bridges

For steel box-girder bridges, the curb/sidewalk face should preferably start beyond the exterior flange of the exterior girder. If this is not practical, a special waterproofing system should be used as described below, to prevent salt-laden deck run-off from invading the joint between the slab and the sidewalk, filtering through the deck and into the unprotected steel box-girder, causing serious internal corrosion. This damaging potential is intensified on wide-deck bridges, and especially in the vicinity of piers, where the concrete decks may have numerous flexural cracks in the negative moment region.

A proprietary product (not on the Designated Sources List) such as "Eliminator" is an appropriate waterproofing system that has been used by Metropolitan Toronto in the recent

past on the Q.E.W. bridge over the Humber River. It is an acrylic-based system manufactured by Sterling Lloyd Products Inc. The supplier and applicator for this system is LAFRENTZ, located in Oakville, Ontario. This waterproofing should start at 150 mm in front of the face of the sidewalk, continue along the top of the deck/sidewalk construction joint and be turned downwards along the vertical joint face under the barrier wall. The waterproofing system should extend the full length of the bridge.

Although more expensive than the standard bridge deck waterproofing system (hot applied asphalt membrane with protection board), the acrylic based system has several advantages in comparison, such as easier application (spray, brush or squeegee), faster curing time, and harder finished product.

9.7.1 Deck Fascia Details

Fascia treatment is categorised by deck type and the presence or absence of a sidewalk, as shown in Figure 9.7.1. Slab-on-girder decks with a sidewalk have a 500 mm high fascia, while those decks without a sidewalk have a 350 mm high fascia. Cast-in-place post-tensioned decks have fasciae from 450 to 600 mm high.

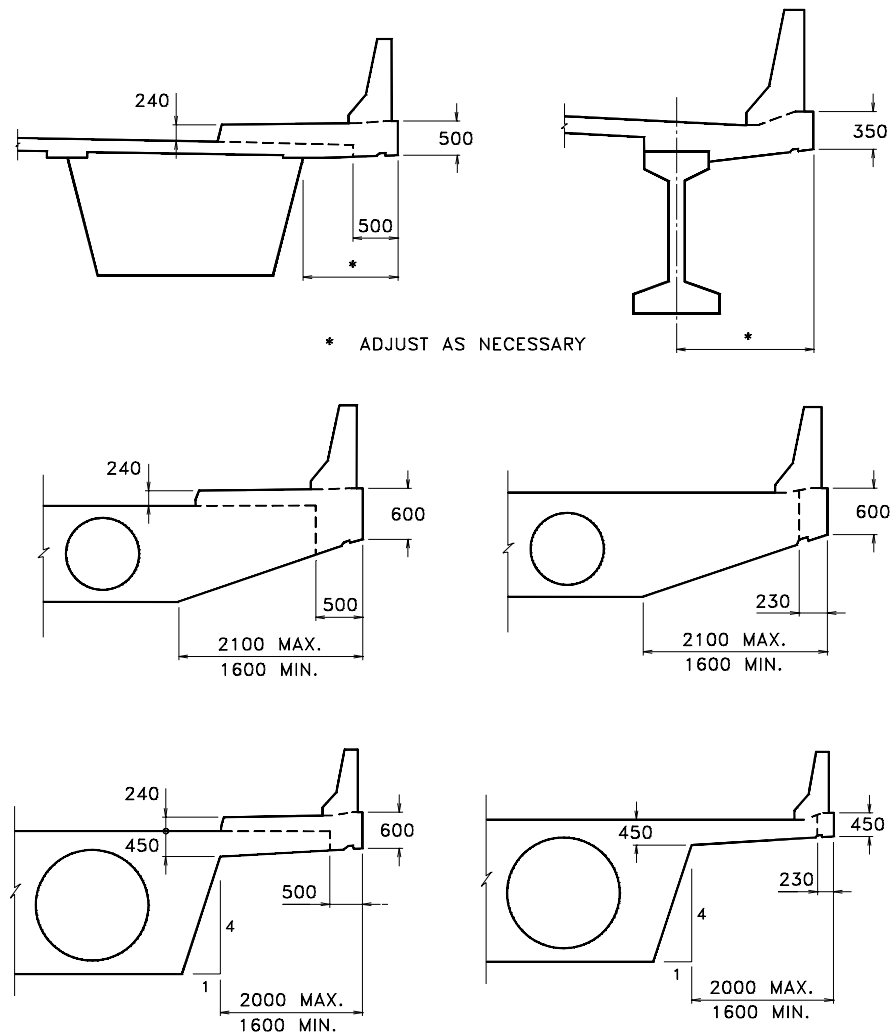


Figure 9.7.1 DECK FASCIA DETAILS

9.8.1 Additional Reinforcement in Skewed Decks

The following detail shows the additional reinforcement required, unless analysis dictates otherwise, for the cantilevered portion of thin slab bridge decks with skews exceeding 20° but less than 45° , and with the cantilever span not exceeding 1.6 m. Skewed decks with cantilever spans less than 0.6 m do not require this additional reinforcement.

SKEW ψ	CANTILEVER	ADDITIONAL REINFORCEMENT
$20^\circ - 35^\circ$	0.6m - 1.2m	6 - 20M
$20^\circ - 35^\circ$	1.2m - 1.6m	6 - 25M
$35^\circ - 45^\circ$	0.6m - 1.2m	8 - 20M
$35^\circ - 45^\circ$	1.2m - 1.6m	8 - 25M

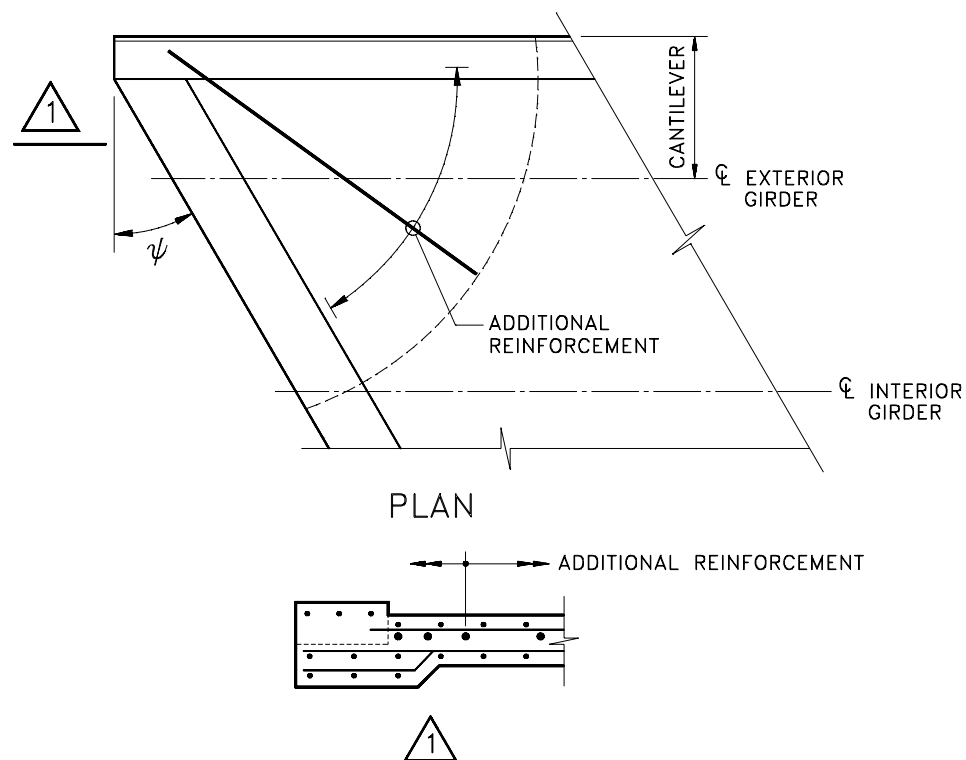


Figure 9.8.1 ADDITIONAL RADIAL REINFORCEMENT FOR SKEWED DECKS

The additional reinforcement shall be placed directly under the top layer of deck reinforcement.

No additional radial reinforcement is required at the obtuse-angle corners of the bridge deck.

9.9.1 Longitudinal Deck Joint Between Structures

Where required in the median, the longitudinal deck joint seal between (twin) structures shall be detailed on drawings as shown in Figure 9.9.1.

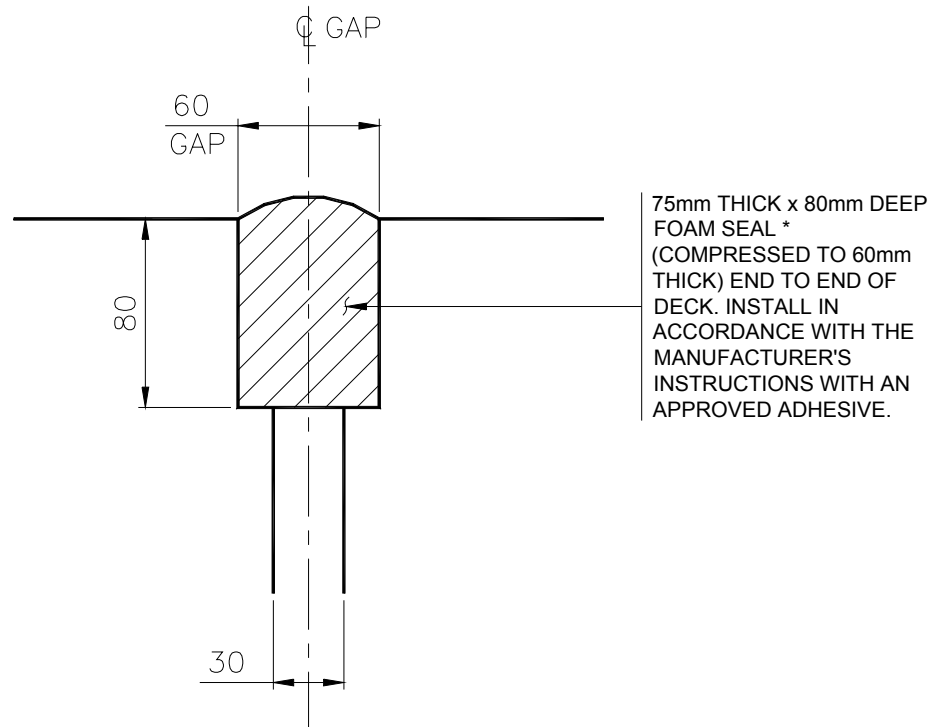


Figure 9.9.1 – Longitudinal Deck Joint

* The foam seal shall consist of material that meets the following requirements:

- Has adequate translational capacity.
- Able to be cut to suit the as built configuration and to be fully continuous over the length of the bridge without splices.
- Has a top surface profile to prevent the accumulation of dirt and dust.
- Provides high resistance to the degrading effects of ultra violet light.
- The seal material has a design and adhesive that will provide a durable watertight joint.
- Proven experience in the field.
- Accompanied by a 2-year product performance warranty.

The foam seal "Evazote 380 ESP with H.A.L.S. (Hindered Amine Light Stabilizers)" supplied by E-Poxy industries is considered to be an acceptable product that meets these requirements.

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10 BARRIERS AND RAILING SYSTEMS

10.1.1 General

- a. Bridge railing shall be selected in accordance with the requirements of Section 12 of the CHBDC, that is, conforming to Performance Levels 1, 2, or 3.

Only the railing appropriate for the performance level of the bridge site should be used

All bridge railings adopted for use by the Ministry originate from the crash-testing programs carried out in the United States. The current standards used by the Ministry are based on crash tests carried out in accordance with the crash test procedures of the National Cooperative Highway Research Program (NHCPR) Report 230 *Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances* (Michie 1981.) Further information about crash tested barrier systems can be found in AASHTO publications, NHCPR Reports, Transportation Research Records, and Federal Highway Administration Memorandums. The Reference subsection of the Commentary to Section 12 of the CHBDC gives a comprehensive listing of pertinent documents, and the following FHWA web site also gives actual details of approved railing systems and memorandums:

<http://safety.fhwa.dot.gov/fourthlevel/hardware/bridgerailings.htm>

Some of the accepted bridge rail designs may also be found in the 1995 AASHTO AGC-ARTBA Joint Committee publication *A Guide to Standardized Highway Barrier Hardware*.

- b. Slip forming of concrete barriers on structures is not permitted.
- c. When curbs are required, for sidewalks, raised medians etc., they shall be 150 mm high.

10.1.2 Determination of Barrier Exposure Index

The barrier exposure index B_e is a number used to determine the required performance level of a barrier at a particular location. This number reflects the traffic volumes and bridge characteristics of a particular site that should result in a barrier type that is most suitable for that situation. Example calculations for this determination, based on Section 12 of the CHBDC, are given below.

From Clause 12.5.2.1.2

$$B_e = \frac{(AADT_1)K_h K_c K_g K_s}{1000}$$

Example 1 – Low Traffic Volume Rural Road

Site Data:

Estimated $AADT_1$ for first year after construction: 1,500 with 10% trucks
Highway Type: two-lane, two-way with Design Speed of 50kph
Highway Curvature: tangent
Highway Grade: 0%

Superstructure Height: roadway passes over a shallow creek with a deck level above normal water level of 4.5 m
 Barrier Clearance: 1 m shoulder on one side and 1 m shoulder plus 1.5 m sidewalk on other

Calculation:

From the code:

	Factor	Table
Hwy. Type Factor K_h	1.2	12.5.2.1.2 (a)
Hwy. Curvature Factor K_c	1.0	12.5.2.1.2 (b)
Hwy. Grade Factor K_g	1.0	12.5.2.1.2 (c)
Superstructure Ht. Factor K_s	0.7	12.5.2.1.2 (d)

From Clause 12.5.2.1.2 $B_e = \frac{1500 \times 1.2 \times 1.0 \times 1.0 \times 0.7}{1000} = 1.260$

From Table 12.5.2.1.3 (a) for worst-case barrier clearance of 1.0 m, 10% trucks and 50 kph Design Speed, B_e of 1.260 is < 32. Site requires a **PL-1** barrier.

Example 2 – Single Lane Ramp

Site Data:

Estimated AADT₁ for first year after construction: 3,100 with 10% trucks
 Highway Type: one-lane, one-way ramp with Design Speed of 80kph
 Highway Curvature: 350 m radius
 Highway Grade: -4%
 Superstructure Height: roadway passes over high occupancy land use with a deck level 12.2 above a highway
 Barrier Clearance: 1 m shoulder on one side and 2.5 m shoulder on other

Calculation:

From the code:

	Factor	Table
Hwy. Type Factor K_h	2.00	12.5.2.1.2 (a)
Hwy. Curvature Factor K_c	3.00 (Outside)	12.5.2.1.2 (b)
Hwy. Grade Factor K_g	1.50	12.5.2.1.2 (c)
Superstructure Ht. Factor K_s	1.52	12.5.2.1.2 (d)

From Clause 12.5.2.1.2 $B_e = \frac{3100 \times 2.0 \times 3.0 \times 1.5 \times 1.52}{1000} = 42.408$

From Table 12.5.2.1.3 (a) for worst-case barrier clearance of 1.0 m, 10% trucks and 80 kph Design Speed, B_e of 42.408 is > 5.4 and < 61.4. Site requires a **PL-2** barrier.

Example 3 – High Traffic Volume Urban Freeway

Site Data:

Estimated AADT₁ for first year after construction: 120,000 with 30% trucks
Highway Type: two-way divided with Design Speed of 120kph
Highway Curvature: 500 m radius
Highway Grade: -2.5%
Superstructure Height: roadway passes over mainline railway tracks with a deck level of 6 m above track bed
Barrier Clearance: 2.5 m shoulders

Calculation:

From the code:

	Factor	Table
Hwy. Type Factor K_h	1.0	12.5.2.1.2 (a)
Hwy. Curvature Factor K_c	1.5	12.5.2.1.2 (b)
Hwy. Grade Factor K_g	1.125	12.5.2.1.2 (c)
Superstructure Ht. Factor K_s	0.8	12.5.2.1.2 (d)

From Clause 12.5.2.1.2 $B_e = \frac{120000 \times 1.0 \times 1.5 \times 1.125 \times 0.8}{1000} = 162$

From Table 12.5.2.1.3 (b) for a barrier clearance of 2.5 m, 30% trucks and 110 kph Design Speed, B_e of 162 is > 13.7. Site requires a **PL-3** barrier.

10.1.3 Performance Levels and Approved Barrier Types

In this section the various performance levels and corresponding bridge railing types that have been successfully crash tested and used by the ministry are described. Note that the railing systems identified in *italics* refer to the name of the railing system that has been crash tested and approved by the FHWA.

1) Performance Level 1 (PL1)

This level is generally used for bridges on low traffic volume roads with little accident history usually on rural highway systems; areas where a small number of heavy vehicles are expected and speeds are either posted or reduced. The minimum crash testing criteria for this level requires the crash testing of a small 1800 lb automobile travelling at 50 mph at an impact angle of 20 deg., and a 5400 lb pickup truck travelling at 45 mph at an impact angle of 20 deg. The minimum required height from top of barrier to the top of the roadway surface at the face of the barrier is 680 mm.

As none of the available crash tested PL-1 systems meet the minimum height and other geometric requirements, none are suitable for sites where pedestrian protection is warranted.

For bridges on low volume roads with an average annual daily traffic (AADT), in both directions, of 400 or less a lower performance level is acceptable. The MTO Bridge Office gives details of its criteria and successful crash test railings in its publication *Guidelines for the Design of Bridges on Low Volume Roads*.

The following barrier types are standards used by the ministry for performance level PL-1:

a. **Box Beam Guide Rail – Side Mount (SS10-40A/B/C)**

This system is based on the crash tested *California Type 115* bridge railing. Its standard structural “W” and “HSS” shapes provide a relatively inexpensive, easy to fabricate and erect railing suitable for vehicular only warrants on low speed and/or low volume highways.

Its open configuration offers a low profile see through rail for maximum visibility, which is desirable in scenic and rural sites.

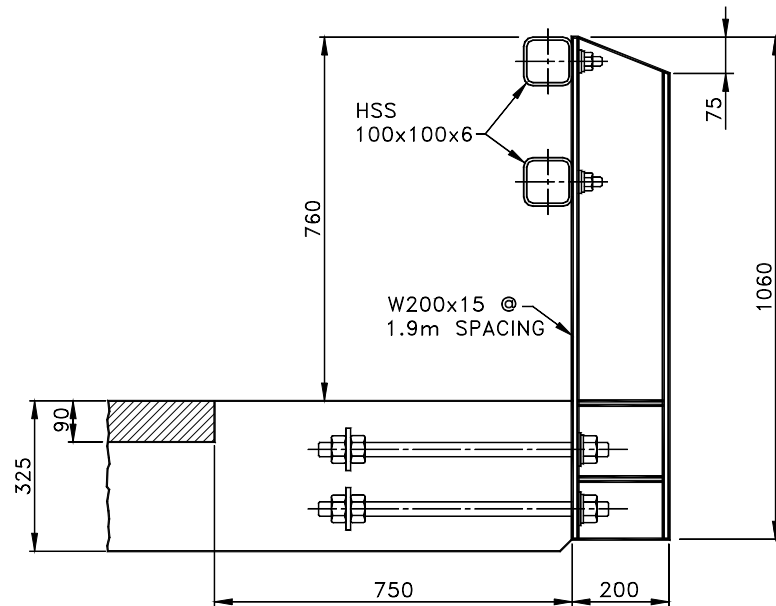


Figure 10.1.3.1a – Box Beam Guide Rail – Side Mount

b. **Thrie Beam Guide Rail – Side Mount (SS10-42A/B)**

This system is based on the crash tested *Oregon Side Mounted Thrie-Beam* bridge railing. The Oregon side mounted system consists of standard “Thrie Beam” elements mounted on standard structural steel posts. It is ideally suited for cast in place and precast slab superstructures with at least 400 minimum slab depth. It should be noted that the “W” shape steel beam guide rail used in Ontario is not the same as the “thrie” beam section. Currently there are no crash tested bridge rail systems available incorporating the “W” shape steel beam.

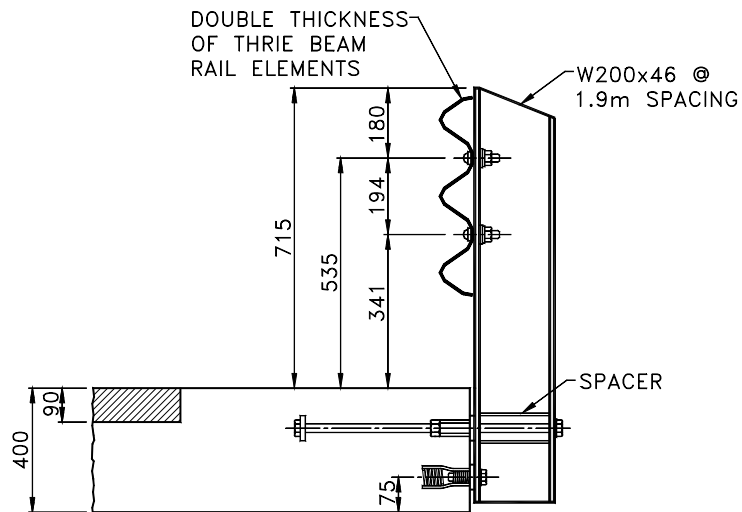


Figure 10.1.3.1b – Thrie Beam Guide Rail – Side Mount

c. Thrie Beam Guide Rail – Timber Deck (SS10-43A/B)

This system, for longitudinal timber bridge decks, is based on the *Steel System-Thrie Beam on Steel Posts* bridge railing crash tested for the US Department of Agriculture Forest Service. This in turn is an adaptation of the *California Thrie Beam* bridge rail modified for timber bridge decks. It consists of a thrie beam connected to standard structural steel “W” shape posts and spacer blocks side mounted on the bridge deck. The steel system is connected to the deck with high strength bars. Material costs are more economical than longitudinal glulam and timber post alternatives.

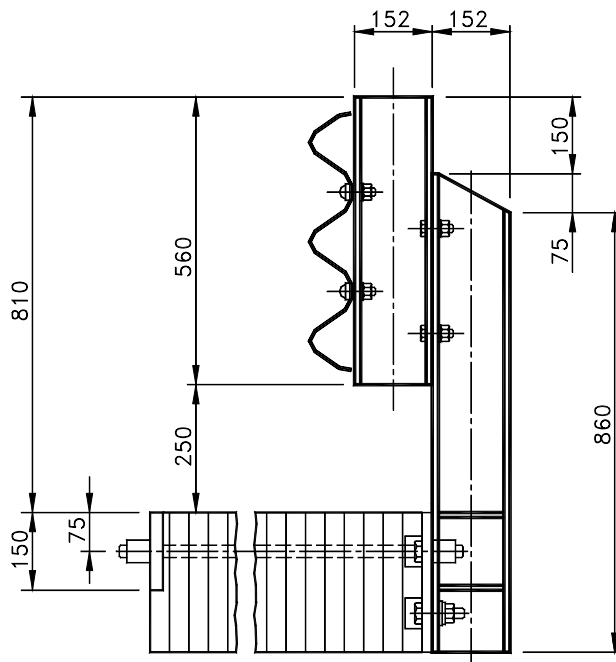


Figure 10.1.3.1c – Thrie Beam Guide Rail – Timber Deck

d. Combination Pedestrian-Traffic Railing (SS110-40/41/42/43)

This system is a combination pedestrian-traffic bridge railing based on the crash tested BR27D railing design developed by the Texas Transportation Institute for use in urban type or other areas where truck traffic is minimal. The railings consist of concrete parapets with metal railings mounted on top of the parapet. The parapets facilitate transfer of post loads into the bridge deck and the metal railing portion permits visibility through the railing. It offers advantages of being suitable for use mounted flush with the deck or on a sidewalk. In addition it offers extra height over other PL1 railing systems (1050 mm compared to 700-800 mm) where it is desirable.

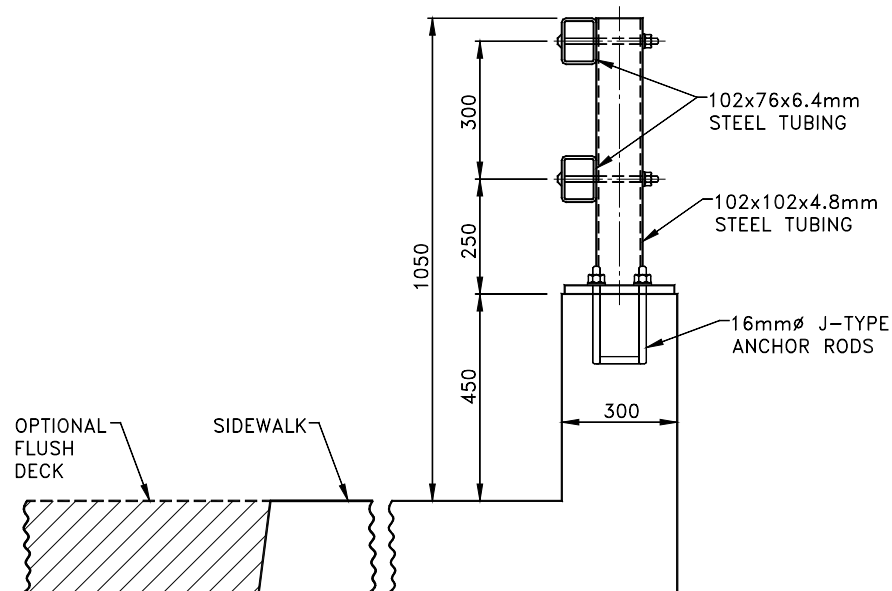


Figure 10.1.3.1d – Combination Pedestrian-Traffic Railing

2) Performance Level 2 (PL2)

This level is generally acceptable for structures on moderate to high traffic volume roads and a wide range of high-speed arterials with low mixtures of heavy vehicles. The minimum crash testing criteria for this level requires the crash testing of a small 1800 lb automobile travelling at 60 mph at an impact angle of 20 deg; a 5400 lb pickup truck travelling at 60 mph at an impact angle of 20 deg. and a 18000 lb medium single unit truck travelling at 50 mph at an impact angle of 15 deg. The minimum required height from top of barrier to the top of the roadway surface at the face of the barrier is 800 mm.

The following barrier types are standards used by the ministry for performance level PL-2:

a) Barrier Wall With Railing – PL2 (SS110-54)

This system is based on the crash tested 32-in (813-mm) *F – Shape Bridge Railing*. It is constructed of reinforced concrete and this type is the most common rigid traffic barrier in use today on both roadways and bridges. F shape does not refer to the shape of the barrier but merely to the crash test designation. Its popularity is based on its generally effective performance as a barrier, particularly with its redirection capabilities, and also its low maintenance costs. The concrete barrier requires virtually no maintenance for most hits. The lower sloped face redirects vehicles without damage under

low-impact conditions. During moderate to severe impacts, some energy is dissipated when the vehicle is lifted off the pavement. The loss of tyre contact with the pavement also aids redirection. In crash tests, the F-shape has proven to be more successful than the New Jersey shape in preventing rollover for small vehicles. The barrier may redirect or contain heavy vehicles but it was not designed for this purpose. Therefore, it is most suitable for highways carrying traffic with low heavy truck volumes.

A handrail has been mounted on top to provide a combination railing for maintenance workers and occasional pedestrians on bridges without sidewalks. It is not to be used on bridges with sidewalks as it provides no extra advantage when compared to a concrete parapet. The handrail also provides some aesthetic benefit.

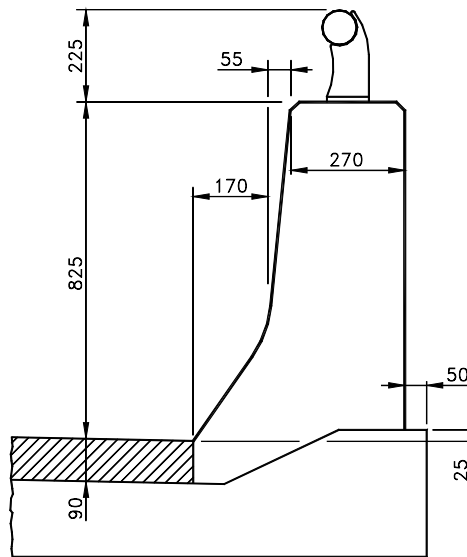


Figure 10.1.3.2a – Barrier Wall with Railing

b) Parapet Wall With Railing – PL2 (SS110-56)

This system is based on the crash tested 32-in (813-mm) *Vertical Concrete Parapet*. It offers a simple to build reinforced concrete alternative to the F shape railing. Vertical concrete walls do not have the energy management feature of the F shape, but crash test have demonstrated that they perform acceptably as traffic barriers. Because vehicles are not lifted or tilted, all four wheels tend to stay on the ground and all the energy absorption upon impact goes into the crushing of the vehicle. Therefore damage to a vehicle on impact with the parapet is likely to be more severe compared to the F shape and redirection not as smooth. Potential rollover is minimised, however, with a vertical face.

A handrail has been mounted on the top to provide a combination railing for maintenance workers and for bridges with sidewalks.

Where width is a premium on bridge decks such as on rehabilitations the narrower width of the parapet provides an advantage over the F shape.

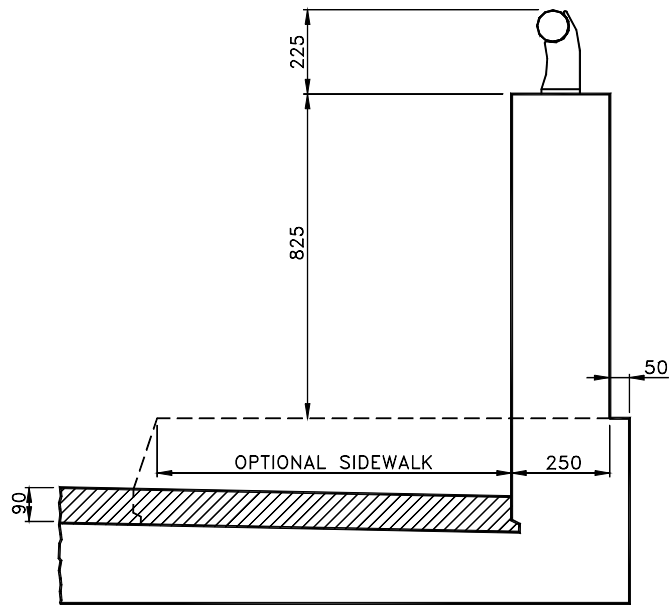


Figure 10.1.3.2b – Parapet Wall with Railing

(c) Box Beam Guide Rail on Curb - PL2 (SS10-50B)

This system is based on the crash tested *Illinois 2399 Curb Mounted Tubular Bridge Rail*. It is constructed of standard structural steel W section posts and two HSS section rails, and offers a good performing light weight and open (“see through”) rail alternative to the concrete barriers outlined above. Even though this rail is acceptable in most applications it is not recommended for use on high speed and limited access highways. For these cases the F shape is recommended, as it is better at redirecting errant vehicles and requires less maintenance. This system has to be used with concrete end posts for connection to the approach railing system as no crash tested transition standard has been developed.

It is suitable for vehicular only applications.

Because it offers one of the most open railings at this performance level it is most suitable for scenic and rural sites.

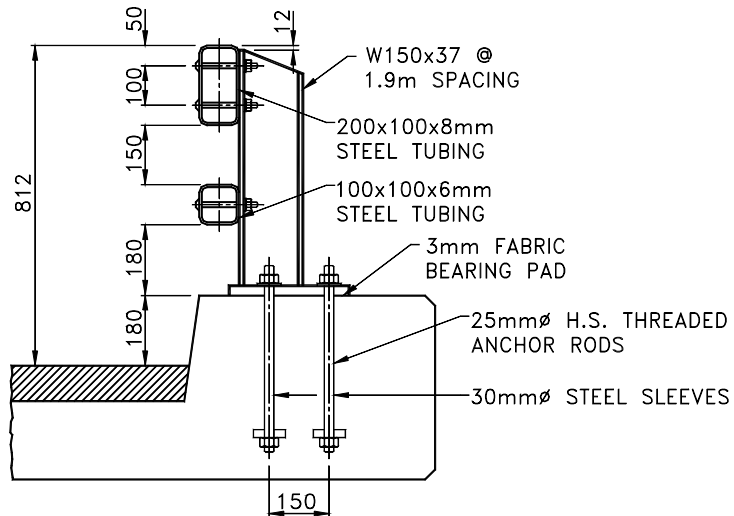


Figure 10.1.3.2c – Box Beam Guide Rail on Curb

3) **Performance Level 3 (PL3)**

This level is generally acceptable for structures on high traffic volume and high-speed highways and freeways with a high volume of mixed heavy vehicles. The minimum crash testing criteria for this level requires the crash testing of a small 1800 lb automobile travelling at 60 mph at an impact angle of 20 deg; a 5400 lb pickup truck travelling at 60 mph at an impact angle of 20 deg. and a 50000 lb Van-Type Tractor –Trailer travelling at 50 mph at an impact angle of 15 deg. The minimum required height from top of barrier to the top of the roadway surface at the face of the barrier is 1050 mm.

At the present time the ministry has only the following barrier type as a standard for performance level PL-3:

a) **Barrier Wall W/O Railing –PL3 (SS110 - 61)**

This system is based on the crash tested 42-in (1.07 m) *F – Shape Bridge Railing*. This railing system is very similar to the F shape PL2 concrete barrier in that the front surfaces and its construction are identical except for its height. The crash test characteristics are similar to the F shape PL2 barrier except that the extra height reduces potential rollover for impacting vehicles.

This railing system is not for use on sidewalks even though it meets the code requirements for pedestrians.

This railing system provides the maximum level of protection, in the event of a collision, for which the ministry has a standard. Barriers crash tested to a higher standard than NHCPR 230 i.e. NHCPR 350 or for special cases such as for heavy trucks are available, and details of these may be found on the FHWA web site.

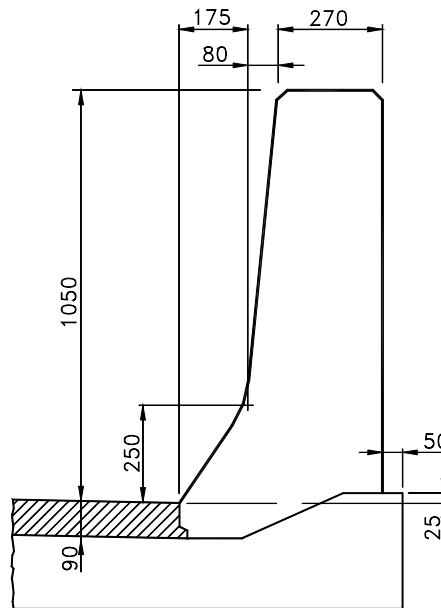


Figure 10.1.3.3 – Barrier Wall W/O Railing – PL-3

10.1.4 Bridges with Pedestrian Sidewalks

Barriers installed on pedestrian sidewalks should have a minimum handrail top at a level of 1050 mm from the top of sidewalk. This handrail provides a safety “grip” for pedestrians in case they slip, prevents people from walking on top and adds some aesthetic value.

The *Parapet Wall With S/W & Railing (SS110-57)* is the only MTO railing system, currently available, for use on the back of sidewalks at the PL2 performance level.

For bridges where barriers at the PL2 performance level are required, and where a sidewalk is on one side only, the *Parapet Wall with Railing-PL2 (SS110-57)* shall be used on the sidewalk side. On the non-sidewalk side the *Parapet Wall with Railing-PL2 (SS110-56)* or *Barrier Wall with Railing-PL2 (SS110-54)* shall be used.

The *Barrier Wall W/O Railing – PL3* railing system is not for use on sidewalks even though it meets the code requirements for pedestrians. A major concern is the barrier’s effectiveness from vehicles vaulting off the curb at this performance level. However, in the uncommon situation where a sidewalk must be located adjacent to a roadway meeting PL3 criteria the following configuration should be considered:

roadway – barrier – sidewalk – pedestrian railing

When this configuration is used interference with sight distance from interchange ramps or crossroads should be reviewed and the approach ends protected as for all barrier systems. Sidewalks located adjacent to PL3 performance level highways should be protected with more substantial protection than a 150-mm high curb.

10.1.5 Unreinforced Concrete Median Barriers and Shoulder Barriers

Unreinforced barrier walls are no longer used on Ministry bridges. The justification for the use of unreinforced concrete median barriers was based entirely on testing and a history of acceptable performance, rather than on their being able to meet structural design requirements. Outside barriers on structures have always had to meet structural design requirements.

Barrier walls are not permitted to be slip-formed on structures because the horizontal reinforcement may produce voids beneath themselves during the slip-forming process. Moisture could collect in these voids and cause problems when the moisture would freeze.

Concrete median barriers on structures must be cast-in-place, and shall not be placed directly on waterproofing membranes and must be provided with lateral restraint to prevent movement. If the membrane is stopped at the barrier, it must be turned up the vertical face at the barrier base and into a groove. With this arrangement, the barrier must be dowelled to the deck so that the membrane is not ruptured by small relative movement. Median barrier details shown on standard drawings SS 110-62/63 meet these requirements and that of CHBDC.

10.2.1 Detailing of Standard Steel Tube Railing

The standard drawings permit detailing using either of two methods:

- a. For structures on which the exact length of parapet or barrier railing can be readily established, e.g. individual structures on tangent, the spacing of the posts shall be shown on the drawings and should be as close as possible to 3 m centre to centre of posts. The rail tubing shall be listed on the Standard Drawing in pieces 6 980 mm in length plus pieces for each side of the bridge of such a length as to give the exact rail length required.

A 6 980 mm tube occupies a length of 7 000 mm because of the 20 mm expansion joint.

Where the non-standard pieces are less than 2 m long, each non-standard piece shall be combined with a 6 980 mm length to give two pieces less than 6980 mm long each.

- b. For other structures, e.g. structures on horizontal curves and structures or retaining walls in groups connected by railing, on which the exact rail length may not be readily established, the post spacing shall be determined by the Contractor and should not be shown on the plans. Rail lengths shall be ordered in 6 980 mm lengths only, with the final length cut to suit by the Contractor. The number of posts shall be listed in the table on the Standard Drawing based on a post spacing of 3 m.

If the curvature of the rail in position is sharper than 150 m radius, at least the first and last sections of the rails should be pre-bent and a note to this effect should be given on the drawings.

10.3.1 Barrier Walls on Fill - Piles

The length of piles for barrier walls on fill shall be determined as follows:

- a. Piles located between the structure and first expansion joint from the structure: Piles 1 m into existing ground, or minimum overall length 3 m and maximum overall length 6 m.
- b. Other piles under barrier walls: Piles 0.5 m into existing ground or minimum overall length 3 m, maximum overall length 5 m.

Because of the high cost of providing piles under barrier walls on fill, the designer should investigate all possible alternatives, such as carrying the normal two-sided concrete highway barrier up to the end of the structure or retaining wall, with the standard transition, where required.

10.4.1 Reinforcement of Concrete Barrier Walls on Structures

The following requirements now apply:

- a. Longitudinal reinforcement in concrete barrier walls must be continuous through construction joints.
- b. Unsupported ends of barrier walls (including those at expansion joints) must, for a distance of 1 m from the unsupported end, be provided with double the amount of reinforcement required for moment for the remainder of the wall. This is normally done by doubling up the number of bars at the unsupported end thereby reducing the spacing to one-half of the normal spacing.

10.4.2 Ducts in Concrete Barrier Walls

The following must be considered for all bridges with ducts in the concrete barrier walls:

- a. Junction boxes should not be located within 1 m from expansion joints, where the vertical steel is at 110 mm spacing.
- b. The deflection cavity for the duct (located at bridge expansion joints) is 143 mm dia. x 150 mm deep. This would interfere with reinforcement if the cover were at the maximum tolerance of 70 ± 20 mm (90 mm). A note should be added stating:

"Adjust end bar spacing to accommodate duct deflection cavity former".
- c. On structures with expansion joint assemblies, especially joints such as modular joints (DSM 9.40.20), where the armouring is carried to the top of the barrier wall, provision must be made to allow the ducts and duct deflection cavity former to be placed.

11 RIGID FRAMES

11.1.1 Rigid Frame, Backfill

Unless the abutment walls have been designed to withstand earth pressure without the deck in place, backfill should not be placed behind them until the deck is constructed. Therefore for most cases a note stating this should be shown on the contract drawings. A standard note to be shown on the General Arrangement drawing is given in 2.5.7.

Normally, rigid frame structures are to be designed assuming simultaneous earth pressure on both sides. It is essential in such cases that the General Arrangement should bear the standard note (given in 2.5.7) under "Construction Notes", on the right-hand side of the drawing.

The standard note, or a shorter note, e.g., from "Backfill" to "abutment", in the above note, must also appear on the preliminary version of the General Arrangement.

When essential due to site access problems, a structure may be designed for earth pressure on one side. This must be stated as a requirement in the Planning Report, or in correspondence before the Preliminary Drawing is approved. When a structure is designed for earth pressure on one side, the General Arrangement shall bear the following note under "Construction Notes" instead of the standard note:

"BACKFILL BEHIND (designate, e.g., "the north", "either", etc.) ABUTMENT MAY BE COMPLETED BEFORE BACKFILLING BEHIND THE OTHER ABUTMENT".

If the design of the abutment wall requires that it be supported during construction (e.g. for stability) a note to this effect should be shown on the General Arrangement drawing under "Construction Notes".

11.1.2 Earth Pressure

Rigid frames and other structures where the deflection of the abutment is prevented by the propping action of the deck shall be designed for the at-rest earth pressure.

11.1.3 Drainage

The provisions of 5.2.1(b) to (e) inclusive apply to the legs of rigid frame structures.

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12 REINFORCING STEEL

12.1.1 Reinforcing Steel Bar Designation

The previously used 6-character bar marks (7 characters for epoxy coated bars) will no longer be used. Reinforcing steel bars will now be designated as illustrated in the following example:

15M @ 300 

which denotes uncoated size 15 metric bars spaced at 300 mm, centre to centre, with the indicated shape. M is the metric bar size identifier.

When the required bar shape is clear on the plan or section, the shape need not be given.

The bar size must always be given, but the bar spacing need not be given if they are specified as "equally spaced" within a clearly defined length or width on the contract drawings.

Where the length over which they apply can be calculated, or the distance is given, the number of bars need not be shown.

The quantity, size and spacing of bars should be given in situations when the extent of the bars is unclear, or it is simpler to do so.

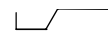
The reinforcing steel schedules and placing drawings, which are prepared by the Contractor, will contain bar marks for identification purposes.

To designate epoxy coated bars, the three characters specifying the bar size, 15M, are prefixed with a letter "C", resulting in C15M.

Some examples of Reinforcing Steel Bar Designation, illustrating the above are as follows:

15M

15M @ 300

15M @ 300 

C15M @ 300

20-15M @ 300

20-C15M @ 300

12.1.2 Coated Reinforcing Steel - Where Required

Reinforcing steel in locations vulnerable to salt-induced corrosion must be epoxy coated. Bars within 100 mm (e.g. cover specified as 125 ± 25 mm) of the following surfaces shall be coated unless the surfaces are permanently covered with at least 500 mm of soil or water (See Fig. 12.1.2(a) to (c)):

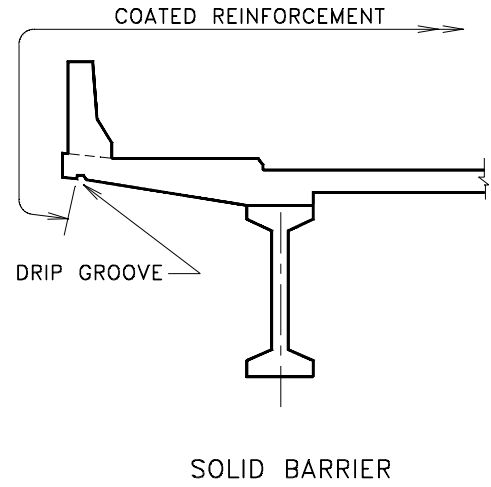
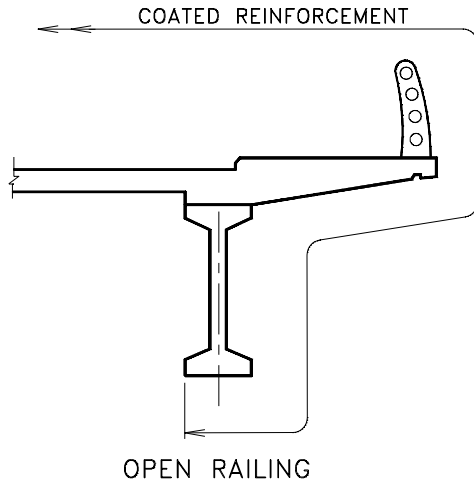
- a. Top surfaces of bridge decks, ballast walls and approach slabs with or without asphalt.
- b. Top and side surfaces of bases for pole and sign structures attached to bridges.
- c. All surfaces of curbs, sidewalks, medians, parapets and barrier walls.
- d. Surfaces of abutments, piers and retaining walls that are exposed to salt splash. This includes all such surfaces within 10 m of an existing or future roadway, measured horizontally from the edge of pavement unless the surface is more than 5m above the roadway or is permanently covered with water or at least 500 mm of soil.
- e. Surfaces of abutments, piers and retaining walls that are exposed to roadway drainage or dripping. This includes surfaces below sealed or unsealed joints, below deck drain outlets, and below overhanging features from which dripping may occur. "Below" should be taken to mean within a vertical cone having an apex angle of 45° or likely to receive run-off from a surface within such a cone.

Stirrups from standard precast I beams, that project into deck slabs that are 200 mm or more in thickness need not be coated.

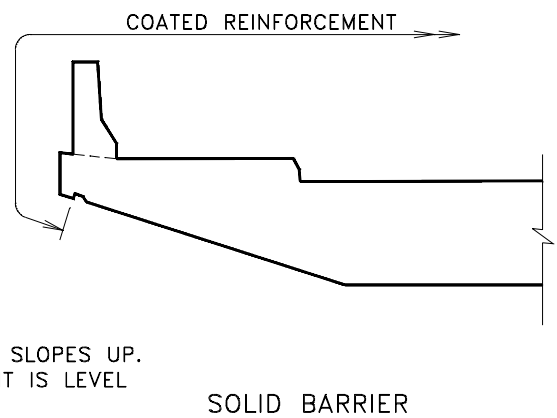
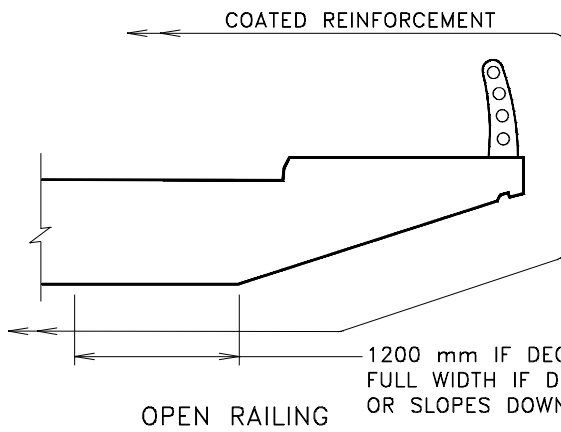
Shear studs cannot be coated and should be provided with 100 mm minimum cover.

Epoxy coating is a fairly expensive process and should be minimised. This often requires an arrangement of reinforcement that minimises the total length of bars that require coating over only a part of their length.

The reinforcing steel in concrete culverts with at least 500 mm of fill height and/or greater than 10 m away from the travelled edge of roadway does not have to be coated even though the culvert ends may be exposed. Deterioration at the ends is much slower, less critical and easier to repair than, for example, on bridge decks, and culverts are frequently extended before deterioration is significant. Coated bars should be considered however, if the exposed length is unusually long.



CONCRETE GIRDER AND SLAB DECKS



1200 mm IF DECK SOFFIT SLOPES UP.
FULL WIDTH IF DECK SOFFIT IS LEVEL
OR SLOPES DOWN

THICK SLABS WITH OR WITHOUT VOIDS

FIG. 12.1.2(a) COATED REINFORCEMENT BELOW JOINTS & RAILINGS

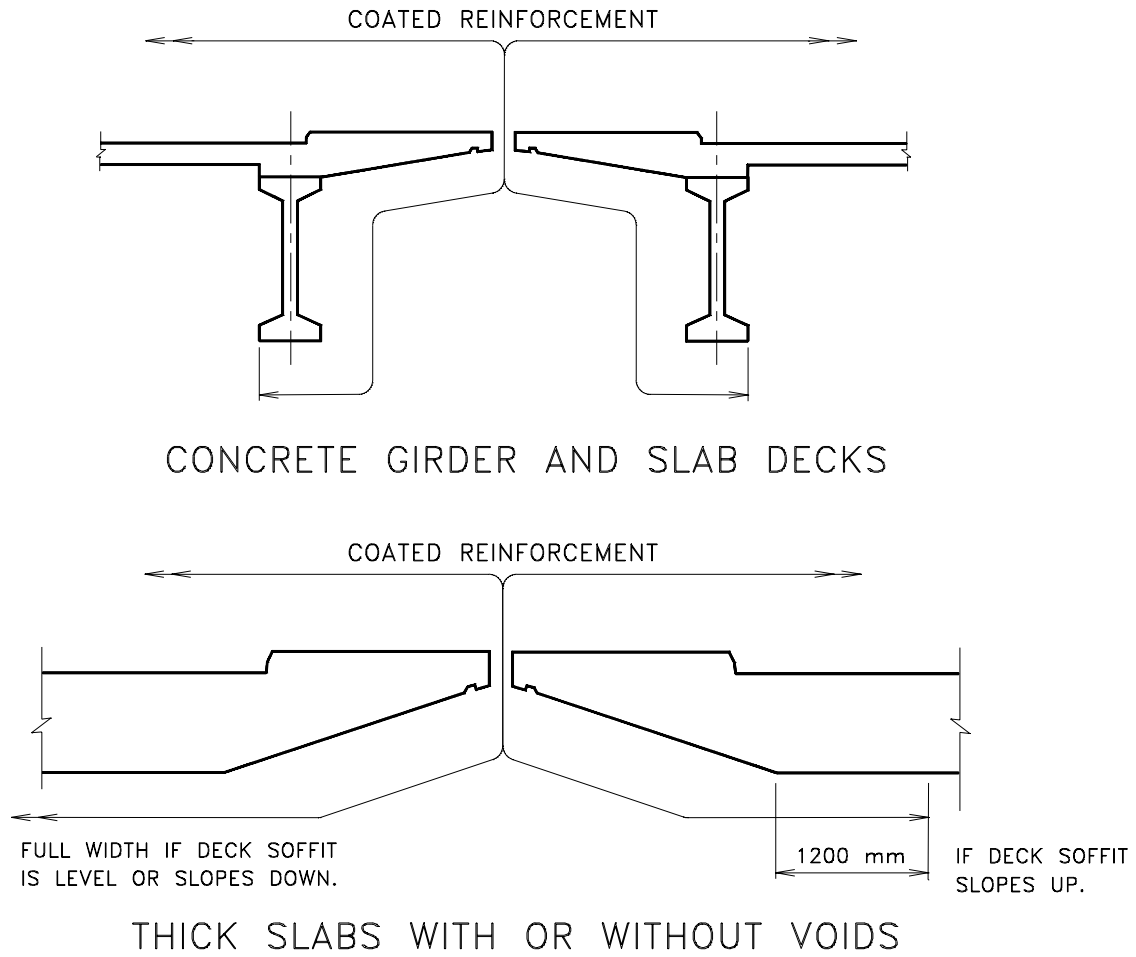


FIG. 12.1.2(b) COATED REINFORCEMENT BELOW JOINTS & RAILINGS

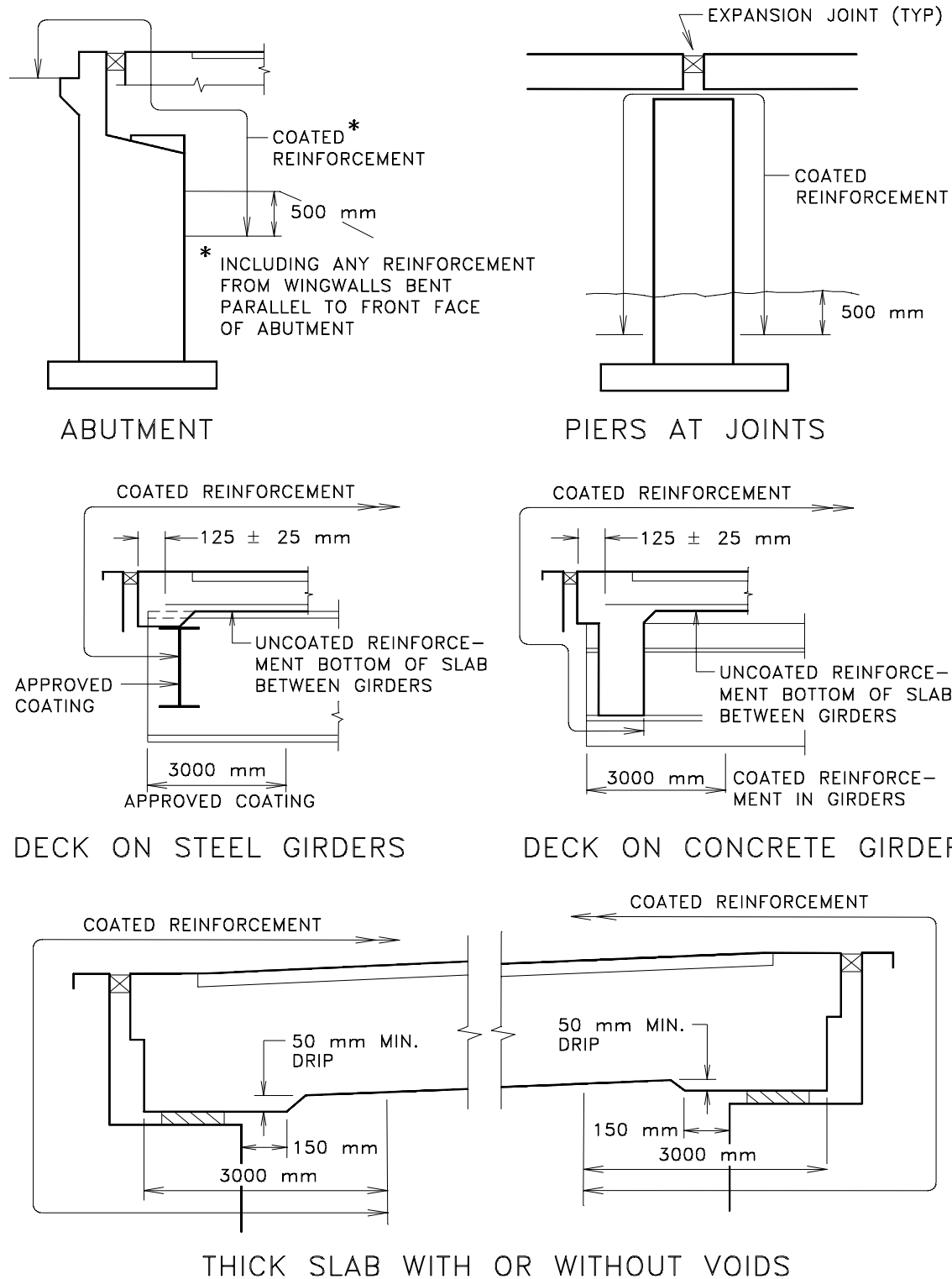


FIG. 12.1.2(c) COATED REINFORCEMENT BELOW JOINTS & RAILINGS

12.1.3 Coated Reinforcing Steel Bar Identification and Dimensions

The prefix "C" is used with the bar identification to denote a coated bar. The "C" should be regarded as an inseparable part of the bar identification.

Coated bars cannot be bent to the standard bend radii used for normal bars, without causing the epoxy coating to crack. For this reason, coated bars must be subject to special bend size and other dimensional limitations if the coating is to remain intact. These limitations are given in this Manual as DA 12-2 to 7. A number of measures can be adopted to avoid problems, e.g.

1. Bent bars can often be avoided where coating is required or the bent portion can be located where coating is not required, in which case, the cracking can be accepted.
2. For new projects, coated bars must be detailed so as to comply with the dimensional limitations. If bars not meeting the limitations are detailed, the Fabricator will substitute bars that do. This could produce placing difficulties during construction and the Fabricator is required to obtain approval for such changes before fabrication.
3. For old stockpiled projects, it is acceptable to leave the detailing and scheduling as it is, and deal with the Fabricator's proposals as they arise. These may include further coating or repair of cracked areas or cutting and lapping of bars.

12.2.1 Reinforcing Steel for Concrete Culverts

Reinforcing Steel and Coated Reinforcing Steel quantities for all cast in place concrete culverts in a contract are combined and included in separate Lump Sum tender items. However, when the total quantity of reinforcing steel for all concrete culverts in a contract is less than 5 tonnes, the reinforcing steel will be included in the concrete item "Concrete in Culverts" and there will be no separate tender item for reinforcing steel. For details of coating see Section 12.1.2.

12.3.1 Reinforcing Steel - Limiting Dimensions

The normal mill length for bar stock is 12 m for size 10, and 18 m for the remainder. These lengths should never be exceeded.

The shipment by truck of bars longer than 15 m requires an uncommon type of vehicle, and should be avoided unless there is a considerable economic advantage.

Bent bars must when laid flat, fit into a rectangle having dimensions not greater than 20 m by 2.4 m.

12.3.2 Use of Size 10 Reinforcing Bars

Size 10 bars are too flexible for most applications, particularly in deck slabs. Size 10 bars shall not be used in deck slabs; their use should be restricted to small sections, such as barrier walls and precast units, where supporting stirrups are close together.

12.3.3 Reinforcing Steel - Availability

Size 45M and 55M reinforcing bars are not commonly called for within the reinforcing steel industry. Fabricators cannot economically justify stocking these bars for which there is a low demand. As a result, a premium may be paid, and delays possible, if these bars are specified. It is recommended that these bar sizes be avoided wherever possible.

Weldable steel is more expensive and not as readily available as non-weldable reinforcing. It should only be specified when necessary for ductility and/or welding of reinforcement.

12.4.1 Splicing of Reinforcing Bars

OPSS 905.03, May 1994, lists and describes the following six types of mechanical connectors for reinforcing steel bars:

a. Filled Sleeve Type	b. Sleeve Swaged Coupler Type
c. Threaded Coupler Type	d. Hot Rolled Thread Bar Coupler Type
e. Forged Bar Coupler Type	f. Form Saver Type

In cases where bars are too congested to permit laps, and full length bars are not practicable, splicing is occasionally unavoidable. Cadweld splices, and Dywidag or Lenton couplers are acceptable for this purpose. They should be specified on the drawings as "Cadweld Series (specify) splice" or as "Dywidag Couplers with hex. nuts" or as "Lenton Standard Couplers". The Cadweld series must be specified to suit the required performance.

Minimum cover requirements apply to the splices and couplers. Cadweld splices are not suitable for epoxy coated bars.

Couplers and splices should not be used where high cyclic stresses exist.

12.5.1 Reinforcing Steel Schedules

For all new design projects, whether designed in-house by MTO or by Consultants, the following procedures shall be adopted.

1. Rebar schedules shall no longer be prepared and provided by the designer as part of the contract documents.
2. Because the contractors will produce their own rebar schedules and placing drawings, individual bar marks shall not be used on the contract drawings, but the new method of Reinforcing Steel Bar Identification detailed in Section 12.1.1 will be used.
3. Review of rebar schedules and placing drawings will be done as detailed in Section 12.5.3.
4. The contract drawings shall be prepared with sufficient rebar detail and other relevant information such that rebar schedules and placing drawings can be independently and correctly produced by the fabricator. For description of detailing, see Section 12.5.2.

Stockpiled projects which have the previously used bar marks detailed on the drawings shall not be converted to the new Bar Identification. In this situation, the drawings were probably prepared in conjunction with the reinforcing steel schedule. The bar shapes and special dimensions would not be detailed on the contract drawings, but would be given in the steel schedule. Under these circumstances, the reinforcing steel schedule should be made available to the Contractor to assist him in preparing his own rebar schedule and placing drawings. However, the Contractor must be made aware that MTO does not guarantee the accuracy of the bar lists, and that the Contractor is responsible for verifying all the data in the schedules.

12.5.2 Detailing Reinforcing Steel on Contract Drawings

1. The structural designer shall ensure that the contract drawings are clearly detailed and that all required detailing information is shown.
2. Reinforcing bars shall be identified according to Section 12.1.1.
3. Minimum detail shall include the size, shape, spacing and placing limits for individual bars, and if necessary, the number of bars. Detailing should be adequate to eliminate ambiguity and misinterpretation. Dimensioned sketches of bars should be provided when required.
4. Lap lengths, (tension or compression) that are not indicated on the contract drawings shall be Class B as per CHBDC. Class C tension lap lengths, where required, shall be indicated on the drawings.
5. Bar hooks, where required, shall be minimum length, as per DA12-2A and DA12-2B of the Structural Manual, unless indicated otherwise.
6. Stirrups shall have minimum hooks, as per DA12-2A and DA12-2B of the Structural Manual, unless indicated otherwise.
7. Reinforcing Steel and Coated Reinforcing Steel are now Lump Sum bid items. Their quantity shall not be listed on the contract Quantities-Structure drawing.
8. The structural designer shall forward the quantities of Reinforcing Steel and Coated Reinforcing Steel items to the Regional Planning and Design (or Highway Engineering) Section, or the Section responsible for contract preparation, and the Estimating Office. These quantities should be accurate to within $\pm 10\%$.
9. The requirements of the above notes 4 to 6 should be given as part of the General Notes on the General Arrangement Drawing of all projects.

12.5.3 Checking of Rebar Schedules and Placing Drawings

For Ministry projects, the Contractor is required to submit six copies of complete bar lists and placing drawings of reinforcing steel a minimum of three weeks before commencing the placing of steel reinforcement.

The distribution of these lists and drawings is as follows:

- 3 sets for Field Office
- 1 set for Regional Construction Office
- 2 sets for Regional Structural Section (one set given to the designer, if necessary)

It is recommended that reinforcing steel schedules and placing drawings be examined by the Regions on a selective basis, for the correct interpretation of the design drawings. The choice

of structure to be checked should be based on the complexity of structure (e.g. post-tensioning, large skews), previous experience with fabricator/contractor, or the structure's critical importance. The Regional Structural Section should decide which structure shall be monitored for review.

When a particular structure is selected, only critical areas and elements such as anchorage zones, cantilevers (decks, retaining walls), columns, areas with large size bars or other locations as determined by the designer need to be checked. Both schedules and placing drawings should be checked together for the size of bars, shape, number and spacing. Individual dimensions and nominal steel need not be checked except where critical lengths, such as embedment lengths or anchorage requirement, etc. are given.

For all structures, and prior to placing of concrete for each component, the Contractor shall submit a Certificate of Conformance to the Contract Administrator. This Certificate shall state that the reinforcing steel has been placed in conformance with the Contract drawings, and shall bear the signature and seal of a Professional Engineer. A Special Provision is required to detail the requirements of the Certificate of Conformance. The present method of field inspection by the Ministry will not be affected by this requirement.

Since the practice of allowing the Contractor to produce reinforcing steel schedules and the elimination of bar marks is relatively new to the Ministry, it seems prudent that the above policy be followed until experience dictates otherwise.

12.5.4 Hooks and Bar Bends

Design Aids DA 12-2A and DA 12-2B (Dec 2001) have been updated to be in conformance with the CHBDC-2000, and also to meet the minimum fabrication requirements of the latest edition of the RSIC Manual of Standard Practice. These two Design Aids have been duplicated as Structural Standard Drawings SS 12-1 and SS 12-2, included in Division 4, Section 1 of this Manual, and shall be attached to the full-size standard drawings in a contract. They are intended for the contractor's use, in order to produce consistently accurate hooks and bar bends in uncoated, epoxy-coated and stainless reinforcing steel.

Design Aids DA 12-3 to DA 12-7 show details of Minimum Bar Bend Dimensions for Grade 400 Coated Bars, Stirrups and Ties. These Design Aids have been produced, based on DA 12-2B, for the benefit of the designer so that the concrete member sizes with realistic and minimum rebar dimensions harmonize.

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13.1.1 Expansion Joints in Decks

Bridge deck expansion joints should be of a sealed type covered by current Structural Office and Bridge Office Standards. The designer should make every effort to minimise the number of expansion joints along the length of structure. In all cases, the expansion gap shall be detailed parallel to the skew of the deck, for the full width of the deck. The resulting exposed acute-angled concrete edges shall be provided with 50 mm chamfers. Designers should also try to avoid skew angles of 32° to 38°, inclusive, to minimise the chances of snagging snow plough blades which are usually set at approximately 35°.

A moveable joint shall be considered an expansion joint even though over a fixed bearing; deflection of the deck can cause closing and opening of the joint (see also 13.1.4).

The gap between the end of a deck and the ballast wall face must meet the requirements of the CHBDC. The gap between the underside of the deck and the bearing seat shall be large enough and accessible enough for cleaning. The drawings must not show the gaps left filled with material used in construction.

When specifying an expansion joint movement rating, only those movements which can occur after the joint has been installed shall be considered. Thus when specifying expansion joint movement ratings for rehabilitation projects, elastic shortening, hydration, shrinkage and creep effects should be neglected. Likewise for new prestressed concrete bridges, creep and shrinkage effects occurring prior to joint installation should be neglected. Joint installation can occur as many as 90 days after the completion of prestressing operations; the actual time to be used by the designer should be determined by consultation with the Region or Bridge Office and indicated on the expansion joint standard.

Expansion joint assemblies are installed by the block-out method either prior to or after the completion of paving operations. Regardless of the method to be used, the designer must account for the compaction of the asphalt wearing surface, by traffic, adjacent to the concrete end dams.

If a concrete strip is detailed as a "dam" between an asphalt wearing surface and an expansion joint, it should be wide enough to permit the placing of concrete through the gap between joint armouring and concrete dam armouring, but should not be less than 500 mm in width, either side of the joint. Weep holes through the deck are required at low points at these dams. Refer to OPSD-3950.00.

Straight reinforcing bars to be placed parallel to expansion joints in the concrete end dams shall consist of:

- (a) for modular joints - two shorter bars lapped between the support boxes; and,
- (b) for all joints - at least two shorter lapped bars which are to be threaded through anchor loops or other in-place reinforcing steel.

Joints should be specified on the drawings by reference to the appropriate "Designated Sources Material List" number and, in the case of DSM 9.40 - Joints, by type. All types, structurally acceptable for the particular project, should be given.

When splicing of joint armouring is unavoidable, such as at construction staging, permissible splice locations must be shown on the drawings. Such locations should be at crown points, if possible, and in no case shall they be located near curbs, barrier walls, wheel paths or at any point where water is likely to pond. On skew bridges, when the approved splice is shown at a crown, the splice in the armouring should be shown parallel to the centreline of the traffic lane and not perpendicular to centreline of joint.

The material for strip seals shall be neoprene. Natural rubber as an alternative material is not permitted as MTO lab tests have found that the effects of ozone and oil swell tests were unacceptable.

13.1.2 Approved Expansion Joint Assemblies

The expansion joint systems approved for use on new and/or rehabilitated structures are listed in Designated Sources for Materials List DSM 9.40 - Joints as follows:

DSM NO. (DIVISION/APPLICATION/PRODUCT)

DSM 9.40.18	Expansion, Injection Systems for Armouring
DSM 9.40.20	Expansion, Modular
DSM 9.40.24	Expansion, Strip Seals Anchored in Concrete, Type A
DSM 9.40.27	Expansion, Strip Seals Anchored in Concrete, Type C
DSM 9.40.30	Expansion, Strip Seals in Elastomeric Concrete, Type A
DSM 9.40.32	Expansion, Strip Seals in Elastomeric Concrete, Type C
DSM 9.40.33	Expansion, Strip Seals in Preformed Retainer

Guidelines for the application of these lists are given in 13.2.1 to 13.2.7.

13.1.3 Expansion Joint Assemblies in Skewed Bridges

A plan and profile must be provided on the drawings for skewed expansion joints. Drawings of fabricated expansion assemblies shall show the following:- crossfalls, break points, lengths between break points, total lengths, skew angles where appropriate, and location of approved splices in armouring where applicable.

When specifying an expansion joint assembly for a skewed bridge, the designer must be aware of the effect of the skew on the rating/performance of the joint.

Joint assemblies in bridges with skews should be carried through sidewalks, medians and barrier walls without horizontal change in direction wherever possible. A 50 mm chamfer shall be detailed on all exposed acute-angled concrete deck edges on skewed bridges.

Where this is not possible the horizontal change in direction should be effected at a location not less than 100 mm and not greater than 600 mm from the edge of the asphalt at the barrier wall.

A horizontal change in direction must NOT be located within 100 mm of a vertical change in direction since simultaneous bending of the seal in two planes is either physically impossible or likely to lead to leakage between the seal and the clamping device. Locating horizontal change as close as is practicable to the barrier wall reduces the modifications necessary to the substructure.

The change in direction shall be effected by means of a 600 mm radius bend. The exact location of the radius bend should appear on the plan view of the structure on the contract drawings and on the contractor's shop drawings.

Structures having a skew of up to 15 degrees and over 45 degrees shall have joint armouring anchorage bars, on the deck side, detailed perpendicular to the expansion joint. Structures skewed from over 15 degrees up to 45 degrees shall use joint armouring anchorage bars, on the deck side, detailed 30 degrees offset from the perpendicular to the expansion joint. Anchorages on the abutment side of the joint shall be placed at right angles to the joint.

13.1.4 Fixed Joints and Longitudinal Joints

Where significant rotational or other movement is anticipated in fixed and longitudinal joints, the joints should be treated as expansion joints. When DSM 9.40 - Joints are used at "Fixed" joints, the "J" dimension shall be set at 50 mm.

Where less than 6mm of total movement is anticipated at fixed joints, an asphaltic plug system may be specified in lieu of DSM 9.40 - Joints.

13.1.5 Cellular Sealants

Closed cell cellular sealants shall be used only in unarmoured longitudinal joints or to replace the seal in existing compression seal expansion joints. Where the existing compression seal has no armouring or where the existing anchorages have to be removed the complete joint should be replaced by DSM 9.40.24 to 9.40.28 or 9.40 joint assemblies as described in 13.2.2.

13.1.6 Completion of Standard Drawings

The following note on the expansion joint standard drawings must be completed by the designer:

"Expansion Joint shall be in accordance with the Designated Sources for Materials List DSM 9.40.____¹ and shall have a rated total movement capacity of _____ mm."²

The movement ratings given in Designated Sources for Materials List DSM 9.40 were established from laboratory testing and to allow seal replacement at 25°C.

Where possible (i.e. where the full rated movement of the joint is not fully utilised), the "J" dimension at various temperatures shown on the standard drawings should be increased by 5 mm. This will have the effect of providing slightly more opening for future replacement of the seal.

1 The designer shall insert the number of the appropriate DSM list or Lists which are acceptable. Where DSM 9.40 - Joints are shown, the acceptable type or types must also be given, except when all types are acceptable.

2 The movement shown should be the required movement as determined by the Designer, calculated at Serviceability Limit States; this movement shall not exceed the rated movement of at least one of the joints in the type specified, and where possible, should not exceed the rated movement of at least one joint from each supplier in the type specified.

13.2.1 Modular Joints

Modular joints are for bridges having total deck joint movements after joint assembly installation, measured parallel to the centreline of the highway, greater than 80mm in one direction.

Available Modular joints can have a minimum of 2 to a maximum of 8 seals. The maximum movement of any one seal, measured parallel to the centreline of the highway, must not exceed 60 mm. Notwithstanding the above, modular joints should only be used as a last resort when no other type of expansion joint is suitable, or when other means cannot be found to accommodate the structure's anticipated movement. Refined methods of analysis are justified to more accurately predict deck joint movement when the use of modular joints is being considered.

Dimensions **F** and **D + F** in Figure 13.2.1(1) vary with the number of seals, and the minimum requirements are:

$$\mathbf{F} = 250 \text{ mm MIN. at } 15^{\circ}\text{C (maintenance requirement)}$$

$$\mathbf{D + F} = 450 \text{ mm MIN. at } 15^{\circ}\text{C (for inspection and maintenance)}$$

For post-tensioned concrete bridges, **F** and **D** should be calculated at 15°C for the required joint size and movements due to creep, shrinkage and temperature fall occurring after time of joint installation. In lieu of more accurate data, the designer may assume joint installation occurs at 90 days after stressing of prestressed concrete bridges.

Dimensions **H** and **B** in Figures 13.2.1 shall be as follows:

						Seldom Used	
No. OF SEALS	2	3	4	5	6	7	8
B (mm)	600	675	750	825	900	975	1050
H (mm) MINIMUM	325	325	325	325	335	340	350

The width of blockout **B** depends only on the number of seals in the joint, while the depth of blockout **H** depends not only on the size of the joint (number of seals) but also on the supplier of the expansion joint.

When the ballast wall is corbelled at the top and the corbel is greater than 100mm, the corbel reinforcing shall be designed by an appropriate method such as "strut and tie".

13.2.2 Modular Joint Selection

1. Calculate the Design Movement "**D**", due to the sum of Creep + Shrinkage + Temperature Range.

Creep and Shrinkage movements are from time of joint installation, i.e., from $t = 90$ days to infinity.

Movements due to Temperature Range are from the sum of temperature fall to temperature rise.

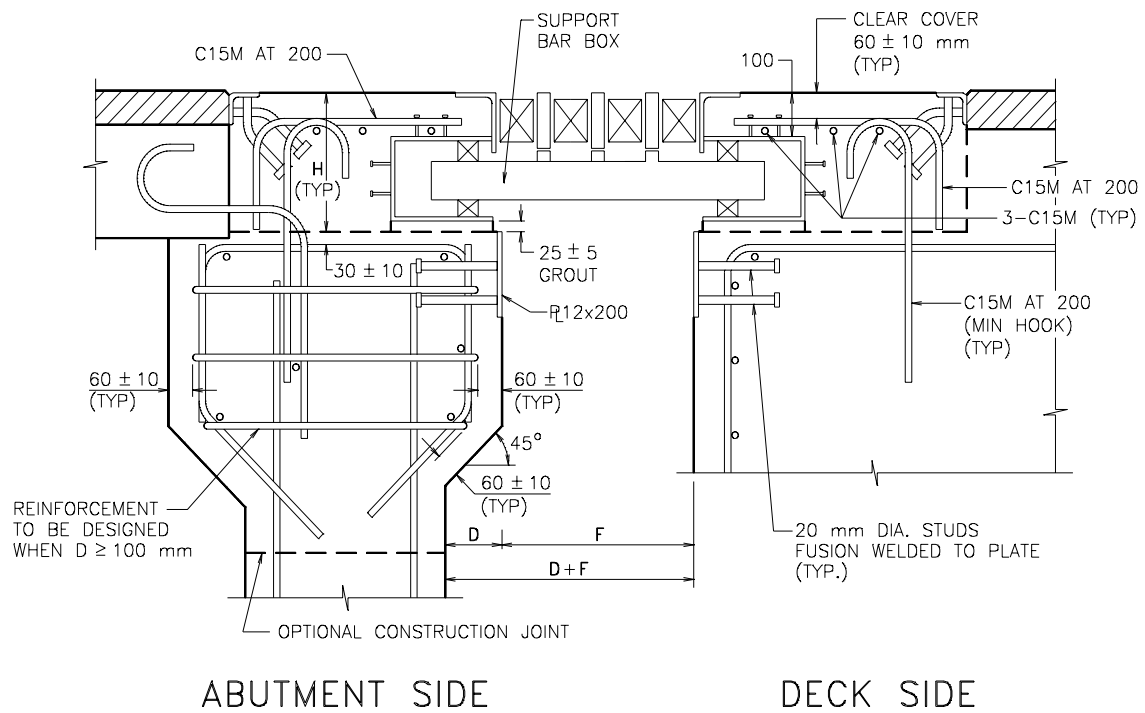


Figure 13.2.1(1) MODULAR JOINT DETAIL - SECTION AT BOXES

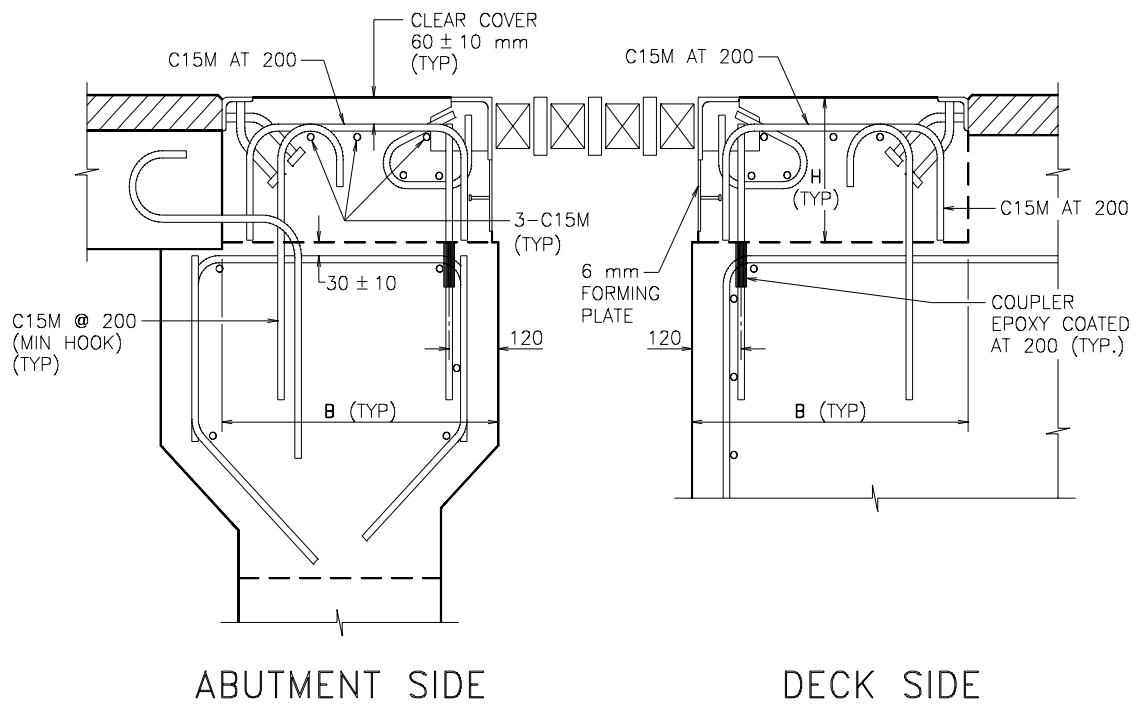


Figure 13.2.1(2) MODULAR JOINT DETAIL - SECTION BETWEEN BOXES

2. Obtain the Serviceability Limit States design movement which is 80% of the above Design Movement.

If the SLS Design Movement > 80 mm, use Modular Joint, otherwise strip seals may be used (see 13.1.2 and 13.2.3 to 13.2.5).
3. For Modular Joints, select joint size (number of seals required) based on the SLS Design Movement, and an allowable movement of 60 mm per seal.
4. Select the dimension "**D**" based on the number of seals required.
5. Select dimensions "**B**" and "**H**".

No. OF SEALS	2	3	4	5	6	7	8
D @ 15°C in mm	200	200	150	100	50	0	0
F @ 15°C in mm	250	250	300	350	400	450	500

13.2.3 Modular Joint Setting

Having selected an expansion joint whose Range of gap rating (max. gap minus min. gap) is greater than or equal to the SLS Design Movement, the "J" gap dimension must be calculated at the assumed 15°C construction temperature.

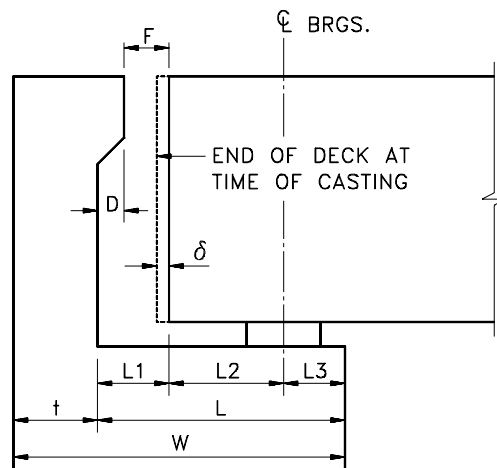
For prestressed concrete structures, "J" at 15°C equals $0.8 (CR + SH + T_{fall})$, where T_{fall} is the movement due to a temperature drop from 15°C to the Minimum Effective Temperature used in design.

The maximum and minimum "J" dimensions must then be shown on the expansion joint drawing.

13.2.4 Abutment Width

Since large movements are expected when modular joints are used, the minimum width of abutment should be based on the distances shown in Figure 13.2.4. The effect of skew and minimum seating requirements for earthquake should also be considered.

At 90 days from casting of the deck, the gap width "F" (see Figure 13.2.4) at the level of the corner protection plates and at a temperature of 15°C shall be equal to "J" but not less than 250 mm.



- δ = the expected movement, at the end of the deck, due to Hydration + Elastic Shortening plus Creep and Shrinkage, from $t=0$ to $t=90$ days
- L1 = Gap between ballast wall and end of deck at 90 days (Min 450 mm)
- L2 = Distance between centreline of bearings and end of deck at 90 days
- L3 = Distance between centreline of bearings and face of abutment
- t = Thickness of ballast wall
- W = $t + (L1 + L2 + L3) = t + L$

Figure 13.2.4

13.2.5 Strip Seal Joints Anchored in Concrete

The joints in this list are for bridges having total deck joint movements calculated at SLS, measured parallel to the centreline of the highway, and after joint assembly installation, of 80 mm or less but not greater than the rated movement of the joint specified.

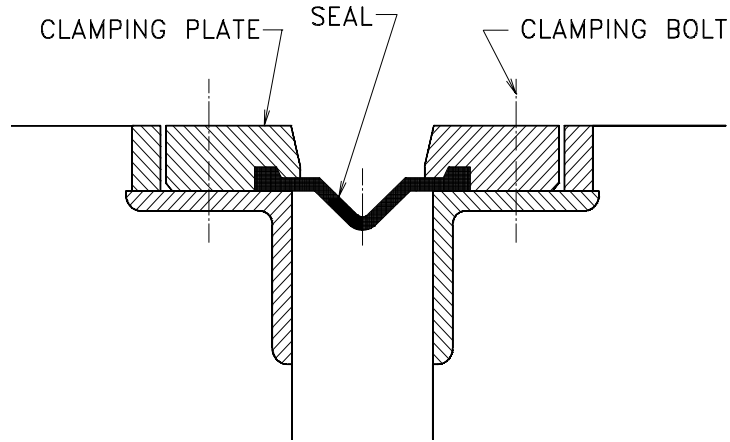
DSM 9.40.24 to 9.40.33 is subdivided into several types based on the method of seal retention used. General guidelines showing the characteristics of each type and the applications for which it should be considered, are given on the following pages.

It is the responsibility of the designer to select the appropriate type or types of joint in accordance with those general guidelines. All types of expansion joints on the DSM List have been approved by the Ministry and should be considered for application on structures. Where further guidance is required the Head, Regional Structural Section should be consulted.

Standard drawing SS 113-11 covers anchorages and armouring for all types of expansion joints listed in DSM 9.40.24 to 9.40.33.

Strip Seals - Type A Steel Plate Clamping Device

Clamping Plate Subject to Direct Wheel Load



Characteristics:

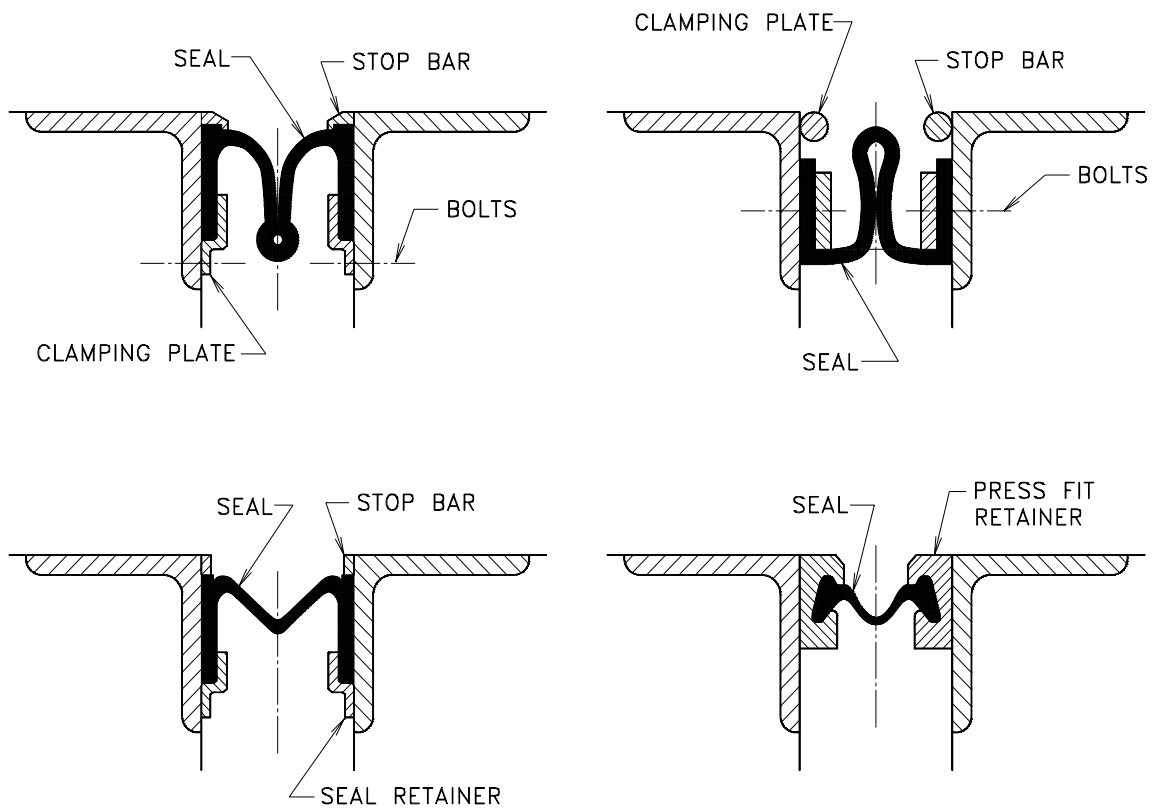
- Clamping plate directly supported.
- Few components.
- Excellent seal retention.
- Easy to install and replace seal.
- Direct access to clamping bolts and seal.
- Seal can be inspected completely and reused, if undamaged.
- Easy to modify for changes in elevation.
- High initial cost.

Application

Type A joints as per DSM 9.40.24 should be considered for all bridges and particularly for bridges which carry high volume freeway/arterial traffic (and especially when there is a high proportion of heavy vehicles).

Only Type A joints should be specified for bridges for which, in the opinion of the Head, Regional Structural Section, it is absolutely essential to minimise the frequency and duration of future lane closures.

Strip Seals - Type C Retainers and Stop Bars



Characteristics

- Good seal retention.
- Can accommodate change in direction in plan.
- Seal cannot be inspected completely without removing the seal.
- Installation or replacement of seal may be difficult.
- Special tools and trained personnel required to install or replace seal for maintenance purposes.
- No direct access to clamping device or retainer and seal.
- Difficult to replace bolted clamping device or damaged retainer.

Application

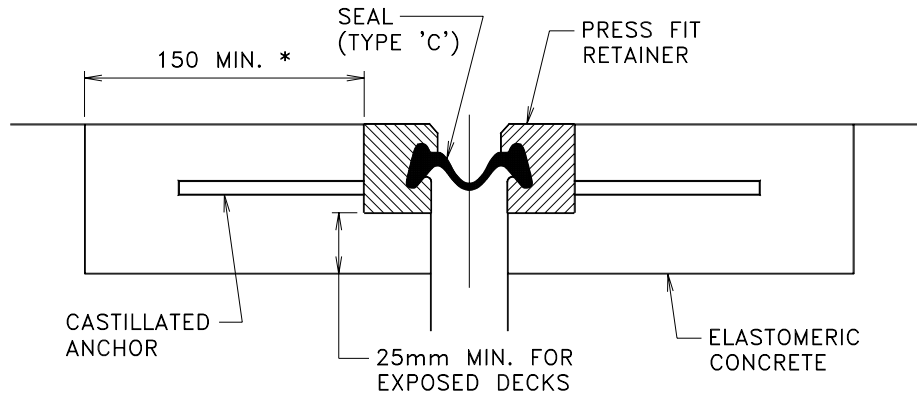
Type C joints as per DSM 9.40.27 should be considered for all bridges but excluding the following:

- a. Bridges which carry high volume freeway/arterial traffic.
- b. Bridges for which, in the opinion of the Head, Regional Structural Section, it is absolutely essential to minimise the frequency and duration of future in-service lane closures.

13.2.6 Strip Seal Joints Anchored in Elastomeric Concrete

The joints in this list are for replacement or modification of existing joints where no satisfactory anchorage exists or can be provided to accommodate DSM 9.40.24 to 9.40.28 joint assemblies.

DSM 9.40.30 lists sources for Type A Steel Plate Clamping Device, and DSM 9.40.32 lists sources for Type C Retainers.



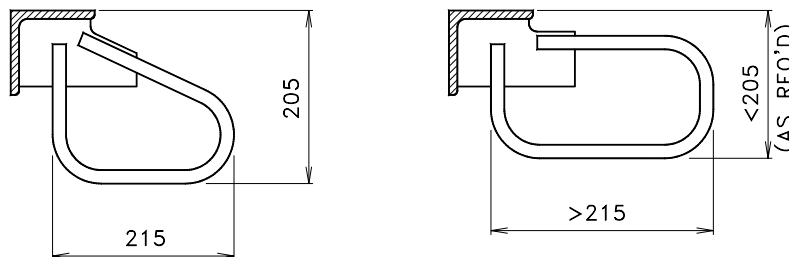
Note (*) :

Dimension on abutment side equal to ballast wall width.

Elastomeric concrete has been used in (strip seal) expansion joint rehabilitation projects by MTO for the past several years, specifically on structures (mainly post-tensioned concrete) where the depth of breakout available is insufficient for the installation of expansion joints anchored in normal concrete. However, its use has exposed inherent disadvantages of joints anchored in elastomeric concrete, some of which are as follows:

- Inadequate durability evidenced in early development of cracks, substantial deterioration in less than 10 years, and the occasional separation of elastomeric concrete from the armouring.
- Approximately 50% higher material cost of elastomeric concrete, as compared to normal concrete, on a per linear metre basis of expansion joint breakout.

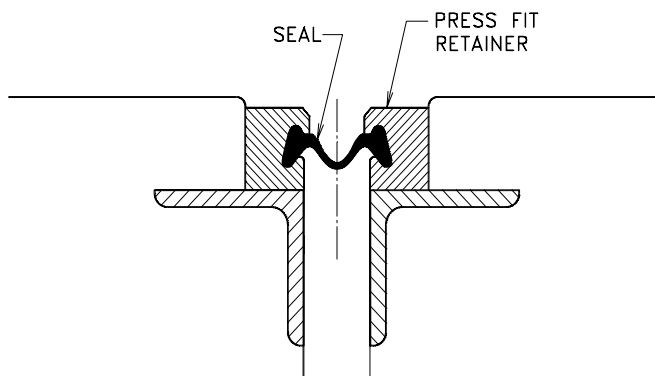
The designer should avoid the use of joints anchored in elastomeric concrete for joint replacement, whenever possible. Where expansion joint breakout depth is a problem (which is the case in most post-tensioned concrete decks), the anchorage assembly should be modified (by reducing the depth and correspondingly increasing the width of the loop anchors) as required, and anchored in normal concrete.



13.2.7 Strip Seals in Preformed Retainer Anchored on Existing Armouring

This list is for decks with concrete overlays where no other expansion joint assembly can be mounted on the existing armouring.

Where the existing anchorage has to be removed, the complete joint should be replaced by DSM 9.40.24, 9.40.27, or 9.40.28 joint assemblies where appropriate.



13.3.1 Rotational Bridge Bearings - General

Sections 13.3.1 to 13.3.9 provide a summary of the features of rotational bearings which could affect bearing selection and identify the bearing design data for rotational bearings that must be shown on the plans. They do not apply to plain or laminated elastomeric bearings.

The Designated Sources For Materials List DSM 9.15.70 for Bridge Rotational Bearings (except elastomeric bearings) will include three types of bearings divided into six classes:

- Class 1A. Pot Bearings Without Uplift Restraint Devices
- Class 2A. Disc Bearings Without Uplift Restraint Devices
- Class 3A. Spherical Bearings Without Uplift Restraint Devices

These bearings may be equipped with sliding surfaces for translation and guides for lateral restraint.

The need for uplift restraint devices is to be avoided and their use requires the approval of the Bridge Office Manager.

The selection of the bearings should be based on the articulation requirements of the structure. The requirements of the bearings such as translations, rotations, uplift, vertical and lateral load capacities and any special requirements must be established during the design process. It is expected that for a majority of bridges, any one of the three types of bearings will satisfy all the structural requirements. However there will be some bridges where, for special reasons, only one or two classes of bearings will be acceptable.

Expansion columns having bearings made of lubricated, unfilled TFE surfaces shall be proportioned based on a design coefficient of friction as given in Table 11.6.3.7 of the CHBDC.

13.3.2 Effect of Rotation on Eccentricity of Axial Load

When a bearing rotates about a horizontal axis there is a shift in the axial load from the centre of bearing and the load becomes eccentric. The shift in the axial load depends on the type of bearing.

For pot and disc bearings, the shift in the axial load from the centre of bearing depends on the properties of the elastomer and is difficult to calculate. Creep of elastomer has a beneficial effect in reducing the shift for disc bearings. Tests indicate that the shift can be 2% to 4% of the diameter of confined elastomer for pot bearings, and 2% to as much as 30% of the diameter of elastomer disc for disc bearings.

For spherical bearings, the shift "e", in the axial load, depends on the coefficient of friction m, and the spherical radius of curvature, "R", and is expressed by:

$$e = mR$$

Given the values of coefficient of friction specified in the CHBDC, and that the spherical radius of curvature "R" is generally between 1.0 and 2.8 times the plan diameter of the TFE curved surface, the shift in the axial load for spherical bearings can be 3% to 17% of the diameter of the TFE curved surface. At Serviceability Limit State Combination 1 loads and maximum rotation, OPSS 1203 limits the shift in the axial load from the centre of the bearing to the following values:

- (a) 3% of the diameter of the confined elastomer for pot bearings;
- (b) 10% of the diameter of the polyether urethane polymer compound for disc bearings;
- (c) 10% of the plan diameter of the curved TFE surface for spherical bearings.

The effects of the maximum permitted shift in the axial load must be considered in the design of the affected structural components. Where it is difficult or costly to provide for the 10% shift, the choice may be restricted to pot bearings.

13.3.3 Rotation Capacity

- (a) About any horizontal axis
 - Pot Bearings Maximum as required but not greater than ± 1
nor less than ± 0.5
 - Disc Bearings Maximum as required but not greater than ± 1
 - Spherical Bearings Maximum as required but not greater than ± 5 nor
less than ± 0.5
- (b) About vertical axis through centre of Bearing
 - Pot Bearings Maximum as required but not less than 1
 - Disc Bearings Maximum as required but not less than 1
 - Spherical Bearings Maximum as required but not less than 1

13.3.4 Translational Capacity

In unrestrained longitudinal direction - Maximum as required but not less than ± 50 mm.

In unrestrained transverse direction - Maximum as required but not less than $\pm 10\text{mm}$.

13.3.5 Orientation of Guided Bearings on Curved Structures

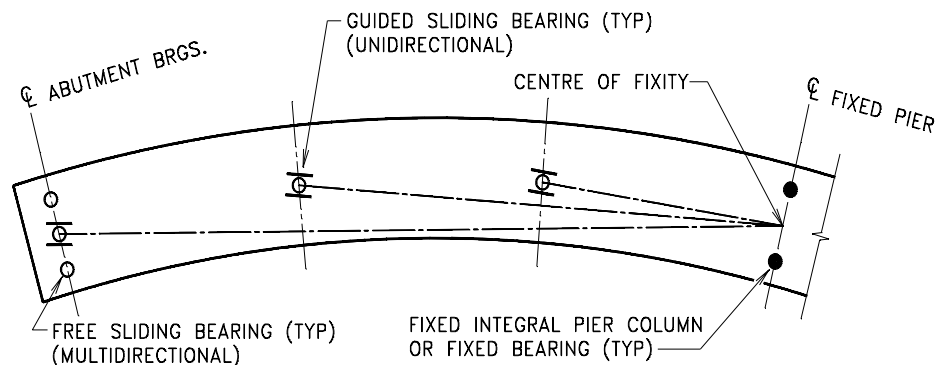
The designer should minimise the number of guided sliding bearings at each support (abutment or pier) location. One is usually adequate, and this should preferably be at the centre of the support. If multiple guided bearings are required at a support, the alignment should be the same for both and set to correspond to a point equidistant from each bearing.

Each support should have a minimum of 3 bearings, which would allow the centre bearing to be guided. If more are required for very wide and/or long-span bridges, odd-number of bearings should be used, again to allow for the centre one (only) to be guided.

In horizontally curved structures, two concepts generally exist when designing for horizontal movements. They are as follows:

- (a) The structure is guided to a Centre of Fixity by aligning guided sliding bearings parallel to the chord drawn from the bearing to the theoretical point of fixity (see Figure 13.3.5(a)). The Centre of Fixity is the point of zero movement of the superstructure for internally induced forces or movements due to temperature change, as well as creep and shrinkage if applicable. It is calculated by taking into consideration the combined stiffness of all supports including shear stiffness of bearings, flexural stiffness of supports (abutment or pier) and rotation of footings due to strain of piles or subsoil.

Bearing orientation A tends to accommodate the direction of the superstructure movements without introducing horizontal forces to the substructure due to these movements. However, for sharp or compound curves, this can introduce distortion at the abutment expansion joint for which it was not designed. One way to avoid this is to place the expansion joint perpendicular to the centroidal chord, but this is not recommended since it will necessitate skewed abutments.

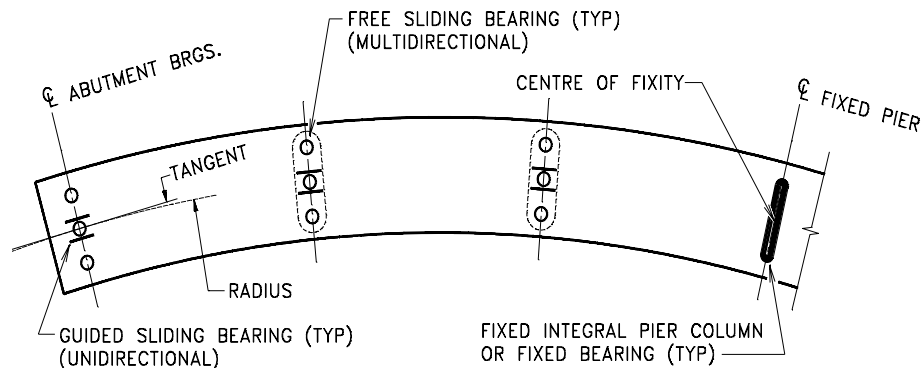


- Guided bearings aligned parallel to chord

Figure 13.3.5(a) BEARING ORIENTATION A

- (b) The structure is guided tangentially by aligning guided sliding bearings tangent to the radius of the structure at the support (abutment or pier) location (see Figure 13.3.5(b)).

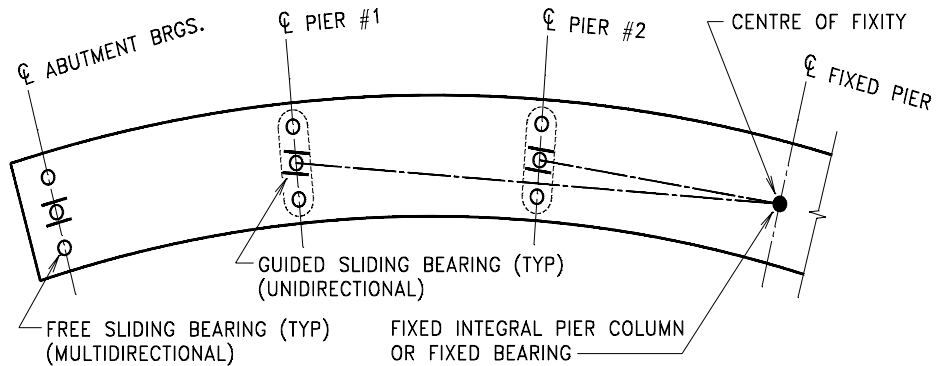
Bearing orientation B relies on the guided bearings to force the horizontal movement in the direction of the longitudinal curved axis of the superstructure, resulting in horizontal forces being transferred to the substructure through these bearings.



- Guided bearings aligned tangent to structure radius

Figure 13.3.5(b) BEARING ORIENTATION B

Both methods of bearing orientation have been used successfully in the past, each with their attendant benefits and disadvantages. The choice of method to use is left to the individual designer. However, since it is always better to minimise the introduction of additional forces or deformations into a structural system, the guided bearings at piers are recommended to be aligned as per Method A, while the guided bearings at abutments are recommended to be aligned as per Method B (see Figure 13.3.5(c)). As a result, because expansion joints at abutments are placed radially, they will expand and contract parallel to the abutment guided bearings without distortion.



- At Abutments, guided bearings are aligned tangent to structure radius
- At Piers, guided bearings are aligned parallel to chord

Figure 13.3.5(c)

Whichever method of bearing orientation is used in design, the designer is encouraged to use a refined method of analysis (such as STRUDL or OMBAS - General Structure) to determine the resulting horizontal movements and corresponding forces transferred to the substructure.

13.3.6 Lateral Load Capacity

(a) Rotational Elements

Lateral Loads as specified on plans in combination with the applicable vertical loads.

(b) Translational Elements with Lateral Restraints and Anchors

For bearings with a capacity of 5000 kN or less	Maximum as required but not less than 10% of factored vertical load
For bearings with a capacity over 5000 kN	Maximum as required but not less than 500 kN plus 5% of factored vertical load in excess of 5000 kN

When more than one bearing with guides for lateral restraint are provided at a pier or abutment, only one bearing shall be considered to resist the transverse loads.

13.3.7 Concrete Bearing Pressure

The average concrete bearing pressure used by the bearing manufacturer to proportion the bearing shall be based on a concrete strength of 30 MPa.

The concrete bearing surfaces above and below the bearing should be proportioned and reinforced when necessary to withstand these pressures and lateral loads.

13.3.8 Installation

As the elevation at the top of bearings is critical especially for bolting diaphragms of steel box girders, the tolerances for the elevation at the top of bearings as specified in OPSS 1203 are as follows:

Concrete deck or girders	2.5 mm
Steel plate girders	1.5 mm
Steel box girders	1.0 mm

The Specification requires all bearings to be bedded over their entire area on an approved grout to achieve the theoretical elevations within these tolerances. The thickness of the bedding grout shall not be less than 5 mm nor more than 15 mm.

The height of bearing assumed in establishing the bearing seat elevation should include 10 mm for grout bedding and the drawings should include the following note:

"Heights of bearings including 10 mm bedding grout assumed in establishing bearing seat elevations are as follows. The contractor shall adjust bearing seat elevations and reinforcing steel to suit actual heights of bearings.

Abutments	 mm
Piers	 mm "

Precautions must be taken by the Designer, when there is the possibility of bearings being installed under extreme cold or hot temperatures and not at 15°C. Either the top plate of a sliding bearing should be made sufficiently large to accommodate all of the maximum expansion and contraction movements, or alternatively the Designer should specify a temperature vs. setting table and corresponding details on the contract drawings.

13.3.9 Bearing Design Data

The Minimum bearing design data provided on the plans shall be as indicated below. Any special requirements must also be specified.

ROTATIONAL BEARING DESIGN DATA

LOCATION	TYPE	LIMIT STATE	COMBINATION	AXIAL LOAD	MAX. VERTICAL LOAD		MAX. ROTATION		MAX. TRANSLATION	
					LONGIT	TRANS	ABOUT HOR. AXIS	ABOUT VERT. AXIS	LONGIT	TRANS
			PERMANENT							
			PERMANENT + TRANSITORY MAX.							
			PERMANENT + TRANSITORY MIN.							
			PERMANENT							
			PERMANENT + TRANSITORY MAX.							
			PERMANENT + TRANSITORY MIN.							
			PERMANENT + EXCEPTIONAL MAX.							
			PERMANENT + EXCEPTIONAL MIN.							

*Applied horizontal loads shall be consistent with applied axial loads

The contract drawings shall include the following note:

"Rotational bearings shall be class/classes listed in DSM List 9.15.70 under the heading "Bearings, Bridge (Rotational)".

"The Contractor shall establish the bearing size such that contact pressure under permanent loads at SLS is not less than 25 MPa."

For bearings subject to uplift, the maximum permitted separation of the bearing components must be specified.

The maximum required translation ("") must be based on the assumption that, at the time of installation, the longitudinal and transverse centrelines of the bearing sliding plate will be set to coincide with the longitudinal and transverse centrelines of the bearing.

The coefficient of friction for PTFE sliding surfaces given in the CHBDC depends on whether the PTFE resin is "filled" or "unfilled". The type of PTFE resin assumed for design must be included with the bearing design data given on the contract

drawings, specifically giving the following note: "UNFILLED PTFE, LUBRICATED DIMPLED SHEET HAS BEEN USED IN THE DESIGN FOR ESTABLISHING THE BEARING COEFFICIENT OF FRICTION." This produces the minimum coefficient of friction. If for some reason the designer wants to transfer a greater than minimum horizontal force to a particular sub-structure, other sliding surface treatments given in the CHBDC may be considered.

13.3.10 Provision for Future Bearing Replacement

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

Connections, e.g. between bearings and shoe plates, must be bolted or make use of accessible screws and should only be welded when future access and removability can be guaranteed.

13.4.1 Elastomeric Bearings - Plain and Laminated

When choosing a bearing, the Designer should give priority to specifying an elastomeric type of bearing because of its superior long term durability.

Elastomeric bearings, plain and laminated must be supplied in accordance with OPSS 1202; bearing sizes should be restricted to the sizes given in OPSS 1202.07.03 as follows:

Size - mm x mm	*Max. SLS Load	*Max. ULS Load
300 x 200	420	600
350 x 250	610	875
400 x 300	840	1200
450 x 350	1100	1575
500 x 400	1400	2000
550 x 450	1730	2475
600 x 500	2100	3000
600 x 600	2520	3600

* Total load, given in kN

To ensure stability the following proportions shall be observed:

plain bearings min. L = 5 ERT
 min. W = 5 ERT

laminated bearing min. L = 3 ERT
 min. W = 3 ERT

where: L = plan dimension in the direction of shear.

W = plan dimension at 90E to direction of shear.

ERT = effective rubber thickness (the sum of the thicknesses of all elastomer laminates with shape factor <12)

Plain elastomeric pads should be used when the thickness is 25 mm or less. Laminated Elastomeric Bearings with thicknesses of 25 mm or less have difficulty in meeting the

Ministry's compression deflection requirements, therefore the minimum thickness for laminated elastomeric bearings shall be 40 mm.
The material for elastomeric bearings shall be shown on the drawings and should be based on where this structure is located as specified below:

South of and including latitude 45°: Natural Rubber or Neoprene

North of latitude 45°: Natural Rubber

13.4.2 Installation of Elastomeric Bearings

Elastomeric bearings are to be installed directly on the bearing seats. No filler or grout is to be shown on the drawings.

Precautions must be taken by the designer when there is the possibility of bearings being installed under extreme cold or hot temperature, and not at 15°C. The translational capacity of the elastomeric bearing shall be greater than the anticipated maximum movement.

For post-tensioned concrete structures, the use of a thicker elastomeric bearing may be a more economical choice than jacking the structure several months after stressing to release the shear deformation in the bearing.

13.4.3 Plain Elastomeric Bearings

Although the CHBDC permits the use of plain elastomeric bearings with thicknesses between 10 mm and 30 mm, the Bearings Committee feels that 10 mm is too thin and 30 mm is too thick. The thickness specified on the contract drawings should be between 15 mm and 25 mm. The shape factor must always be checked.

13.4.4 Strip Bearings for Box and Hollow Slab Girders

Strip plain elastomeric bearings may be used for precast box or hollow slab girder bridges. Thickness of the strip bearing shall not exceed 25 mm and only the durometer hardness needs to be considered.

13.4.5 Elastomeric Bearing Design Data

For elastomeric bearings, the minimum required design data provided on the contract drawings shall include the following:

- (a) SLS permanent load and SLS permanent plus maximum transitory load.
- (b) SLS maximum required translation and rotation.
- (c) ULS permanent load and ULS permanent plus maximum transitory load.
- (c) Maximum and minimum allowable design range of shear stiffness for the chosen bearing.

The rotational capacity of any bearing specified or supplied shall be not less than ± 0.5 degrees, and the edge vertical deflection under maximum rotation and vertical load shall not exceed 14% of the elastomer stiffness.

13.4.6 Laminated Elastomeric Bearing with Slider Plate Assembly

This type of bearing might be the most cost effective choice for a long and relatively lighter structure. For durability reasons, a slider plate should not be used when a thicker bearing without a slider plate would be adequate.

13.5.1 Supply of Additional Sample Bearings for Testing

To ensure the quality of the bearings supplied, standard special provision 910 requires that additional bearings be supplied by the Contractor for testing purposes.

The Bearings Work Group has given the following general guidelines for determining the number of bearings to be supplied for testing purposes, based on the number of bearings required for each structure and the bearing type. Note that each structure requires at least one bearing for testing if the bearings are of the plain or laminated elastomeric type.

I Elastomeric-Plain Bearings

- a. Less than 40 bearings on the structure.
1 bearing of the most common size.
- b. More than 40 bearings on the structure.
1 bearing per 40 bearings.

II Elastomeric-Laminated Bearings

- a. Less than 20 bearings on the structure.
1 bearing of the thickest size.
- b. More than 20 bearings on the structure.
1 bearing of the thickest size and
1 bearing of the most common size.
The two bearings supplied may be of the same size.
- c. More than 40 bearings on the structure.
The general guideline of 1 bearing for testing per 20 bearings on the structure should be followed, with a mixture of the various thicknesses and sizes being considered.

At the present time no additional bearings are required to be supplied for all other approved bearings (i.e. disc, pot, spherical).

Twin structures shall be considered as separate structures.

13.6.1 Shear Rate of Elastomeric Bearings

When an elastomeric bearing undergoes shear deformation due to horizontal loading, a horizontal force develops which is transferred to the substructure. The magnitude of this force is a function of the shear stiffness (shear rate) of the bearing at the time of displacement and the amount of bearing deformation.

The amount of deformation (due to superstructure movement) depends on the combined effects of the following internally induced forces:

- (a) heat of hydration (of concrete)
- (b) elastic shortening (of concrete due to prestressing)
- (c) shrinkage (of concrete)
- (d) creep (of concrete under prestress and permanent loads)
- (e) temperature change (produces expansion / contraction)

Note: - (a) to (d) are applicable to concrete structures only
- (e) is applicable to all structures

The shear stiffness of any elastomeric bearing depends partly on the hardness of the elastomer (constant) but mainly on the ambient temperature. The shear stiffness is fairly constant for all temperatures above freezing, but increases rapidly with decreasing temperatures. At -40°C , the shear rate can be two and a half times the stiffness at construction temperature.

In light of this, and since the various structure movements occur not only at its different ages (important when considering creep and shrinkage) but also at a variety of temperatures, the designer must use the bearing shear stiffnesses that correspond to each condition.

For bearing design data given on the contract drawings, the designer should give the shear rate that is neither $K_{\min}$ nor $K_{\max}$, but the stiffness that corresponds to a maximum allowable shear rate at 20°C and 55 durometer hardness.

Bearing catalogues now provide the minimum (at 20°C) and maximum (at -40°C) shear rates for all laminated elastomeric bearings available. Combining the appropriate shear rates, as indicated in the following paragraphs, with the various horizontal movements will result in a combined effective shear force which neither underestimates nor overestimates bearing behaviour. This then produces the force that is transferred to the substructure for which it must be designed.

- (A) For post-tensioned concrete structures, the superstructure may be jacked up at approximately three months after construction to relieve all horizontal shear deformation that has taken place in the bearing pad until that time, returning the bearing to its originally undeformed configuration. This eliminates all bearing movement due to heat of hydration, elastic shortening and a substantial proportion of the total shrinkage and creep. After the three-month jacking, the bearing must be designed for the remaining portion of shrinkage and creep, and all of the temperature change.

Because shrinkage and creep occur throughout the year, it is reasonable to use the average of $K_{\min}$ and $K_{\max}$ for those displacements, where $K_{\min}$ is the bearing shear rate at 20°C and $K_{\max}$ is the shear rate at -40°C .

For temperature drop, even though the bearing will experience a sustained shear deformation over an extended period of time during cold temperatures, $K_{\max}$ should not be used. Rather, since in responding to temperature drop, the bearing stiffness starts at near $K_{\min}$ and increases to $K_{\max}$ at the lowest temperature, the arithmetic mean of the full range of K values should be used. Therefore, for temperature drop, $K=1.35 \times (K_{\min})$ for natural rubber elastomers, and $K=1.60 \times (K_{\min})$ for neoprene elastomers should be used.

For temperature rise, since such bridge movements occur at a temperature above 15°C , $K=K_{\min}$ should be used.

(B) For post-tensioned concrete structures which are not jacked up after three months to relieve bearing deformations, all five internally induced forces produce horizontal deformation of bearings. The magnitude of this total force depends on the appropriate shear stiffness of bearing being used for each of the horizontal movements of bridge superstructure. The following are recommended:

- (a) heat of hydration: $K=K_{min}$
- (b) elastic shortening: $K=K_{min}$
- (c) shrinkage: $K=0.5x(K_{min}+K_{max})$
- (d) creep: $K=0.5x(K_{min}+K_{max})$
- (e) temperature drop: $K=1.35x(K_{min})$, [rubber]; $K=1.60x(K_{min})$, [neoprene]
- temperature rise: $K=K_{min}$

(C) For structural steel superstructures, only temperature change is a relevant internally induced force that causes shear deformation of bearings. The following is recommended:

- for temperature drop: $K=1.35x(K_{min})$, [rubber]; $K=1.60x(K_{min})$, [neoprene]
- for temperature rise: $K=K_{min}$

(D) For precast prestressed concrete bridges, heat of hydration and elastic shortening occur during fabrication, hence they do not affect bearing behaviour. We can assume that shrinkage (including differential) and creep-induced bridge movements will transfer horizontal forces to the substructure of a magnitude based on the average of K_{min} and K_{max} .

As with the other types of structures, bearing response to temperature rise should be based on K_{min} . For temperature drop, $K=1.35x(K_{min})$ [for rubber] and $K=1.60x(K_{min})$ [for neoprene] are recommended.

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14 CULVERTS

14.1.1 Culverts - General

Preparation of contract documents for culverts, both concrete and corrugated metal, is to be carried out by the Regional Offices.

In cases where standard structures cannot be utilised, the Regions may request the Bridge Office to carry out all or part of the design work.

14.1.2 Concrete Culverts, Cast-in-Place

All cast-in-place concrete culverts should be designed and detailed in accordance with the latest version of the "Concrete Culvert Design and Detailing Manual". In this manual the designs meet the requirements of CHBDC and the "Exceptions" in Division 1 of the Structural Manual and contains information necessary to complete the standard drawings and quantities for concrete box and open footing culverts. The following culvert types are covered:

- a) Rigid frame open footing culverts with fill heights of 0.6 m to 6.0 m and sizes 2.0 m span x 1.25 m height to 6.0 m span x 4.0 m height.
- b) Rigid frame box culverts with fill heights of 0.6 m to 6.0 m and sizes 2.0 m span x 1.25 m height to 6.0 m span x 4.0 m height.
- c) Non-rigid frame box culverts with fill heights of 0.6 m to 5.0 m and sizes 1.25 m span x 1.25 m height to 1.5 m span x 1.5 m height.

Reduced scale copies of the standard drawings for all of the above culvert types are available in the "Concrete Culvert Design and Detailing Manual."

14.1.3 Concrete Culverts, Precast

Precast concrete box culverts with spans greater than 3 m shall meet the requirements of the CHBDC.

Design specifications for precast culverts with spans up to and including 3 m are being prepared in OPSS 1821 "Material Specifications for Precast Reinforced Box Culverts and Box Sewers." Until these design specifications are available such culverts should be designed to CHBDC requirements and the "Exceptions" given in Division 1 of this manual.

Precast box culverts utilising reinforcing bars shall be supplied with haunch reinforcement.

14.1.4 Epoxy Coated Bars in Culverts

The use of epoxy coated bars in culverts is restricted to those having less than 500 mm of fill height (See 12.1.2).

14.1.5 Earth Pressure

Culverts and other structures where the deflection of the side walls is prevented by the propping action of slabs shall be designed for the at-rest earth pressure.

14.1.6 Design of Concrete Culverts

Beneficial effects of axial compression in flexural reinforced concrete culvert components shall be taken into account in proportioning the reinforcement whenever it is practical to do so.

14.2.1 Corrugated Steel Pipe Products - Dimensioning

Except for lengths, all dimensions of corrugated steel pipe products must be given in metric units, for both metric and imperial contracts.

14.2.2 Length of Structural Plate Pipes and Pipe Arches

The overall length of structural plate pipe and pipe arches shall always be detailed on the drawings and the dimension shown shall always be a multiple of 610 mm, + 100 mm.

e.g. 61 100 mm is acceptable

This permits the use of an integral number of standard plate sizes without cutting, and ensures that the hook bolts in the cut-off walls will be aligned with the standard circumferential holes in the plates.

The spacing of hook bolts, if these are required, should be a multiple of 244 mm, to match the spacing of standard holes in the plates.

14.2.3 Corrugated Metal Culverts

All soil steel structures, 3.0 m and over, must be designed to the CHBDC. Site preparation must be such that there is an area on each side of the structure equal to half of the culvert span which will require engineered soils.

The use of pipe arches is restricted to sites at which round pipes cannot be accommodated. Other alternatives should also be considered.

It is necessary that the installation of pipe arches make provision for the necessary backfill volume of engineered soils (granular 'A') on each side of the structure and that each installation have cut-off walls or headwalls on each end.

Metal culverts under 3.0 m are not controlled by this procedure; but attention is drawn to the importance of site preparation and the use of properly compacted backfill material.

15 WOOD STRUCTURES

15.1.1 Metric Sizes

Finished metric sizes for dressed wood are as follows:

Joist & Plank Lumber mm.	Rough Lumber mm.	Surfaced Timber mm.
38	51	-
64	76	-
89	102	-
114	127	114
140	152	140
165	178	165
184	203	191
-	229	216
235	254	241
-	279	-
286	305	292
337	356	343
387	406	394
-	457	445
-	508	495

The "Metric Handbook for Canadian Softwood Lumber" gives metric sizes for all available sections as shown above and for all other available lumber products.

15.2.1 Pressure Preservative Treatment for Wood

The Bridge Office policy with regard to the use of wood preservatives is as follows and shall be specified, as shown below, on the drawings.

Note:

Preservative Treatment of Wood

Treatment shall be in accordance with CSA Standard 080 M 1983 and AWP Standards M1, M2 and M4. Only oil-borne preservatives shall be used. The type and retention shall be as follows:

- (1) Piling - Creosote - 130 kg/m (8#/cf) retention.
- (2) All other wood:
Creosote - 130 kg/m (8#/cf) retention
or
Pentachlorophenol - 6 kg/m (0.4#/cf) retention.

Chromated Copper Arsenate and Ammoniacal Copper Arsenate are acceptable preservatives for wood, other than piling, at a retention of 6 kg/m (0.4#/cf) but are not available at present in oil borne form.

The standard construction note for handling of treated wood and treating of field exposed surfaces is given in 2.5.7.

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16 MISCELLANEOUS

16.1.1 Approach Slabs on Structures

- a. 6000 mm approach slabs shall be used on structures on all paved roads and on roads that are to be paved in the near future.
- b. Provision shall be made for the future installation of 6000 mm approach slabs on all structures having concrete or steel decks.
- c. Approach slabs are to have a wearing surface with no waterproofing. See 2.5.7 for note to appear on the Plan on the General Arrangement drawing.
- d. 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 must also carry the note:

"The approach slabs are not part of this contract".

Stock-piled projects that carry a note having the same intent as the above but with different wording, must be altered.

16.2.1 Transportation and Fabricator Handling of Structural Components

Prefabricated, indivisible structural components that exceed (including the vehicle) any of the following limitations of

Length 19.0 m
Width 3.5 m
Height 2.6 m
Weight 30,000 kg

shall require Special Oversize/Overweight hauling permit(s) in order to be transported by highway carrier over King's highways.

For the following two categories: (A) Routine Oversize/Overweight loads, and (B) Non Routine Oversize/Overweight Loads, pre-approval for King's highways transportation is not required. The Length/Width/Height/Weight limitations for both categories are detailed in S.P. No. 109F16, to which reference should be made.

Components exceeding the limitations of categories (A) or (B) require that the designer of the Load obtain transportation approvals from the Weight and Load Engineer, Carrier safety and Enforcement Branch, MTO and other authorities, all as required by S.P. No. 109F16.

As early as possible in the design stage, written request for road-transportation shall be submitted to the Oversize / Overweight Permits Office for an in-principle special permit approval. The request shall indicate the number of components, dimensions, weights, travel origin and destination, duration of transportation and required travel frequency. The transportation approvals may take up to four (4) weeks and may not be granted if transportation is deemed to be unsafe.

16.3.1 Services Carried Through or Under Bridges

Fluid carrying pipe lines are not normally allowed to be carried through or under bridges, unless approved. This includes oil and gas pipelines, sanitary and storm sewers and water mains.

Electrical power lines may be carried through, under, or over bridges provided the voltage does not exceed 44,000 volts.

The following details show approved details for the accommodation of non-MTO utilities on bridges. Bridges shall be used for this purpose only as a last resort and after the proponent has carried out a cost-benefit analysis. Schemes other than those shown below may be considered, subject to the approval of MTO.

Utility plant is not allowed in sidewalks or to be directly suspended from thin deck slabs. Also, utility hardware must not be placed in a location which prohibits routine inspection of structural components. In any case, it must not be located below the underside of girders for slab-on-girder bridges, or the underside of deck for slab-type bridges.

Steel components that make up the duct support system for bridges having an all concrete superstructure or for bridges having painted steel girders, must be stainless steel, galvanised, metallised or painted. For bridges having atmospheric corrosion resistant (ACR) steel girders, all non-embedded components may be fabricated of the same steel. Attachments shall not be welded to flanges of superstructure girders.

Where necessary, the designer must also provide special details to allow the utility ducts to pass through the abutment ballast walls and superstructure diaphragms. These details must ensure (i) that the ducts will be able to accommodate all prescribed structure movements including deck jacking for bearing repairs, (ii) that fill material from behind the ballast walls is not washed out, and (iii) any settlement that may occur in the fill material behind the ballast wall does not impose undue stresses on the ducts. When the utilities are run through a steel box girder, the designer must consider the location of intermediate cross-braces, vertical stiffeners in pier diaphragms, access openings in pier diaphragms, etc. All utilities shall be designed with provisions to allow for future bridge maintenance.

Duct Location For Structures With Accessible Cavities Or Spaces Located Within The Limits Of The Perimeter Of The Superstructure

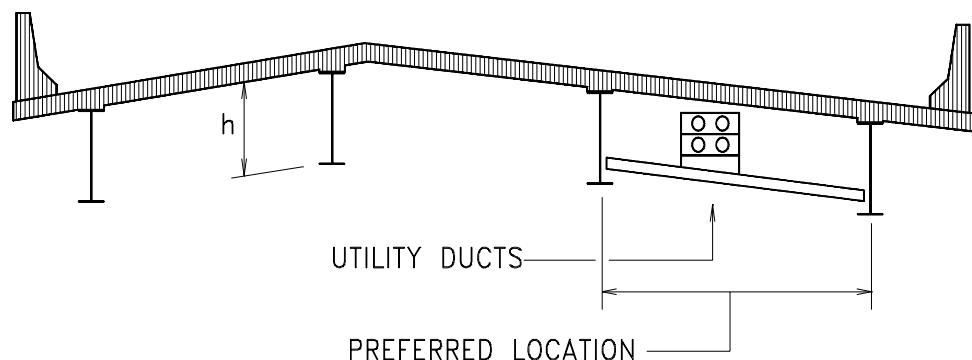


Figure 16.3.1(a) Slab-on-concrete/steel I type girders

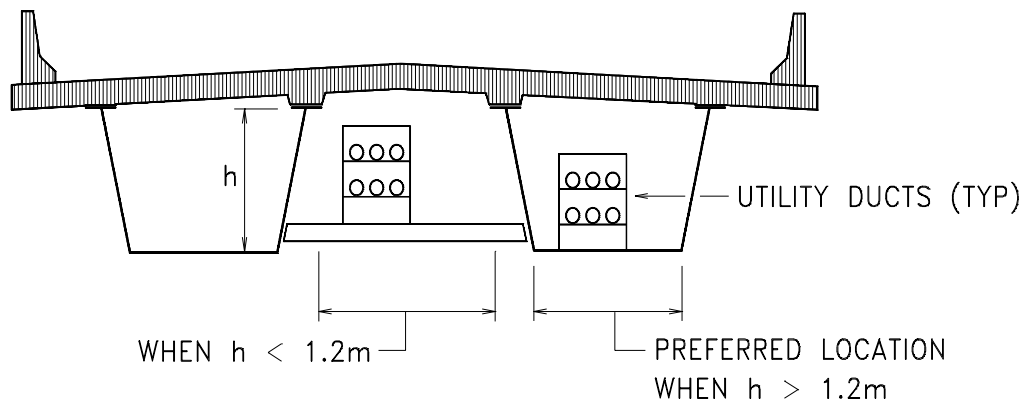


Figure 16.3.1(b) Slab-on-concrete/steel box girders

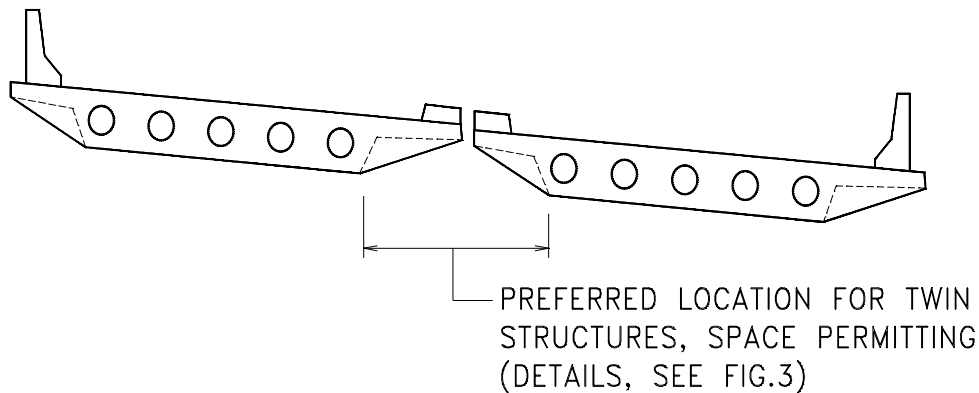


Figure 16.3.1(c) Twinned round-voided post-tensioned decks

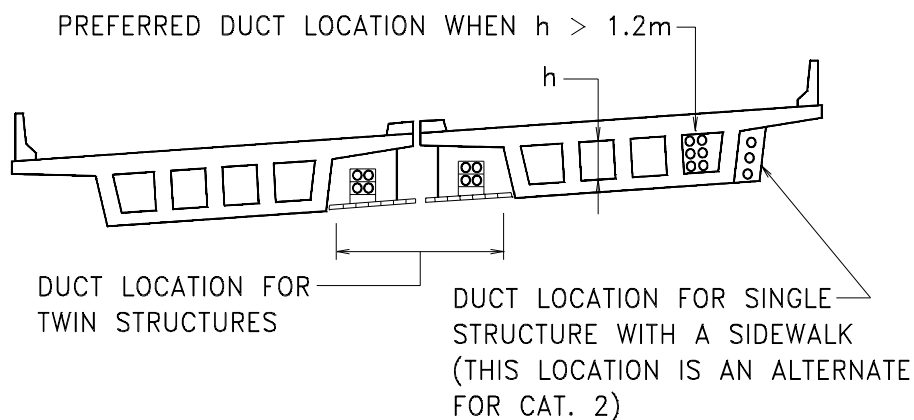
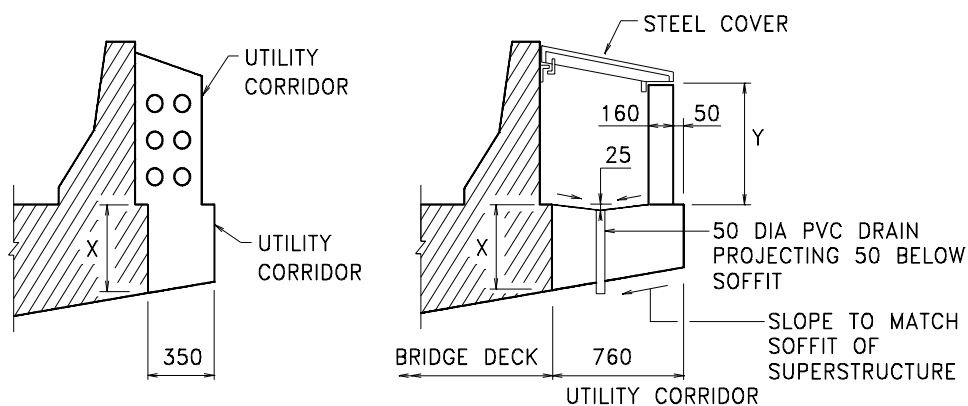


Figure 16.3.1(d) Trapezoidal box post-tensioned decks

Duct Location In Slab Type Structures, Superstructures Without Accessible Cavities

Projects covered by this criteria require the construction of a non-structural utility corridor outside of and adjacent to the barrier wall. Structure types included in this category include rigid frames, culverts, and solid or round-voided post-tensioned concrete structures. This scheme is best suited for structures without sidewalks or situated in non-urban locations because of the possibility of pedestrians climbing up onto the utility corridor.



SMALL NUMBER OF DUCTS
NON-ACCESSIBLE

LARGE NUMBER OF DUCTS
PERMANENTLY ACCESSIBLE

Structure Type	Sidewalk	X	Y
Slab	with	500	625
	without	350	825
CIP, P/T deck: - tapered cantilever	with	600	625
	without	600	825
	with	600	625
	without	450	825

Figure 16.3.1 (e) Cantilevered utility corridor

Electrical lighting ducts (incl. MTO utilities) shall not exceed 50 mm diameter, except when installed in barrier walls, and shall be placed in accordance with the following criteria:

1. In abutments, ballast walls, slab piers and rigid frames, members must be greater than 500 mm in thickness and spacing of adjacent ducts shall be not less than 6 m.
2. In round columns:
 - Maximum of one electrical duct, outside diameter not exceeding 50 mm, may be placed at centre of columns having a diameter of 1000 mm or more.
 - Junction boxes are not permitted in columns.
 - Entrance/exit of duct at the bottom of columns to be through the footing
 - Entrance/exit of duct at the top of columns:
 - for fixed-fixed columns (no bearings at top) the duct shall pass straight through the deck-column interface.
 - for columns with bearings at top, duct may pass through side of column, provided it does not interfere with reinforcement or bearing assembly and is kept clear of the zone of high stress near the bearings.

3. In post-tensioned decks:

- Ducts and junction boxes may be placed in post-tensioned decks provided they are located in solid portions of the deck adjacent to supports; they must be placed above the bottom mat of reinforcement (and transverse post-tensioning, when present).
- Longitudinal ducts placed in post-tensioned decks may be located inside deck voids, but in any case shall not be located in web portions of the deck.

When ducts are to be placed in such elements, positive drainage of the embedded duct work must be assured and the ducts must be located behind the steel reinforcement. The structural engineer shall also give consideration to the location of junction boxes necessary in such installations and to the potential effects of duct bursting on the structural integrity of the component(s). Electrical ducts left unused would need to be grouted as is required with unused post-tensioning ducts.

Other services may be provided for only as a result of specific approvals and agreements. These, and approval for exemption from above restrictions, should be referred to the Assistant Deputy Minister of Operations.

16.3.2 Electrical Grounding of Structures

Railings attached to a structure must be electrically grounded if the structure carries electrical power conductors or provision for them. The Electrical Engineering Section details the grounding.

Bridges having lighting ducts require grounding but those carrying only telephone ducts or cables do not.

High level bridges may require lightning protection. Such structures should be referred to the Traffic Management and Engineering Office, Electrical Engineering Section.

16.4.1 Temporary Modular Bridges

The use of Temporary Modular (TM) bridges (Brand Names: Acrow, Bailey, etc.,) is primarily for installations generated from emergencies in the ministry or municipalities, or for seasonal use as detours at ministry construction sites. The use of TM bridges for permanent structures is discouraged.

The responsibility and control of the supply, maintenance and inventory of all Temporary Modular bridge components under the jurisdiction and ownership of MTO is governed by the following Policy, Planning & Standards Directive PLNG-B-007 (formerly Quality and Standards Directive QST-B-20 "*Temporary Modular Bridges*"), revised August 2000:

"Temporary Modular Bridges":	(A)	<i>Issuing Priorities</i>
	(B)	<i>Supply, Maintenance and Inventory Control</i>
	(C)	<i>Retrieval, Rental and Disposal "</i>

A summary of the contents of this document is given in this subsection.

Municipality Needs

The ministry will only supply TM bridges to municipalities when there is a real emergency. The ministry will not supply TM bridges to municipalities for detour purposes when new bridges are being constructed or for bridge rehabilitations.

Municipalities that need TM bridge components in an emergency shall initially attempt to obtain their requirements from the private sector. If the private sector is unable to supply the components within one week from the date of the emergency, the municipality may request them from the ministry. When submitting a request to the ministry, the municipality must provide evidence that they were unable to obtain the TM components in a timely manner. The procedure to follow when making an emergency request and details of a legal TM bridge rental/purchase agreement that has to be entered into is given in Directive PLNG –B-007.

Other Agencies

The ministry will not supply TM bridges to any other jurisdictions.

Issuing Priorities

The Regional Director for the region in which the TM bridge is required is responsible for approving the release of TM bridges. The Manager of Operations, Northern Region, North Bay is responsible for issuing TM bridges. Issuing priorities will be in the following order of precedence:

1. Emergency needs on the ministry's Provincial Highway System
2. Emergency needs in municipalities
3. Detour needs on the ministry's Provincial Highway System

Emergency Needs

To ensure a responsible supply of TM bridge components sufficient to meet most anticipated emergencies, the ministry maintains and services an emergency stock of TM components at a stockyard in Northern Region.

An emergency may exist due to failure of any bridge component or collapse of an existing bridge that leads to a permanent closure of the bridge. An emergency requiring the issue of TM bridge components is deemed to exist when one or more of the following conditions are satisfied:

- (a) An existing structure has been damaged beyond immediate repair.
- (b) The road is completely closed to traffic due to damage to the bridge.
- (c) The shortest detour that exists exceeds what would normally be considered reasonable for purposes of emergency response (i.e. fire, police and ambulance.)
- (d) The time required to make repairs to the existing structure is considered excessive.
- (e) Alternative solutions, other than the supply of TM bridge components are not viable.

The approval procedure for emergency requests at ministry and municipality sites, prior to the release of TM bridge components, shall be as detailed in Directive PLNG-B-007.

Detours and Non-Emergency Needs

At present, the ministry maintains and services a portion of the existing inventory of TM bridge components for use as detours at ministry construction sites. This stock is kept separate from that intended for emergency use. However, this detour stock may be depleted by attrition with time and the supply of TM components for operational needs will eventually be obtained from the private sector.

The need for a TM bridge and sufficient information for design must be clearly stated in a Structural Planning Report. This will be the Structural Planning Report for a structural project

for which a TM detour is required or, if the TM bridge is not part of such a project, a separate report.

TM bridge drawings, quantities, and appropriate tender items are to be prepared by the Regional Structural Section responsible for the project. Site numbers should be assigned with the latter B added, e.g. 34-2221/B in the title block (see Section 2.5.1.)

During the preliminary stages of design, the Regional Structural Section shall obtain confirmation from the Manager of Operations, Northern Region that the required quantities of TM bridge components will be available when needed. If this is not possible then consideration shall be given to alternative designs from the private sector.

The approval procedure, bills of materials and requests for the release of TM bridge components shall be as detailed in Directive PLNG-B-007.

16.5.1 Noise Barrier Design

Noise Barriers shall be designed in accordance with the Canadian Highway Bridge Design Code as light slender structures not unusually sensitive to wind action, and CSA-Z107.9, Standard for Certification of Noise Barriers. Wind loads and ice accretion loads on panels shall be as prescribed for sign panels. Reference wind pressures for a 25 year return period shall be used.

In the calculation of section properties and strength for cold formed steel members, for which the provisions of the CHBDC are not applicable, the requirements of CSA-S136 for ultimate limit state design shall apply.

In evaluating or designing a structure on which a noise barrier is to be mounted, the above criteria shall apply, except that the local reference wind pressure shall always be used and a gust factor of 2.0 is sufficient for the relatively rigid structures (e.g. retaining walls), to which noise barriers are generally attached.

16.6.1 Rock Protection

In the OPS specifications, the material previously specified as "random rip-rap" is now "rock protection".

The following will therefore apply:

- a. All drawings presently being prepared for contracts should use the description "Rock Protection".
- b. In the planning report a statement will have to be made as to whether broken rock can be used as rock protection or not.

Stockpiled projects should be updated to comply with the above.

16.6.2 Slopes

The normal slope for permanent earth cuts and fill is 2:1. For appearance, to facilitate grass cutting or to dispose of surplus or unsuitable (e.g. organic) fill, slopes flatter than this are often used; but these reasons do not justify increasing the length of a bridge nor the height of an abutment.

Slopes steeper than 2:1 are not used under bridges without surface protection. Rock fill can be stable at a slope of 1.25:1 and slopes with rock protection can be stable at 1.5:1. Such intended slopes must be approved by the Foundations Section at the preliminary design stage.

16.6.3 Slope Paving

All inclined earth surfaces bounded by the vertical projection of the deck onto the ground below shall be paved up to the face of the abutment.

16.7.1 High Mast Lighting Poles

The design of high mast lighting poles and their foundations shall be based on the requirements of the Canadian Highway Bridge Design Code. A reference wind pressure for a 50-year return period shall be used. AASHTO's "Standard Specifications for Structural Supports for Highway Signs, Luminaries and Traffic Signals" shall also be used for design requirements, for example fatigue design, not specified by CHBDC. However, where there is a conflict the CHBDC shall govern.

On Ministry projects pole heights of 25, 30, 35, 40 and 45m are currently used and the design for these high mast lighting supporting structures has already been carried out. In April 2002 MTO issued standard drawings showing details of these structures. General details are shown on the following MTO standards:

MTOD-2453.000 High Mast Lighting Pole 25m, 30m and 35m 8-sided pole
MTOD-2453.030 High Mast Lighting Pole 40m and 45m 12-sided pole

High mast lighting poles are supplied by a Vendor of Record (Equipment Supply Contractor) for Ministry contracts. Details of the supply can be found in Special Provision No. 630F02.

Compared to previous standards, the circular pole cross-sections have been eliminated and the octagonal cross-sections have been replaced by 8 and 12-sided sections with the cross-sectional width of the poles at the base increasing significantly. The diameters of the base plates have also changed, and a single base plate has replaced the double base plate.

The installation of high mast poles is covered by OPSS 630 "Construction Specification for the Installation of High Mast Poles" and related special provisions.

High mast lighting supports have fixed based support systems that do not yield or break away upon impact. The large mass of the poles and the potential safety consequences of them falling to the ground necessitate a fixed base design. Since fixed base systems are rigid obstacles they should not be used in the roadside clear zone unless protected by a barrier.

16.7.2 High Mast Lighting Pole Foundations

An anchorage assembly detailed on the following standards connects the high mast lighting support to a concrete caisson foundation:

MTOD-2218.010 High Mast Lighting Pole Anchorage Assembly
MTOD-2453.050 High Mast Lighting Pole Anchorage Assembly Details

The design of foundations shall be based on the requirements stated in the above clause 16.7.1.

Foundations for high mast poles may be designed according to the Bridge Office publication BRO-006, "Guidelines for the Design of High Mast Pole Foundations, third edition," which is based on the requirements of clause 16.7.1 above. This guide presents the design of concrete caisson type foundations in a variety of soil conditions including rock and layered soils. The designs and design methods given in this document are modeled on short piles with a

rotational limit of 0.005 radians at the ground surface, and the theoretical analyses given in papers by Bengt Broms and others.

The following standard drawings for high mast pole foundations that are ground or median mounted are available for use:

- SS116-50 High Mast Lighting Pole Footing Ground Mounted
- SS116-51 High Mast Lighting Pole Footing Median Tall Mounted (Symmetrical)
- SS116-52 High Mast Lighting Pole Footing Median Tall Mounted (Asymmetrical)

The construction of foundations for high mast poles is covered by OPSS 631 "Construction Specification for Concrete Footings for High Mast Poles" and related special provisions.

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17 COMPUTER APPLICATIONS

17.1.1 List of Computer Programs

The programs supported currently by the Bridge Office are listed below:

PROGRAM	DESCRIPTION	MAINFRAME	UNIX	PC
BR00310	Bridge Quantities		A	R
BR00340	Reinforcing Steel Bar Schedule		A	A
BR00470	Highway Geometry		A	R
BR00810	Bridge Detailing Functions		A	R
BR0013010 to BR0013014	Ontario Structure Clearance and Load restriction Information System (OSCLIS)	A		
OMBAS	Ontario Modular Bridge Analysis System (OHBDC)		A	A
CANBAS	CANadian Bridge Analysis System (CHBDC)		A	A
OBMS	Ontario Bridge Management System: Restricted to MTO and consultants hired by MTO			A

Notes:

1. **A** currently available. **R** conversion to PC platform under review.
2. BR0013010 to BR0013014 (OSCLIS) are on the MTO mainframe computer.
3. **OMBAS/CANBAS** PC versions are available for licensed use on individual machines. Please contact the Bridge Office for details.
4. The final version of **OMBAS** will be 6.4.X.
5. **CANBAS** is available currently as a beta version till the production version is available in late 2002.

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18.1.1 Sign Support Structures

The design and detailing of standard sign support structures is to be carried out in accordance with the latest version of the Sign Support Manual.

The Sign Support Manual contains information needed to prepare the contract drawings, tender quantities and special provisions for standard sign supports. The design of the Cantilever, Tri-Chord, and CMS sign supports meets the requirements of the CHBDC.

All non-standard sign supports must be custom designed.

The standard sign supports covered by the Sign Support Manual are:

Overhead Sign Supports

a. Cantilever Static Sign Supports

A sign support which cantilevers from the roadside over the closest driving lane and shoulder for static signs. This sign support is suitable for signboard sizes up to and including 25 m², depending on the eccentricity of the centreline of the signboard. The structure is constructed of galvanised structural steel and consists of an overhead truss supported on a single leg column with a concrete caisson type foundation.

b. Tri-Chord Static Sign Supports

A sign support that crosses the highway with spans ranging from 14 m to 36 m inclusive for static signs. This sign support is suitable for signboard sizes up to and including 45 m². The structure is constructed of galvanised structural steel and consists of a three-chord overhead truss supported on single leg columns with concrete caisson type foundations.

c. Monotube Sign Supports

A sign support that crosses the highway with spans ranging from 13.5 m to 24 m inclusive for lane designation signs. This sign support is suitable for signboard sizes up to and including 1200 mm by 1200 mm. The structure is constructed of galvanised structural steel and consists of four tapered, octagonal monotubes with concrete caisson type foundations

d. Changeable Message Sign Supports (CMS)

A sign support that crosses the highway with spans ranging from 17.6 m to 34 m inclusive for changeable message sign systems. This sign support is suitable for sign systems up to and including 40 m². The structure is constructed of an overhead aluminum truss supported on galvanised structural steel columns with concrete caisson type foundations

e. Bridge Mounted Sign Supports

A sign support mounted on bridges for static signs ranging in depth from 1525 mm to 2740 mm inclusive and width as required. The structure is constructed of aluminium struts and arms connected to the bridge by stainless steel anchors.

Roadside Sign Supports

f. Steel Column Sign Supports

A roadside static sign support that can be used for signs ranging in depth from 1200 mm to 3600 mm and in width from 3000 mm to 7200 mm. The structure is constructed of galvanised structural steel and consists of rolled steel crossarms and vertical columns embedded in concrete filled augured holes.

They can be of the breakaway or non-breakaway type.

g. Timber Sign Supports

A roadside static sign support that can be used for signs ranging in depth from 1200 mm to 2700 mm and in width from 2700 mm to 6000 mm. The structure is constructed of pressure preservative treated Douglas Fir or Jack Pine and consists of signboards connected by steel connector plates to two or more timber posts embedded in concrete filled augured holes.

All timber sign supports are of the breakaway type.

Reduced scale copies of the standard drawings for all of the above structures are available in the "Sign Support Manual."

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STRUCTURAL MANUAL

DIVISION 3 - DESIGN AIDS

December 2003

DECEMBER 2003	LIST OF DESIGN AIDS	DA 1-1

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NOTE: Design Aids marked "**WITHDRAWN**" are under review for future inclusion in this manual.

q = EQUIVALENT FLUID PRESSURE (kPa)
 p = UNIT PRESSURE AT DEPTH H METRES
 P = TOTAL FORCE PER LINEAR METRE OF WALL
 M = OVERTURNING MOMENT (kN-m)

AT SERVICEABILITY LIMIT STATES

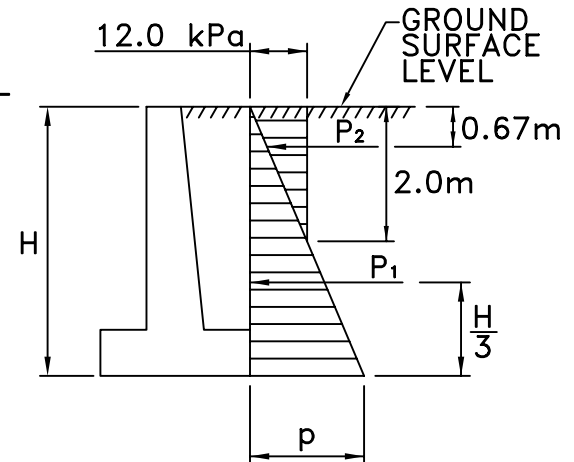
$$\alpha_E = 1.0$$

$$q = 7.0 \times (H)$$

AT ULTIMATE LIMIT STATES

$$\alpha_E = 1.25$$

$$q = 1.25 \times (7.0) \times (H)$$



$$\phi = 30^\circ$$

$$P = P_1 + P_2$$

– RESULTS INCLUDE THE EFFECT OF COMPACTION SURCHARGE

SLS			
H (m)	p (kPa)	P (kN)	M (kN-m)
1.0	7	13	6
1.5	11	19	14
2.0	14	26	25
2.5	18	33	40
3.0	21	44	59
3.5	25	55	84
4.0	28	68	114
4.5	32	83	152
5.0	35	100	197
5.5	39	118	251
6.0	42	138	315

ULS			
H (m)	p (kPa)	P (kN)	M (kN-m)
1.0	9	16	8
1.5	13	24	18
2.0	18	33	31
2.5	22	43	50
3.0	26	55	74
3.5	31	69	105
4.0	35	85	143
4.5	40	104	190
5.0	44	125	246
5.5	48	148	314
6.0	53	173	394

THE METHOD OF EQUIVALENT FLUID PRESSURES IS LIMITED TO A MAXIMUM HEIGHT OF 6.0m. FOR RETAINING WALLS WITH HEIGHTS > 6.0m, THE EARTH PRESSURE DISTRIBUTION SHALL BE ESTABLISHED BY A GEOTECHNICAL ENGINEER.

q = EQUIVALENT FLUID PRESSURE (kPa)
 p = UNIT PRESSURE AT DEPTH H METRES
 p^1 = UNIT PRESS. DUE TO BACKFILL AT DEPTH H
 p^2 = UNIT PRESSURE DUE TO 0.8m SURCHARGE
 P = TOTAL FORCE PER LINEAR METRE OF WALL
 M = OVERTURNING MOMENT (kN-m)

AT SERVICEABILITY LIMIT STATES

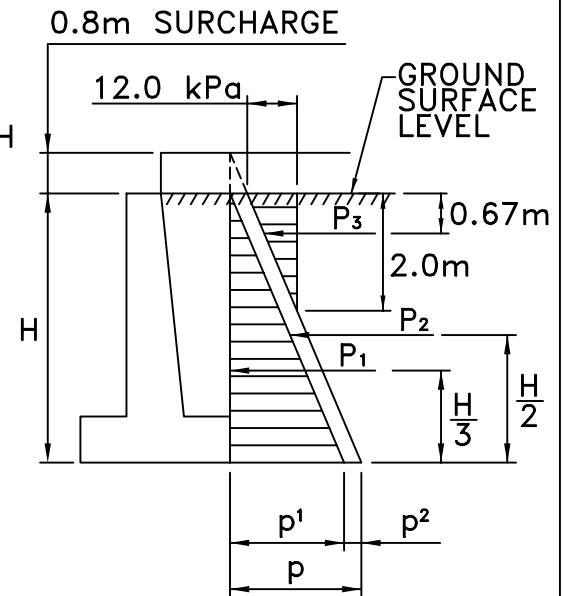
$$\alpha_E = 1.0$$

$$q = 7.0 \times (H)$$

AT ULTIMATE LIMIT STATES

$$\alpha_E = 1.25$$

$$q = 1.25 \times (7.0) \times (H)$$



$$\phi = 30^\circ$$

$$P = P_1 + P_2 + P_3$$

$$p = p^1 + p^2$$

– RESULTS INCLUDE THE EFFECT OF COMPACTION SURCHARGE

SLS			
H (m)	p (kPa)	P (kN)	M (kN-m)
1.0	13	18	9
1.5	11	28	20
2.0	20	37	37
2.5	23	48	58
3.0	27	60	85
3.5	30	75	118
4.0	34	90	159
4.5	37	108	209
5.0	41	128	268
5.5	44	149	337
6.0	48	172	417

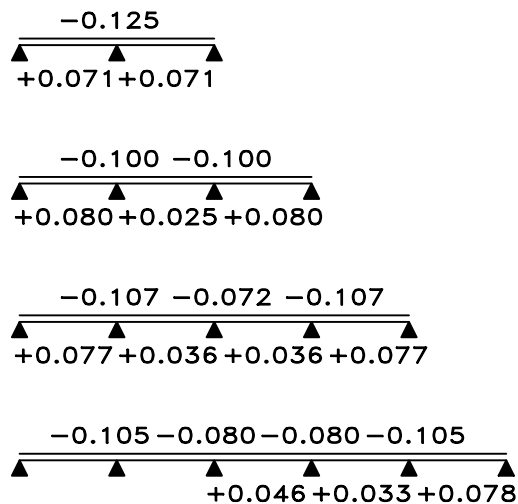
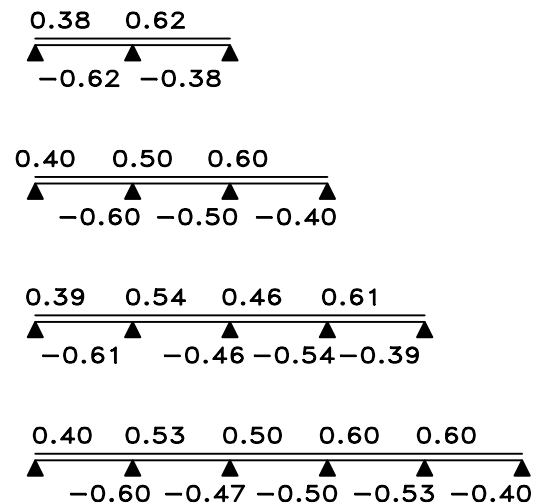
ULS			
H (m)	p (kPa)	P (kN)	M (kN-m)
1.0	16	23	11
1.5	14	35	25
2.0	25	46	46
2.5	29	60	73
3.0	34	75	106
3.5	38	94	148
4.0	43	113	199
4.5	46	135	261
5.0	51	160	335
5.5	55	186	421
6.0	60	215	521

THE METHOD OF EQUIVALENT FLUID PRESSURES IS LIMITED TO A MAXIMUM HEIGHT OF 6.0m. FOR RETAINING WALLS WITH HEIGHTS > 6.0m, THE EARTH PRESSURE DISTRIBUTION SHALL BE ESTABLISHED BY A GEOTECHNICAL ENGINEER.

BENDING MOMENT = coefficient $\times w \times L^2$
 SHEAR FORCE = coefficient $\times w \times L$
 w = U.D.L.
 L = SPAN

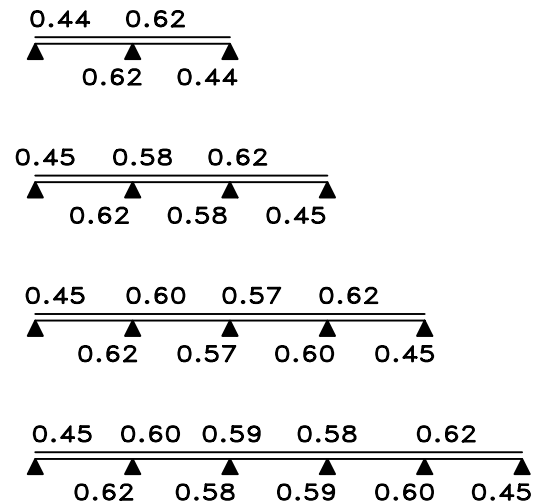
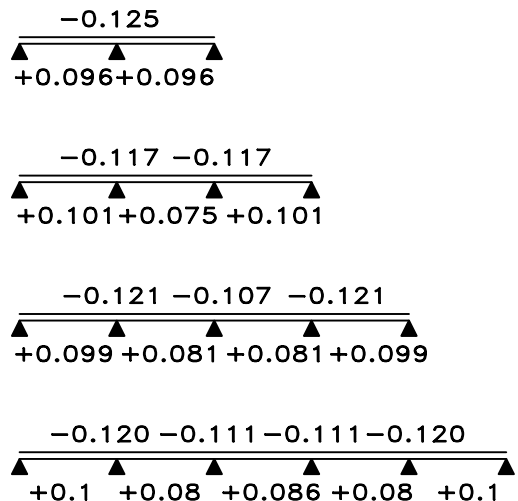
COEFFICIENTS FOR MAXIMUM BENDING MOMENT AND SHEAR

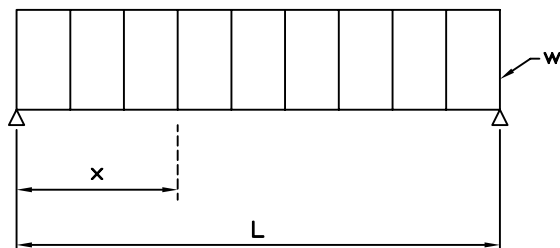
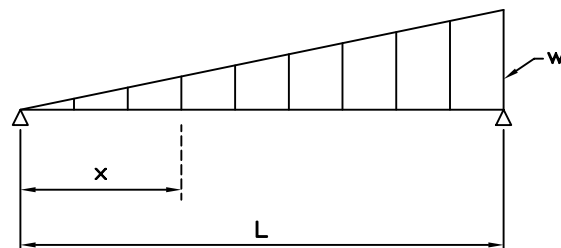
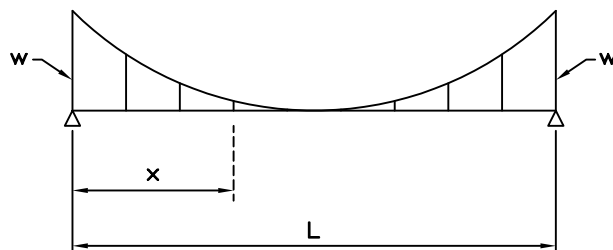
DEAD LOAD (ALL SPANS LOADED)

BENDING MOMENTSHEAR

LIVE LOAD

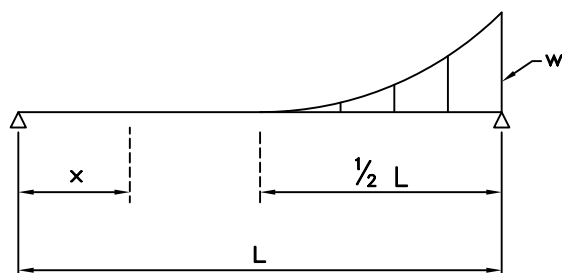
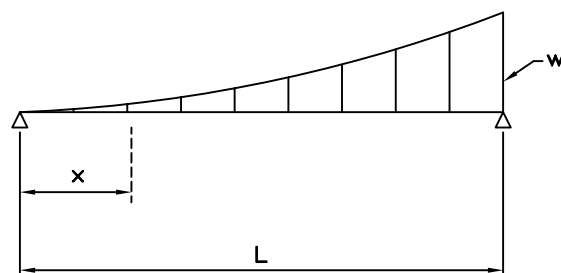
(SEQUENCE OF U.D.L. LOADED SPAN(S) TO GIVE MAX. BENDING MOMENT OR SHEAR)



UNIFORM LOADTRIANGULAR LOADSYMMETRICAL PARABOLIC LOAD

$\frac{x}{L}$	$\propto$		
	UNIFORM LOAD	TRIANGULAR LOAD	SYMMETRICAL PARABOLIC LOAD
0.05	0.02375	0.00831	0.007165
0.10	0.04500	0.01650	0.012300
0.15	0.06375	0.02444	0.015831
0.20	0.08000	0.03200	0.018133
0.25	0.09375	0.03906	0.019531
0.30	0.10500	0.04550	0.020300
0.35	0.11375	0.05119	0.020665
0.40	0.12000	0.05600	0.020800
0.45	0.12375	0.05981	0.020831
0.50	0.12500	0.06250	0.020833
0.55	0.12375	0.06394	0.020831
0.60	0.12000	0.06400	0.020800
0.65	0.11375	0.06256	0.020665
0.70	0.10500	0.05950	0.020300
0.75	0.09375	0.05469	0.019531
0.80	0.08000	0.04800	0.018133
0.85	0.06375	0.03931	0.015831
0.90	0.04500	0.02850	0.012300
0.95	0.02375	0.01544	0.007165

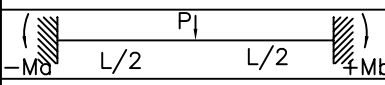
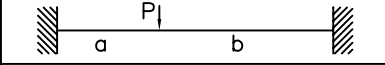
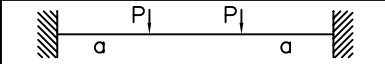
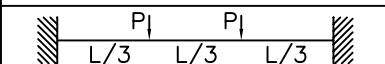
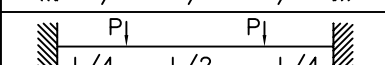
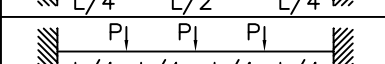
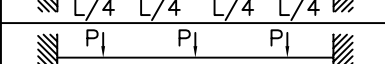
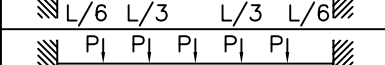
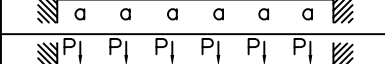
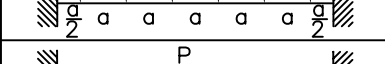
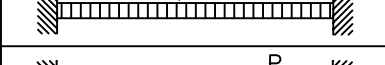
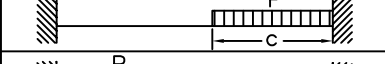
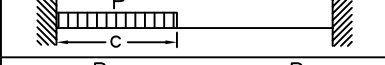
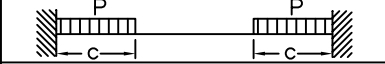
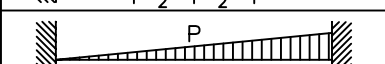
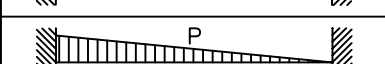
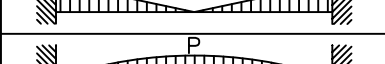
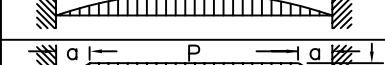
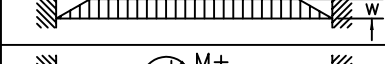
$$\text{MOMENT} = \propto wL^2$$

PARABOLIC LOAD APARABOLIC LOAD B

	$\propto$	
$\frac{x}{L}$	PARABOLIC LOAD A	PARABOLIC LOAD B
0.05	0.001042	0.004165
0.10	0.002083	0.008325
0.15	0.003125	0.012457
0.20	0.004167	0.016533
0.25	0.005208	0.020507
0.30	0.006250	0.024325
0.35	0.007292	0.027915
0.40	0.008333	0.031200
0.45	0.009375	0.034082
0.50	0.010417	0.036458
0.55	0.011456	0.038207
0.60	0.012467	0.039200
0.65	0.013373	0.039290
0.70	0.014050	0.038325
0.75	0.014323	0.036132
0.80	0.013967	0.032533
0.85	0.012706	0.027332
0.90	0.010217	0.020325
0.95	0.006123	0.011290

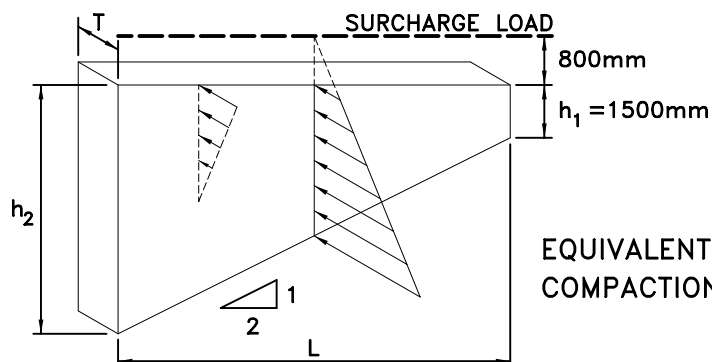
$$\text{MOMENT} = \propto wL^2$$

SIMPLY SUPPORTED	FIXED END
	$\frac{3}{16} PL$
	$\frac{Pa}{2} \left[1 - \left(\frac{a}{L} \right)^2 \right]$
	$1.5 Pa \left(\frac{b}{L} \right)$
	$\frac{PL}{3}$
	$\frac{9}{32} PL$
	$\frac{15}{32} PL$
	$\frac{19}{48} PL$
	$\frac{Pa}{8} (n^2 - 1)$ n=number of spaces
	$\frac{Pa}{8} (n^2 + 0.5)$ n=number of loads
	$\frac{PL}{8}$
	$\frac{Pc}{8} \left(2 - \frac{c}{L} \right)^2$
	$\frac{Pc}{8} \left[2 - \left(\frac{c}{L} \right)^2 \right]$
	$\frac{3}{4} Pc \left(1 - \frac{2}{3} \frac{c}{L} \right)$
	$\frac{Pa}{8L^2} \left[4b(a+L) - c^2 \right]$
	$\frac{3}{16} PL \left[1 - \frac{1}{3} \left(\frac{c}{L} \right)^2 \right]$
	$\frac{PL}{7.5}$
	$\frac{7}{60} PL$
	$\frac{PL}{6.4}$
	$\frac{3}{16} PL$
	$0.15 PL$
	$\frac{PL^2}{8} \left[1 - 2 \left(\frac{a}{L} \right)^2 - \left(\frac{a}{L} \right)^3 \right]$
	$\frac{M}{2} \left[1 - 3 \left(\frac{a}{L} \right)^2 \right]$
	$\frac{3EI}{L^2} \Delta$

	FIXED END	FIXED END
M_a	TYPE OF LOADING	M_b
$\frac{PL}{8}$		$\frac{PL}{8}$
$P_a \left(\frac{b}{L} \right)^2$		$P_b \left(\frac{a}{L} \right)^2$
$P_a \left(\frac{L-a}{L} \right)$		$P_a \left(\frac{L-a}{L} \right)$
$\frac{2}{9} PL$		$\frac{2}{9} PL$
$\frac{3}{16} PL$		$\frac{3}{16} PL$
$\frac{5}{16} PL$		$\frac{5}{16} PL$
$\frac{19}{72} PL$		$\frac{19}{72} PL$
$\frac{Pa}{12} (n^2 - 1)$ n=number of spaces		$\frac{Pa}{12} (n^2 - 1)$ n=number of spaces
$\frac{Pa}{12} (n^2 + 0.5)$ n=number of loads		$\frac{Pa}{12} (n^2 + 0.5)$ n=number of loads
$\frac{PL}{12}$		$\frac{PL}{12}$
$\frac{P_c}{2} \left[\frac{2}{3} \times \frac{c}{L} - \frac{1}{2} \left(\frac{c}{L} \right)^2 \right]$		$\frac{P_c}{2} \left[1 - \frac{4}{3} \times \frac{c}{L} + \frac{1}{2} \left(\frac{c}{L} \right)^2 \right]$
$\frac{P_c}{2} \left[1 - \frac{4}{3} \times \frac{c}{L} + \frac{1}{2} \left(\frac{c}{L} \right)^2 \right]$		$\frac{P_c}{2} \left[\frac{2}{3} \times \frac{c}{L} - \frac{1}{2} \left(\frac{c}{L} \right)^2 \right]$
$\frac{P_c}{2} \left(1 - \frac{2}{3} \frac{c}{L} \right)$		$\frac{P_c}{2} \left(1 - \frac{2}{3} \frac{c}{L} \right)$
$\frac{Pa}{12L^2} \left[12ab^2 + c^2(L-3b) \right]$		$\frac{Pa}{12L^2} \left[12a^2b + c^2(L-3a) \right]$
$\frac{PL}{8} \left[1 - \frac{1}{3} \left(\frac{c}{L} \right)^2 \right]$		$\frac{PL}{8} \left[1 - \frac{1}{3} \left(\frac{c}{L} \right)^2 \right]$
$\frac{PL}{15}$		$\frac{PL}{10}$
$\frac{PL}{10}$		$\frac{PL}{15}$
$\frac{PL}{9.6}$		$\frac{PL}{9.6}$
$\frac{PL}{8}$		$\frac{PL}{8}$
$\frac{PL}{10}$		$\frac{PL}{10}$
$\frac{wL^2}{12} \left[1 - 2 \left(\frac{a}{L} \right)^2 + \left(\frac{a}{L} \right)^3 \right]$		$\frac{wL^2}{12} \left[1 - 2 \left(\frac{a}{L} \right)^2 + \left(\frac{a}{L} \right)^3 \right]$
$M \frac{b}{L} (2 - 3 \frac{b}{L})$		$M \frac{a}{L} (2 - 3 \frac{a}{L})$
$\frac{6EI}{L^2} \Delta$		$\frac{6EI}{L^2} \Delta$

$$M_f = (0.091L^4 + 1.675L^3 + 16.547L^2)/h_2$$

$$V_f = (0.365L^3 + 5.031L^2 + 33.094L)/h_2$$



$$f'_c = 30\text{MPa}$$

$$f_y = 400\text{MPa}$$

CONCRETE COVER = 70mm

$$\phi = 30^\circ$$

EQUIVALENT FLUID PRESSURE = $8k = 7.0\text{ kPa}$

COMPACTION SURCHARGE = 12.0 kPa @ SURFACE

= 0.0 kPa @ 1.7m DEPTH

T=400mm

L (m)	Vf (kN/m)	Mf (kN.m/m)	req'd As mm ²	SPACING (mm)		
				15M	20M	25M
1.50	28	19	620	325		
2.00	36	32	620	325		
2.50	44	48	620	325		
3.00	51	67	620	325		
3.50	59	89	800	250	375	
4.00	67	113	1010	175	275	450
4.50	76	140	1260	150	225	375
5.00	84	170	1560		175	300
5.50	93	203	1880		150	250
6.00	102	239	2260			200
6.50	111	278	2660			175
7.00	121	321	3120			150
7.50	131	367	3670	30M @ 175		
8.00	141	416	4250	30M @ 150		
8.50	151	469	4960	35M @ 200		
9.00	162	526	5730	35M @ 175		
9.50	173	587	6620	35M @ 150		

L length of wingwall

Vf the factored shear force due to lateral pressure at the fixed end (U.L.S.) per unit height of wall

Mf the factored moment due to lateral pressure at the fixed end (U.L.S.) per unit height of wall

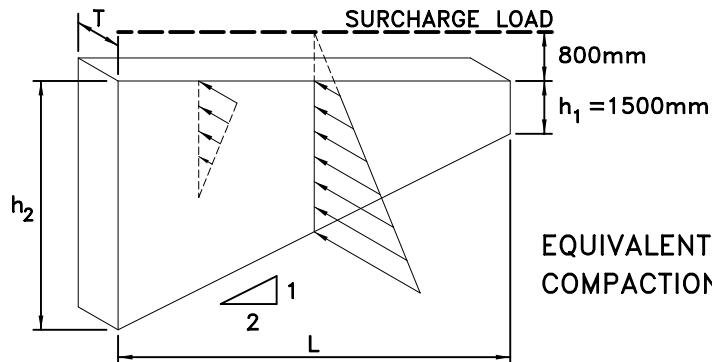
$$\alpha_{ULS} = 1.25$$

NOTES:

- THIS TABLE TO BE READ IN CONJUNCTION WITH DRAWING NUMBER SS105-2.
- CODE: CHBDC - 2000

$$M_f = (0.091L^4 + 1.675L^3 + 16.547L^2)/h_2$$

$$V_f = (0.365L^3 + 5.031L^2 + 33.094L)/h_2$$



$$f'_c = 30\text{MPa}$$

$$f_y = 400\text{MPa}$$

CONCRETE COVER = 70mm

$$\phi = 30^\circ$$

EQUIVALENT FLUID PRESSURE = $8k = 7.0\text{ kPa}$

COMPACTION SURCHARGE = 12.0 kPa @ SURFACE

= 0.0 kPa @ 1.7m DEPTH

T=450mm

L (m)	Vf (kN/m)	Mf (kN.m/m)	req'd As mm ²	SPACING (mm)		
				15M	20M	25M
1.50	28	19	680	300		
2.00	36	32	680	300		
2.50	44	48	680	300		
3.00	51	67	680	300		
3.50	59	89	690	275	425	
4.00	67	113	870	225	325	
4.50	76	140	1090	175	275	450
5.00	84	170	1330		225	375
5.50	93	203	1600		175	300
6.00	102	239	1900		150	250
6.50	111	278	2220			225
7.00	121	321	2600			175
7.50	131	367	3000			150
8.00	141	416	3450	30M @ 200		
8.50	151	469	3950	30M @ 175		
9.00	162	526	4500	30M @ 150		
9.50	173	587	5120	35M @ 175		
10.00	184	652	5810	35M @ 150		

L length of wingwall

Vf the factored shear force due to lateral pressure at the fixed end (U.L.S.) per unit height of wall

Mf the factored moment due to lateral pressure at the fixed end (U.L.S.) per unit height of wall

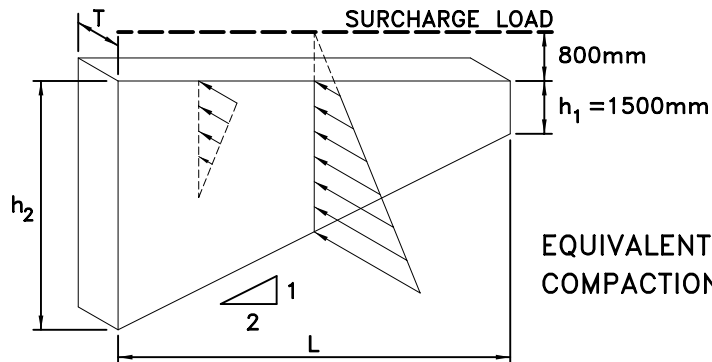
$$\alpha_{ULS} = 1.25$$

NOTES:

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- CODE: CHBDC - 2000

$$M_f = (0.091L^4 + 1.675L^3 + 16.547L^2)/h_2$$

$$V_f = (0.365L^3 + 5.031L^2 + 33.094L)/h_2$$



$$f'_c = 30\text{MPa}$$

$$f_y = 400\text{MPa}$$

CONCRETE COVER = 70mm

$$\phi = 30^\circ$$

EQUIVALENT FLUID PRESSURE = $8k = 7.0\text{ kPa}$

COMPACTION SURCHARGE = $12.0\text{ kPa @ SURFACE}$

= $0.0\text{ kPa @ }1.7\text{m DEPTH}$

T=500mm

L (m)	Vf (kN/m)	Mf (kN.m/m)	req'd As mm ²	SPACING (mm)		
				15M	20M	25M
1.50	28	19	740	275	400	
2.00	36	32	740	275	400	
2.50	44	48	740	275	400	
3.00	51	67	740	275	400	
3.50	59	89	740	275	400	
4.00	67	113	770	250	375	
4.50	76	140	950	200	300	
5.00	84	170	1160	150	250	425
5.50	93	203	1390		200	350
6.00	102	239	1650		175	300
6.50	111	278	1930		150	250
7.00	121	321	2250			200
7.50	131	367	2590			175
8.00	141	416	2960			150
8.50	151	469	3380	30M @ 200		
9.00	162	526	3830	30M @ 175		
9.50	173	587	4330	30M @ 150		
10.00	184	652	4880	35M @ 200		

L length of wingwall

Vf the factored shear force due to lateral pressure at the fixed end (U.L.S.) per unit height of wall

Mf the factored moment due to lateral pressure at the fixed end (U.L.S.) per unit height of wall

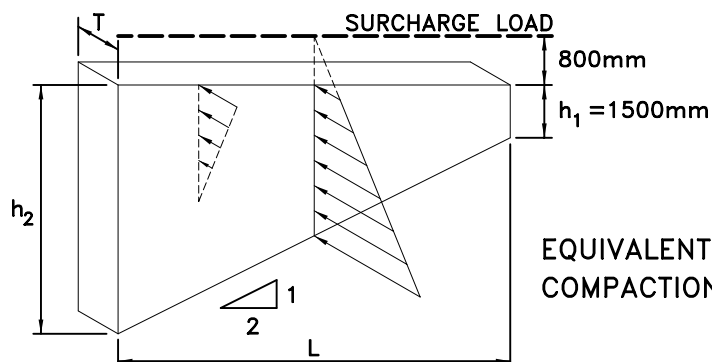
$$\alpha_{ULS} = 1.25$$

NOTES:

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- CODE: CHBDC - 2000

$$M_f = (0.091L^4 + 1.675L^3 + 16.547L^2)/h_2$$

$$V_f = (0.365L^3 + 5.031L^2 + 33.094L)/h_2$$



$$f'_c = 30\text{MPa}$$

$$f_y = 400\text{MPa}$$

CONCRETE COVER = 70mm

$$\phi = 30^\circ$$

EQUIVALENT FLUID PRESSURE = $8k = 7.0\text{ kPa}$

COMPACTION SURCHARGE = 12.0 kPa @ SURFACE

= 0.0 kPa @ 1.7m DEPTH

T=550mm

L (m)	Vf (kN/m)	Mf (kN.m/m)	req'd As mm ²	SPACING (mm)		
				15M	20M	25M
1.50	28	19	800	250	375	
2.00	36	32	800	250	375	
2.50	44	48	800	250	375	
3.00	51	67	800	250	375	
3.50	59	89	800	250	375	
4.00	67	113	800	250	375	
4.50	76	140	850	225	350	
5.00	84	170	1030	175	275	450
5.50	93	203	1240	150	225	400
6.00	102	239	1460		200	325
6.50	111	278	1710		175	275
7.00	121	321	1990			250
7.50	131	367	2280			200
8.00	141	416	2610			175
8.50	151	469	2960			150
9.00	162	526	3350	30M @ 200		
9.50	173	587	3780	30M @ 175		
10.00	184	652	4240	30M @ 150		

L length of wingwall

Vf the factored shear force due to lateral pressure at the fixed end (U.L.S.)
per unit height of wall

Mf the factored moment due to lateral pressure at the fixed end (U.L.S.)
per unit height of wall

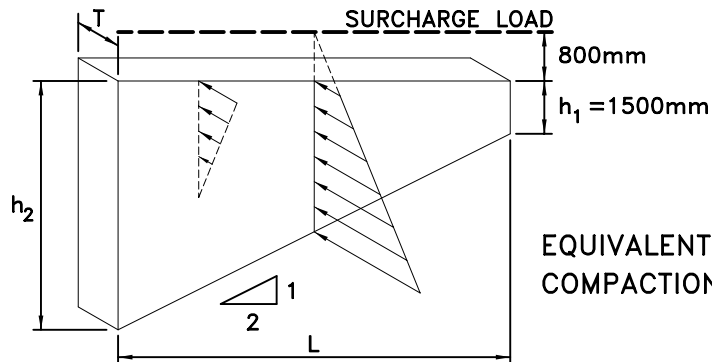
$$\alpha_{ULS} = 1.25$$

NOTES:

- THIS TABLE TO BE READ IN CONJUNCTION WITH DRAWING NUMBER SS105-2.
- CODE: CHBDC - 2000

$$M_f = (0.091L^4 + 1.675L^3 + 16.547L^2)/h_2$$

$$V_f = (0.365L^3 + 5.031L^2 + 33.094L)/h_2$$



$$f'_c = 30\text{MPa}$$

$$f_y = 400\text{MPa}$$

CONCRETE COVER = 70mm

$$\phi = 30^\circ$$

EQUIVALENT FLUID PRESSURE = $8k = 7.0\text{ kPa}$

COMPACTION SURCHARGE = 12.0 kPa @ SURFACE

= 0.0 kPa @ 1.7m DEPTH

T=600mm

L (m)	Vf (kN/m)	Mf (kN.m/m)	req'd As mm ²	SPACING (mm)		
				15M	20M	25M
1.50	28	19	860	225	350	
2.00	36	32	860	225	350	
2.50	44	48	860	225	350	
3.00	51	67	860	225	350	
3.50	59	89	860	225	350	
4.00	67	113	860	225	350	
4.50	76	140	860	225	350	
5.00	84	170	930	200	300	
5.50	93	203	1110	175	250	425
6.00	102	239	1310		225	375
6.50	111	278	1530		175	325
7.00	121	321	1780		150	275
7.50	131	367	2050			225
8.00	141	416	2330			200
8.50	151	469	2640			175
9.00	162	526	2990			150
9.50	173	587	3360	30M @ 200		
10.00	184	652	3760	30M @ 175		

L length of wingwall

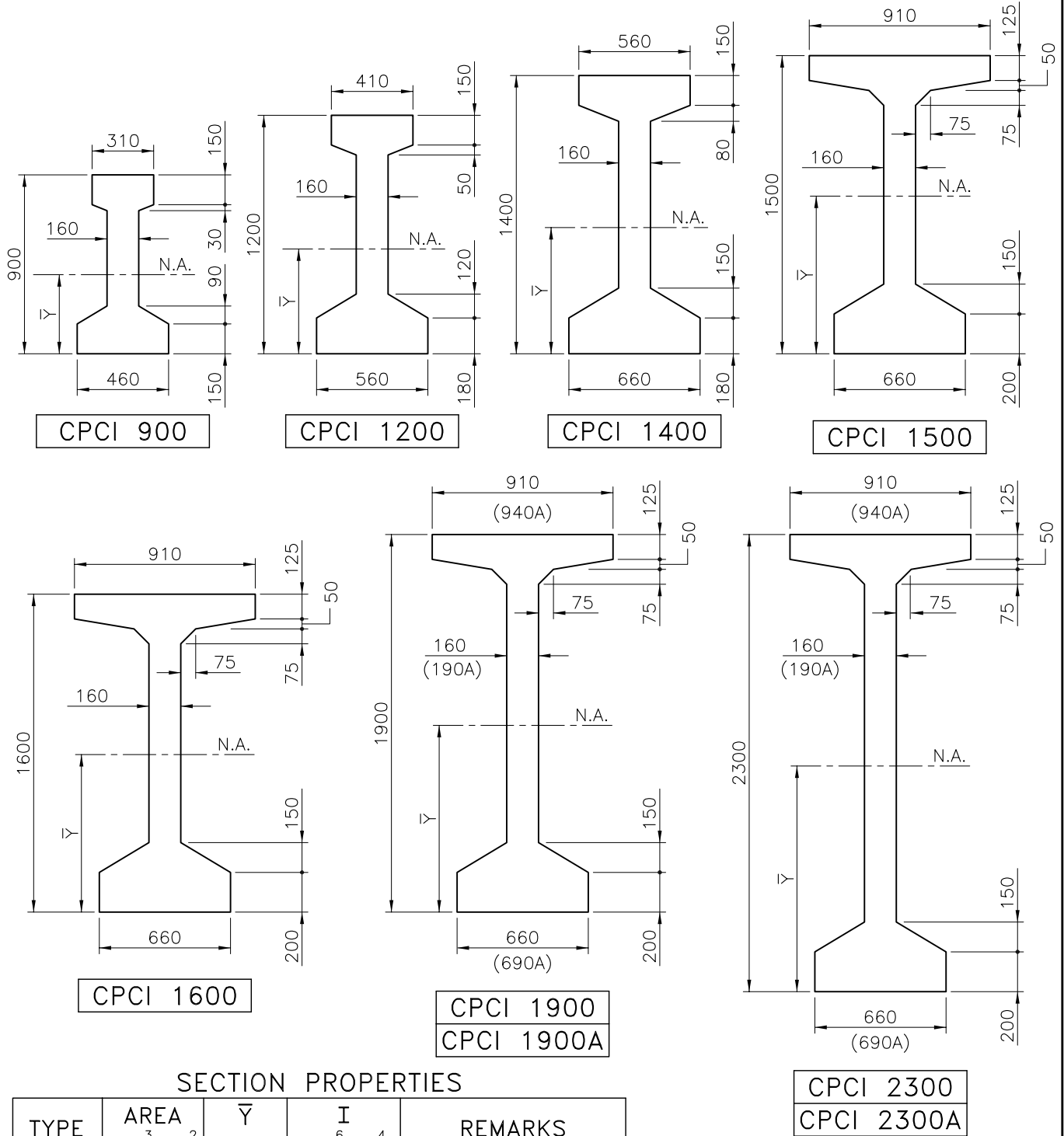
Vf the factored shear force due to lateral pressure at the fixed end (U.L.S.) per unit height of wall

Mf the factored moment due to lateral pressure at the fixed end (U.L.S.) per unit height of wall

$$\alpha_{ULS} = 1.25$$

NOTES:

- THIS TABLE TO BE READ IN CONJUNCTION WITH DRAWING NUMBER SS105-2.
- CODE: CHBDC - 2000

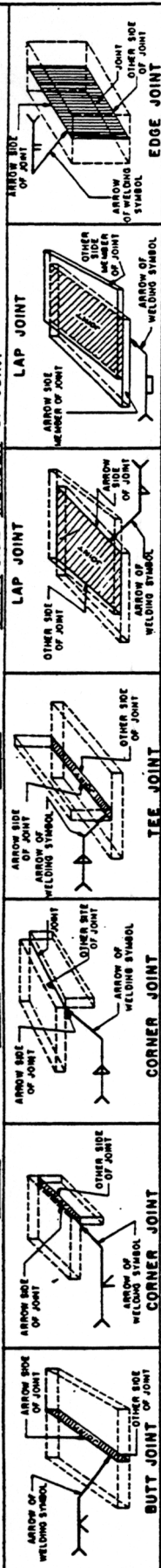


SECTION PROPERTIES

TYPE	AREA $\times 10^3 \text{ mm}^2$	$\bar{Y}$ mm	I $\times 10^6 \text{ mm}^4$	REMARKS
900	227	400	19943	
1200	332	529	55369	
1400	427	638	104925	
1500*	499	745	151383	*OPTIONAL
1600	515	793	178129	
1900	563	940	274135	
1900A	620	941	291289	POST-TENSIONED
2300	627	1136	441935	
2300A	696	1137	472367	POST-TENSIONED

*OPTIONAL.
SEE SUBSECTION 7.2.2
OF THE STRUCTURAL MANUAL.

BASIC JOINTS—IDENTIFICATION OF ARROW SIDE AND OTHER SIDE OF JOINT AND ARROW-SIDE AND OTHER-SIDE MEMBER OF JOINT



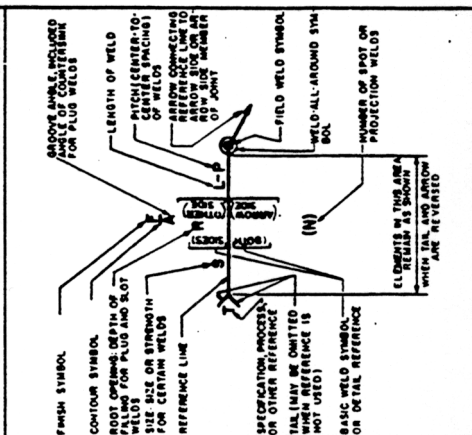
BASIC WELD SYMBOLS AND THEIR LOCATION SIGNIFICANCE

LOCATION SIGNIFICANCE	GROOVE						BACK OR BACKING		FLANGE	
	FLAT	FLARE-V	FLARE-V	FLARE-V	FLARE-V	FLARE-V	BACKING	SURFACING	EDGE	CORNER
ARROW SIDE										
OTHER SIDE										
BOTH SIDES										
NO ARROW SIDE OR OTHER SIDE SIGNIFICANCE										

TYPICAL WELDING SYMBOLS

BACK OR BACKING WELD SYMBOL	SINGLE-V-GROOVE WELDING SYMBOL INDICATING ROOT PENETRATION	PROJECTION WELDING
SURFACING WELD SYMBOL INDICATING BUILT-UP SURFACE	DOUBLE-BEVEL-GROOVE WELDING SYMBOL	SEAM WELDING
DOUBLE FILLET WELDING SYMBOL	WELDING SYMBOLS FOR COMBINED WELDS	FLASH OR UPSET WELDING SYMBOL
CHAIN INTERMITTENT FILLET WELDING SYMBOL	PLUG WELDING SYMBOL	SQUARE-GROOVE WELDING SYMBOL
STAGGERED INTERMITTENT FILLET WELDING SYMBOL	SLOT WELDING SYMBOL	FLARE-V-AND FLARE-BEVEL-GROOVE WELDING SYMBOL
SINGLE-V-GROOVE WELDING SYMBOL	SPOT WELDING SYMBOL	EDGE-AND CORNER-FLANGE WELD SYMBOLS

SUPPLEMENTARY SYMBOLS		CONTOUR	
WELD ALL-AROUND	WELD THRU	FLUSH	CONVEX



SUPPLEMENTARY SYMBOLS USED WITH WELDING SYMBOLS

WELD ALL-AROUND SYMBOL	WELD THRU SYMBOL	FLUSH-CONTOUR SYMBOL	CONVEX-CONTOUR SYMBOL

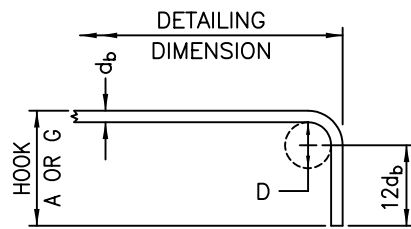
AS RECOMMENDED BY AMERICAN WELDING SOCIETY

PROPERTIES OF REINFORCING STEEL BARS

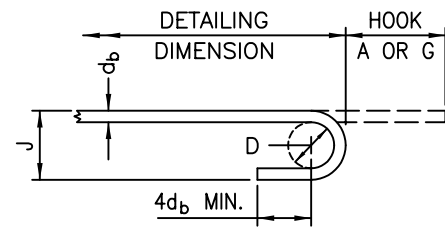
BAR SIZE	MASS (kg/m)	NOMINAL DIMENSIONS		
		DIAMETER (mm)	CROSS SECTIONAL AREA (mm ²)	PERIMETER (mm)
10M	0.785	11.3	100	35.5
15M	1.570	16.0	200	50.1
20M	2.355	19.5	300	61.3
25M	3.925	25.2	500	79.2
30M	5.495	29.9	700	93.9
35M	7.850	35.7	1000	112.2
45M	11.775	43.7	1500	137.3
55M	19.625	56.4	2500	177.2

STEEL AREA PER METRE WIDTH (mm²)

SPACING (mm)	BAR SIZE							
	10M	15M	20M	25M	30M	35M	45M	55M
80	1250	2500	3750	6250	8750	12500		
100	1000	2000	3000	5000	7000	10000	15000	
120	833	1667	2500	4167	5833	8333	12500	20833
125	800	1600	2400	4000	5600	8000	12000	20000
140	714	1429	2143	3571	5000	7143	10714	17857
150	667	1333	2000	3333	4667	6667	10000	16667
160	625	1250	1875	3125	4375	6250	9375	15625
175	571	1143	1714	2857	4000	5714	8571	14286
180	556	1111	1667	2778	3889	5556	8333	13889
200	500	1000	1500	2500	3500	5000	7500	12500
220	455	909	1364	2273	3182	4545	6818	11364
225	444	889	1333	2222	3111	4444	6667	11111
240	417	833	1250	2083	2917	4167	6250	10417
250	400	800	1200	2000	2800	4000	6000	10000
260	385	769	1154	1923	2692	3846	5769	9615
275	364	727	1091	1818	2545	3636	5455	9091
280	357	714	1071	1786	2500	3571	5357	8929
300	333	667	1000	1667	2333	3333	5000	8333
350	286	571	857	1429	2000	2857	4286	7143
400	250	500	750	1250	1750	2500	3750	6250
450	222	444	667	1111	1556	2222	3333	5556
500	200	400	600	1000	1400	2000	3000	5000



STANDARD 90° HOOK



STANDARD 180° HOOK

MINIMUM BENDING PIN DIAMETER, D, mm

BAR SIZE	STEEL GRADE	
	400R ⁽²⁾	400W
10M	70	60
15M	100	90
20M	120	100
25M	150	150
30M	250	200
35M	300	250
45M	450 ⁽¹⁾	400
55M	600 ⁽¹⁾	550

(1) Special fabrication is required for bends exceeding 90° for bars of these sizes and grade.

(2) For stainless steel, with $F_y = 420$, use the same D as for 400R.

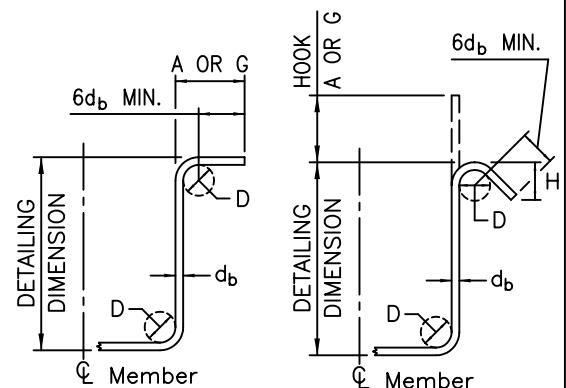
STANDARD HOOK DIMENSIONS

BAR SIZE	90° HOOKS		180° HOOKS			
	A OR G (mm)		A OR G (mm)		J (mm)	
	400R	400W	400R	400W	400R	400W
10M	180	180	140	130	90	80
15M	260	250	180	170	130	120
20M	310	300	220	200	160	140
25M	400	400	280	280	200	200
30M	510	490	400	350	310	260
35M	610	590	480	430	370	320
45M	790	770	680	630	540	490
55M	1030	1010	900	850	710	660

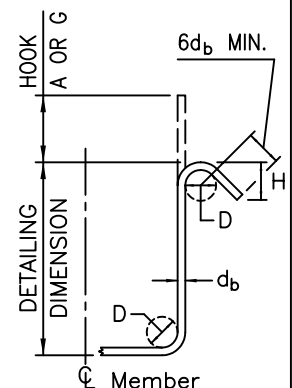
NOTE: All Hook Dimensions are according to the CHBDC-2000.

MINIMUM STIRRUP AND TIE HOOK DIMENSIONS

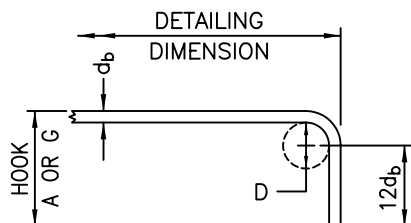
BAR SIZE	BAR DIAM. d_b (mm)	PIN DIAM. D(mm)	90°	135°	
			A OR G (mm)	A OR G (mm)	H (approx.) (mm)
10M	11.3	45	100	100	70
15M	16.0	65	140	140	100
20M	19.5	80	180	175	115
25M	25.2	100	230		



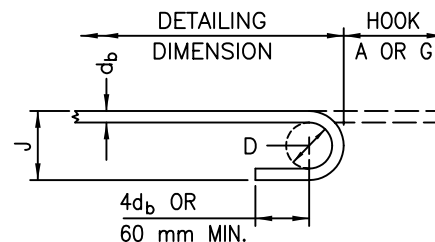
MIN. 90° HOOK



MIN. 135° HOOK



STANDARD 90° HOOK



STANDARD 180° HOOK

MINIMUM BENDING PIN
DIAMETER, D, mm

BAR SIZE	STEEL GRADE 400
10M	80
15M	120
20M	160
25M	200
30M	240
35M	350
45M	450
55M	550

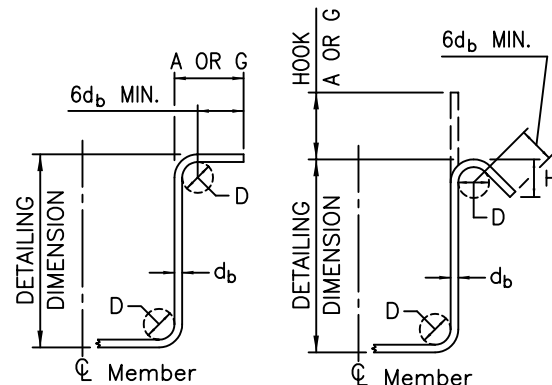
STANDARD HOOK DIMENSIONS

BAR SIZE	90° HOOKS	180° HOOKS	
	A OR G (mm)	A OR G (mm)	J (mm)
10M	190	210 *	110 *
15M	270	260 *	160 *
20M	330	300 *	200
25M	430	330	250
30M	510	460 *	300
35M	640	700 *	430 *
45M	790	850 *	540
55M	1010	1050 *	680 *

NOTE: Tabulated values for Standard Hook Dimensions are the larger of calculated (according to CHBDC-2000) and Reinforcing Steel Institute of Canada (RSIC) requirements. Value marked by '*' indicate RSIC recommended minimum.

MINIMUM STIRRUP AND TIE HOOK DIMENSIONS

BAR SIZE	BAR DIAM. d_b (mm)	PIN DIAM. D(mm)	90°	135°	
			A OR G (mm)	A OR G (mm)	H (approx.) (mm)
10M	11.3	90	180	190	120
15M	16.0	130	190	220	130
20M	19.5	160	230	260	150
25M	25.2	200	280		

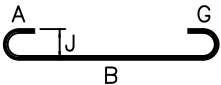
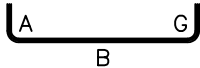
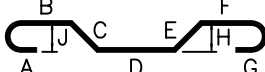
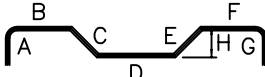
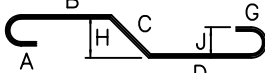
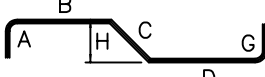
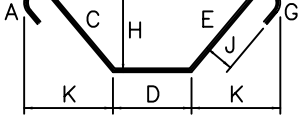


MIN. 90° HOOK

MIN. 135° HOOK

NOTE. Value marked by '**' indicates RSIC recommended minimum based on the average of 90° and 180° hooks.

MINIMUM BAR BEND DIMENSIONS FOR GRADE 400 COATED BARS

BAR TYPE	BAR SHAPE DETAILS	DETAILING DIMENSIONS														
		SIZE	A	B	C	D	E	F	G	H	J	K	L	O	R	
1		10M	210	250					210		110					
		15M	260	350					260		160					
		20M	300	450					300		200					
		25M	330	550					330		250					
2		10M	190	250					190							
		15M	270	350					270							
		20M	330	450					330							
		25M	430	550					430							
3		10M	210	250	250	250	250	250	210		110					
		15M	260	350	350	350	350	350	260		160					
		20M	300	450	450	450	450	450	300		200					
		25M	330	550	550	550	550	550	330		250					
4		10M	190	250	250	250	250	250	190							
		15M	270	350	350	350	350	350	270							
		20M	330	450	450	450	450	450	330							
		25M	430	550	550	550	550	550	430							
5		10M	210	250	250	250			210		110					
		15M	260	350	350	350			260		160					
		20M	300	450	450	450			300		200					
		25M	330	550	550	550			330		250					
6		10M	190	250	250	250			190							
		15M	270	350	350	350			270							
		20M	330	450	450	450			330							
		25M	430	550	550	550			430							
7		10M	210		250	250	250		210		110					
		15M	260		350	350	350		260		160					
		20M	300		450	450	450		300		200					
		25M	330		550	550	550		330		250					

NOTES

• All dimensions are out-to-out of a bar except A and G on minimum 180° and 135° hooks.

• All bar types that have end segment(s) which are not minimum 90° or 180° bends, have specified lengths that meet RSIC minimum physical requirements.

NOTES

- All dimensions are out-to-out of a bar except A and G on minimum 180° and 135° hooks.
- All bar types that have end segment(s) which are not minimum 90° or 180° bends, have specified lengths that meet RSIC minimum physical requirements.

MINIMUM BAR BEND DIMENSIONS FOR GRADE 400 COATED BARS

BAR TYPE	BAR SHAPE DETAILS	DETAILING DIMENSIONS														
		SIZE	A	B	C	D	E	F	G	H	J	K	L	O	R	
8		10M	190	250					190							
		15M	270	350					270							
		20M	330	450					330							
		25M	430	550					430							
9		10M													50	
		15M													75	
		20M													100	
10		10M	130		130										50	
		15M	190		190										75	
		20M	230		230										100	
11		10M	210	250		130					110				50	
		15M	260	350		190					160				75	
		20M	300	450		230					200				100	
		25M	330	550		300					250				125	
12		10M	210	250	250	130					110					
		15M	260	350	350	190					160					
		20M	300	450	450	230					200					
		25M	330	550	550	300					250					
13		10M		130		250			210		110				50	
		15M		190		350			260		160				75	
		20M		230		450			300		200				100	
		25M		300		550			330		250				125	
14		10M	130	250	250	250	130									
		15M	190	350	350	350	190									
		20M	230	450	450	450	230									
		25M	300	550	550	550	300									

NOTES

- All dimensions are out-to-out of a bar except A and G on minimum 180° and 135° hooks.
- All bar types that have end segment(s) which are not minimum 90° or 180° bends, have specified lengths that meet RSIC minimum physical requirements.

NOTES

- All dimensions are out-to-out of a bar except A and G on minimum 180° and 135° hooks.
- All bar types that have end segment(s) which are not minimum 90° or 180° bends, have specified lengths that meet RSJC minimum physical requirements.

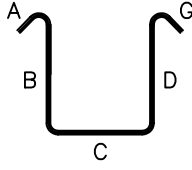
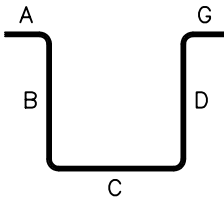
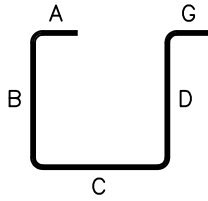
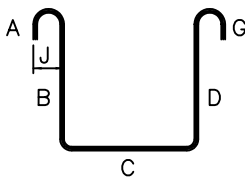
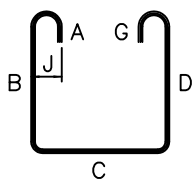
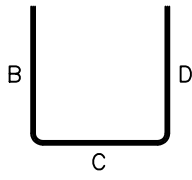
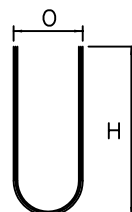
BAR TYPE	BAR SHAPE DETAILS	DETAILING DIMENSIONS														
		SIZE	A	B	C	D	E	F	G	H	J	K	L	O	R	
16		10M	190	250	250	130										
		15M	270	350	350	190										
		20M	330	450	450	230										
		25M	430	550	550	300										
17		10M		190	250	190										
		15M		270	350	270										
		20M		330	450	330										
		25M		430	550	430										
18		10M	210	250	190						110					
		15M	260	350	270						160					
		20M	300	450	330						200					
		25M	330	550	430						250					
19		10M		130	250	130										
		15M		190	350	190										
		20M		230	450	230										
		25M		300	550	300										
20		10M		190	250	190										
		15M		270	350	270										
		20M		330	450	330										
		25M		430	550	430										
21		10M	210	250	190						110					
		15M	260	350	270						160					
		20M	300	450	330						200					
		25M	330	550	430						250					

NOTES

• All dimensions are out-to-out of a bar except A and G on minimum 180° and 135° hooks.

• All bar types that have end segment(s) which are not minimum 90° or 180° bends, have specified lengths that meet RSIC minimum physical requirements.

MINIMUM BAR BEND DIMENSIONS FOR GRADE 400 COATED STIRRUPS

BAR TYPE	BAR SHAPE DETAILS	DETAILING DIMENSIONS														
		SIZE	A	B	C	D	E	F	G	H	J	K	L	O	R	
S1		10M	190	250	250	250			190							
		15M	220	350	350	350			220							
		20M	260	450	450	450			260							
		25M	290	550	550	550			290							
S4		10M	180	250	250	250			180							
		15M	190	350	350	350			190							
		20M	230	450	450	450			230							
		25M	260	550	550	550			260							
S5		10M	180	250	250	250			180							
		15M	190	350	350	350			190							
		20M	230	450	450	450			230							
		25M	260	550	550	550			260							
S7		10M	210	250	250	250			210		110					
		15M	260	350	350	350			260		160					
		20M	300	450	450	450			300		200					
		25M	330	550	550	550			330		250					
S9		10M	210	250	250	250			210		110					
		15M	260	350	350	350			260		160					
		20M	300	450	450	450			300		200					
		25M	330	550	550	550			330		250					
S10		10M		180	250	180										
		15M		190	350	190										
		20M		230	450	230										
		25M		260	550	260										
S11		10M								110				110		
		15M								140				160		
		20M								180				200		
		25M								230				250		

NOTES

• All dimensions are out-to-out of a bar except A and G on minimum 180° and 135° hooks.

• All bar types that have end segment(s) which are not minimum 90° or 180° bends, have specified lengths that meet RSIC minimum physical requirements.

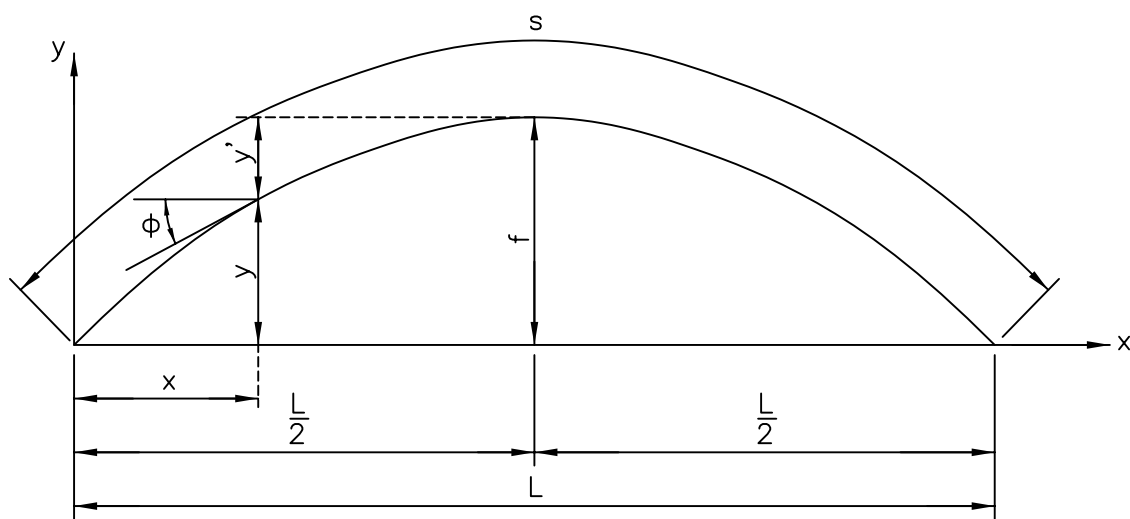
NOTES

- All dimensions are out-to-out of a bar except A and G on minimum 180° and 135° hooks.
- All bar types that have end segment(s) which are not minimum 90° or 180° bends, have specified lengths that meet RSIC minimum physical requirements.

NOTES

- All dimensions are out-to-out of δ bar except A and G on minimum 180 and 135° hooks.
- All bar types that have end°segment(s) which are not minimum 90 or 180 bends, have specified lengths that meet RSIC minimum physical requirements.

JUNE 2002		IMPERIAL—SI CONVERSION FACTORS		DA 16—5	
SUBJECT		CONVERSION FACTORS		SUBJECT	
AREA		CONVERSION FACTORS		SUBJECT	
LINEAR		1 in. = 25.4 mm 1 ft. = 304.8 mm = 0.3048 m 1 mile = 1.609 km		AREA	
VOLUME		1 in. ³ = 16.3871 cm ³ = 16387.1 mm ³ 1 ft. ³ = 0.0283 m ³ 1 yd. ³ = 0.7646 m ³		MASS	
MASS DENSITY		1 lb./ft. ³ = 16.018 kg/m ³ 1 lb./in. ³ = 27.680 kg/cm ³		FORCE	
SPEED		1 mi./h = 1.609 km/h 1 ft./s = 0.305 m/s		1 lb. = 4.44822 N 1 kip = 4448.22 N 1 N = 0.10197 kg 1 kg = 9.8067 N 1 kg = 2.2046 lb.	
LOADS (U.D.L.)		1 kip/in. = 175.1 N/mm 1 kip/ft. = 14.5939 kN/m 1 lb./ft. = 14.5939 N/m		BENDING MOMENT OR TORSION	
PRESSURE OR STRESS OR MODULUS OF ELASTICITY		1 psi = 6.89476 kPa 1 ksi = 6.89476 MPa 1 psf = 47.88 Pa 1 ksf = 47.88 kPa		MOMENT OF INERTIA	
				SECTION MODULUS	
				TEMPERATURE	
				°C = 5/9 (°F - 32)	
				THERMAL EXPANSION	
				in./in. °F = 5.556 x 10 ⁻¹ m/m °C	
				Thermal Coefficient of Linear Expansion for Concrete : 6.667 x 10 ⁻⁶ /°F = 12 x 10 ⁻⁶ /°C	
				NOTE:	
				1 Pa = 1 N/m ² 1 kip = 1000 lb. 1 ton = 2000 lb. 1 metric tonne = 1000 kg	



$$y = \frac{4f}{L^2} (L-x)x$$

$$\tan \phi = \frac{dy}{dx} = \frac{4f}{L^2} (L-2x)$$

$$s = \left\{ \sqrt{1 + \left(\frac{4f}{L} \right)^2} + \frac{L}{4f} \log_e \left[\frac{4f}{L} + \sqrt{1 + \left(\frac{4f}{L} \right)^2} \right] \right\} \times \frac{L}{2}$$

$\frac{x}{L}$	y	y'
0.05	0.19f	0.81f
0.10	0.36f	0.64f
0.15	0.51f	0.49f
0.20	0.64f	0.36f
0.25	0.75f	0.25f
0.30	0.84f	0.16f
0.35	0.91f	0.09f
0.40	0.96f	0.04f
0.45	0.99f	0.01f
0.50	1.00f	0.00f

$\frac{f}{L}$	s	$\frac{f}{L}$	s
0.05	1.0066L	0.55	1.5560L
0.10	1.0261L	0.60	1.6353L
0.15	1.0571L	0.65	1.7166L
0.20	1.0982L	0.70	1.7997L
0.25	1.1478L	0.75	1.8842L
0.30	1.2043L	0.80	1.9700L
0.35	1.2667L	0.85	2.0570L
0.40	1.3337L	0.90	2.1449L
0.45	1.4047L	0.95	2.2338L
0.50	1.4789L	1.00	2.3234L

STRUCTURAL MANUAL

DIVISION 4 - STRUCTURAL STANDARD DRAWINGS SECTION 1

December 2003

JUNE 2002	LIST OF STRUCTURAL STANDARD DRAWINGS	SS 1-1

STANDARD DRAWINGS - SMALL SIZE DRAWINGS

The following standard drawings are issued in 216 x 279mm format, for attachment on a full-size standard drawing sheet.

Digital file names are established by removing one "S" from the Standard Drawing name prefix, and adding the extension "DWG". The drawing files are in AutoCAD.

The layer named UPDATE in the digital file, contains a Revision Information Sheet for the current revision of the drawing.

1. INDEX OF STRUCTURAL STANDARD DRAWINGS

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List of Structural Standard Drawings	SS 1-2
List of Structural Ontario Provincial Standard Drawings	SS 1-3
List of Structural Ontario Provincial Standard Drawings	SS 1-4

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Structural Standard Drawings - Layering Structure	SS 2-1

3. PILES

4. FOOTINGS

5. ABUTMENTS, WINGWALLS AND RETAINING WALLS

Retaining Walls – Rustication Details	SS 5-2

6. PIERS

7. PRESTRESSED CONCRETE

8. STRUCTURAL STEEL

9. DECK AND CURBS

Provision for Future Light Pole Base	SS 9-15
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10. BARRIERS AND RAILING SYSTEMS

Thrie Beam Guide Rail - Rail Detail	SS 10-20
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Box Beam Guide Rail - Side Mount - Assembly Detail (PL-1)	SS 10-40A
Box Beam Guide Rail - Side Mount - Post and Attachment Details (PL-1)	SS 10-40B
Box Beam Guide Rail - Side Mount - Rail and Splice Detail (PL-1)	SS 10-40C
Thrie Beam Guide Rail - Side Mount - Assembly Detail (PL-1)	SS 10-42A
Thrie Beam Guide Rail - Side Mount - Post and Attachment Details (PL-1)	SS 10-42B
Thrie Beam Guide Rail - Timber Deck - Assembly Detail (PL-1)	SS 10-43A
Thrie Beam Guide Rail - Timber Deck - Post and Attachment Details (PL-1)	SS 10-43B
Box Beam Guide Rail On Curb - Assembly Detail (PL-2)	SS 10-50A
Box Beam Guide Rail On Curb - Post and Base Detail (PL-2)	SS 10-50B
Box Beam Guide Rail On Curb - Rail and Splice Detail (PL-2)	SS 10-50C

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12. REINFORCING STEEL

Hook Dimensions for Uncoated Bars	SS 12-1
Hook Dimensions for Coated Bars	SS 12-2

13. BEARING ASSEMBLIES AND EXPANSION JOINTS

14. CULVERTS

15. WOOD STRUCTURES

16. MISCELLANEOUS

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ONTARIO PROVINCIAL STANDARD DRAWINGS - STRUCTURAL

The following Structural OPSD's can be found in the "Ontario Provincial Standards for Roads and Municipal Services" - Volume 3, as of November 2001:

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2. GENERAL DESIGN AND DRAUGHTING

Falsework Clearances OPSD-4602.000

3. PILES

Splice and Driving Shoe Details for Steel H-Piles OPSD-3301.000

Splice and Driving Shoe Details for Steel Tube Piles OPSD-3302.000

Details of Driving Shoe and Pile Head Protection for Wood Piles OPSD-3303.000

Oslo Points for HP310 H-Piles OPSD-3304.000

Steel Sheet Piles Anchorage Details OPSD-3320.000

4. FOOTINGS

Contours of Frost Depths for Northern Ontario OPSD-3400.010

Contours of Frost Depths for Southern Ontario OPSD-3400.011

5. ABUTMENTS, WINGWALLS AND RETAINING WALLS

Minimum Granular Backfill Requirements - Abutments OPSD-3501.000

Minimum Granular Backfill Requirements - Retaining Walls OPSD-3504.000

Rock Backfill Requirements - Abutments OPSD-3505.000

Retaining Wall and Abutment Wall Drain Detail OPSD-3506.000

Wingwall Sleeve Details for Perforated Drain OPSD-3515.000

Concrete Toe Details OPSD-4066.010

6. PIERS

Cutwater Details OPSD-3601.00

7. PRESTRESSED CONCRETE

Positive Moment Connection CPCI 900, 1200, 1400, 1500, 1600, 1900 and 2300 OPSD-3701.000

8. STRUCTURAL STEEL

Access Hatch for Structural Steel Box Girder OPSD-3810.00

9. DECK AND CURBS

Modification of Round, Vertical Deck Drain for Addition of Asphalt to Existing Structures

..... OPSD-3901.01

Modification of Scupper Deck Drain for Addition of Asphalt to Existing Structures OPSD-3901.02

Modification of Inclined Deck Drain for Addition of Asphalt to Existing Structures OPSD-3901.03

Deck Drain Details - Downspout without Catch Basin OPSD-3902.01

Deck Drain Details - Transverse Bar Opening OPSD-3902.020

Deck Drain Details - Longitudinal Bar Opening OPSD-3902.030

Deck Drain Details - Sloped Downspout without Catch Basin OPSD-3902.04

Median Deck Drains Details - Transverse Bar Opening OPSD-3902.05

Median Deck Drains Details - Longitudinal Bar Opening OPSD-3902.06

Drip Detail OPSD-3904.040

DECEMBER 2003	LIST OF STRUCTURAL OPS DRAWINGS	SS 1-4

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Bridge Deck Waterproofing Details at Active Wide Cracks Greater Than 2mm and Construction Joints	OPSD-3906.030
Lighting Pole Base Details for Structures with Parapet Walls	OPSD-3911.000
Lighting Pole Base Details for Structures with Concrete Barrier Walls.....	OPSD-3912.000
Support and Tie Down for Round Void Forms	OPSD-3920.000
Drainage Outlets in Round Void Forms	OPSD-3921.000
Drainage Outlets in Trapezoidal Void	OPSD-3921.010
Supports for Reinforcing Steel, for Slab Depth Greater Than 300mm	OPSD-3922.000
Supports for Reinforcing Steel, for Slab Depth 300mm or Less	OPSD-3922.010
Method of Obtaining Screed Elevations for Steel Girders	OPSD-3930.00
Method of Obtaining Screed Elevations for Precast Concrete Girders.....	OPSD-3930.010
Drainage of Asphalt Wearing Surface on New Decks	OPSD-3950.00
Drainage of Asphalt Wearing Surface on Existing Decks	OPSD-3951.00
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Culverts - Warning Message Lettering Details	OPSD-4040.06
15. WOOD STRUCTURES	
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Steel Pedestal Detail 150mm High Aluminum Truss Sign Support	OPSD-4810.000
Steel Pedestal Detail 300 – 1200mm High Aluminum Truss Sign Support	OPSD-4810.500
8 Bolt Anchorage Insert for Steel Sectional Sign Support	OPSD-4820.000

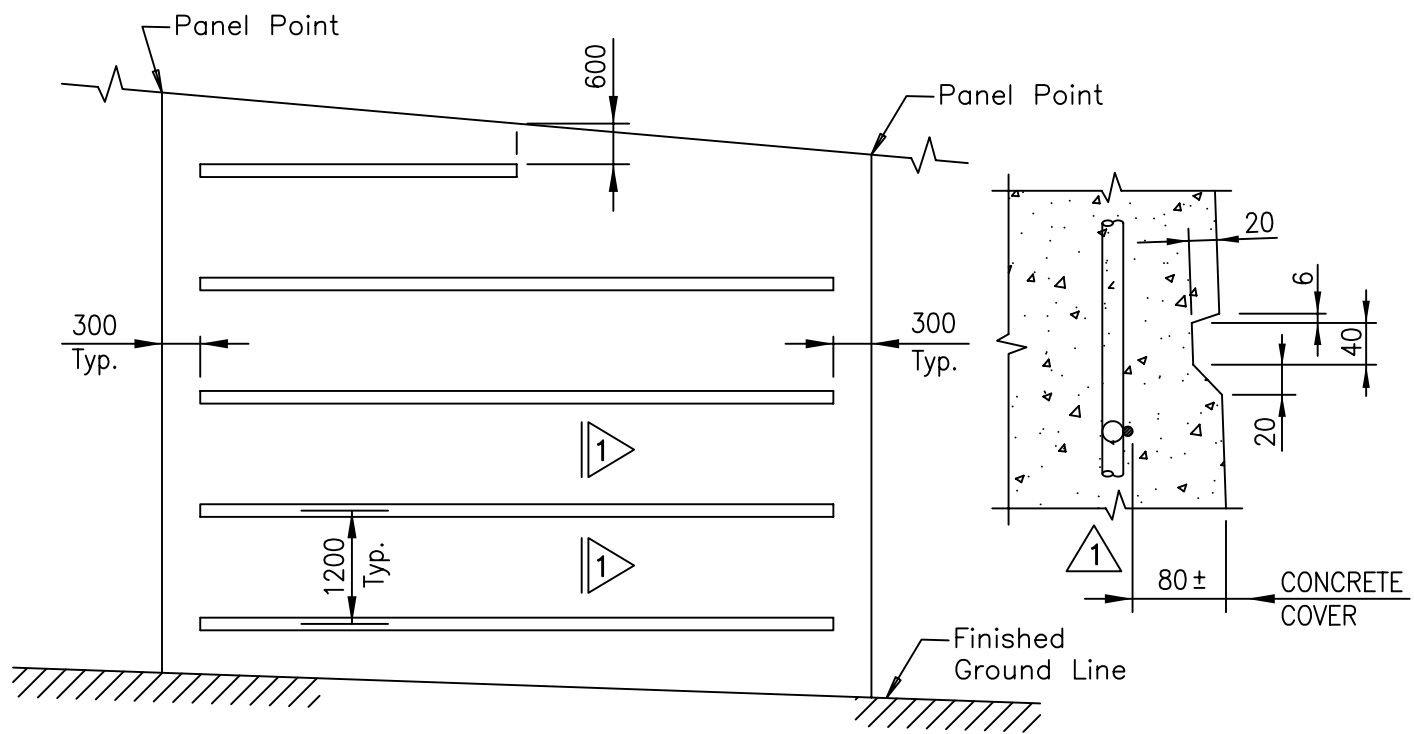
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551–559	(Varies)	Continuous	FRAME AND TITLE BLOCK	(Varies)
560	8	Continuous	WORK LINE	(Not to be plotted)
561	1 (Red)	Continuous	MISCELLANEOUS	_____
562	3 (Green)	Continuous	STRUCTURAL STEEL 1	_____
563	7 (White)	Continuous	REINFORCING STEEL 1	_____
564	9	Continuous	STRUCTURAL STEEL 2	_____
565	2 (Yellow)	Continuous	CONCRETE/STEEL OUTLINE	_____
566	6 (Magenta)	Continuous	REINFORCING STEEL 2	_____
567	1 (Red)	CL1	CENTRE LINE 1	_____ (20,-1.5,1.5,-1.5)
568	1 (Red)	CL2	CENTRE LINE 2	_____ (10,-1,1,-1)
569	3 (Green)	DL1	HIDDEN LINE 1	_____ (1,-0.5)
570	4 (Cyan)	DL2	DASH LINE 2	_____ (1.25,-1)
571	7 (White)	DL3	DASH LINE 3	_____ (2.5,-1)
572	10	DL4	HIDDEN LINE 2	_____ (3,-1)
574	3 (Green)	DL6	PHANTOM LINES	_____ (10,-1,1,-1,1,-1)
575	6 (Magenta)	DL7	REINFORCING STEEL 3	_____ (7,-2)
580	4 (Cyan)	Continuous	DIMENSION LINE	_____
580	4 (Cyan)	–	DIMENSION TEXT 2.0 mm	DIMENSION TEXT
581	3 (Green)	–	TEXT 2.0 mm	TEXT
582	7 (White)	–	TEXT 2.5 mm	TEXT
583	2 (Yellow)	–	TEXT 3.0 mm	TEXT
584	11	–	TEXT 3.5 mm	TEXT
585	13	–	TEXT 4.5 mm	TEXT
589	9	–	DIMENSION TEXT 2.5 mm	DIMENSION TEXT
590	8	Continuous	HATCH BOUNDARY	(Not to be plotted)
591	1 (Red)	Continuous	HATCH 1	_____
592	3 (Green)	Continuous	CONCRETE PATTERN	_____
593	4 (Cyan)	Continuous	HATCH 2	_____
594	5 (Blue)	Continuous	HATCH 3	_____
596	9	Continuous	HATCH 4	_____
UPDATE	7 (White)	–	REVISION INFORMATION SHEET	(Not to be plotted)

		COLOUR No.	1	2	3	4	5	6	7	8	9	10	11	12	13	14
PEN THICKNESS	0.25mm															
	0.35mm															
	0.50mm															
	0.70mm															

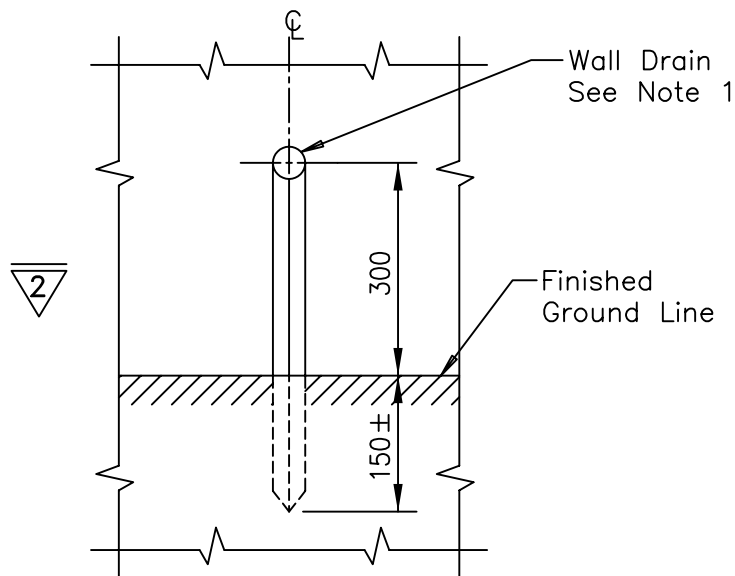
STRUCTURAL STANDARD DRAWINGS LAYERING STRUCTURE

Date MARCH 1997 Rev

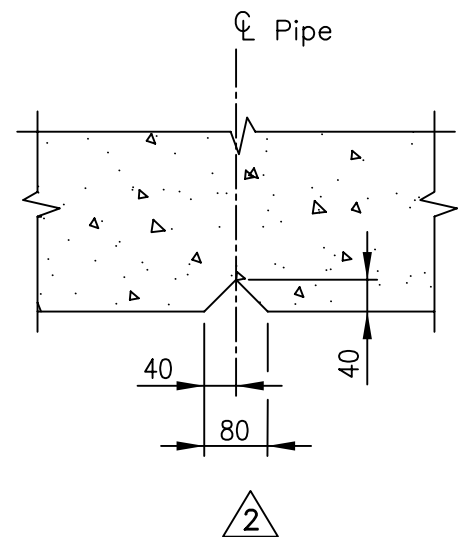
SS2–1



RETAINING WALL GROOVES



GROOVE BELOW DRAINS



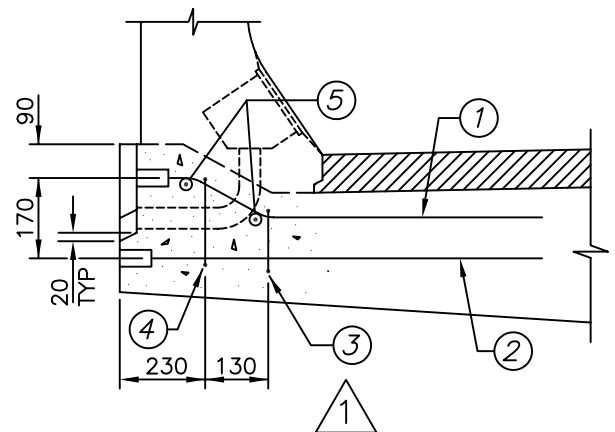
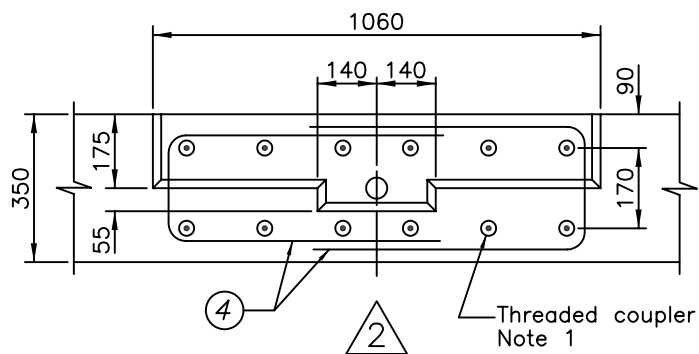
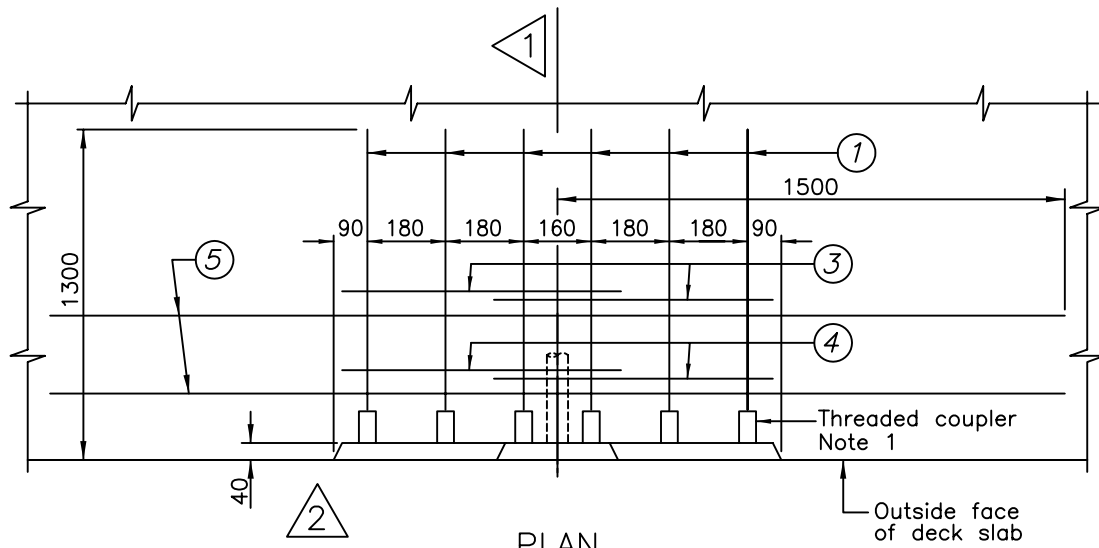
NOTES:

- 1) Where specified wall drains shall be installed as per OPSD 3506.000.
- A) All rustication grooves shall be horizontal.
- B) All horizontal form panel joints shall coincide with grooves and be completely hidden by them.
- C) Horizontal rustication grooves shall be used on fill or toe walls only and when the exposed face exceeds 3000mm.
- D) All dimensions are in millimeters unless otherwise specified.

RETAINING WALL — RUSTICATION DETAILS

Date JUNE 2002 Rev

SS5-2



MARK	SIZE	SHAPE
①	C20M	
②	C20M	

MARK	SIZE	SHAPE
③	C15M	
④	C15M	
⑤	C15M	

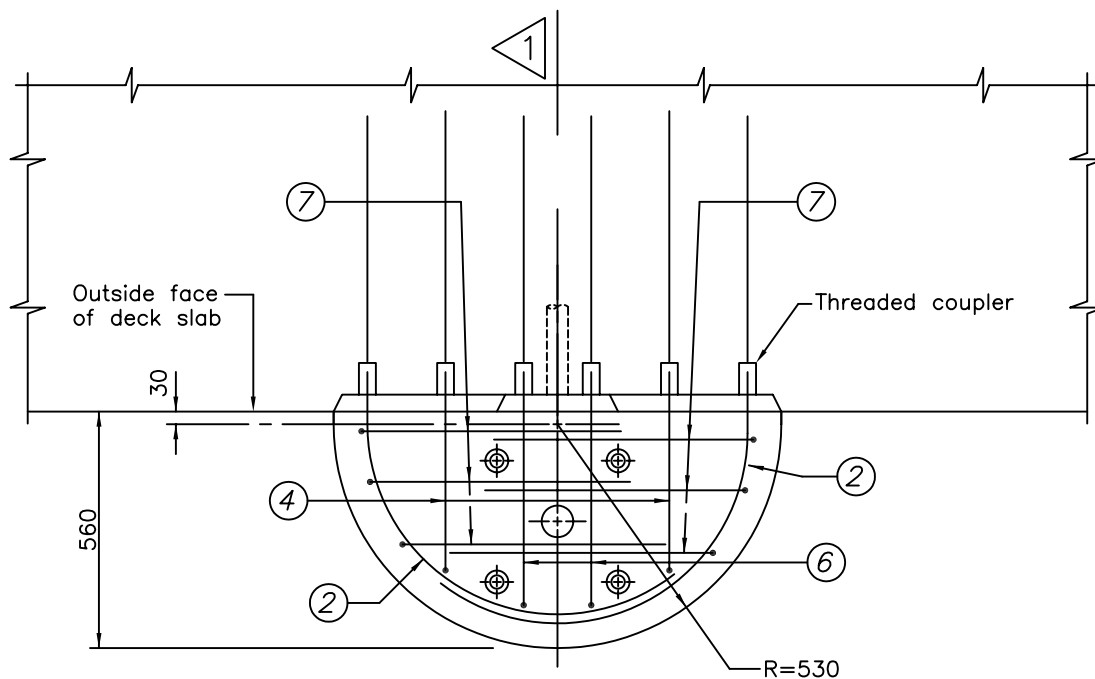
NOTES:

- 1 Threaded couplers shall be hot dip galvanized according to CSA Standard G164-M92.
- A Reinforcing steel for deck and barrier wall not shown.
- B Cover to reinforcing steel 70 ± 20 mm.
- C Read in conjunction with SS9-16 and bridge electrical details.
- D Provide galvanized cap-in to threaded couplers and cap on electrical conduit for protection against obstructions until base is cast.
- E All dimensions are in millimetres or metres unless otherwise shown.

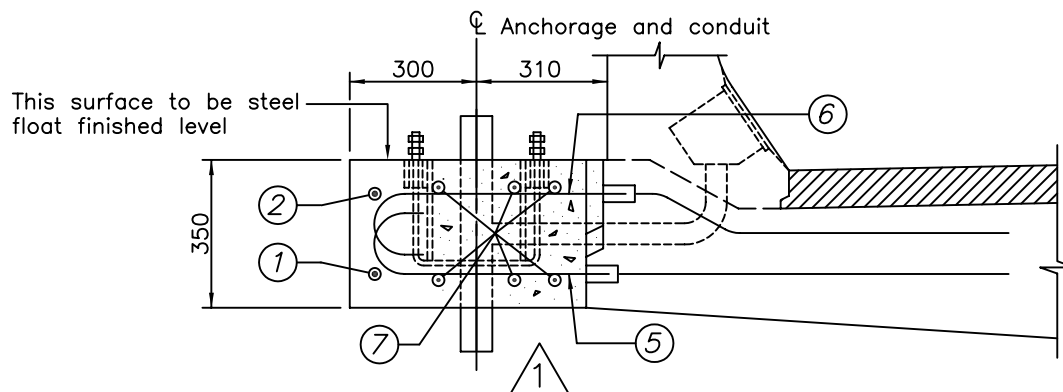
PROVISION FOR FUTURE LIGHT POLE BASE

Date MARCH 1997 Rev

SS9-15



PLAN
(Barrier wall not shown)



MARK	SIZE	SHAPE
①	C20M	
②		

A = 30 for ①
A = 70 for ②

MARK	SIZE	SHAPE
③	C20M	
④		
⑤		
⑥		
⑦	C15M	

B = 406 for ③ B = 480 for ⑤
B = 446 for ④ B = 520 for ⑥

NOTES:

A Reinforcing steel for deck and barrier wall not shown.

B Cover to reinforcing steel 70 ± 20 mm.

C Read in conjunction with SS9-15 and bridge electrical details.

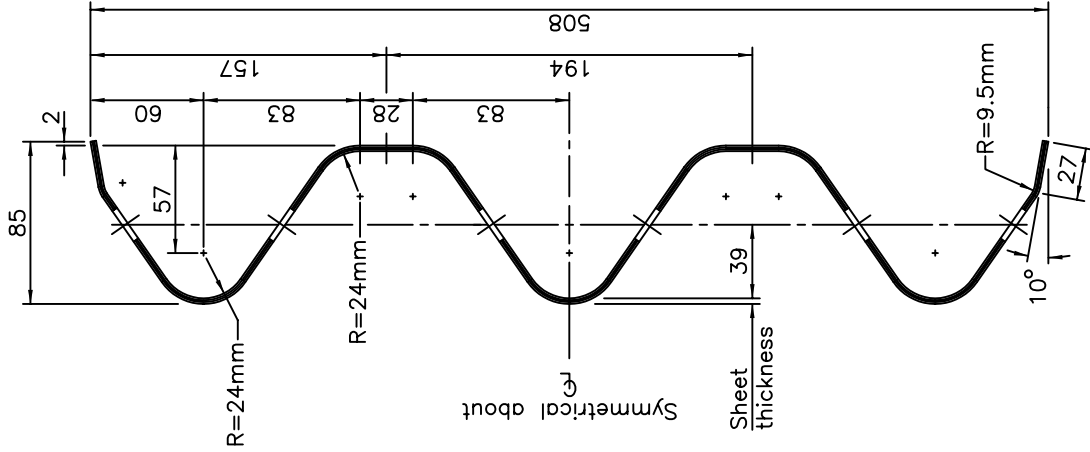
D Strength of anchorage based on 4-32mm dia studs equally spaced on 406mm Bolt Circle Diameter.

E All dimensions are in millimetres or metres unless otherwise shown.

RETROFIT LIGHT POLE BASE

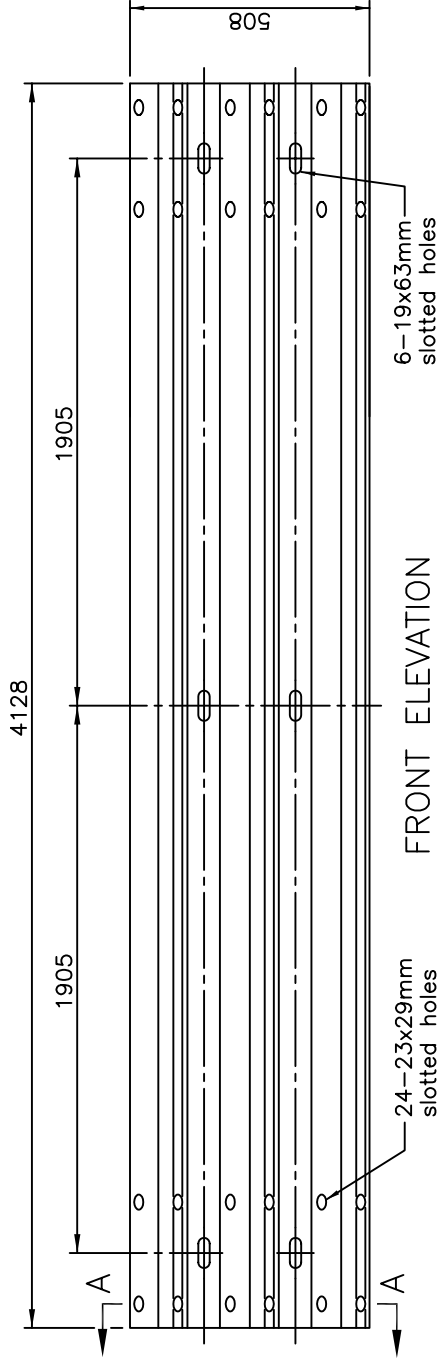
Date MARCH 1997 Rev

SS9-16

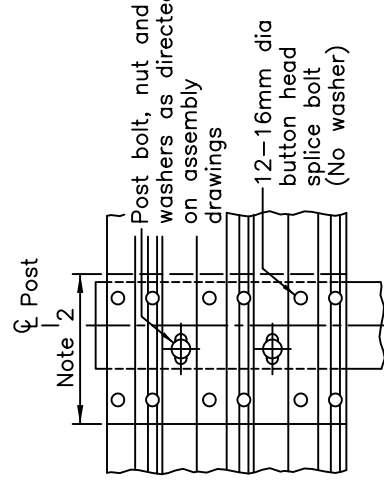


ENLARGED SECTION A-A

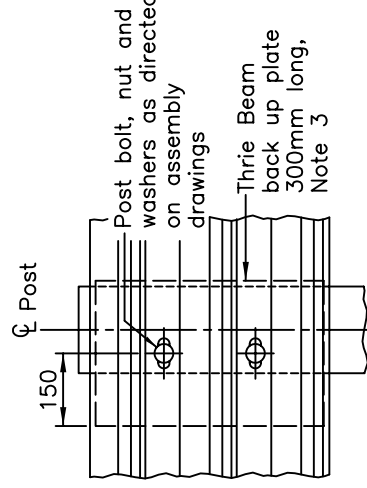
Note 1



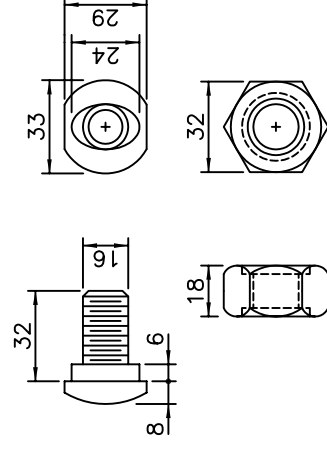
FRONT ELEVATION



SPLICE DETAIL



THRIE BEAM
BACK UP PLATE
DETAIL



SPLICE BOLT AND NUT

NOTES:

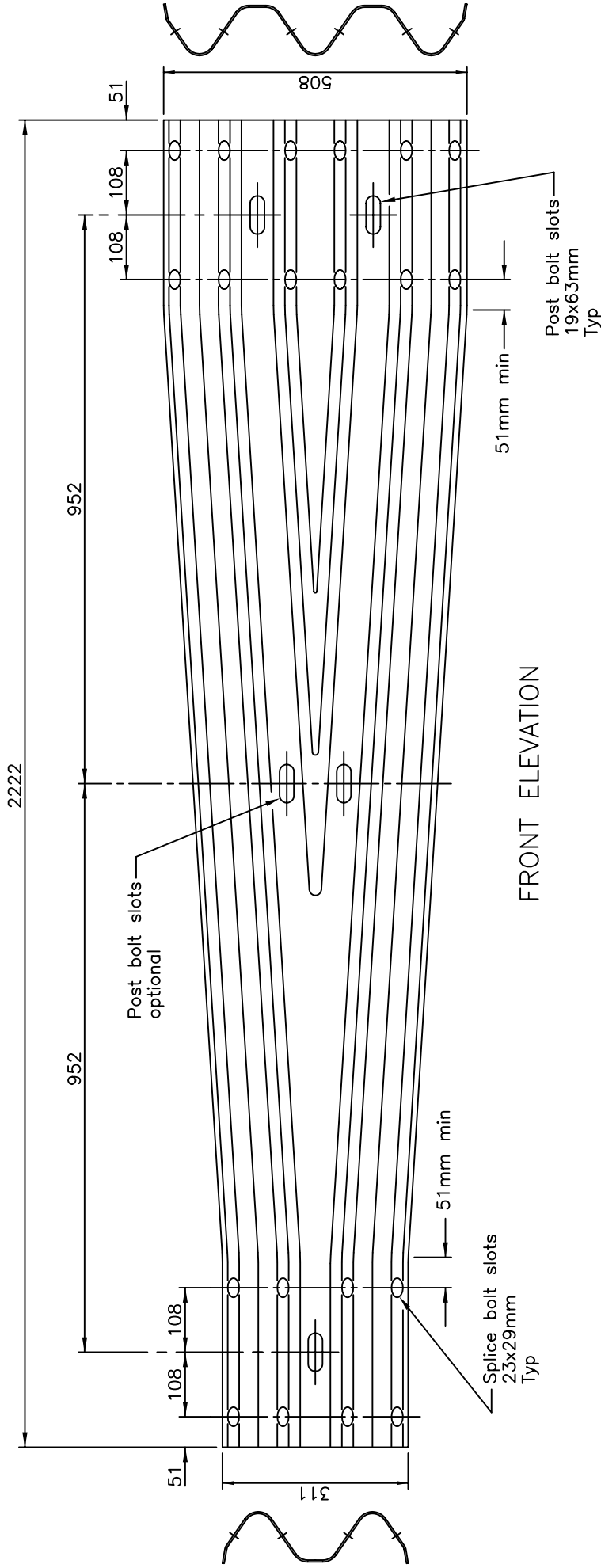
- 1 Thickness of rail shall be as shown on assembly drawings.
- 2 Overlap guide rail elements in direction of traffic flow.
- 3 The 300mm section of Thrie Beam (back up plate) shall be installed at intermediate posts only.

A Thrie Beam shall conform to AASHTO Specification M180 Type 2.

B All dimensions subject to manufacturing tolerances unless otherwise indicated.

C All dimensions are in millimetres or metres unless otherwise shown.

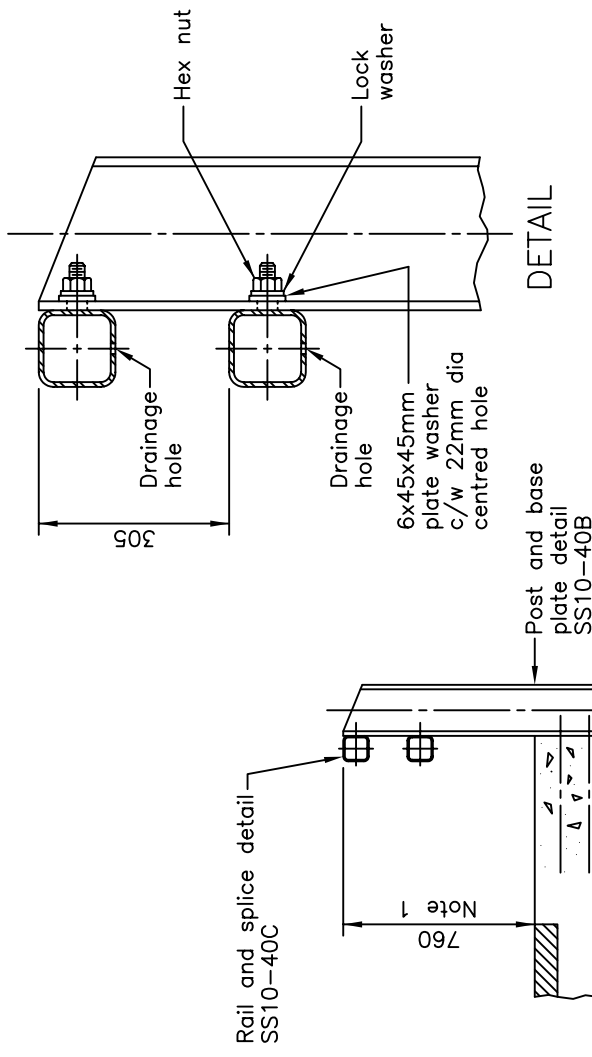
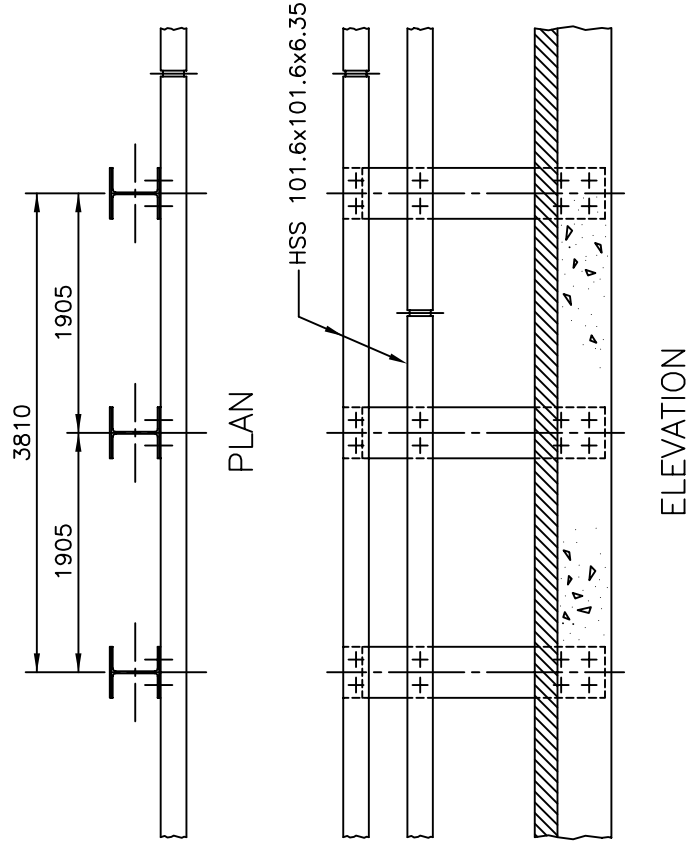
THRIE BEAM GUIDE RAIL		
RAIL DETAIL		
Date	JUNE 1993	Rev
SS10-20		



NOTES:

- A This Standard to be read in conjunction with SS10-20 and OPSD 902.01.
- B Overlap guide rail elements in direction of traffic flow.
- C Guide rail elements shall conform to AASHTO Specification M180 Type 2.
- D All dimensions subject to manufacturing tolerances unless otherwise indicated.
- E All dimensions are in millimetres or metres unless otherwise shown.

TRANSITION RAIL STEEL BEAM GUIDE RAIL TO THRIE BEAM		Date	JUNE 1993	Rev
		SS10-21		



ELEVATION

SIDE VIEW

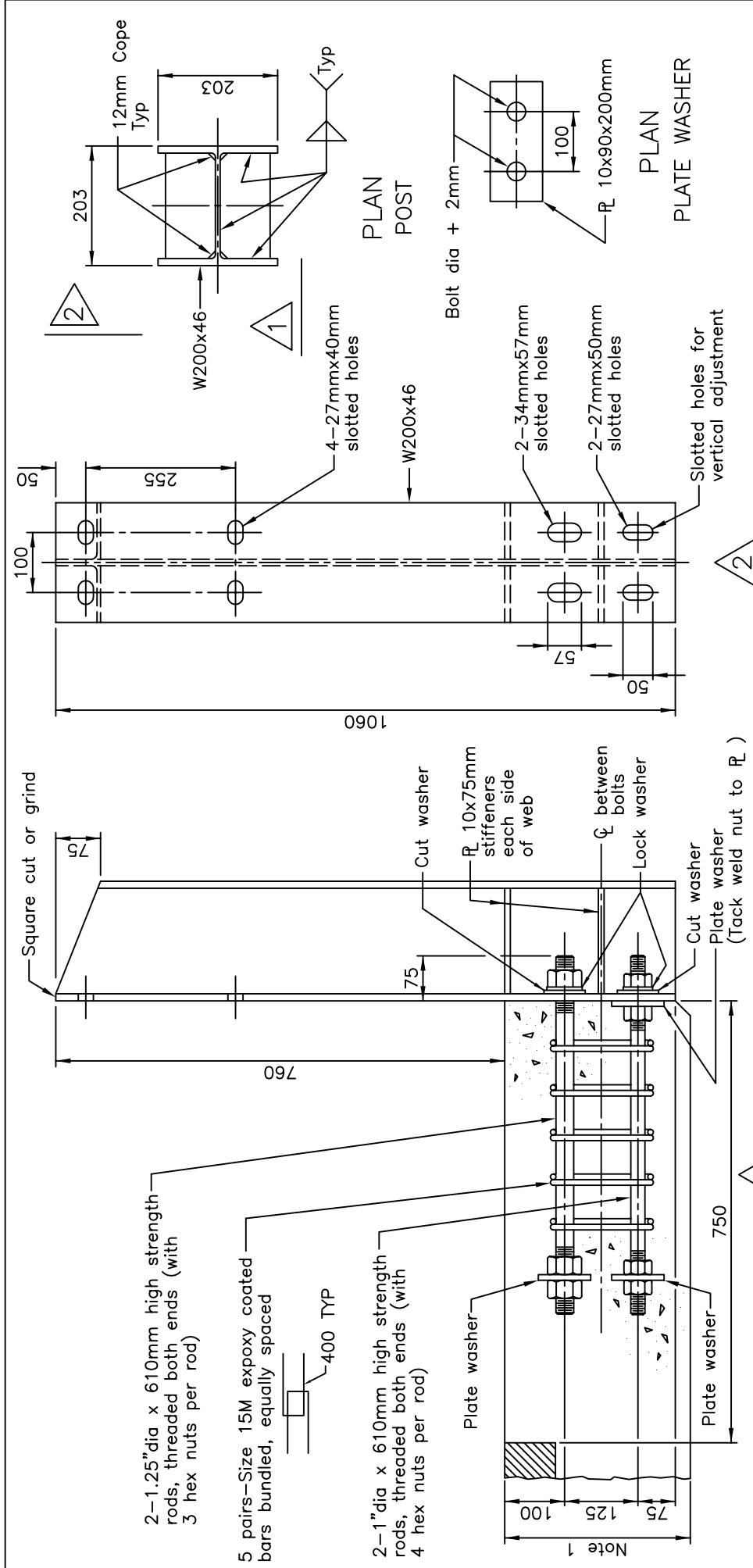
NOTES:

- 1 Mounting height shall be measured to top of guide rail.
- A This Standard to be read in conjunction with SS10-40B and SS10-40C.
- B Posts shall be vertical. Tubing shall be parallel to vertical alignment.
- C Tubing shall be shop bent to fit horizontal curve when curve radius is less than 290 m.
- D Guide rails shall be continuous over not less than 3 intermediate posts, with a minimum length of 3 panels, except as noted.
- E No more than one rail is to be spliced in each panel, except as noted.
- F Space posts to provide 500mm clear between expansion joints and $\varnothing$ of post.
- G Rail joints in top and bottom beams at deck expansion joints shall provide allowance for movement equal to width of deck joint with corresponding increase of length of sleeve.
- H Stud bolt nuts shall be torqued to 237 N.m.
- J All dimensions are in millimetres or metres unless otherwise shown.

BOX BEAM GUIDE RAIL – SIDE MOUNT
ASSEMBLY DETAIL
PERFORMANCE LEVEL 1

Date JUNE 1993 Rev

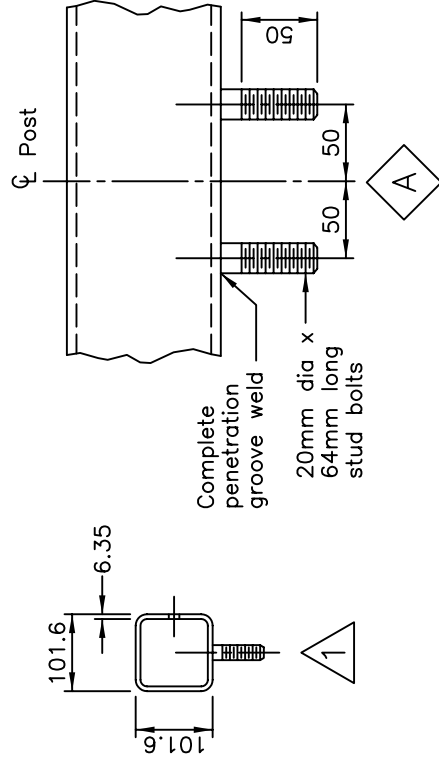
SS10-40A



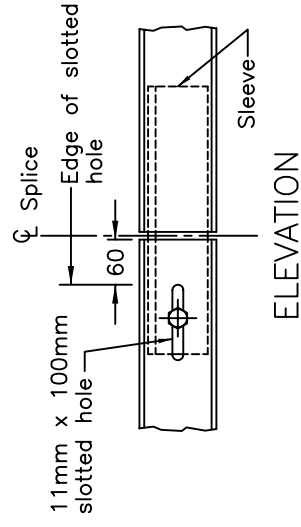
NOTES:

- 1 Minimum deck thickness: 325mm.
- A This Standard to be read in conjunction with SS10-40A and SS10-40C.
- B W200x46 shall conform to CSA Standard G40.20/40.21-98 Grade 350W.
- C Steel plate shall conform to CSA Standard G40.21-M92 Grade 300W.
- D Reinforcing steel shall conform to CSA Standard G30.18-M92 Grade 400.
- E Anchor rods shall conform to ASTM Specification A449. They shall be fitted with matching heavy hex nuts and washers.
- F Anchor bolt nuts shall be wrench tight.
- G The anchorage assembly (including bolts, nuts, and washers), and posts, shall be hot dip galvanized in conformance to CSA Standard G164-M92.
- H All dimensions are in millimetres or metres unless otherwise shown.

BOX BEAM GUIDE RAIL – SIDE MOUNT		Date	NOV 2001	Rev	1
POST AND ATTACHMENT DETAILS					
PERFORMANCE LEVEL 1					
SS10-40B					

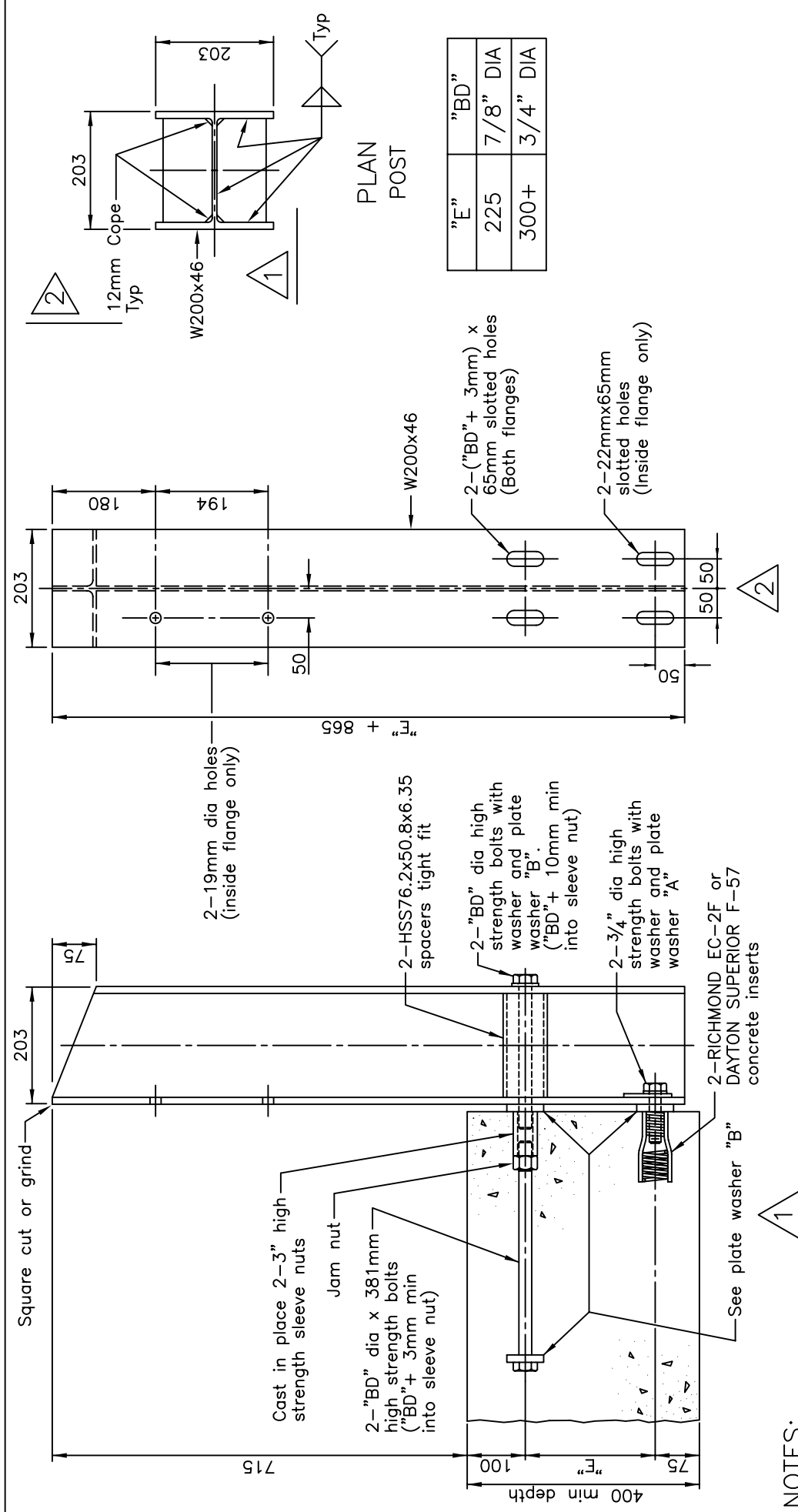


- A This Standard to be read in conjunction with SS10-40A and SS10-40B.
- B HSS guide rail shall conform to CSA Standard CAN3-G40.21-M92 Grade 350W.
- C Steel plate shall conform to CSA Standard G40.21-M92 Grade 300W.
- D Stud bolts shall conform to ASTM Specification A449.
- E 10mm nut may be replaced with drilled and tapped hole in sleeve.
- F Welding shall conform to CSA Standard W59-M89 and shall be done by a welder qualified under CSA Standard W47-M83.
- G HSS guide rail, rail splices and fasteners shall be hot dip galvanized after shop fabrication in conformance to CSA Standard G164-M92.
- H All dimensions are in millimetres or metres unless otherwise shown.



BOX BEAM GUIDE RAIL – SIDE MOUNT RAIL AND SPLICE DETAIL

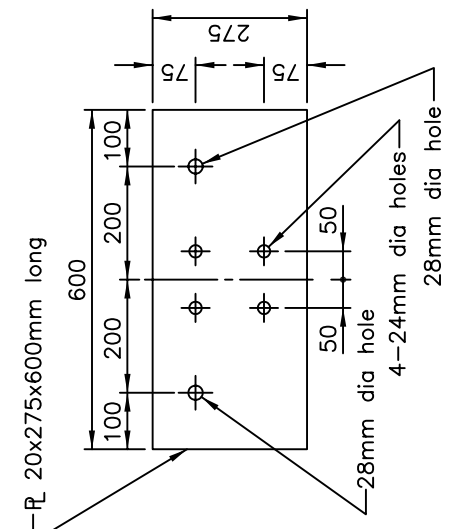
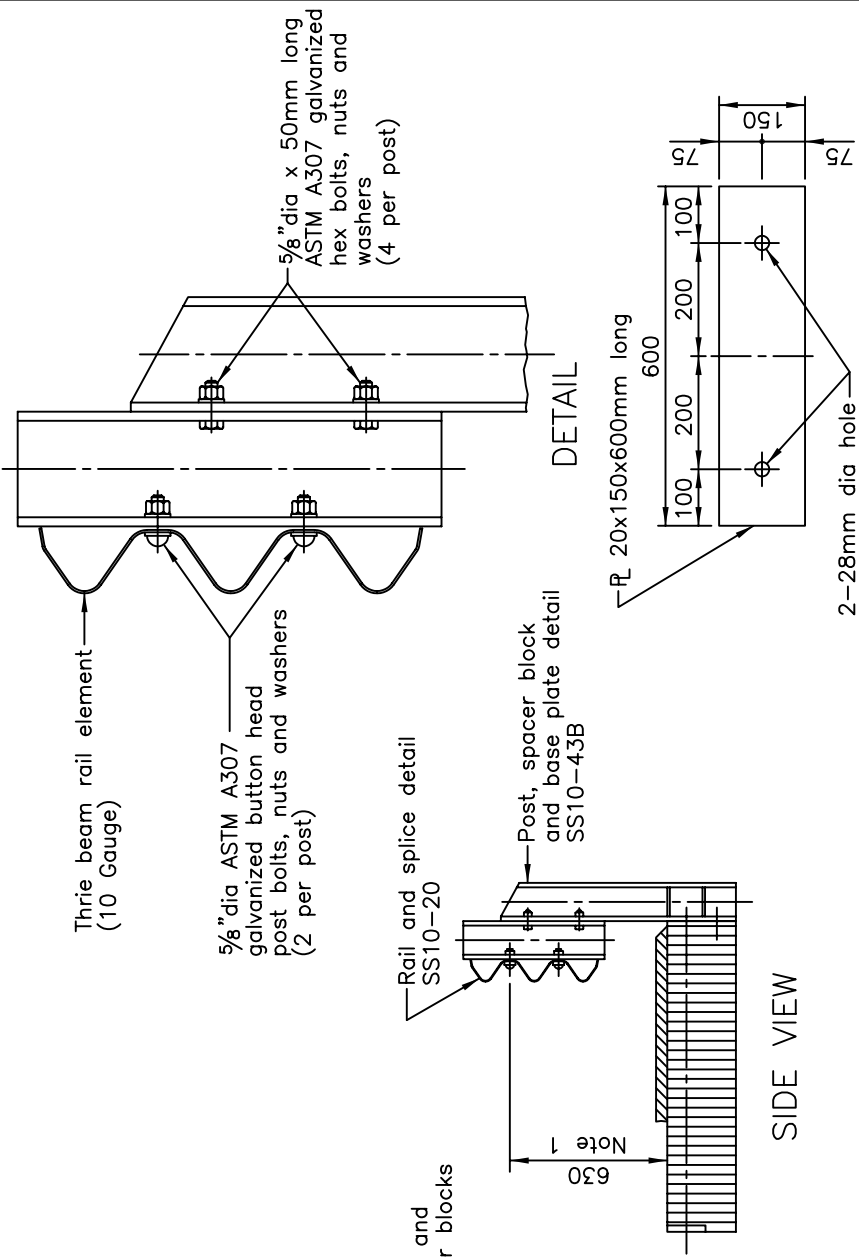
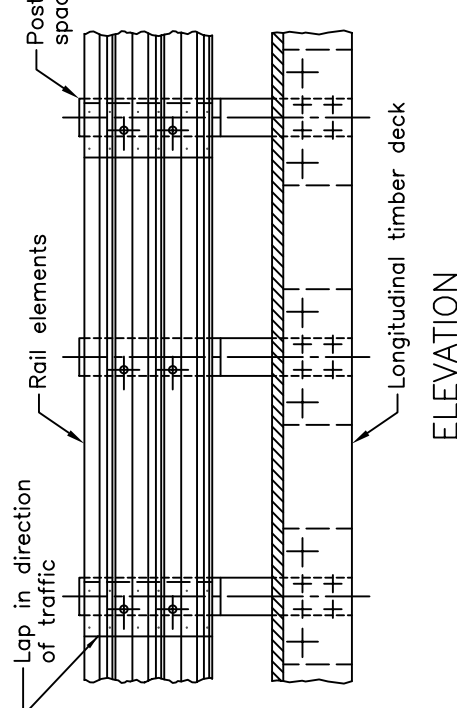
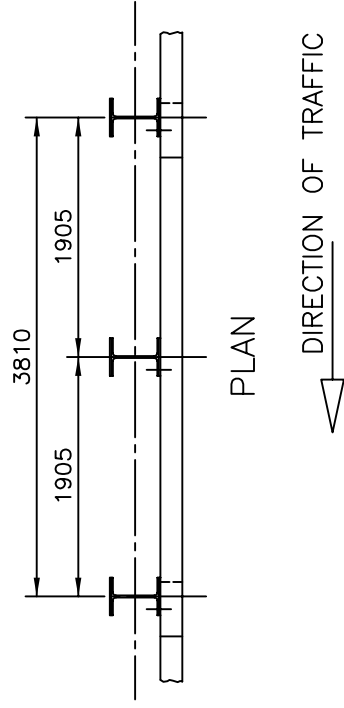
SS10-42A



NOTES:

- A This Standard to be read in conjunction with SS10-42A and SS10-20.
- B W200x46 shall conform to CSA Standard G40.20/40.21-98 Grade 350W.
- C Steel plate shall conform to CSA Standard G40.21-M92 Grade 300W.
- D Anchor bolts shall conform to ASTM Specification A325. They shall be fitted with matching sleeve nuts, heavy hex nuts and washers.
- E Turn upper high strength post bolts 2/3 turn past snug tight condition.
- F Tighten lower high strength post bolts 1/3 turn past snug condition.
- G The anchorage assembly, (including bolts, nuts, and washers), and posts, shall be hot dip galvanized in conformance to CSA Standard G164-M92.
- H All dimensions are in millimetres or metres unless otherwise shown.

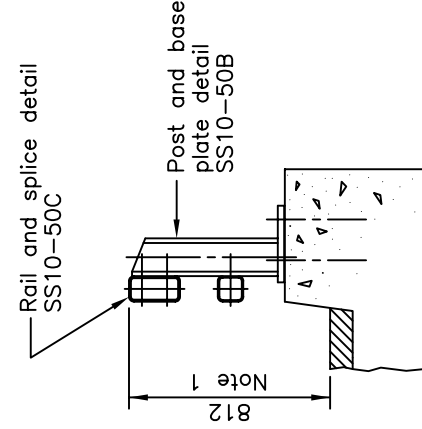
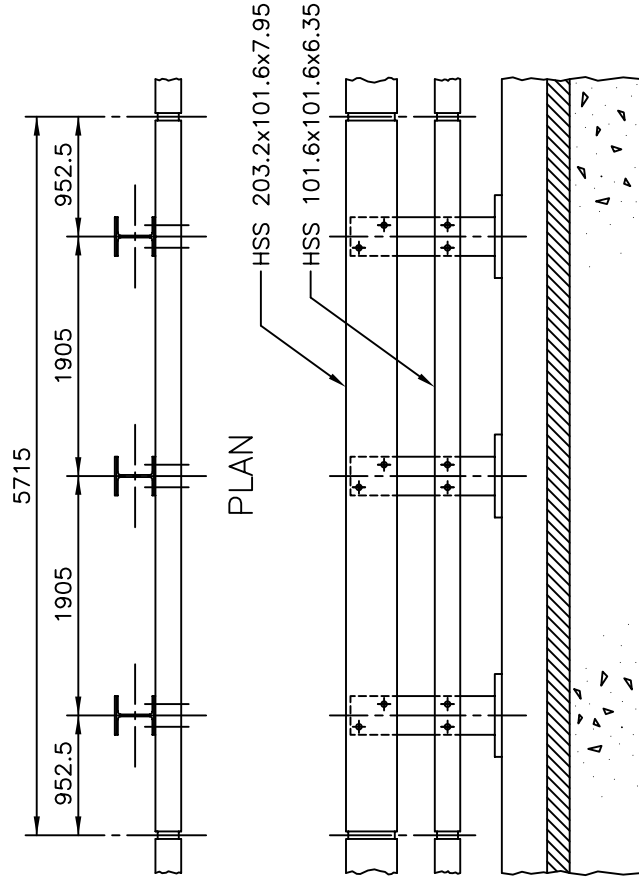
THRIE BEAM GUIDE RAIL - SIDE MOUNT		Date	NOV 2001	Rev	1
POST AND ATTACHMENT DETAILS					
		PERFORMANCE LEVEL 1			
		SS10-42B			



NOTES:

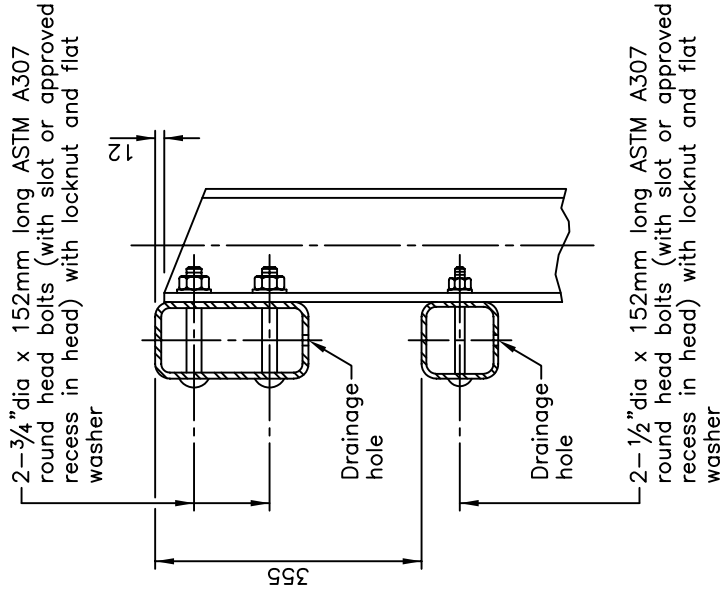
- 1 Mounting height of thrie beam above deck shall be measured to centre line of top bolt.
- A This Standard to be read in conjunction with SS10-43B and SS10-20.
- B Railing shall be fabricated to the horizontal and vertical alignment of the structure.
- C Posts and spacer blocks shall be installed normal to grade.
- C Field ream bolt holes in rail at splice locations. Repair damaged galvanizing with zinc-rich paint.
- D All dimensions are in millimetres or metres unless otherwise shown.

THRIE BEAM GUIDE RAIL – TIMBER DECK			Date	JUNE 1993	Rev
ASSEMBLY DETAIL					
			PERFORMANCE LEVEL 1		
			SS10-43A		



ELEVATION

SIDE VIEW

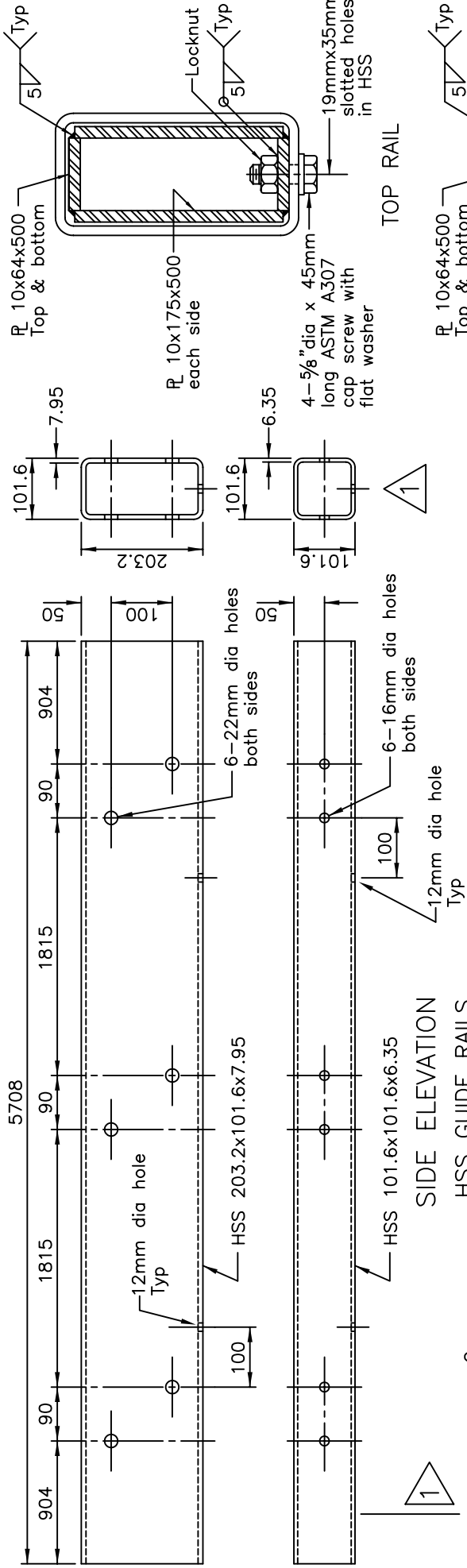


DETAIL

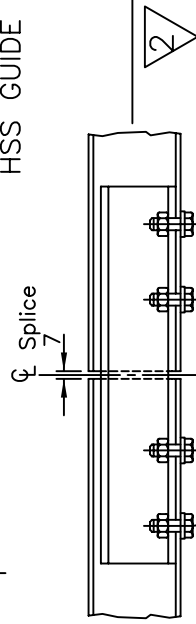
NOTES:

- 1 Mounting height shall be measured to top of guide rail.
- A This Standard to be read in conjunction with SS10-50B and SS10-50C.
- B Posts to be set vertical.
- C All dimensions are in millimetres or metres unless otherwise shown.

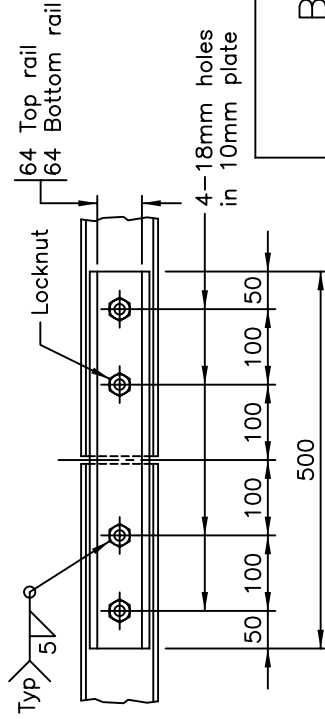
BOX BEAM GUIDE RAIL ON CURB			
ASSEMBLY DETAIL			
PERFORMANCE LEVEL 2			
Date	JUNE 1993	Rev	
SS10-50A			



SIDE ELEVATION
HSS GUIDE RAILS



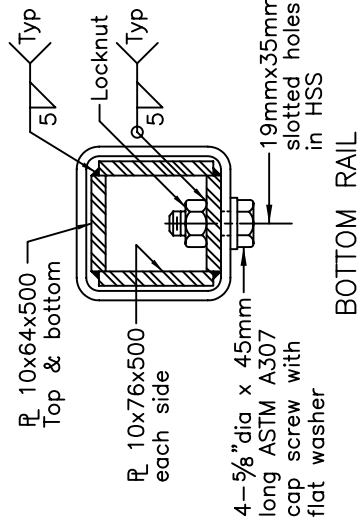
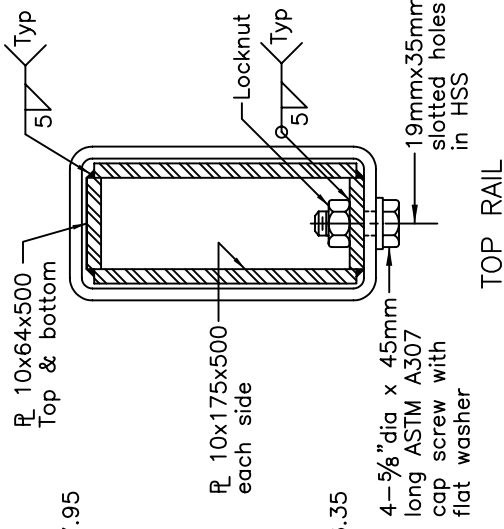
SECTION
SPLICE



NOTES:

- This Standard to be read in conjunction with SS10-50A and SS10-50B.
- HSS guide rails shall conform to CSA Standard CAN3-G40.21-M92 Grade 350W.
- Steel plate shall conform to CSA Standard G40.21-M92 Grade 300W.
- Holes in HSS guide rails may be drilled in the field.
- Welding shall conform to CSA Standard W59-M89 and shall be done by a welder qualified under CSA Standard W47-M83.
- HSS guide rails, rail splices and fasteners shall be hot dip galvanized in conformance to CSA Standard G164-M92.
- All dimensions are in millimetres or metres unless otherwise shown.

SECTIONS AT RAIL SPLICE



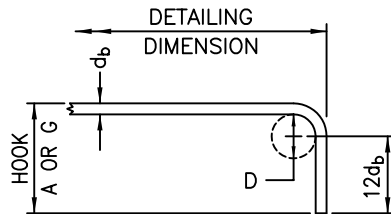
BOX BEAM GUIDE RAIL ON CURB

RAIL AND SPLICE DETAIL

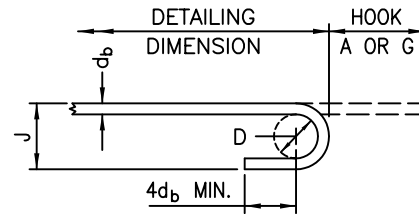
PERFORMANCE LEVEL 2

Date	JUNE 1993	Rev

SS10-50C



STANDARD 90° HOOK



STANDARD 180° HOOK

MINIMUM BENDING PIN DIAMETER, D, mm

BAR SIZE	STEEL GRADE	
	400R ⁽²⁾	400W
10M	70	60
15M	100	90
20M	120	100
25M	150	150
30M	250	200
35M	300	250
45M	450 ⁽¹⁾	400
55M	600 ⁽¹⁾	550

(1) Special fabrication is required for bends exceeding 90° for bars of these sizes and grade.

(2) For stainless steel, with $F_y = 420$, use the same D as for 400R.

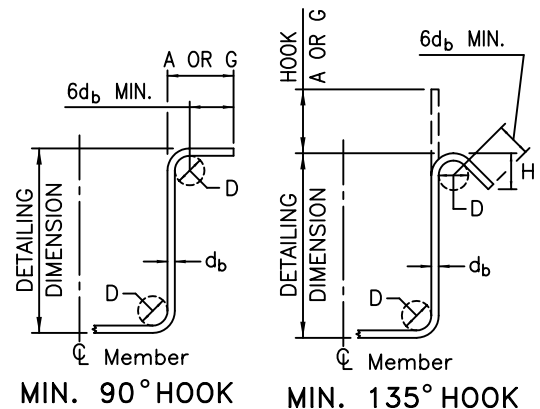
STANDARD HOOK DIMENSIONS

BAR SIZE	90° HOOKS		180° HOOKS			
	A OR G (mm)		A OR G (mm)		J (mm)	
	400R	400W	400R	400W	400R	400W
10M	180	180	140	130	90	80
15M	260	250	180	170	130	120
20M	310	300	220	200	160	140
25M	400	400	280	280	200	200
30M	510	490	400	350	310	260
35M	610	590	480	430	370	320
45M	790	770	680	630	540	490
55M	1030	1010	900	850	710	660

NOTE: All Hook Dimensions are according to the CHBDC-2000.

MINIMUM STIRRUP AND TIE HOOK DIMENSIONS

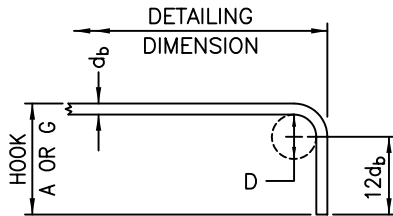
BAR SIZE	BAR DIAM. d_b (mm)	PIN DIAM. D(mm)	90°	135°	
			A OR G (mm)	A OR G (mm)	H (approx.) (mm)
10M	11.3	45	100	100	70
15M	16.0	65	140	140	100
20M	19.5	80	180	175	115
25M	25.2	100	230		



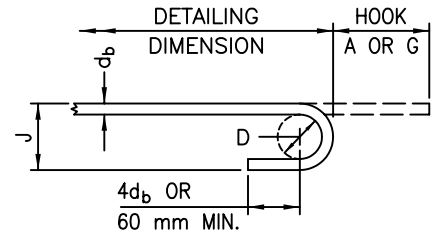
HOOK DIMENSIONS
FOR UNCOATED BARS

Date JUNE 2002 Rev

SS12-1



STANDARD 90° HOOK



STANDARD 180° HOOK

MINIMUM BENDING PIN
DIAMETER, D, mm

BAR SIZE	STEEL GRADE 400
10M	80
15M	120
20M	160
25M	200
30M	240
35M	350
45M	450
55M	550

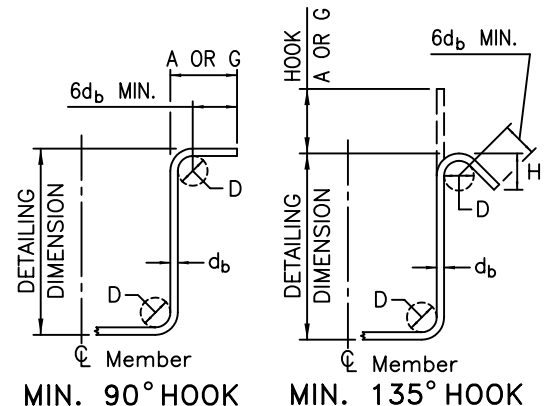
STANDARD HOOK DIMENSIONS

BAR SIZE	90° HOOKS	180° HOOKS	
	A OR G (mm)	A OR G (mm)	J (mm)
10M	190	210 *	110 *
15M	270	260 *	160 *
20M	330	300 *	200
25M	430	330	250
30M	510	460 *	300
35M	640	700 *	430 *
45M	790	850 *	540
55M	1010	1050 *	680 *

NOTE: Tabulated values for Standard Hook Dimensions are the larger of calculated (according to CHBDC-2000) and Reinforcing Steel Institute of Canada (RSIC) requirements. Value marked by '*' indicate RSIC recommended minimum.

MINIMUM STIRRUP AND TIE HOOK DIMENSIONS

BAR SIZE	BAR DIAM. d_b (mm)	PIN DIAM. D(mm)	90°	135°	
			A OR G (mm)	A OR G (mm)	H (approx.) (mm)
10M	11.3	90	180	190	120
15M	16.0	130	190	220	130
20M	19.5	160	230	260	150
25M	25.2	200	280		

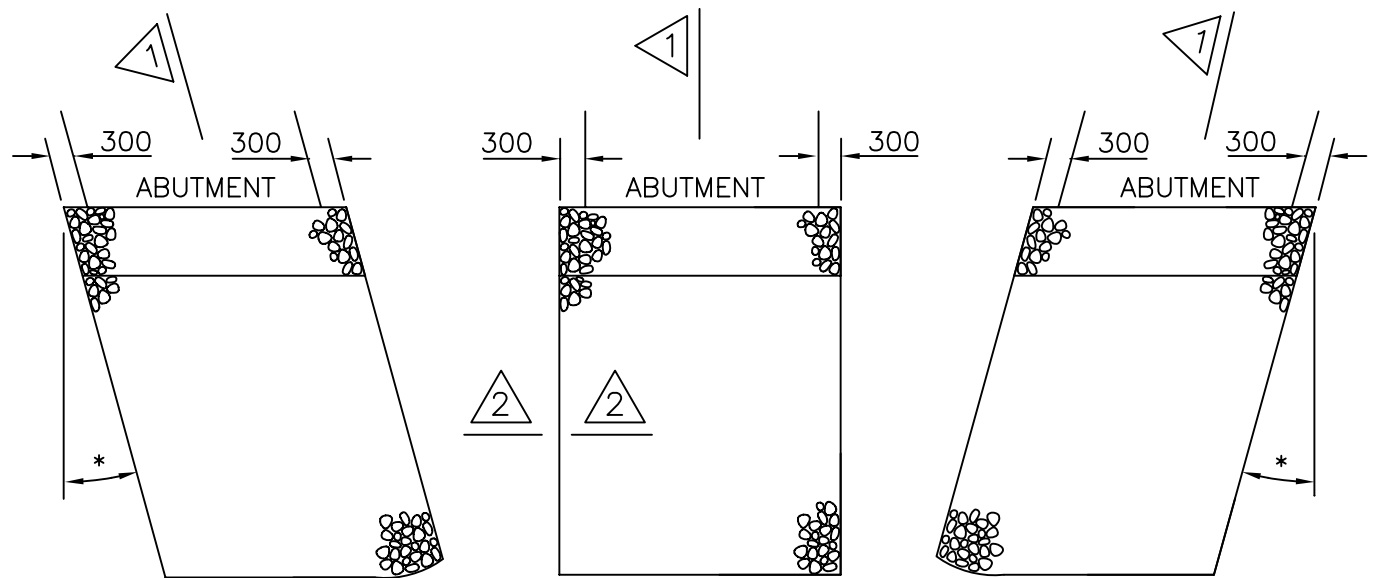


NOTE. Value marked by '**' indicates RSIC recommended minimum, based on the average of 90° and 180° hooks.

HOOK DIMENSIONS
FOR COATED BARS

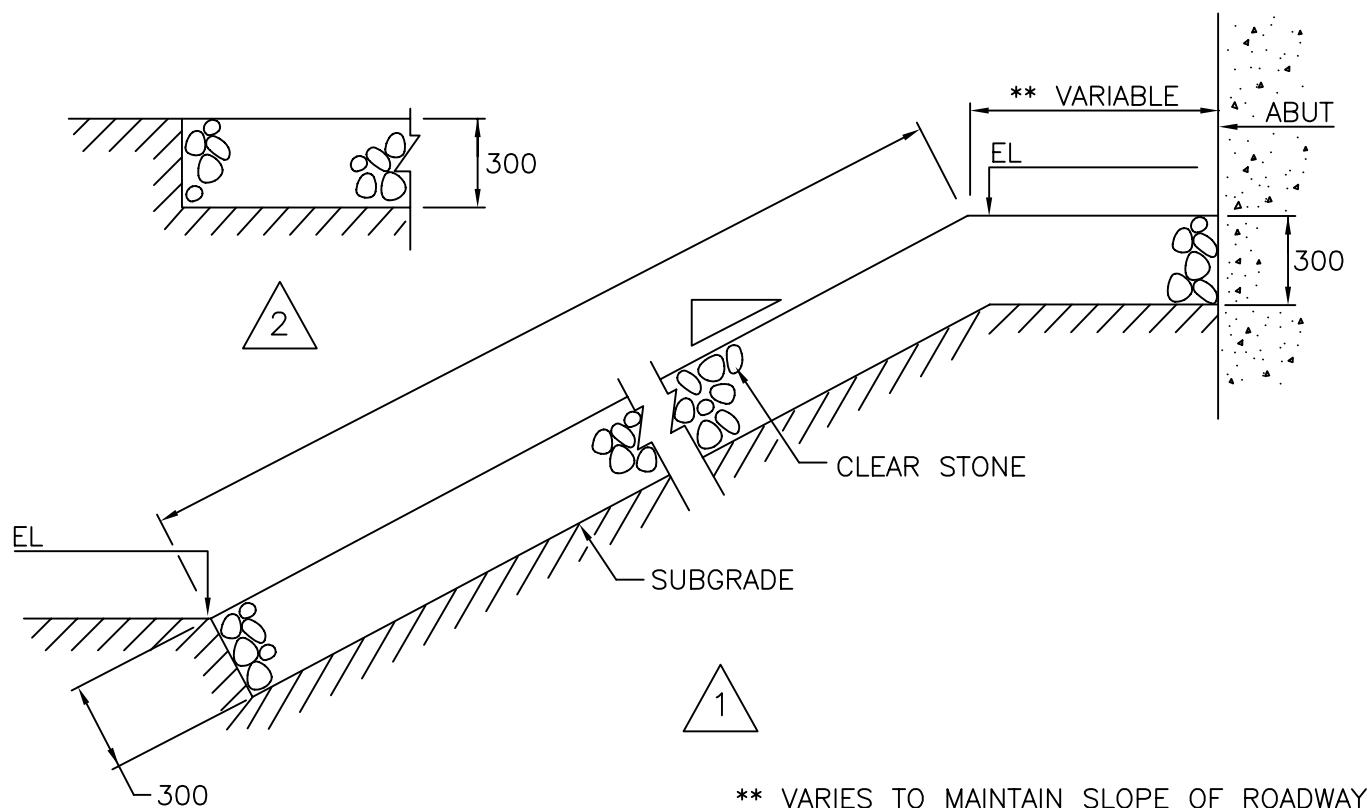
Date JUNE 2002 Rev

SS12-2



* SKEW TO BE SAME AS BRIDGE

SLOPE PROTECTION LAYOUTS



** VARIES TO MAINTAIN SLOPE OF ROADWAY CUT THROUGH THE BRIDGE

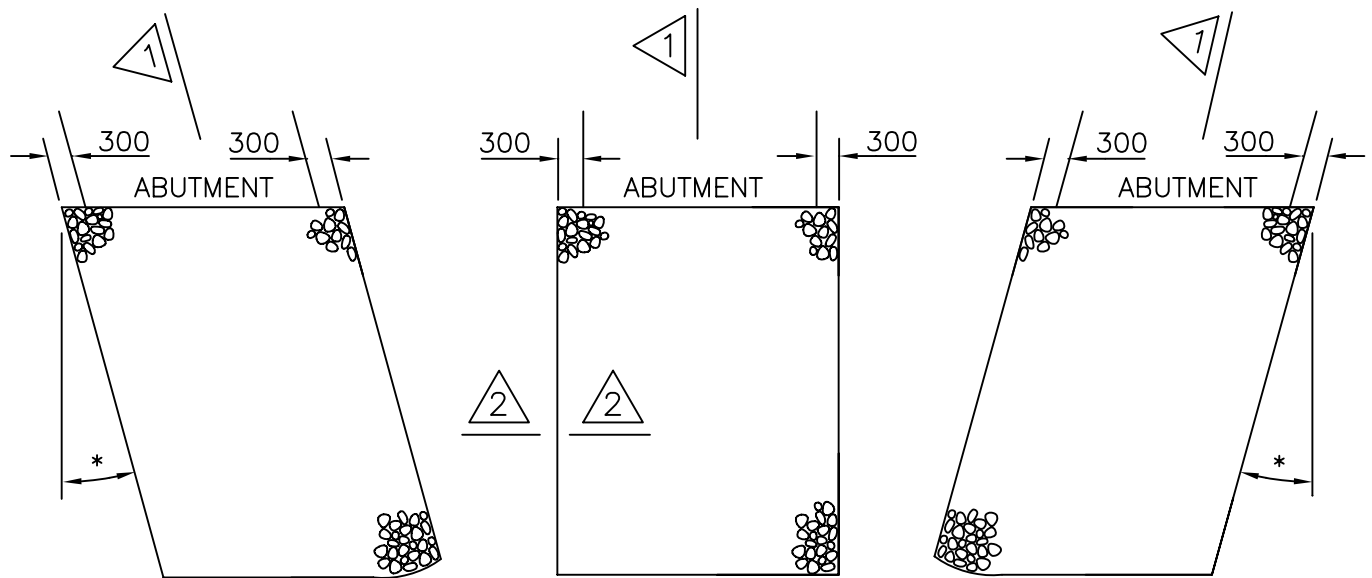
NOTES:

1 All dimensions are in millimetres or metres unless otherwise shown.

ROCK SLOPE PROTECTION WITH BERM

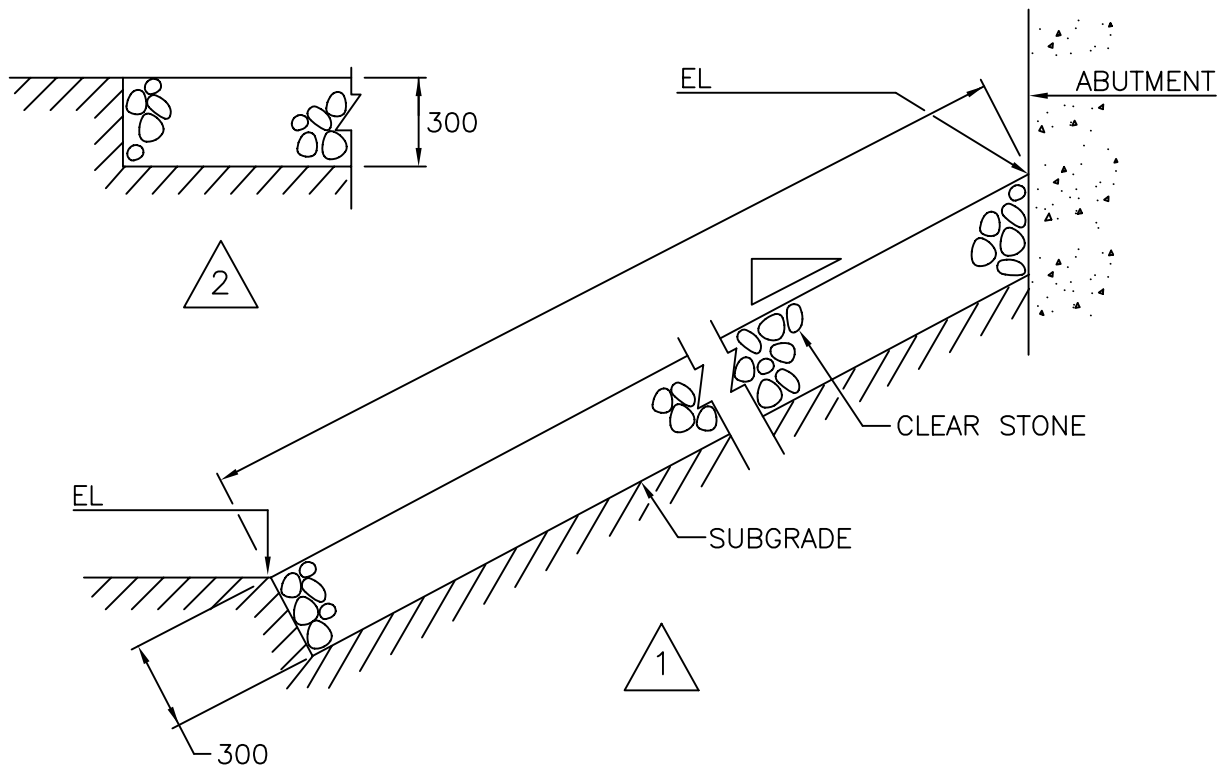
Date	MARCH 1997	Rev	

SS16-20



* SKEW TO BE SAME AS BRIDGE

SLOPE PROTECTION LAYOUTS

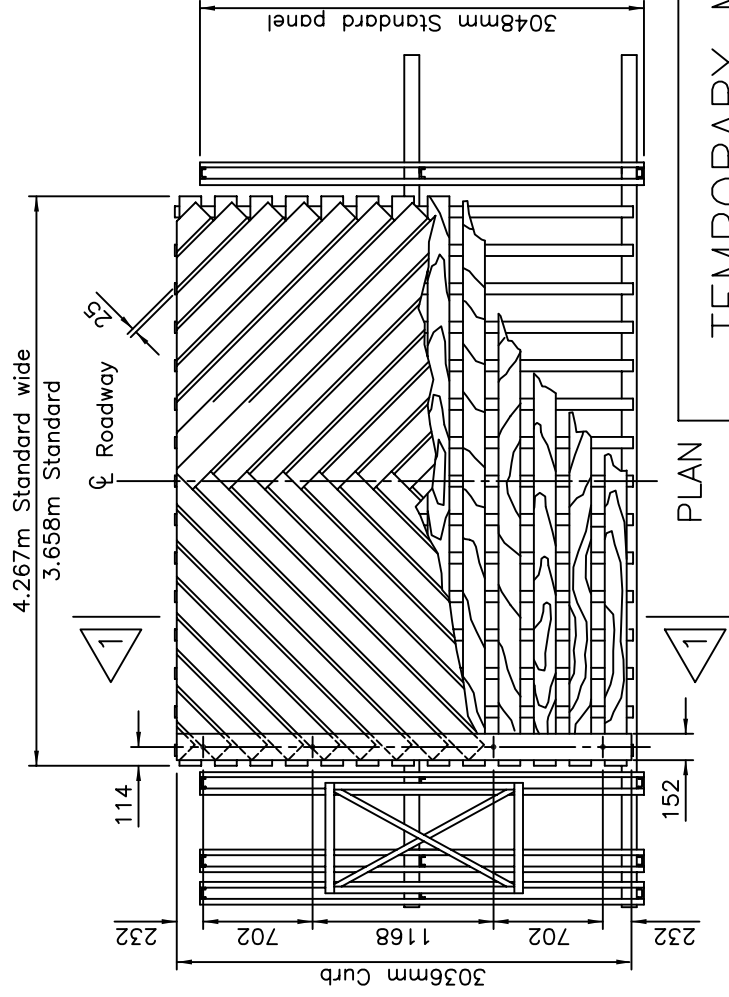
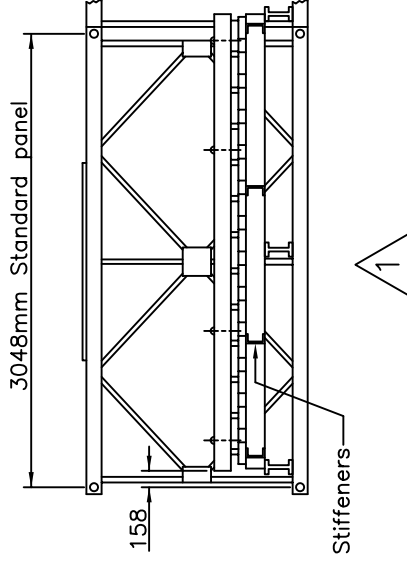
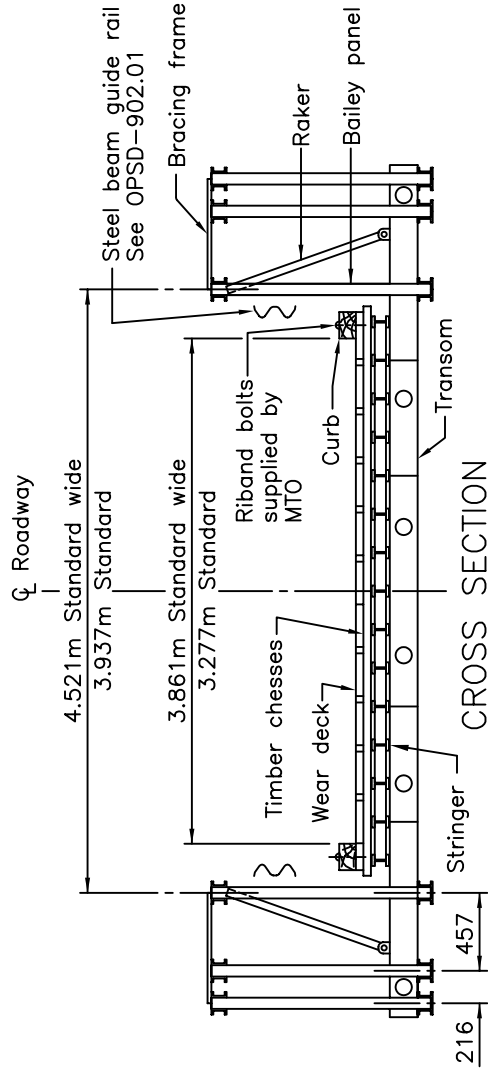


NOTES:

1 All dimensions are in millimetres or metres unless otherwise shown.

ROCK SLOPE PROTECTION WITHOUT BERM

Date	MARCH 1997	Rev	
SS16-21			



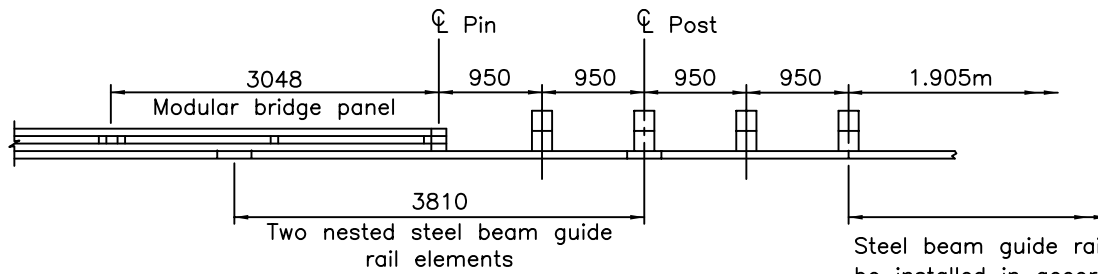
NOTES:

- A Timber chesses to be jack pine or red pine and consists of:
- Standard Wide – 38x140mm x 4.267m
 - Standard – 38x140mm x 3.658m
- B Wear deck to be jack pine or red pine and consists of:
- Standard Wide – 38x140mm x 2.845m
 - Standard – 38x140mm x 2.438m
- Deck to be spiked with 125mm galvanized ardox nails driven on a batter.
- C Curbs: 152x102mm x 3.036m with bolt holes 25mm dia to be drilled by the Contractor.
- D All dimensions are in millimetres or metres unless otherwise shown.

TEMPORARY MODULAR BRIDGE TIMBER DECKING FOR STANDARD AND STANDARD WIDE BRIDGES

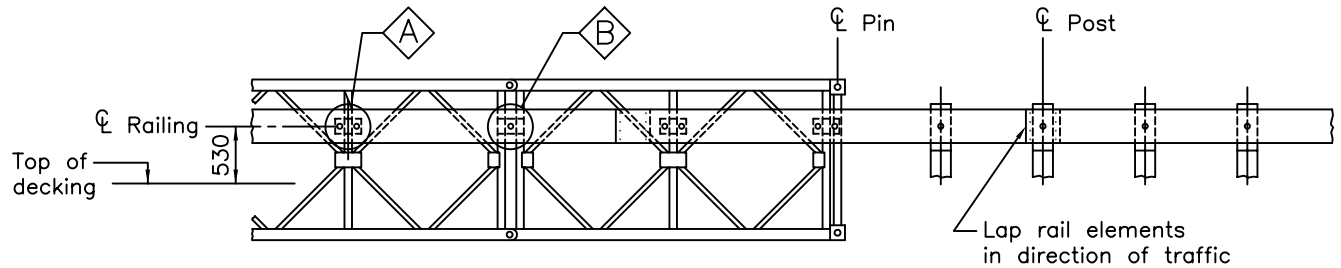
Date	MARCH 1997	Rev

SS16-40

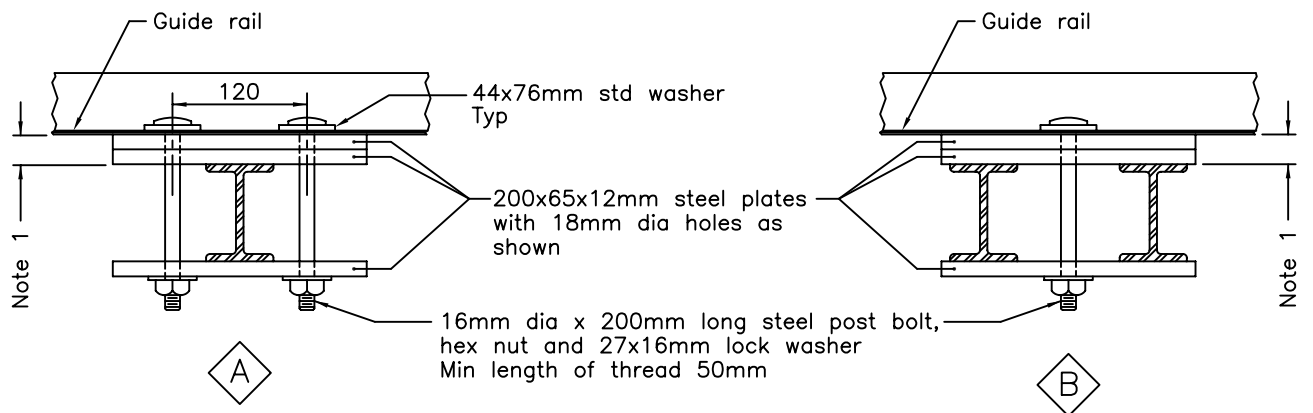


PLAN

Steel beam guide rail within this section shall be installed in accordance with OPSD-902.05 and the requirements of this contract



FRONT ELEVATION



NOTES:

- 1 An alternate hollow structural section, timber or other suitable spacer, bearing uniformly on the vertical leg of the bridge panel, may be used provided sufficient horizontal clearance for traffic is maintained. Where bridges have end panels with end posts, the spacer shall have adequate depth to ensure a smooth guide rail alignment.
- A This standard to be used in conjunction with OPSD-902.01.
- B Provide 23x29mm slotted holes in guide rail to suit details A and B.
- C For details of posts, offset blocks, bolts and washers see OPSD-902.03.
- D Alternate details shall be subject to approval by the Contract Administrator.
- E All connection elements, including steel post bolts, nuts and washers shall be hot dip galvanized according to CSA Standard G164-M92.
- F All dimensions are in millimetres or metres unless otherwise shown.

STEEL BEAM GUIDE RAIL FOR TEMPORARY MODULAR BRIDGES

Date MARCH 1997 Rev

SS16-41

STRUCTURAL MANUAL

DIVISION 4 - STRUCTURAL STANDARD DRAWINGS SECTION 2

December 2003

DECEMBER 2003	LIST OF STRUCTURAL STANDARD DRAWINGS	SS 101-1

STANDARD DRAWINGS - FULL SIZE DRAWINGS

The following standard drawings are issued in 914 x 610mm format, for use as full-size contract drawings. They require additional information before they can be included in the contract documents.

Digital file names are established by removing one "S" from the Standard Drawing name prefix, and adding the extension "DWG". The drawing files are in AutoCAD.

The layer named UPDATE in the digital file, contains a Revision Information Sheet for the current revision of the drawing.

1. INDEX OF STRUCTURAL STANDARD DRAWINGS

List of Structural Standard Drawings	SS 101-1
List of Structural Standard Drawings	SS 101-2
List of Structural Standard Drawings	SS 101-3

2. GENERAL DESIGN AND DRAUGHTING

3. PILES

Pile Driving Control	SS 103-11
Splice and Driving Shoe Details for Steel H-Piles	SS 103-12

4. FOOTINGS

5. ABUTMENTS, WINGWALLS AND RETAINING WALLS

Retaining Wall Details.....	SS 105-1
Wingwall Details for Bridges	SS 105-2

6. PIERS

7. PRESTRESSED CONCRETE

Prestressed Girders and Bearings (CPCI 900)	SS 107-1
Prestressed Girders and Bearings (CPCI 1200)	SS 107-2
Prestressed Girders and Bearings (CPCI 1400)	SS 107-3
Prestressed Girders and Bearings (CPCI 1600)	SS 107-4
Prestressed Girders and Bearings (CPCI 1900)	SS 107-5
Prestressed Girders and Bearings (CPCI 2300)	SS 107-6

8. STRUCTURAL STEEL

9. DECK AND CURBS

10. BARRIERS AND RAILINGS SYSTEMS

Railing for Barrier/Parapet Wall	SS 110-21
Two Tube Railing on Parapet – PL1	SS 110-40
Two Tube Railing on Parapet for Sidewalk – PL1	SS 110-41
Parapet Wall for Two Tube Railing – PL1.....	SS 110-42
Parapet Wall with Sidewalk for Two Tube Railing – PL1.....	SS 110-43
Barrier Wall with Railing - PL2 - Stainless Steel Rebar	SS 110-54
Parapet Wall with Railing - PL2 - Stainless Steel Rebar	SS 110-56
Parapet Wall on Sidewalk with Railing - PL2 - Stainless Steel Rebar	SS 110-57

DECEMBER 2003	LIST OF STRUCTURAL STANDARD DRAWINGS	SS 101-2

Barrier Wall without Railing - PL3 - Stainless Steel Rebar	SS 110-61
Reinforced Concrete Median Barrier Wall on Structures – Type I	SS 110-62
Reinforced Concrete Median Barrier Wall on Structures – Type II	SS 110-63

11. RIGID FRAME

12. REINFORCING STEEL

13. BEARING ASSEMBLIES AND EXPANSION JOINTS

Strip Seal Expansion Joint Assembly for Barrier Walls	SS 113-10
Strip Seal Expansion Joint Assembly for Barrier Walls - with Drainage System	SS 113-11
Strip Seal Expansion Joint - Type "A" - Details	SS 113-12
Strip Seal Expansion Joint - Type "C" - Details	SS 113-13
Expansion Joint Drainage System - Catch Basin and Downpipe - Details	SS 113-14
Strip Seal Expansion Joint Assembly for Parapet Walls	SS 113-15
Strip Seal Expansion Joint Assembly for Parapet Walls - with Drainage System	SS 113-16
Modular Expansion Joint with Injection Hose System - Assembly	SS 113-20
Modular Expansion Joint with Injection Hose System - Details I	SS 113-21
Modular Expansion Joint with Injection Hose System - Details II	SS 113-22

14. CULVERTS

15. WOOD STRUCTURES

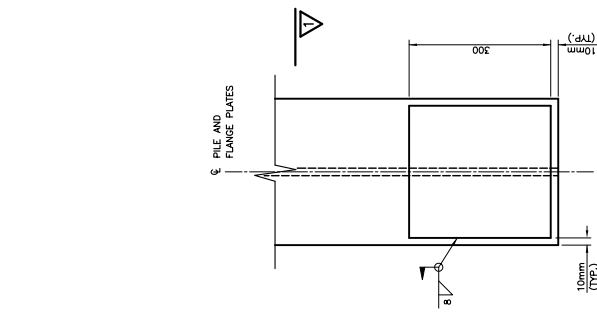
16. MISCELLANEOUS

6000mm Approach Slab	SS 116-1
Details of Concrete Slope Paving	SS 116-10
As Constructed Elevations and Dimensions	SS 116-40
High Mast Lighting Pole Footing – Ground Mounted	SS 116-50
High Mast Lighting Pole Footing – Median Tall Wall Mounted (Symmetrical)	SS 116-51
High Mast Lighting Pole Footing – Median Tall Wall Mounted (Asymmetrical)	SS 116-52

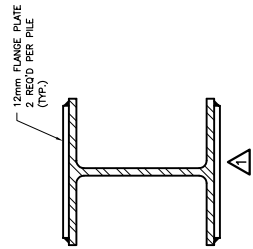
17. COMPUTER APPLICATIONS

18. SIGN SUPPORTS

- GENERAL NOTES:
- PILE ENDS TO BE SPICED SHALL BE CUT SQUARE PERPENDICULAR TO CENTRELINE OF PILE.
 - FLANGE AND SPICE PLATES SHALL BE ACCORDING TO CSA G40.20/G40.21-98 GRADE 300W.
 - WELDING SHALL BE ACCORDING TO CSA-W59M.
 - THIS STANDARD APPLIES TO H-PILE SIZES HP310x79, HP310x110, AND HP310x132.
 - HANDLING HOLES SHALL ONLY BE MADE IN THE PORTION OF THE PILE TO BE CUT OFF IN THE PORTION OF THE PILE IN THE CONCRETE PILE CAP.



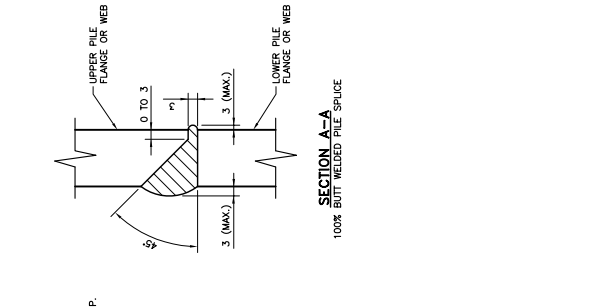
ELEVATION



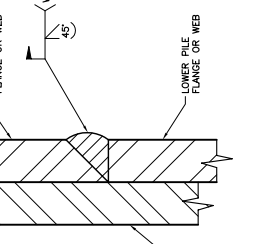
PILE DRIVING SHOES

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

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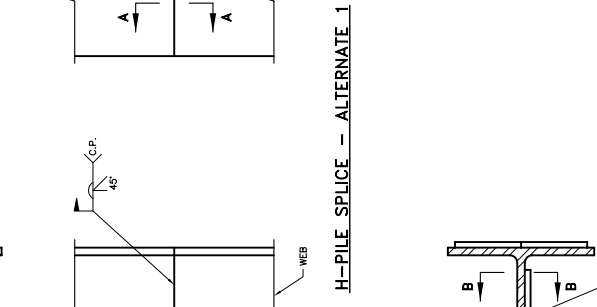
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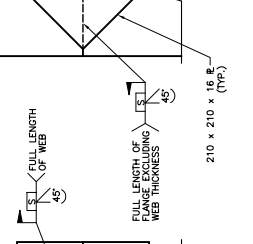
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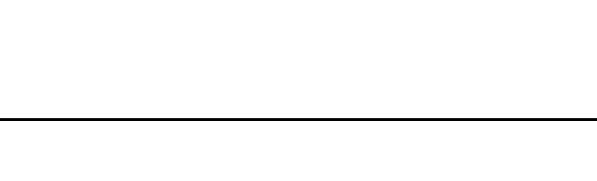
ELEVATION



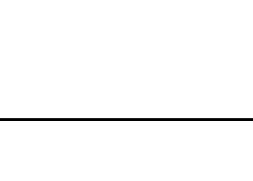
PILE DRIVING SHOES

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ELEVATION



PILE DRIVING SHOES

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
WP No

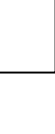
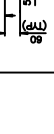
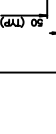
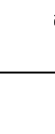
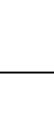
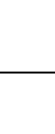
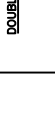
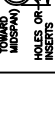
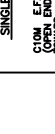
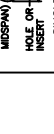
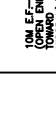
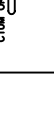
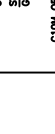
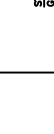
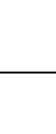
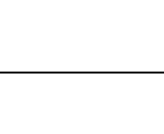
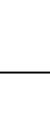
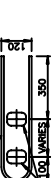
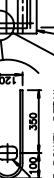
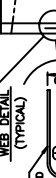
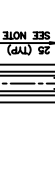
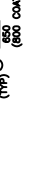
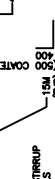
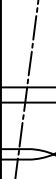
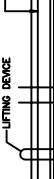
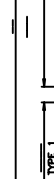
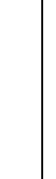
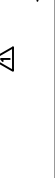
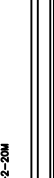
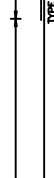
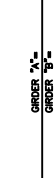
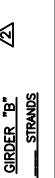
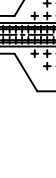
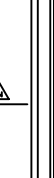
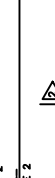
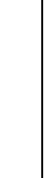
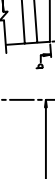
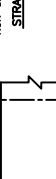
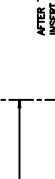
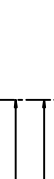
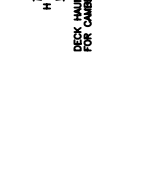
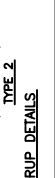
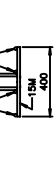
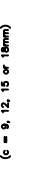
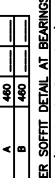
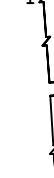
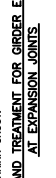
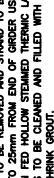
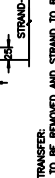
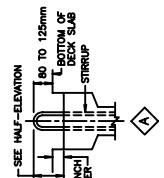
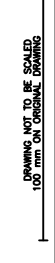
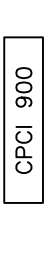
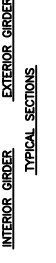
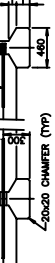
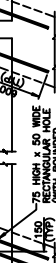
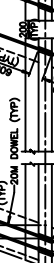
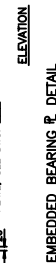
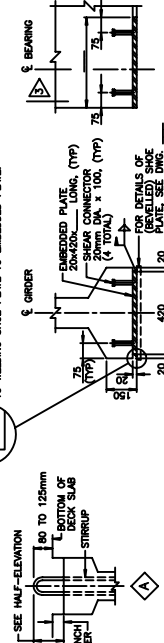
SHEET

PRESTRESSED GIRDERS AND BEARINGS

NOTES TO DESIGNER

1. FOR INTEGRAL ABUTMENT TYPE BRIDGES, THIS STANDARD DRAWING WILL NEED TO BE MODIFIED AS REQUIRED.
2. ANCHORAGE DETAIL SHALL BE AS SHOWN OR MAY BE USED ONLY IF STRUCTURE GRADE EXCEEDS 2.8% AND/OR IF ROTATIONAL BEARINGS ARE USED. THEY ARE THEN USED INSTEAD OF GIRDER SOFFIT BLOUTOOTS.
3. THE "NOTES TO DESIGNER" SHALL BE DELETED FROM THIS DRAWING PRIOR TO ISSUING OF THE CONTRACT.

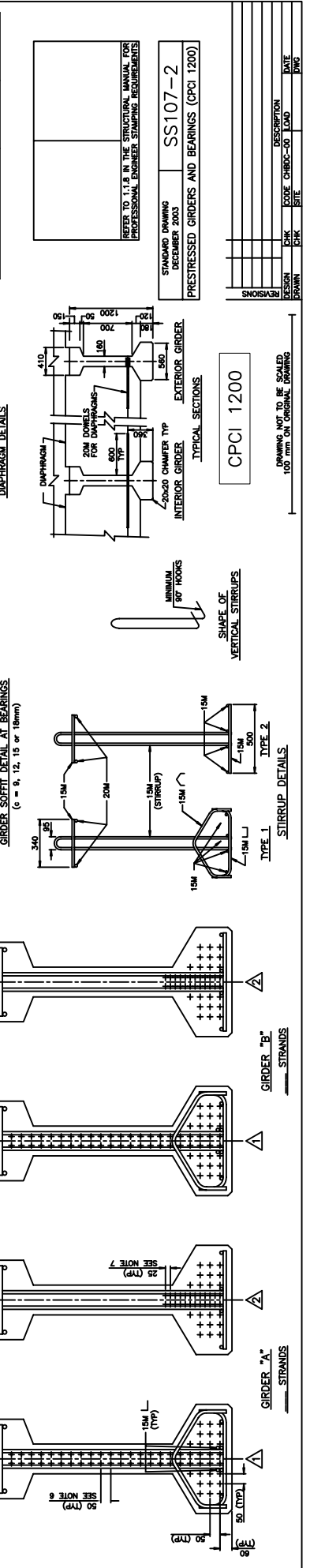
1. PRESTRESSING STEEL SHALL BE LOW-RELAXATION SEVEN WIRE STRANDS, SIZE DESIGNATION ____ GRADE 1860.
2. MINIMUM BREAKING STRENGTH OF STRAND ____ kN.
3. JACKING FORCE PER STRAND ____ kN.
4. FORCE PER STRAND AFTER ALL LOSSES ____ kN.
5. THE ELAPSED TIME INTERVAL BETWEEN JACKING OF STRANDS AND TRANSFER SHALL NOT BE LESS THAN 16 HOURS.
6. PRESTRESSING STRANDS SHALL BE PRESTRESSED IN THE GIRDERS AT A MINIMUM OF 150mm IN THE VICINITY OF RECTANGULAR HOLES OR INSERTS FOR 200mm DOWELS.
7. THE VERTICAL SPACING OF PRESTRESSING STRANDS IN THE WEB DRAWING MAY BE INCREASED TO SUIT THE FABRICATOR'S REQUIREMENTS. THE MAXIMUM SPACING SHALL BE 50mm. THE FABRICATOR SHALL BE RESPONSIBLE FOR ADJUSTING THE NUMBER OF STRANDS TO MAINTAIN THE DESIRED FORCE EFFECT IS PRODUCED AS REQUIRED BY THE DESIGN.
8. CLASS OF CONCRETE ____ MPa.
9. CONCRETE STRENGTH AT TRANSFER ____ MPa.
10. REINFORCING STEEL SHALL BE IN ACCORDANCE WITH CAN/CSA STANDARD G30.18-402.
11. CLEAR COVER TO REINFORCING STEEL 30 + 10mm/-5mm.
12. REINFORCING STEEL WITH A CLEAR COVER OF LESS THAN 20mm SHALL BE LOCATED WITHIN 100mm FROM THE ORDER LINE AND LOCATED WITHIN 100mm FROM THE ORDER LINE.
13. FOR BRIDGES ON GRADES EXCEEDING 3%, THE ENDS OF GIRDERS SHALL BE CAST SO THAT THEY ARE VERTICAL WHEN ERECTED.
14. AT ENDS OF GIRDERS WHICH ARE NOT TO BE ENCASED IN CONCRETE, STRANDS SHALL BE STRESSED AND COATED, AND GIRDERS SHALL BE PAINTED WITH TWO COATS OF ASPHALTIC PAINT.
15. DOWEL INSERTS SHALL BE ZINC PLATED OR GALVANIZED IN ACCORDANCE WITH CSA STANDARD G104-402.
16. DOWEL INSERTS SHALL BE CAPABLE OF DEVELOPING A FORCE IN TENSION OF 20 kN BE SUBJECTED TO SERVICE LOADS.
17. FOR THE DIAPHRAGMS, THE PRECASTER WILL SUPPLY THE THREADED INSERTS AND THE MATCHING DIAPHRAGM DOWELS FOR THE EXTERIOR GIRDERS ONLY.
18. IF THE EXTERIOR GIRDERS ARE TO MATCH INSERTS, THEY SHALL BE TYPED AT ONE END TO MATCH INSERTS.
19. IF ANCHORED EMBEDDED STEEL BEARING PLATES ARE REQUIRED AT GIRDER ENDS, THEY SHALL HAVE A THICKNESS OF 10mm AND BE LOCATED WITHIN 100mm FROM THE ORDER LINE. FABRICATION IN ACCORDANCE WITH CSA STANDARD G104-402. ALL DAMAGED GALVANIZED SURFACES SHALL BE COATED WITH TWO COATS OF ZINC-RICH PAINT.
20. FOR CONNECTIONS TO CONTINUOUS GIRDERS SEE 0650-3701.000.



CONT No WP No	
PRESTRESSED GIRDERS AND BEARINGS	SHEET

PRESTRESSING STEEL SHALL BE LOW-RELAXATION SEVEN WIRE STRANDS, SIZE DESIGNATION ____ GRADE 1860.
MINIMUM BREAKING STRENGTH OF STRAND ____ kN.
JACKING FORCE PER STRAND ____ kN.
FORCE PER STRAND AFTER ALL LOSSES ____ kN.
THE ELAPSED TIME INTERVAL BETWEEN JACKING OF STRANDS AND TRANSFER SHALL NOT BE LESS THAN 15 HOURS.

1. FOR INTEGRAL ABUTMENT TYPE BRIDGES, THIS STANDARD DRAWING WILL NEED TO BE MODIFIED AS REQUIRED.
2. ANCHORED EMBEDDED STEEL BEARING PLATES SHALL BE USED ONLY IF STRUCTURE GRADE EXCEEDS 2.5% AND/OR IF ROTATIONAL BEARINGS ARE USED. THEY ARE THEN USED INSTEAD OF DESIGNER SOFT BLOCKOUTS.
3. THE "NOTES TO ORDER" SHALL BE DELETED FROM THIS DRAWING PRIOR TO ISSUING OF THE CONTRACT.

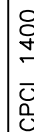
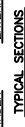
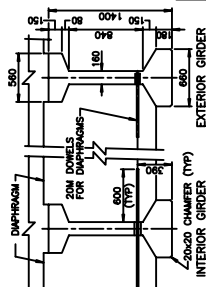
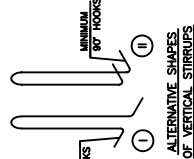
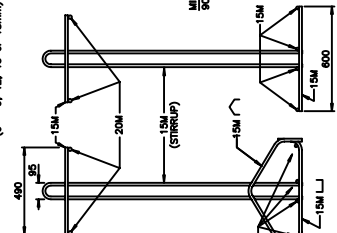
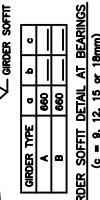


CONT No WP No		SHEET
PRESTRESSED GIRDERS AND BEARINGS		

1. FOR INTEGRAL ABUTMENT TYPE BRIDGES, THIS STANDARD DRAWING WILL NEED TO BE MODIFIED AS REQUIRED. ANCHORED EMBEDDED STEEL BEARING PLATES SHALL BE USED ONLY IF STRUCTURE GRADE EXCEEDS 2.5% AND/OR IF ROTATIONAL BEARINGS ARE USED. THEY ARE THEN USED INSTEAD OF GRADER SOFT BLOCKOUTS.

4. FORCE PER STRAND AFTER ALL LOSSES ——— kN.
5. THE ELAPSED TIME INTERVAL BETWEEN JACKING OF STRANDS AND TRANSFER SHALL NOT BE LESS THAN 15 HOURS.
6. PRESTRESSING STRANDS SHALL BE SPACED VERTICALLY AT A MINIMUM OF 150mm IN THE VICINITY OF RECTANGULAR HOLES OR INSERTS FOR 20M DOWELS.

2. ANCHORED EMBEDDED STEEL BEARING PLATES SHALL BE USED ONLY IF STRUCTURE GRADE EXCEEDS 2.5% AND/OR IF ROTATIONAL BEARINGS ARE USED. THEY ARE THEN USED INSTEAD OF GIRDER SOFTT BLOCKOUTS.



BEARING DATA		REQUIREMENTS AT SERVICEABILITY	
		UNIT STATES	LOADING
		ADJUSTMENTS	PIERS
DEAD LOAD (kN)		—	—
TOTAL LOAD (kN)		—	—
MOVEMENT (mm)		+/-	+/-
MAXIMUM SHEAR RATE (kN/mm)		—	—
BEARING SIZE (mm)		x	x
NUMBER REQUIRED		—	—
REMARKS	NOTE		

STANDARD DRAWING DECEMBER 2003	SS107-3
PRESTRESSED GIRDERS AND BEARINGS (CPCI 1400)	

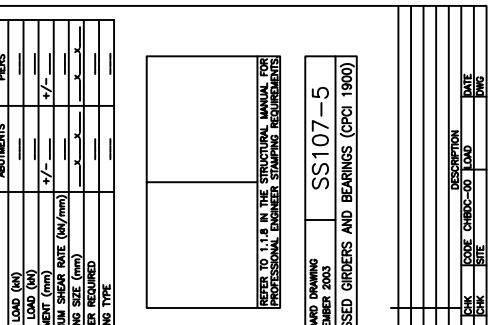
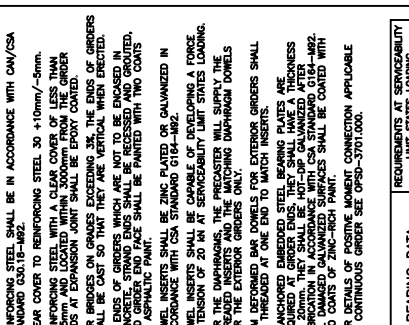
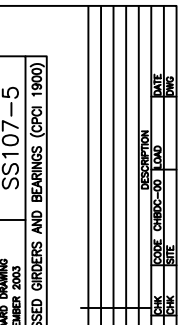
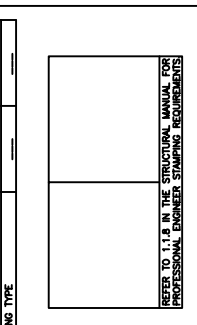
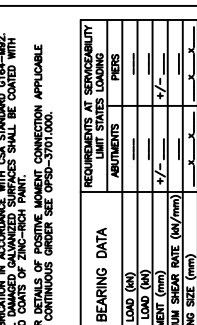
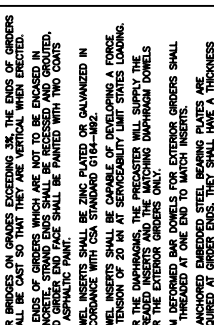
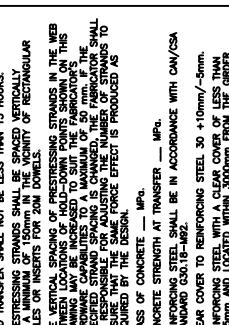
[illegible]

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

CONT No WP No	
RESTRESSED GIRDERS AND BEARINGS	
SHEET	

RESTRESSED GIRDERS AND BEARINGS

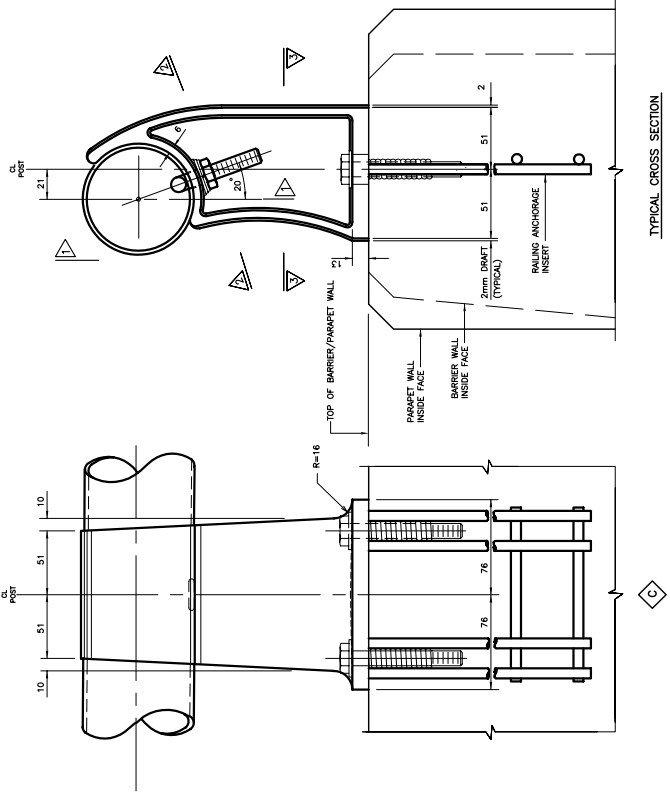
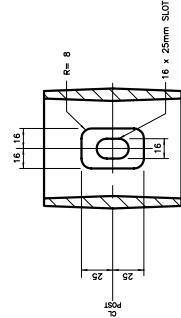
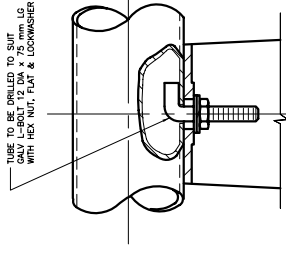
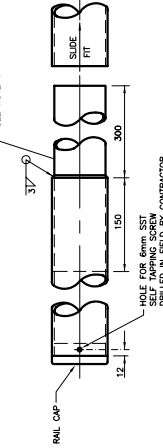
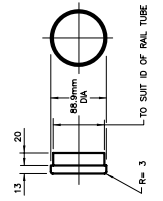
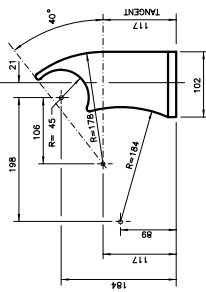
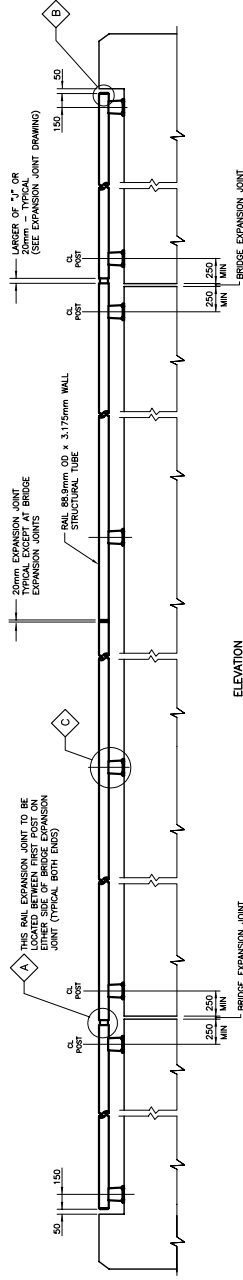
- STRESSING STRAND SHALL BE LOW-RELAXATION SEVEN WIRE STRANDS, SIZE DESIGNATION ____ GRADE 1860.
MINIMUM BREAKING STRENGTH OF STRAND ____ MN.
STRESSING STRANDS PER STRAND AFTER ALL LOSSES ____ MN.
ELAPSED TIME BETWEEN MOVING OF STRANDS ____ MIN.
STRESSING STRANDS SHALL BE STAGED VERTICALLY
A MINIMUM OF 150mm IN THE SPACING OF RECTANGULAR
OR INSERTS FOR 200 DOWELS
VERTICAL LOCATIONS OF STRESSING STRANDS IN THE WEB
OF ALL JOINTS OF ALL CONCRETE POINTS SHALL BE THE
SAME
STRESSING STRANDS SHALL BE ENOUGH TO SUIT THE FABRICATOR'S
DESIGN AND BE INCREASED TO SUIT THE FABRICATOR'S
DESIGN
STRESSING STRAND SPACING IS CHANGED, THE FABRICATOR SHALL
BE RESPONSIBLE FOR ADJUSTING THE NUMBER OF STRANDS TO
BE USED TO MEET THE DESIGN FORCE EFFECT IS PROVIDED
BY THE DESIGN.



BEARING DATA		REQUIREMENTS AT SERVICEABILITY LIMIT STATES		PIERS
ABUTMENTS				
LOAD (kN)				
LOAD (kN)				
ELEMENT (mm)		+/-	+/-	
MIN. SHEAR RATE (kN/mm)				
ING SIZE (mm)		x	x	x
ER REQUIRED				
ING TYPE				

WARD DRAWING NUMBER 2003	SS107-5
DESIGNED GIRDEBS AND BEARINGS (CPCI 1900)	

[illegible]



MATERIAL	POST SPACING mm	
	MINIMUM	MAXIMUM
STEEL	2 500	3 500
ALUMINUM	2 000	2 500

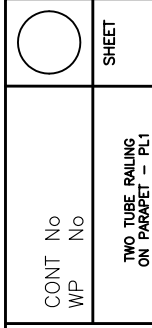
ITEM	NO. REQ'D	LINEAR mm	BENDING RADIUS	LOCATION
POST AND ANCHORAGE	----	----	----	----
END CAP	----	----	----	----
88.9 OD PIPE WITH SPICE (6990mm LG)	----	----	----	----
88.9 OD PIPE WITHOUT SPICE (_____mm LG)	----	----	----	----
88.9 OD PIPE WITH SPICE (_____mm LG)	----	----	----	----

APPLICABLE STANDARD DRAWINGS	OPSD-4019.000	SINGLE RAIL ANCHORAGE INSERTS

STANDARD DRAWING DECEMBER 2004	SS110-21
RAILING FOR BARRIER/PARAPET WALL	

[illegible]

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING



1. THIS STANDARD TO BE READ IN CONJUNCTION WITH SS10-10.
2. RAIL ELEVATOR AND RAILS SHALL BE FOLLOWING TENSILE/COMPRESSION STRENGTH SPECIFICATIONS IN ACCORDANCE WITH CSA G40.21-14 GRADE 350W, CLASS H.
3. RAIL SHALL BE SUPPLIED WITH SPICE IN LENGTHS OF 7315 mm (EXCLUDING SPICE) (EXCEPT AS NOTED).
4. GALVANIZING ON MATING SURFACES OF RAILS TO HAVE UNIFORM THICKNESS NOT EXCEEDING 0.15 mm TO ENSURE SLIDING FIT.
5. POSTS AND RAILS SHALL BE GALVANIZED IN ACCORDANCE WITH CAN/CSA G40.21-14 GRADE 350W SHALL BE DONE AFTER FABRICATION.
6. ELECTRODES SHALL BE LOW HYDROGEN SPECIFICATION E7018, E7016 OR E7018.
7. POST AND ANCHORAGE TO INCLUDE ALL BOLTS AND WASHERS.
8. END CAP TO BE USED SELF TAPING FASTENERS.
9. RAIL ENDS AND WASHERS FOR TAPPING INTO RAILS TO POSTS SHALL BE GALVANIZED (CSA G40.21-14-M).
10. ALTERNATE RAIL CAPS MAY BE USED WITH APPROVAL OF ENGINEER.
11. RAILS SHALL BE RESISTOR TO FOLLOW RAIL CURVATURE WHERE RADIUS IS LESS THAN 1500 METRES.
12. RAIL POSTS SHALL BE SET PERPENDICULAR TO GRADE.
13. WHERE LAYOUT OF POSTS IS NOT SHOWN, POST LOCATION SHALL BE DETERMINED BY THE CONTRACTOR.
14. RAIL MAY BE CUT AS REQUIRED IN FIELD. CUT TO BE SURFACE TREATED WITH ZINC RICH PAINT.
15. WHEN RAIL IS CUT, THE CUT END SHALL NOT BE LEFT AS A CONTINUOUS AND POST SPACING DETERMINED WITH REFERENCE TO EXISTING POSTS.
16. GROUT SHALL NOT BE USED UNDER BASE PLATES.
17. HOLES FOR HSB RAILINGS SHALL BE DRILLED IN FIELD.



STANDARD DRAWING DECEMBER 2003	SS110-40
TWO TUBE RAILING ON PARAPET - PL1	

[illegible]

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

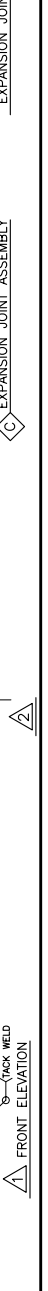
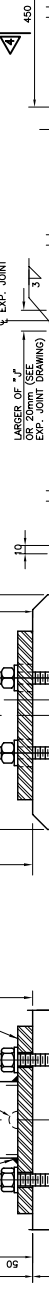
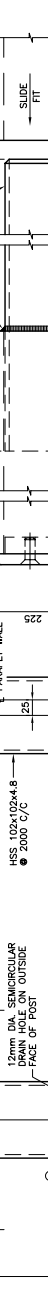
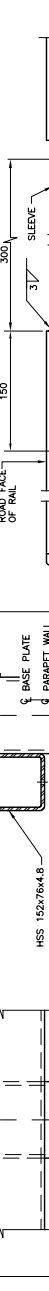
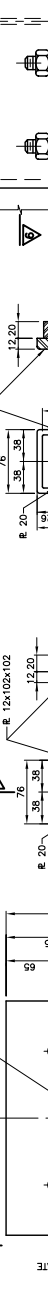
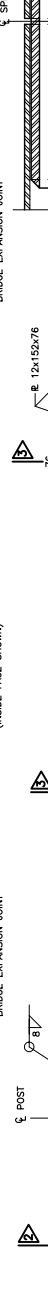
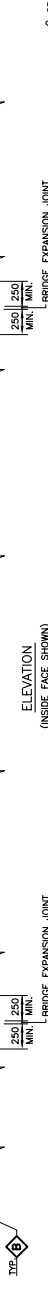
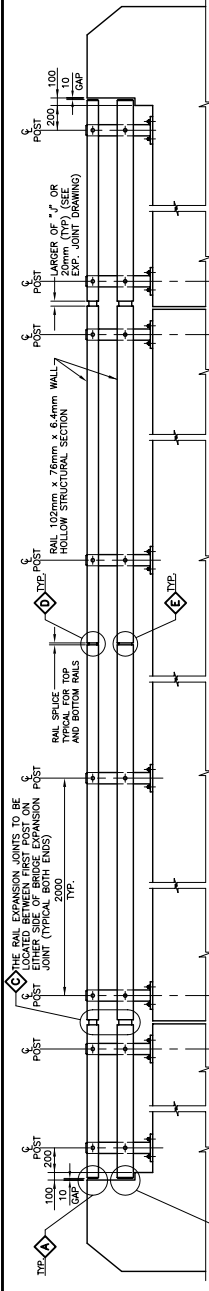
CONT No WP No	SHEET
TWO TUBE RAILING ON PARAPET FOR SIDEWALK - PL1	

NOTES:

- THIS STANDARD TO BE READ IN CONJUNCTION WITH SS110-43.
- RAIL ELEMENTS AND POSTS SHALL BE FOLLOW STRUCTURAL GRADE 350N, CLASS H, CONFORMANCE WITH CSA G40.21-14.
- RAIL SHALL BE SUPPLIED WITH SPICE IN LENGTHS OF 7315 mm (EXCLUDING SPICES) EXCEPT AS NOTED.
- GALVANIZING ON MATING SURFACES OF RAILS TO HAVE UNIFORM THICKNESS NOT EXCEEDING 0.15 mm TO ENSURE SLEEVING FIT.
- POSTS SHALL BE GALVANIZED IN ACCORDANCE WITH CSA G164-14. ALL GALVANIZING SHALL BE DONE AFTER FABRICATION.
- ELECTRODES SHALL BE A LOW HYDROGEN SPECIFICATION E7015, E7016 OR E7018.
- POST AND ANCHORAGE TO INCLUDE ALL BOLTS AND WASHERS.
- END CAP TO INCLUDE SST SELF TAPPING FASTENERS.
- BOLTS, NUTS AND WASHERS FOR FASTENING STEEL RAILING TO POSTS SHALL BE GALVANIZED (CSA G164-14).
- ALTERNATE RAIL CAPS MAY BE USED WITH APPROVAL OF ENGINEER.
- RAILS SHALL BE PRESENT TO FOLLOW ROAD CURVATURE WHERE RADIUS IS LESS THAN 150 METRES.
- RAIL POSTS SHALL BE SET PERPENDICULAR TO GRADE.
- WHERE LAYOUT OF POSTS IS NOT SHOWN, POST LOCATION SHALL BE DETERMINED BY THE CONTRACTOR.
- RAIL MAY BE CUT AS REQUIRED IN FIELD. CUT TO BE SURFACE TREATED WITH ZINC RICH PAINT.
- WHEN CONNECTING TO EXISTING RAILING, RAIL MUST BE WELDED TO EXISTING POSTS. WELDING SHALL BE DETERMINED WITH REFERENCE TO EXISTING POSTS.
- GROUT SHALL NOT BE USED UNDER BASE PLATES.
- HOLES FOR HSS RAILINGS SHALL BE DRILLED IN FIELD.

- LOOKUP FOR HSS RAIL WITH SPICE GIVEN IN TABLE DOES NOT EXCEED 300 mm PER SPICE OR 6
- SPICING OF RAIL TUBES MAY BE DONE BY WELDING ON SPICE PLATE OR BY SWEDEING OF RAIL END

ITEM	NO. REQ'D	LINEAR (mm)	BENDING (mm)	LOCATION
POST WITH TOP CAP AND ANCHORAGE				
END CAP				
HSS 102x76x6.4 WITH SPICE (7315 mm LO)				
HSS 102x76x6.4 WITH SPICE (7315 mm LO)				
HSS 102x76x6.4 WITH SPICE (7315 mm LO)				



STANDARD DRAWING DECEMBER 2003	SS110-41
TWO TUBE RAILING ON PARAPET FOR SIDEWALK - PL1	

DATE	BY	CHECKED	DESCRIPTION	DATE
2003-12-10	CHK	CHK	STRUCT	2003-12-10
2003-12-10	CHK	CHK	STRUCT	2003-12-10

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

EXPANSION JOINT SLEEVE

EXPANSION JOINT ASSEMBLY

EXPANSION JOINT ASSEMBLY

EXPANSION JOINT ASSEMBLY

EXPANSION JOINT ASSEMBLY

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

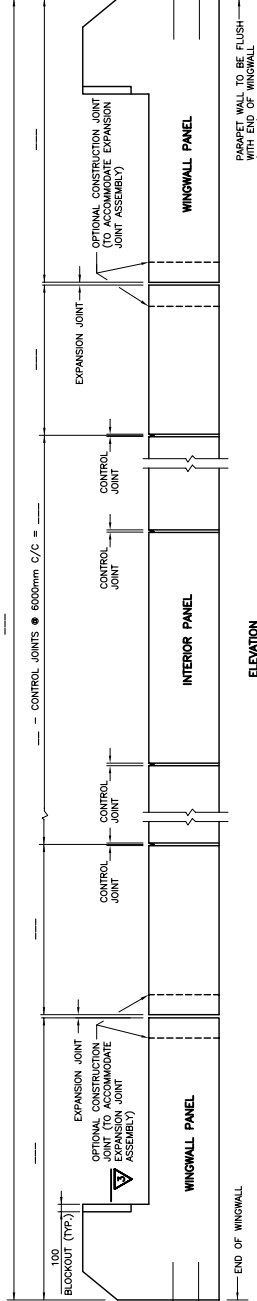
CONT No
WP No

PARAPET WALL FOR
TWO TUBE RAILING - PL1

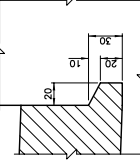
(TYPICAL)
REINFORCEMENT DRAWING
FOR ANCHORAGE DETAILS SEE

NOTES:

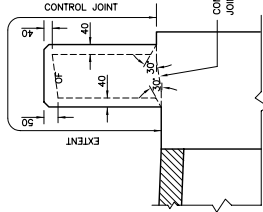
- CHASE REQUIRED ON HIGH AND LOW SIDE OF CROSSFALL.
- CONCRETE COVER TO REINFORCING STEEL 70 ± 20mm.
- REINFORCING STEEL SHALL BE STAINLESS TYPE WITH A MINIMUM YIELD STRENGTH OF 420 MPa.
- BAR LAP SPLICE FOR HORIZONTAL REINFORCEMENT MUST NOT LAP THROUGH JOINTS.
- ANCHORAGE OF JOINTS TO BE 700mm.
- LENGTH OF HORIZONTAL BAR TO SUIT CONTRACTOR'S OPERATIONS.
- CONTROL JOINT TO BE FORMED.
- SAMCUTS NOT PERMITTED.
- CONTROL JOINT FORM HARDWARE NOT TO BE LEFT IN PLACE.
- OPTIONAL CONSTRUCTION JOINTS TO BE LOCATED WITHIN LIMITS OF CONCRETE DAMS ON DECK OR BALLAST WALL.
- LEGEND
EF DENOTES EACH FACE
IF DENOTES INSIDE FACE
OF DENOTES OUTSIDE FACE



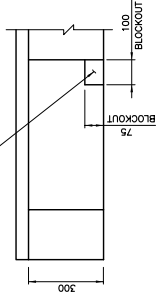
ELEVATION



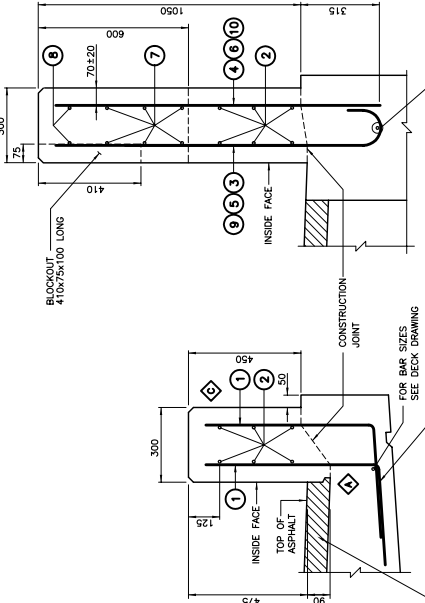
CHASE DETAIL



EXTENT OF CONTROL JOINT IN PARAPET WALL



BLOCKOUT SECTION
TYPICAL ON WINGWALLS



CONCRETE PARAPET ON WINGWALL

FOR BAR SIZES
SEE WINGWALL DRAWING

FOR BAR SIZES
SEE DECK DRAWING
BARS TO BE LAPPED
WITH DECK TRANSVERSE
BOTTOM REINFORCING

CONCRETE PARAPET ON DECK

TYPICAL DIMENSIONS & REINFORCEMENT

- 6-BAR ② EQUALLY SPACED OF
- 6-BAR ③ EQUALLY SPACED IF
- 1-BAR ④ EF

CONCRETE PARAPET ON WINGWALL

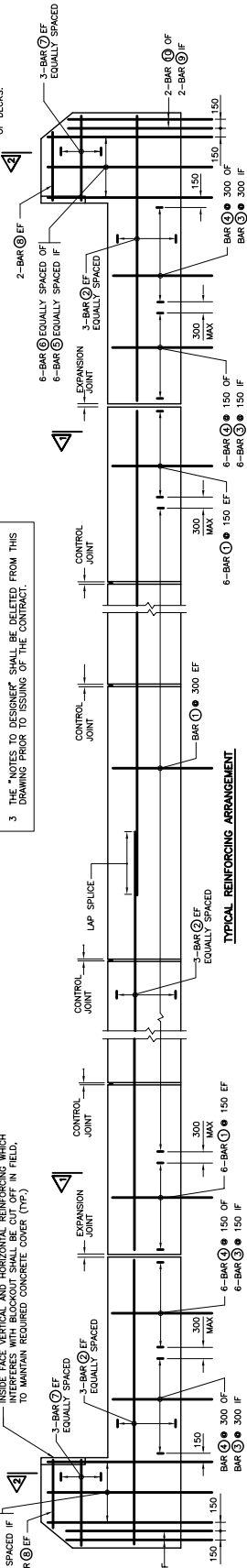
TYPICAL REINFORCEMENT

- INSIDE FACE VERTICAL AND HORIZONTAL REINFORCING WHICH INTERFERES WITH BLOCKOUT SHALL BE CUT OFF IN FIELD, TO MAINTAIN REQUIRED CONCRETE COVER (TYP.)
- THESE CHANGES ARE MADE
- THE "NOTES TO DESIGNER" SHALL BE DELETED FROM THIS DRAWING PRIOR TO ISSUING OF THE CONTRACT.

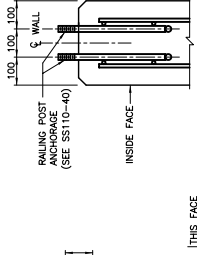
- WHEN RECOMMENDED BY THE REGION FOR LOW VOLUME ROADS EPOXY COATED REINFORCING STEEL SHALL REPLACE STAINLESS STEEL. THE REASON FOR THIS CHANGE AND THE APPLICABLE PORTION OF THE BAR MARK TABLE SHOULD BE DELETED.
- THE DRAWING SHALL BE IDENTIFIED AS "MODIFIED" WHEN THESE CHANGES ARE MADE
- THE "NOTES TO DESIGNER" SHALL BE DELETED FROM THIS DRAWING PRIOR TO ISSUING OF THE CONTRACT.

CONTROL JOINT DETAIL

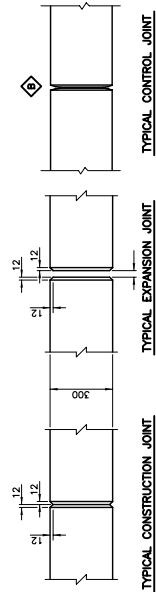
* DIMENSION VARIES FOR DIFFERENT TYPES OF DECK.



TYPICAL REINFORCING ARRANGEMENT



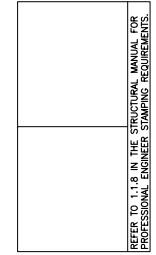
POST ANCHORAGE DETAIL



TYPICAL CONSTRUCTION JOINT

TYPICAL EXPANSION JOINT

TYPICAL CONTROL JOINT



APPLICABLE STANDARD DRAWINGS
SS110-40 TWO TUBE RAILING ON PARAPET - PL1

STANDARD DRAWING
DECEMBER 2003
SS110-42
PARAPET WALL FOR
TWO TUBE RAILING - PL1

REVISIONS	DATE	BY	CHK	DESCRIPTION	DATE

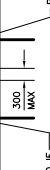
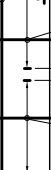
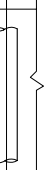
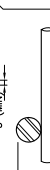
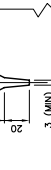
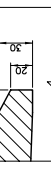
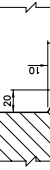
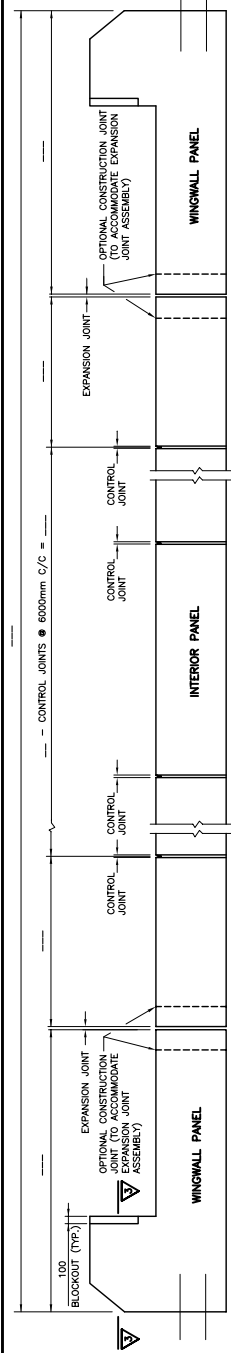
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No WP No	
PARAPET WALL WITH SIDEWALK FOR TWO TUBE RAILING - PL1	SHEET

NOTES

1. CROSSFALL REQUIRED ON HIGH AND LOW SIDE OF CROSSFALL.
2. CONCRETE COVER TO REINFORCING STEEL 70 $\pm$ 20mm.
3. REINFORCING STEEL SHALL BE STAINLESS TYPE WITH A MINIMUM YIELD STRENGTH OF 420 MPa.
4. BAR LAP SPACING FOR HORIZONTAL REINFORCEMENT MUST NOT LAP THROUGH CONTROL JOINT.
5. MINIMUM BAR LAP SPICE TO BE 700mm.
6. LENGTH OF HORIZONTAL BAR TO SUIT CONTRACTOR'S OPERATIONS.
7. CONTROL JOINT TO BE FORMED.
8. SAWCUT NOT PERMITTED.
9. CONTROL JOINT FROM BACKSIDE NOT TO BE LEFT IN PLACE.
10. CONCRETE JOINTS ON HEDGE OR BALLAST WALLS TO BE FORMED WITHIN 100mm OF FACE.

BAR MARK	STAINLESS	SPY-COATED
①	S15M 	S15M 
②	S15M 	S15M 
③	S15M 	S15M 
④	S15M 	S15M 
⑤	S15M 	S15M 
⑥	S15M 	S15M 
⑦	S15M 	S15M 
⑧	S15M 	S15M 
⑨	S15M 	S15M 
⑩	S15M 	S15M 



APPLICABLE STANDARD DRAWINGS
SS110-41 TWO TUBE RAILING ON PARAPET FOR SIDEWALK-PL1

STANDARD DRAWING DECEMBER 2003	SS110-43

REVISIONS		DATE		BY	DESCRIPTION			
DESIGN	CHK	CHK	CODE	CHBDC-00	LOAD	DATE	DWG	
DRAWN	CHK	CHK	SITE					

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

WP	No

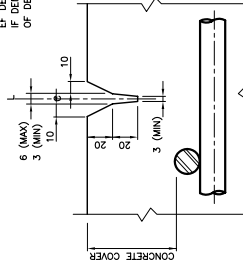
PARAPET WALL WITH RAILING - PL2	SHEET

NOTES:

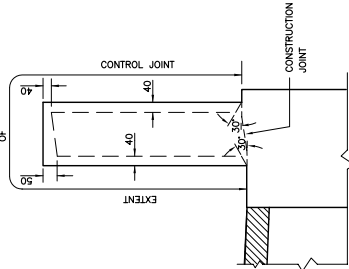
1. CHASE REQUIRED ON HIGH AND LOW SIDE OF CROSSFALL
2. CONCRETE COVER TO REINFORCING STEEL 75±0mm.
3. REINFORCING STEEL SHALL BE STAINLESS STEEL WITH A MINIMUM YIELD STRENGTH OF 420 MPa.
4. BAR LAP SPACING FOR HORIZONTAL REINFORCEMENT MUST NOT LAP THROUGH JOINT.
5. MINIMUM BAR LAP SPACE TO BE 550mm.
6. LENGTH OF HORIZONTAL BAR TO SUIT CONTRACTOR'S OPERATIONS.
7. LENGTHS NEED NOT MATCH DISTANCE BETWEEN CONTROL JOINTS.
8. CONTROL JOINT TO BE FORMED.
9. SAWCUTS NOT PERMITTED.
10. CONTROL JOINT FORM HARDWARE NOT TO BE LEFT IN PLACE.
11. OPTIONAL CONSTRUCTION JOINTS TO BE LOCATED WITHIN LIMITS OF CONCRETE DAMS ON DECK OR BALLAST WALL.
12. LEGEND

EF DENOTES EACH FACE
IF DENOTES INSIDE FACE
OF DENOTES OUTSIDE FACE

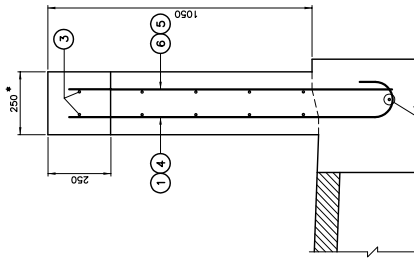
BAR MARK	SIZE	SHAPE
①	S15M	
②	S15M	STRAIGHT
③	S15M	STRAIGHT
④	S15M	
⑤	S15M	STRAIGHT
⑥	S15M	STRAIGHT



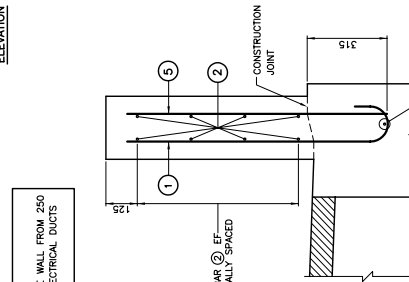
CONTROL JOINT DETAIL



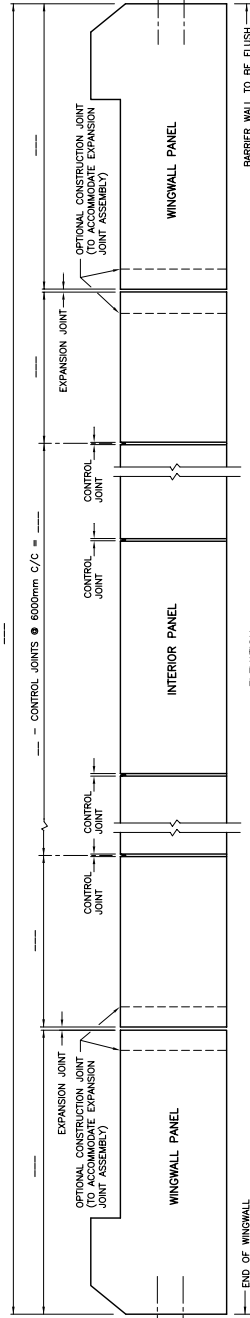
EXTENT OF CONTROL JOINT IN PARAPET WALL



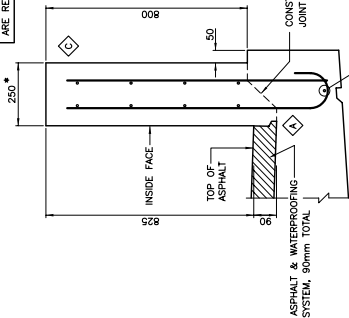
—FOR BAR SIZES —
OF MINIMUM 10MM



 FOR BAR SIZES
SEE DEFG COMMING

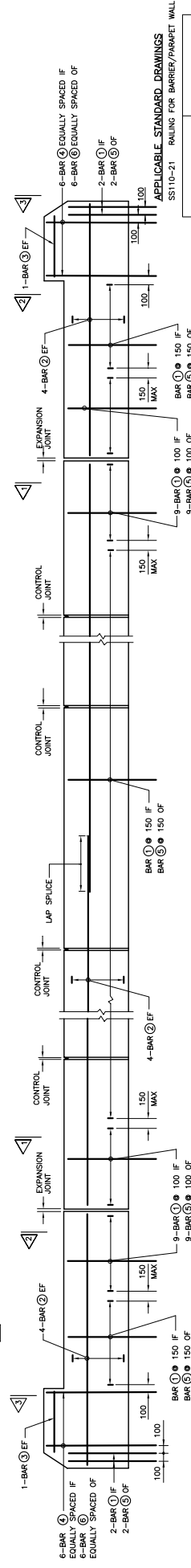


*** NOTE TO DESIGNER**
INCREASE THE THICKNESS OF THE WALL FROM 250 TO 300mm WHEN 75mm DIA. ELECTRICAL DUCTS ARE REQUIRED.



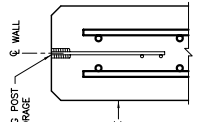
CONCRETE PARAPET ON DECK

TYPICAL DIMENSIONS
(FOR BAR NUMBERS SEE )

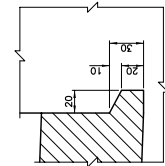


APPLICABLE STANDARD DRAWINGS

TYPICAL REINFORCING ARRANGEMENT

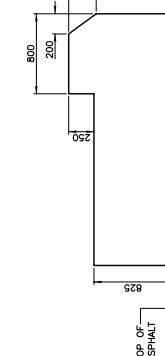


◀



◁ ▷

THIS FACE
PERPENDICULAR



DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

POST ANCHORAGE DETAIL

CHASE DETAIL

ELEVATION OF PARAPET ON WINGWALL

STANDARD DRAWING DECEMBER 2003	SS110-56
PARAPET WALL WITH RAILING - PL2 STAINLESS STEEL REBAR	

[illegible]

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

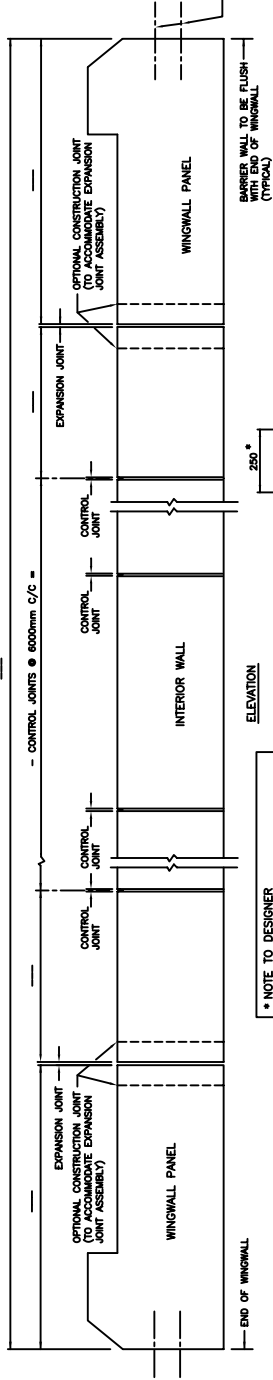
CONT No WP No	SHEET
PARAPET WALL WITH S/W & RAILING PL2 - STAINLESS STEEL REBAR	

NOTES:

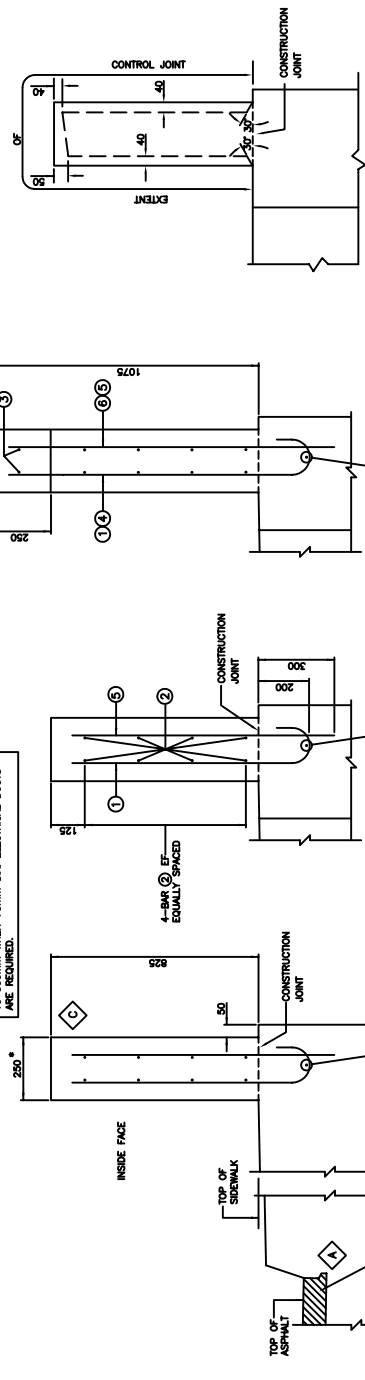
- CHASE REQUIRED ON HIGH AND LOW SIDE OF CROSSFALL
- CONCRETE COVER TO REINFORCING STEEL 75-20mm.
- REINFORCING STEEL SHALL BE STAINLESS TYPE WITH A MINIMUM YIELD STRENGTH OF 420 MPa.
- BAR LAP SPICE FOR HORIZONTAL REINFORCEMENT MUST NOT LAP THROUGH CONTROL JOINT.
- MINIMUM BAR LAP SPICE TO BE 500mm.
- LENGTH OF HORIZONTAL BAR TO SUIT CONTRACTOR'S OPERATIONS. BAR LENGTHS NEED NOT MATCH DISTANCE BETWEEN CONTROL JOINTS.
- CONTROL JOINT TO BE FORMED.
- SAW CUTS NOT PERMITTED.
- CONTROL JOINT FORM HARDWARE NOT TO BE LEFT IN PLACE.
- OPTIONAL CONSTRUCTION JOINTS TO BE LOCATED WITHIN LIMITS OF CONCRETE DAMS ON DECK OR BALLAST WALL
- LEGEND

EF DENOTES EACH FACE
OF REINFORCING STEEL
OF DENOTES OUTSIDE FACE

BAR MARK	SIZE	SHAPE
1	S15M	13-1-48
2	S15M	STRAIGHT
3	S15M	STRAIGHT
4	S15M	13-1-48
5	S15M	STRAIGHT
6	S15M	STRAIGHT



* NOTE TO DESIGNER
INCREASE THE THICKNESS OF THE WALL FROM 250
TO 300mm WHEN 75mm DIA ELECTRICAL DUCTS
ARE REQUIRED.



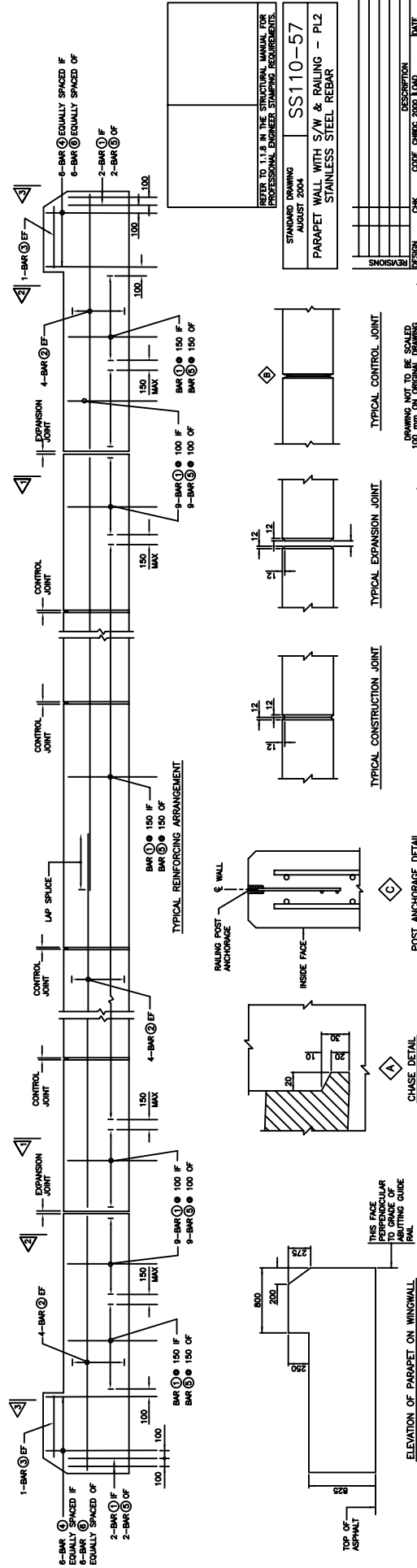
EXTENT OF CONTROL JOINT IN PARAPET WALL

CONTROL JOINT DETAIL

CONCRETE PARAPET ON WINGWALL
TYPICAL REINFORCEMENT

CONCRETE PARAPET ON DECK
TYPICAL DIMENSIONS (FOR BAR NUMBERS SEE (A))

APPLICABLE STANDARD DRAWINGS
SS110-21 RAILING FOR BARRIER/PARAPET WALL



STANDARD DRAWING AUGUST 2004	SS110-57
PARAPET WALL WITH S/W & RAILING - PL2 STAINLESS STEEL REBAR	

REFER TO 1.1.9 IN THE STRUCTURAL MANUAL FOR
PROFESSIONAL ENGINEERING REQUIREMENTS.

DESIGN	CHK	CODE	CHRG	2000	LOAD	DATE
SS110-57						

METRIC

MEASUREMENTS ARE IN METRES

CONT No

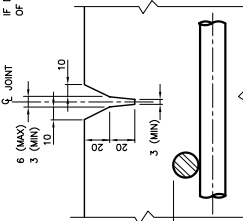
SHEET

NOTES:

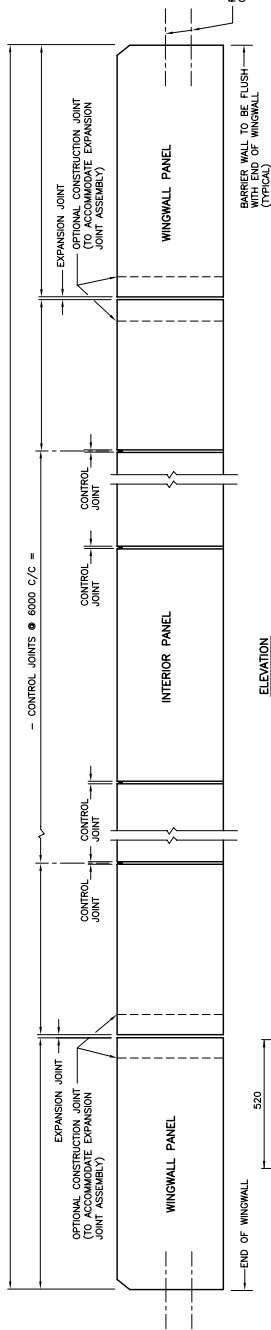
1. CHASE REQUIRED ON HIGH AND LOW SIDE OF CROSSFALL
2. CONCRETE COVER TO REINFORCING STEEL 70 ±20mm.
3. REINFORCING STEEL SHALL BE STAINLESS STEEL WITH A MINIMUM YIELD STRENGTH OF 420 MPA.
4. BAR LAP SPACING FOR HORIZONTAL REINFORCEMENT MUST NOT LAP THROUGH CONTROL JOINT.
5. MINIMUM BAR LAP SPACE TO BE 550mm, UNLESS OTHERWISE SHOWN.
6. LENGTH OF HORIZONTAL BAR TO SUIT CONTRACTOR'S OPERATIONS.
7. BAR LENGTHS NEED NOT MATCH DISTANCE BETWEEN CONTROL JOINTS.
8. CONTROL JOINT TO BE FORMED.
9. SCAFFOLDS NOT PERMITTED.
10. CONTROL JOINT FORM HARDWARE NOT TO BE LEFT IN PLACE.
11. OPTIONAL CONSTRUCTION FORMS TO BE LOCATED WITHIN LIMITS OF CONCRETE DAMS ON DECK OR BALLAST WALL.

11. LEGEND
EF DENOTES EACH FACE
IF DENOTES INSIDE FACE
OF DENOTES OUTSIDE FACE

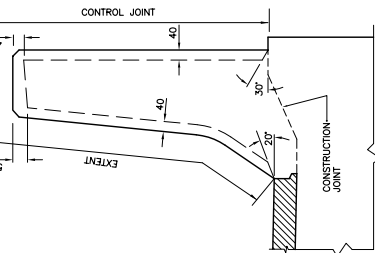
BAR MARK	SIZE	SHAPE
①	S15M	
②	S15M	
③	S15M	
④	S15M	
⑤	S15M	
⑥	S15M	STRAIGHT
⑦	S15M	STRAIGHT



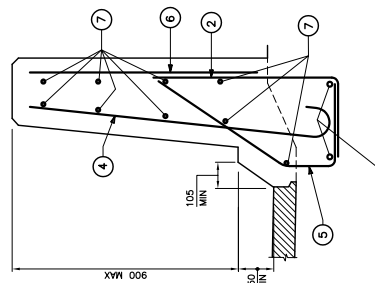
CONTROL JOINT DETAIL



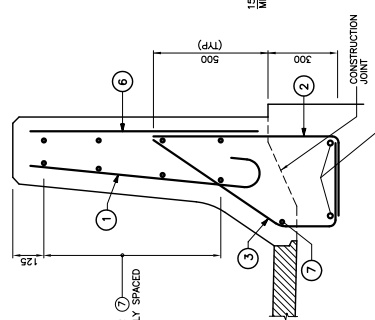
ELEVATION



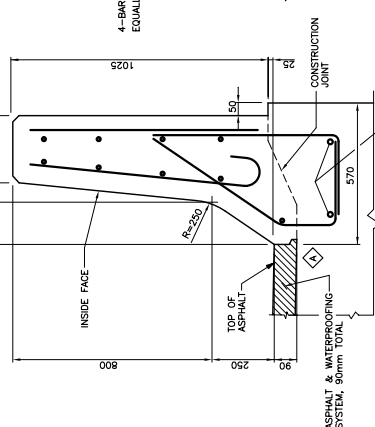
EXTENT OF CONTROL JOINT IN BARRIER WALL



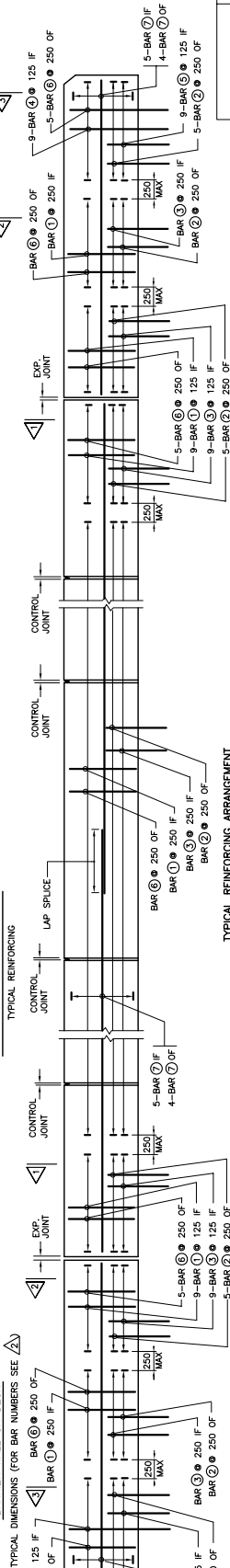
BARRIER WALL ON WINGWALL



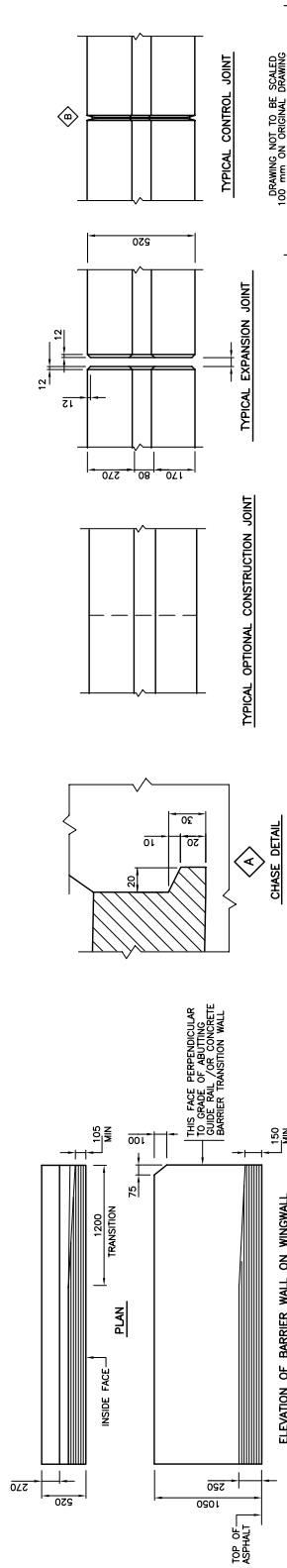
FOR BAR SIZES



BARRIER WALL ON DECK



TYPICAL REINFORCING ARRANGEMENT



BARRIER TRANSITION WALL

ELEVATION OF BARRIER WALL ON WINGWALL

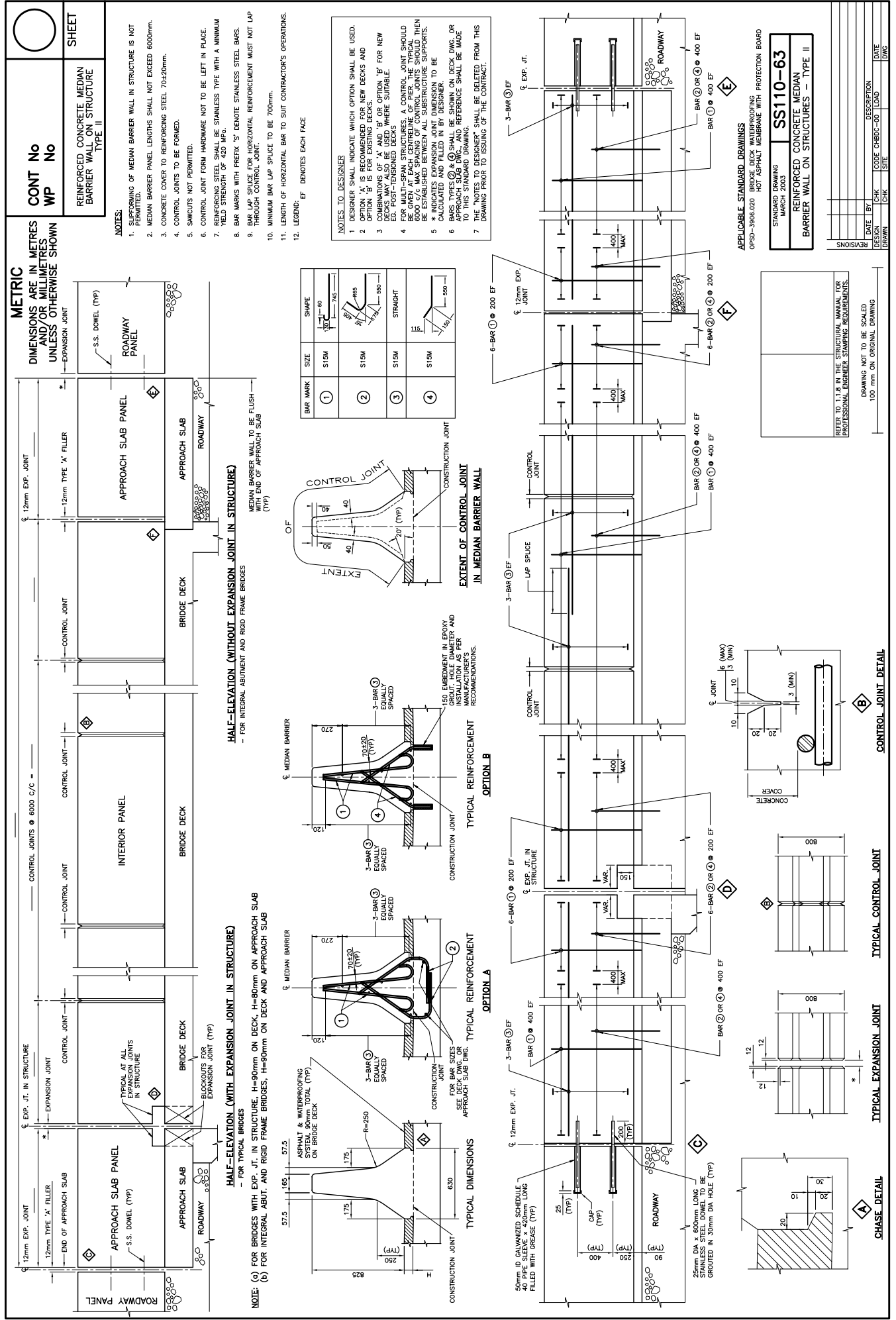
TYPICAL OPTIONAL CONSTRUCTION JOINT

TYPICAL EXPANSION JOINT

TYPICAL CONTROL JOINT

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

[illegible]



Technical drawing showing a cross-section of a wall and a blockout (DECK). The drawing includes dimensions and labels:

- WALL
- BLOCKOUT (DECK)
- FRONTAL CONTAINMENT
- FR-D-707

CONCRETE ANCHOR
MODULATE DUCTS

Technical drawing of a mechanical part showing dimensions 20 and 30 MIN.

SEAL CONTINUOUS THROUGH CORNER (TYP)

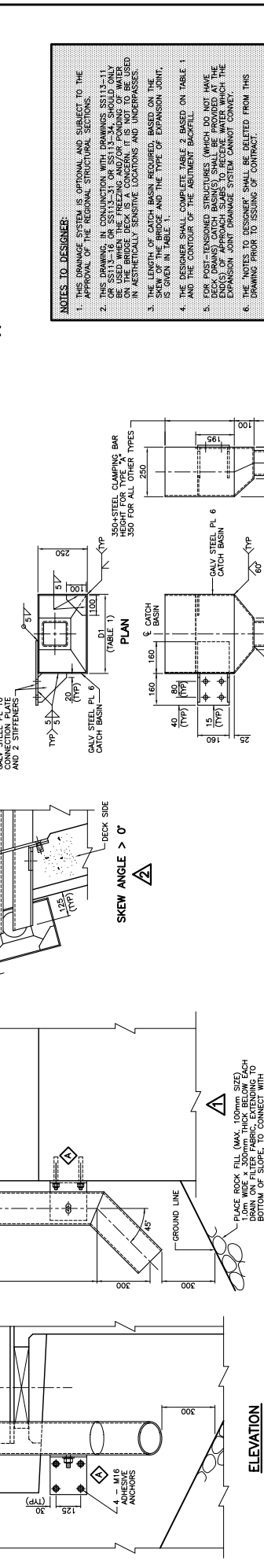
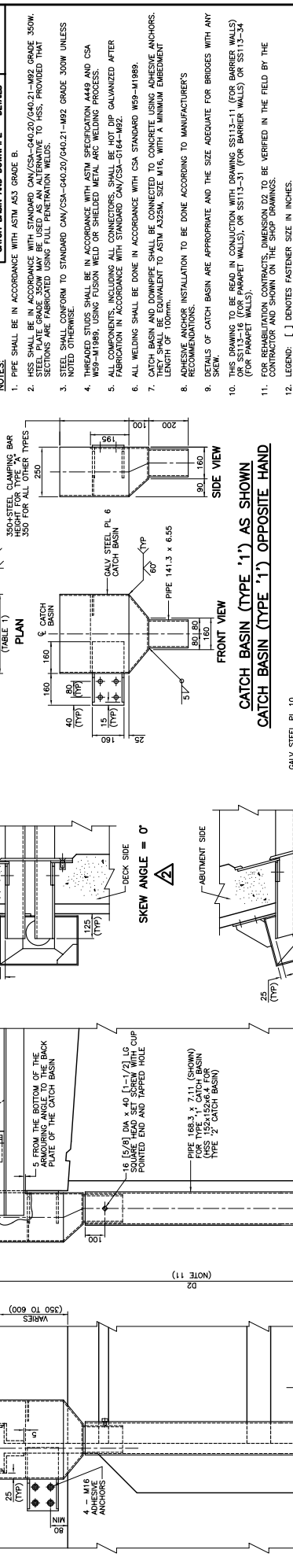
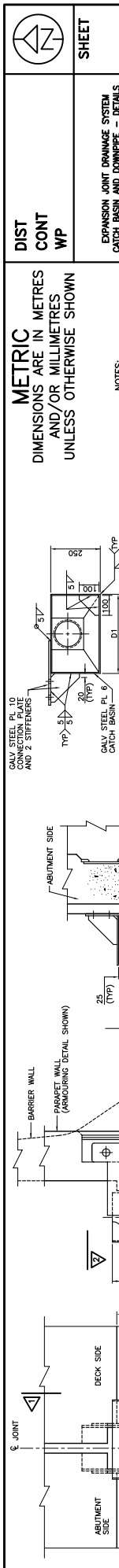
MODTIME

MODAL

Model:

100

DETAILS



POSITIVE DRAINAGE.

(PLOW GUARD AND CLAMPING BAR FOR TYPE 'X,' NOT SHOWN)

4 - M16 ADHESIVE ANCHORS

L 200x200x10 185 LG BRACKET

L-19 [3/4] DIA. x 185 LG THREADED STUD WELDED TO PLATE (NOTE 4)

HEAVY HEX NUT AND STRUCTURAL PLATE

4 - M16 ADHESIVE ANCHORS

L 200x200x10 185 LG BRACKET

L-19 [3/4] DIA. x 185 LG THREADED STUD WELDED TO PLATE (NOTE 4)

HEAVY HEX NUT AND STRUCTURAL PLATE

TABLE 1

SKIEW ANGLE	LENGTH OF CATCH BASIN, DT (mm)
0° TO 15°	500
15° TO 30°	550
30° TO 45°	600
>45°	700

200 160 90

HSS 127x127x6.4

5

FRONT VIEW

SIDE VIEW

CATCH BASIN (TYPE '2') AS SHOWN

CATCH BASIN (TYPE '2') OPPOSITE HAND

[illegible]

ALTERNATE DOWNPIPE CONNECTION TO CONCRETE

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

LIMITS OF LOCAL WIDENING WHEN REQUIRED BASED ON SKEW AND "H". FOR DETAILS, SEE NECK DRAWING(S), PARAPET WALL DRAWING(S) AND ABUTMENT DRAWING(S).

C

1. EXPANSION JOINT SHALL BE IN ACCORDANCE WITH THE DESIGNATED SOURCES FOR MATERIALS LIST DSM 9.40, TYPE ____ DETAILS SHOWN FOR OTHER TYPES ARE NOT APPLICABLE.

2. EXPANSION JOINT ASSEMBLY CONSTRUCTION AND MATERIAL SHALL BE ACCORDING TO SPSS 920 AND OFSS 1210, AND AS SPECIFIED IN THE CONTRACT DOCUMENTS

3. JOINT ASSEMBLIES SHALL BE COMPLETELY SHOP ASSEMBLED (EXCEPT FOR SEALS) AND PRESET TO DIMENSION "J" FOR 15°C AND ADJUSTED IN THE FIELD TO SUIT INSTALLATION TEMPERATURE.

4. JOINT ASSEMBLY INSTALLATION TEMPERATURE SHALL BE TAKEN AS MEAN SHADE AIR TEMPERATURE AT SITES (SUCH) BEFORE TO JOINT INSTALLATION AS FOLLOWS

7. FIELD SPLICES IN JOINT ASSEMBLY ARE ONLY PERMITTED AT STAGED CONSTRUCTION, AND/OR AS SHOWN ON THE CONTRACT DRAWINGS.

8. SETTING ANGLES SHALL BE FLAME CUT ACCORDING TO OPSS 920, BUT IN NO CASE PRIOR TO CONCRETE REACHING INITIAL SET

PREFORMED SEALS SHALL HAVE MINIMUM THICKNESS OF 5mm OR AS PER DSM.

1. PREFORMED SEALS SHALL BE INSTALLED AFTER JOINT ASSEMBLY HAS BEEN CAST IN PLACE, STYROFOAM OR FILLER BETWEEN DECK AND BALLAST WALL REMOVED, AND EXPANSION GAP CLEARED OF ALL DEBRIS.

5. RAISED MEDIANS ON STRUCTURES SHALL BE PROVIDED WITH SLIDING PLATES, PLATE THICKNESS AND ANCHORAGES SHALL CONFORM TO SIDEWALK DETAILS, AND SHALL ACCOMMODATE

STRUCTURE.

ALL JOINT ANCHORAGES SHALL BE DETAILED ON WORKING DRAWINGS PERPENDICULAR TO THE

3. THIS DRAWING TO BE READ IN CONJUNCTION WITH DRAWING SS113-12 OR SS113-13.

LEGEND: [] DENOTES FASTENER SIZE IN INCHES

19. 19 [3/4] DIAMETER BOLTS SHALL BE TENSIONED TO 125KN AND BE IN ACCORDANCE WITH WITH ASTM A325. ALL BOLTS USED IN 25 DIA. x 50 LONG SLOTTED HOLES SHALL BE INSTALLED WITH OVERSIZE WASHERS

5. CLAMPING BAR BOLTS FOR TYPE 'A' JOINTS SHALL BE EITHER 19 [3/4] DIAMETER FLAT COUNTERSINK HEAD CAR BOLTS TENSIONED TO 140kN AND IN ACCORDANCE WITH ASTM F493, OR IN ACCORDANCE WITH ASTM F493.

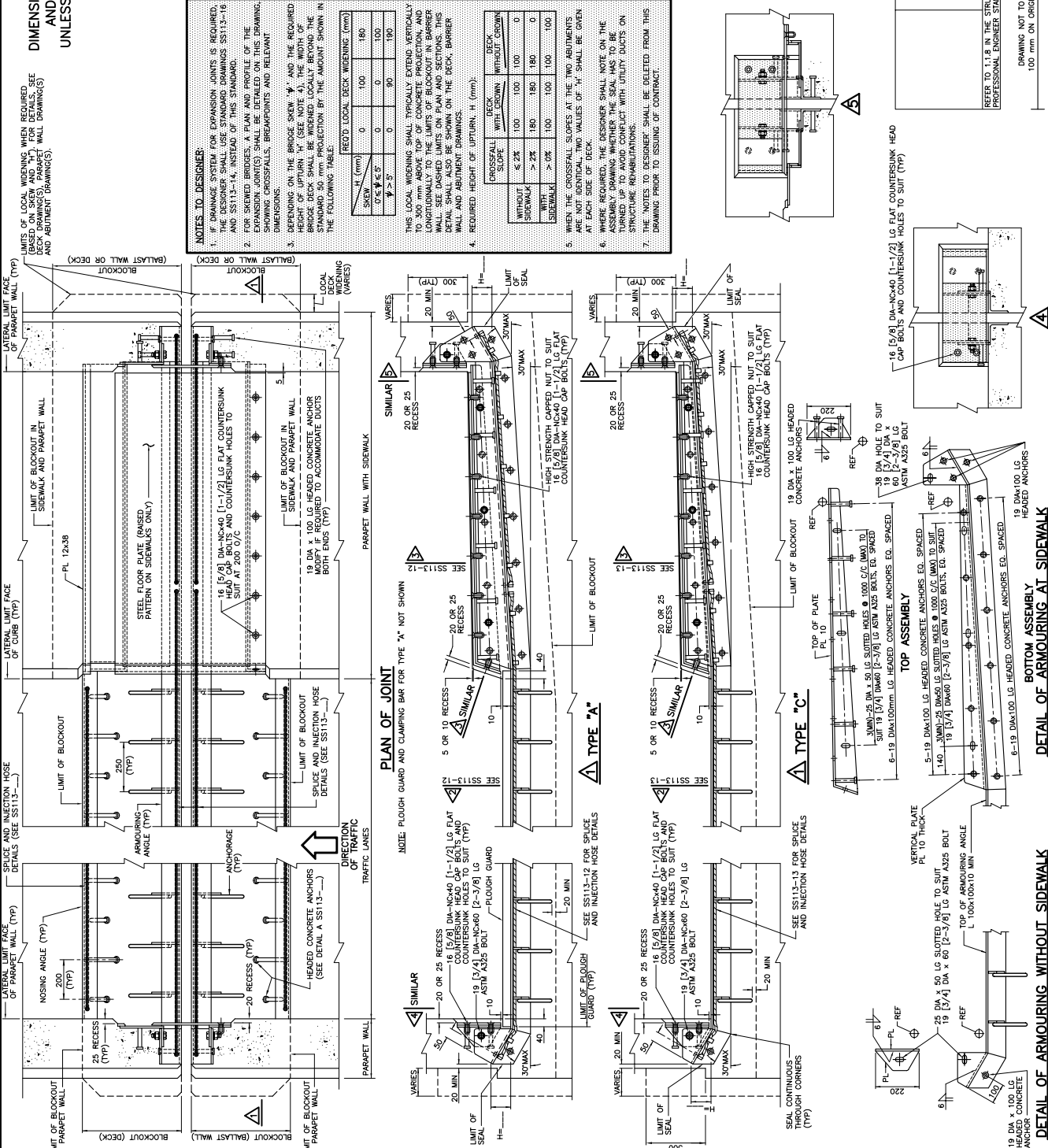
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				DESIGN	
				CHECK	
				DRAWN	
				CHECK	
				SITE	
				CODE CHBDC-00	
				LOAD	
				DWG	

	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80	81	82	83	84	85	86	87	88	89	90	91	92	93	94	95	96	97	98	99	100	101	102	103	104	105	106	107	108	109	110	111	112	113	114	115	116	117	118	119	120	121	122	123	124	125	126	127	128	129	130	131	132	133	134	135	136	137	138	139	140	141	142	143	144	145	146	147	148	149	150	151	152	153	154	155	156	157	158	159	160	161	162	163	164	165	166	167	168	169	170	171	172	173	174	175	176	177	178	179	180	181	182	183	184	185	186	187	188	189	190	191	192	193	194	195	196	197	198	199	200	201	202	203	204	205	206	207	208	209	210	211	212	213	214	215	216	217	218	219	220	221	222	223	224	225	226	227	228	229	230	231	232	233	234	235	236	237	238	239	240	241	242	243	244	245	246	247	248	249	250	251	252	253	254	255	256	257	258	259	260	261	262	263	264	265	266	267	268	269	270	271	272	273	274	275	276	277	278	279	280	281	282	283	284	285	286	287	288	289	290	291	292	293	294	295	296	297	298	299	300	301	302	303	304	305	306	307	308	309	310	311	312	313	314	315	316	317	318	319	320	321	322	323	324	325	326	327	328	329	330	331	332	333	334	335	336	337	338	339	340	341	342	343	344	345	346	347	348	349	350	351	352	353	354	355	356	357	358	359	360	361	362	363	364	365	366	367	368	369	370	371	372	373	374	375	376	377	378	379	380	381	382	383	384	385	386	387	388	389	390	391	392	393	394	395	396	397	398	399	400	401	402	403	404	405	406	407	408	409	410	411	412	413	414	415	416	417	418	419	420	421	422	423	424	425	426	427	428	429	430	431	432	433	434	435	436	437	438	439	440	441	442	443	444	445	446	447	448	449	450	451	452	453	454	455	456	457	458	459	460	461	462	463	464	465	466	467	468	469	470	471	472	473	474	475	476	477	478	479	480	481	482	483	484	485	486	487	488	489	490	491	492	493	494	495	496	497	498	499	500	501	502	503	504	505	506	507	508	509	510	511	512	513	514	515	516	517	518	519	520	521	522	523</

ADDITIONAL ENGINEER STAFFING REQUIREMENTS:

HEADED CONCRETE ANCHOR

DETAIL OF ARMOURING WITHOUT SIDEWALK



STEEL FLOOR PLATES
PL 12 THICK (PLAN)
BARRIER WALL ONLY
BOTH ENDS (TYP)

LIMIT OF BLOCKOUT
IN SIDEWALK

10 MIN
(TYPICAL)

M20 GALV FLAT HD SOCKET SCREW
(LENGTH TO SUIT) AND COUNTERSINK
HOLES TO SUIT AT 200 O/C (TYPICAL)

FIELD SPRUCE AND INJECTION
ROSE DETAILS SEE SS113-21

DIRECTION OF TRAFFIC

LIMIT OF BLOCKOUT
IN BARRIER WALL

SUPPORT BOX DETAILS
SEE SS113-22 (TYPICAL)

M20 GALV FLAT HD SOCKET SCREW
(LENGTH TO SUIT) AND COUNTERSINK
HOLES TO SUIT (TYPICAL)

PLAN OF JOINT AT SIDEWALK

M20 GALV FLAT HD SOCKET SCREW
(LENGTH TO SUIT) AND COUNTERSINK
HOLES TO SUIT (TYPICAL)

180 MIN
20 MIN
30 MAX
30 MIN
SEE NOTE 24
SECTION 21-21
SUPPORT BOX

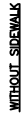
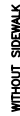
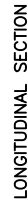
[illegible][illegible]

SETTING DEVICE AND DIMENSIONS

600	750	900
BALLAST WALL		
325	325	325
325	325	335
1/4" (MIN)		
B mm		

CONT No WP No	
6000 mm APPROACH SLAB	SHEET

1. CLEAR COVER TO REINFORCING STEEL 70 ± 20 mm EXCEPT AS NOTED.
2. LAYOUT OF REINFORCING STEEL WILL BE SIMILAR FOR LEFT HAND AND ZERO DEGREE SKEW.
3. BARS MARKED WITH PREFIX C DENOTE COATED BARS.
4. WATERPROOFING AT JOINT BETWEEN BRIDGE AND APPROACH SLAB TO BE IN ACCORDANCE WITH OFSD-3908.020.
5. WATERPROOFING FOR BRIDGES WITHOUT EXPANSION JOINTS (WATERPROOFING OF APPROACH SLAB ELEMENTS) TO BE IN ACCORDANCE WITH OFSD-3908.030.



FOR BRIDGES WITH EXPANSION JOINTS

SECTION AT WINGWALL

**DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING**

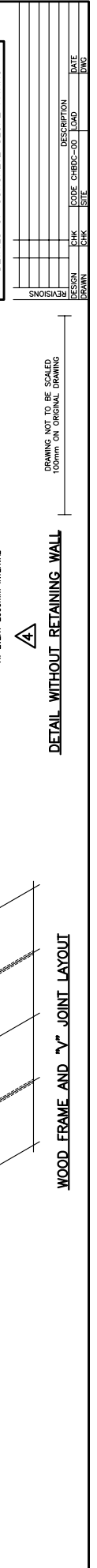
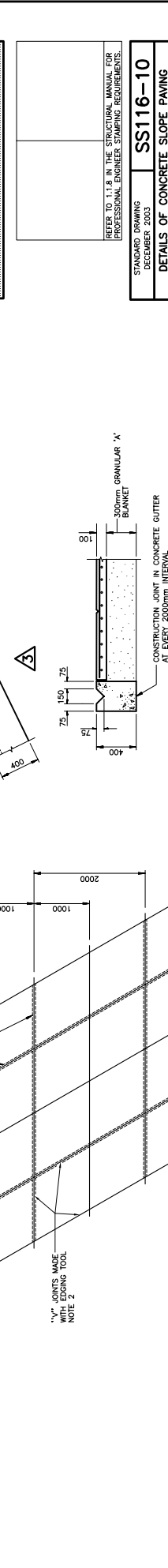
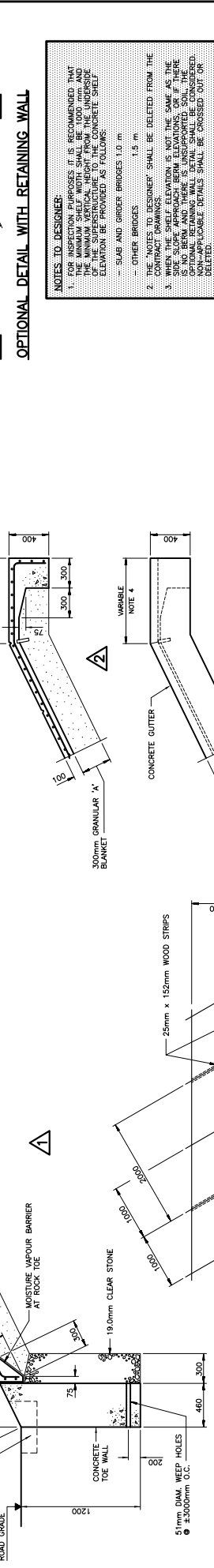
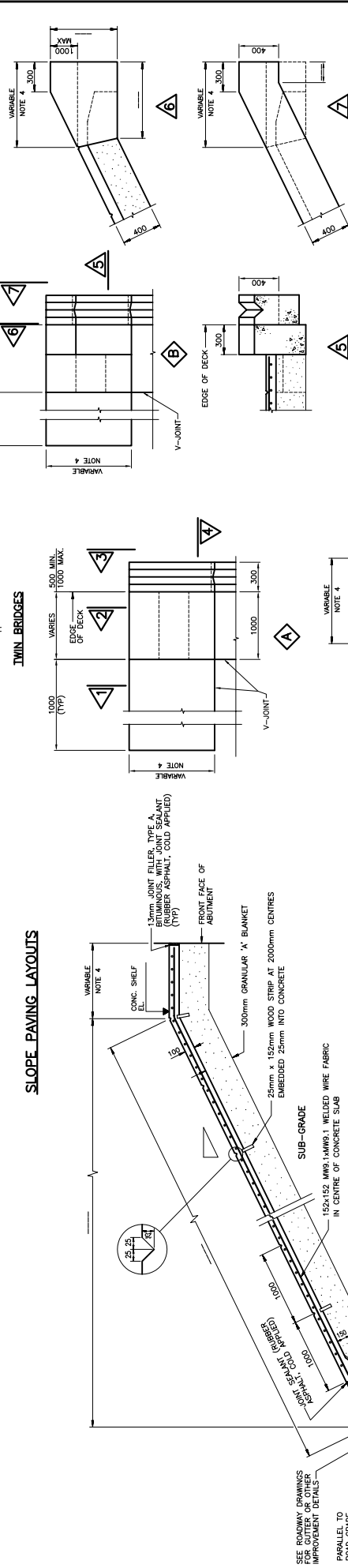
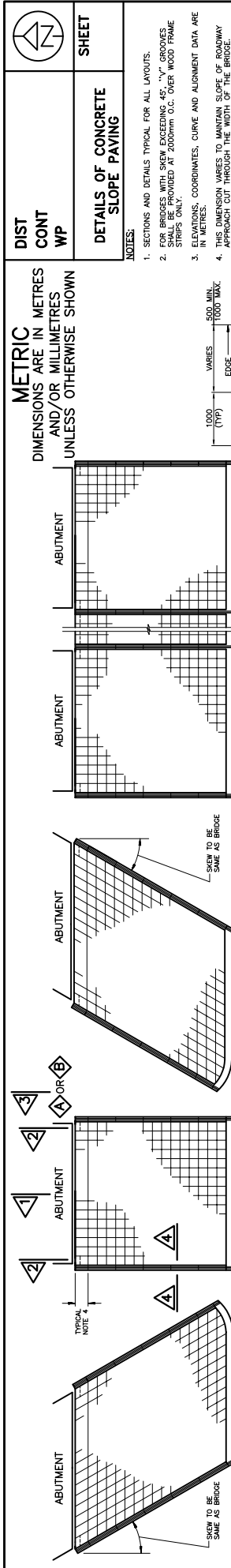
APPLICABLE STANDARD DRAWINGS

OPSD-3906.020	BRIDGE DECK WATERPROOFING - HOT APPLIED ASPHALT MEMBRANE WITH PROTECTION BOARD
OPSD-3906.030	BRIDGE DECK WATERPROOFING DETAILS AT ACTIVE WIDE CRACKS GREATER THAN 2mm AND CONSTRUCTION JOINTS

REFER TO 1.1.8 IN THE STRUCTURAL MANUAL FOR PROFESSIONAL ENGINEER STAMPING REQUIREMENTS

STANDARD DRAWING JUNE 2002	SS116-1
6000 mm APPROACH SLAB	

[illegible]



METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

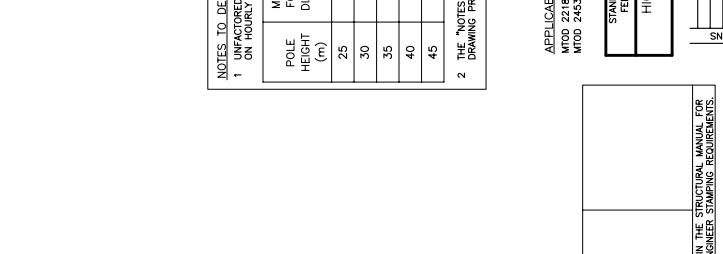
CONT

UNLESS OTHERWISE SHOWN

[illegible]

6 ANCHOR TEMP

- NOTES:**



NOTES TO DE
1 UNFACTORED
ON HOURLY

POLE HEIGHT (m)	M F D
25	
30	
35	
40	
45	

2 THE "NOTES DRAWING PR

APPLICABLE

MTOD 2218
MTOD 2453

STAN	HII
FEL	

DESIGN	
DRAWN	
REVISIONS	

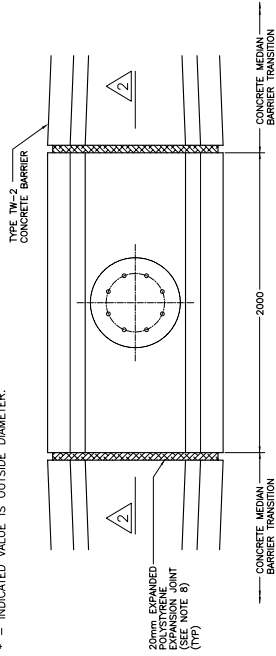
QUANTITIES

TOTAL NO. OF POLES THIS SHEET =

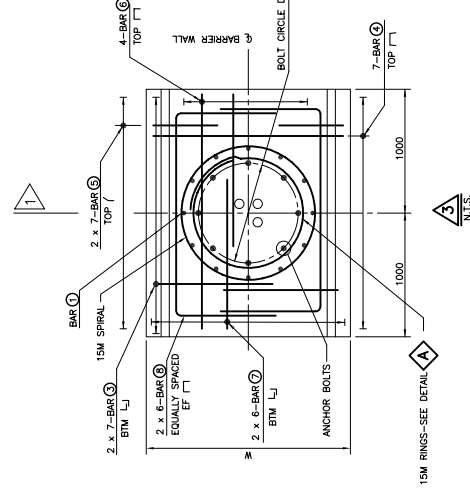
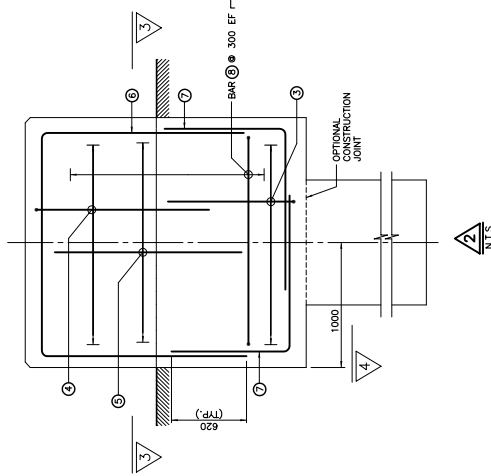
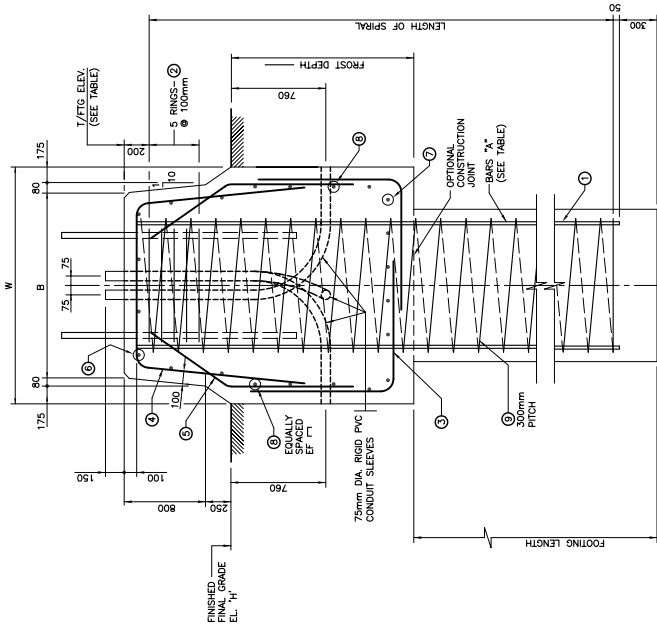
FOOTING DATA

POLE No.	LOCATION	HEIGHT (m)	T/FTG ELEVATION (m)	FOOTING DIAMETER (m)	FOOTING LENGTH (m)	REINFORCING BARS "A" No.	SIZE	LENGTH (m)	SPIRAL LENGTH (m)	DIAM. (m)	B (mm)	EL. "H" (m)	W (mm)

NOTE * - INDICATED VALUE IS OUTSIDE DIAMETER.



PLAN



METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

NOTES TO DESIGNER

1 UNFACTORED LOADS PROVIDED IN TABLE BELOW ARE BASED ON HOURLY MEAN WIND PRESSURE $q = 595 \text{ Pa}$.

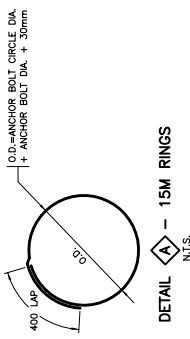
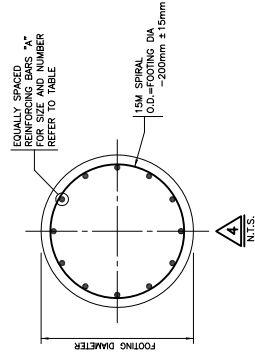
POLE HEIGHT (m)	FOOTING DIAMETER (m)	REINFORCING BARS "A"		B min. (mm)	UNFACTORED LOADS AT BASE OF H/HP	
		MAX. SIZE			MOMENT (kN-m)	SHEAR (kN)
25	1.22	30M		1350	380	29
30	1.37	30M		1500	570	38
35	1.37	30M		1500	815	50
40	1.52	35M		1650	895	48
45	1.52	35M		1650	1155	56

2 INDICATE FROST DEPTH.

3 THE "NOTES TO DESIGNER" SHALL BE DELETED FROM THIS DRAWING PRIOR TO ISSUING OF THE CONTRACT.

2 INDICATE FROST DEPTH. THE FROST DEPTH* SHALL BE OBTAINED FROM THIS DRAWING PRIOR TO ISSUING OF THE CONTRACT.

- NOTES:
- 1 CLASS OF CONCRETE SHALL BE 30 MPa.
 - 2 CLEAR COVER TO REINFORCING STEEL SHALL BE 100±25mm.
 - 3 REINFORCING STEEL SHALL BE GRADE 400.
 - 4 BARS PREFIXED WITH "C" DENOTES EPOXY COATED BARS.
 - 5 THE REINFORCING STEEL ASSEMBLY SHALL BE CAREFULLY FABRICATED TO ENSURE THE ANCHORAGE ASSEMBLY WILL FIT.
 - 6 EACH ANCHORAGE ASSEMBLY SHALL BE PROVIDED WITH A TEMPLATE.
 - 7 ANCHORAGE ASSEMBLY SHALL BE INSTALLED VERTICALLY. NO ADJUSTMENTS SHALL BE ALLOWED AFTER CONCRETE IS PLACED IN FOOTING.
 - 8 FOR FINISHED GRADE ELEVATION, REFER TO GRADING DRAWINGS.
 - 9 EXPANDED POLYSTYRENE INSULATION JOINT SHALL BE LOCATED TO GSSB 61.02 AND BE HELD IN PLACE WITH LIGHT GALVANIZED NAILS.
 - 10 IF SOIL CONDITIONS ENCOUNTERED IN FIELD ARE DIFFERENT FROM THOSE SHOWN IN FOUNDATION INVESTIGATION AND DESIGN REPORT, WORK SHALL BE HALTED AND THE CONTRACT ADMINISTRATOR NOTIFIED IMMEDIATELY.



BAR MARK	SIZE	SHAPE
1	15M	STRAIGHT
2	15M	
3	15M	
4	C15M	
5	C15M	
6	C15M	
7	15M	
8	C15M	
9	15M	SPIRAL

TOTAL No. OF POLES THIS SHEET =	

	POLE	T/FTG	FOOTING	FO

NOTE * - INDICATED VALUE IS OUTSIDE DIAMETER.



CONT

2 INDICATE FROST DEPTH.

3 THE "NOTES TO DESIGNER" SHALL BE DELETED FROM THIS
DRAWING PRIOR TO ISSUING OF THE CONTRACT.



APPLICABLE STANDARD DRAWINGS

MTOD 2218.010	HIGH MAST LIGHTING POLE ANCHORAGE ASSEMBLY
MTOD 2453.050	HIGH MAST LIGHTING POLE ANCHORAGE ASSEMBLY DETAILS
OPSD 911.380	GUIDE RAIL SYSTEM, CONCRETE BARRIER PERMANENT TRANSITION INSTALLATION AT PIERS AND POLES

STANDARD DRAWING FEBRUARY 2003	SS116-52
HIGH MAST LIGHTING POLE FOOTING	
MEDIAN TALL WALL MOUNTED (ASYMMETRICAL)	

[illegible]

REFER TO 1.1.8 IN THE STRUCTURAL MANUAL FOR PROFESSIONAL ENGINEER STAMPING REQUIREMENTS.

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING