SECTION 9 – DECKS AND CURBS

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9 BRIDGE DECKS

9.1 General

This section covers requirements for bridge decks, including sidewalks and curbs.

9.2 Deck Geometry

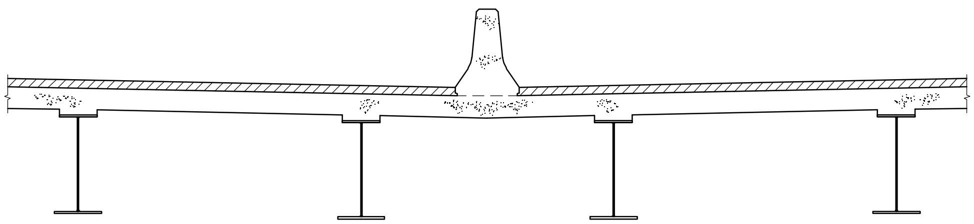
The travelled width of the bridge deck typically matches that of the approach roadway, although some exceptions are possible depending on roadway class. Consideration is also made to future rehabilitation of the bridge. Requirements can be found in the MTO Design Supplement for TAC Geometric Design Guide (GDG) for Canadians Roads.

9.2.1 Bridge Deck Width

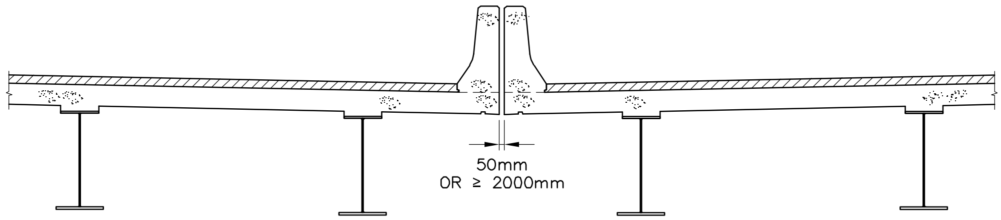
When traffic on a wide bridge is separated by a median curb or barrier, the bridge may be designed as a single wide bridge, or as twin bridges with longitudinal separation at the median. Closely spaced bridges lead to run-off and deicing salts on the cantilever overhangs between them and may promote a microclimate with high time of wetness. From the perspectives of durability and structural stiffness, for bridge deck widths up to 35 m, the preferred approach is to have a transversely continuous structure, see Figure 9.2-1.

Where twin bridges are used, they shall be separated by less than 50 mm or more than 2 m apart. The 2 m minimum permits inspection with the Bridgemaster. Bridges carrying bidirectional traffic with a deck width less than 35 m shall be designed without longitudinal joints. Decks wider than 35 m and twin parallel bridges (typically with high skew or curvature) with cumulative width less than 35 m, require approval of the Structural Section.

As the bridge deck gets wider, transverse restraint due to thermal loads and shrinkage must be considered. The deck may require additional transverse reinforcement to control cracking.



a) Transversely Continuous Bridge



b) Twined Bridge Configuration

Figure 9.2-1 – Closely Spaced Parallel Bridges vs. Continuous Span Carrying Both Directions of Travel

9.2.2 Sidewalk Widths on Bridges

Sidewalk widths on bridges shall comply with the *Accessibility for Ontarians with Disabilities Act*, S.O. 2005, c.11, and the Transportation Association of Canada. Geometric Design Guide for Canadian Roads: 2017, in conjunction with the MTO Design Supplement. For designs of new bridges with sidewalks or bridge rehabilitations where the sidewalks are rehabilitated, unless otherwise permitted in section 80.31 of *O. Reg. 413/12*, the minimum clear sidewalk width shall be 1500 mm. In addition, the minimum clear sidewalk width shall:

- Be maintained over the entire length of the bridge and transition smoothly to the approaches;
- Not be reduced below 1500 mm by appurtenances such as guiderail connections to the bridge;
- Apply to the clear surface of the sidewalk excluding surfaces of other elements such as curbs or tops of walls that are separated from the sidewalk by longitudinal joints.

The requirements above are illustrated in Figure 9.2-2 for greater clarity. These requirements are typically met using an 1800 mm wide sidewalk (measured at asphalt level and with the standard curb face slope of 50 mm horizontal to 150 mm vertical) using standard MTO details on the bridge approaches.

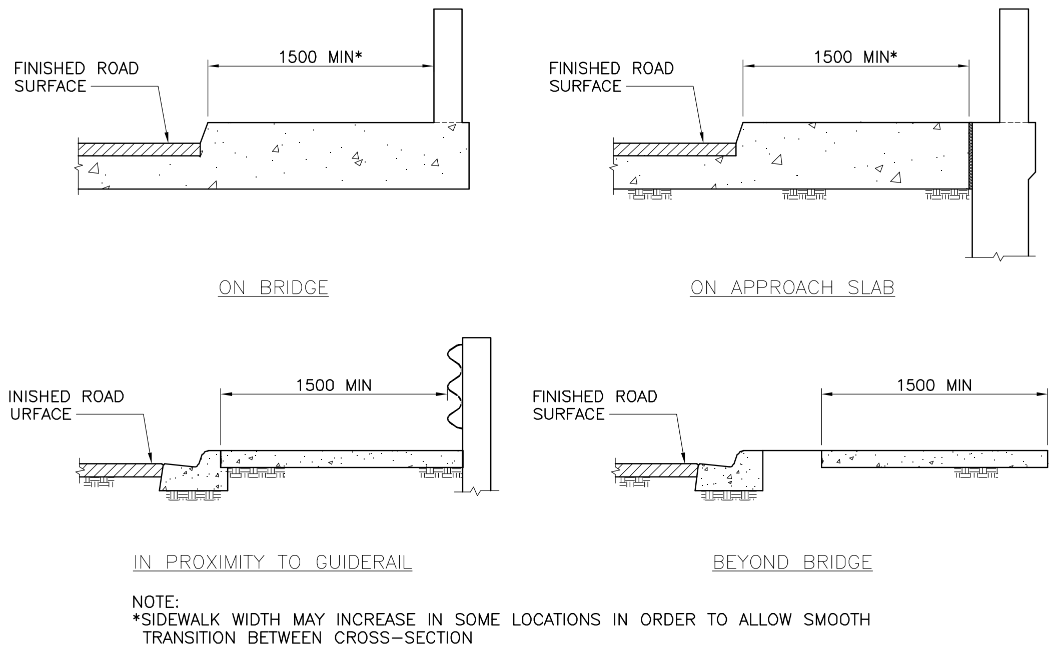


Figure 9.2-2 – Sidewalk Width

9.2.3 Deck Fascia Details

Fascia treatment is categorised by deck type and the presence or absence of a sidewalk as shown in Figure 9.2-3. Slab on girder decks with a sidewalk or curbs supporting open railings shall have a 500 mm to 600 mm high fascia, depending on the barrier impact loading. Decks without a sidewalk supporting concrete barriers shall have a 350 mm or 400 mm high fascia, depending on the barrier impact loading. Cast in place post-tensioned decks shall have fasciae from 350 to 600 mm high.

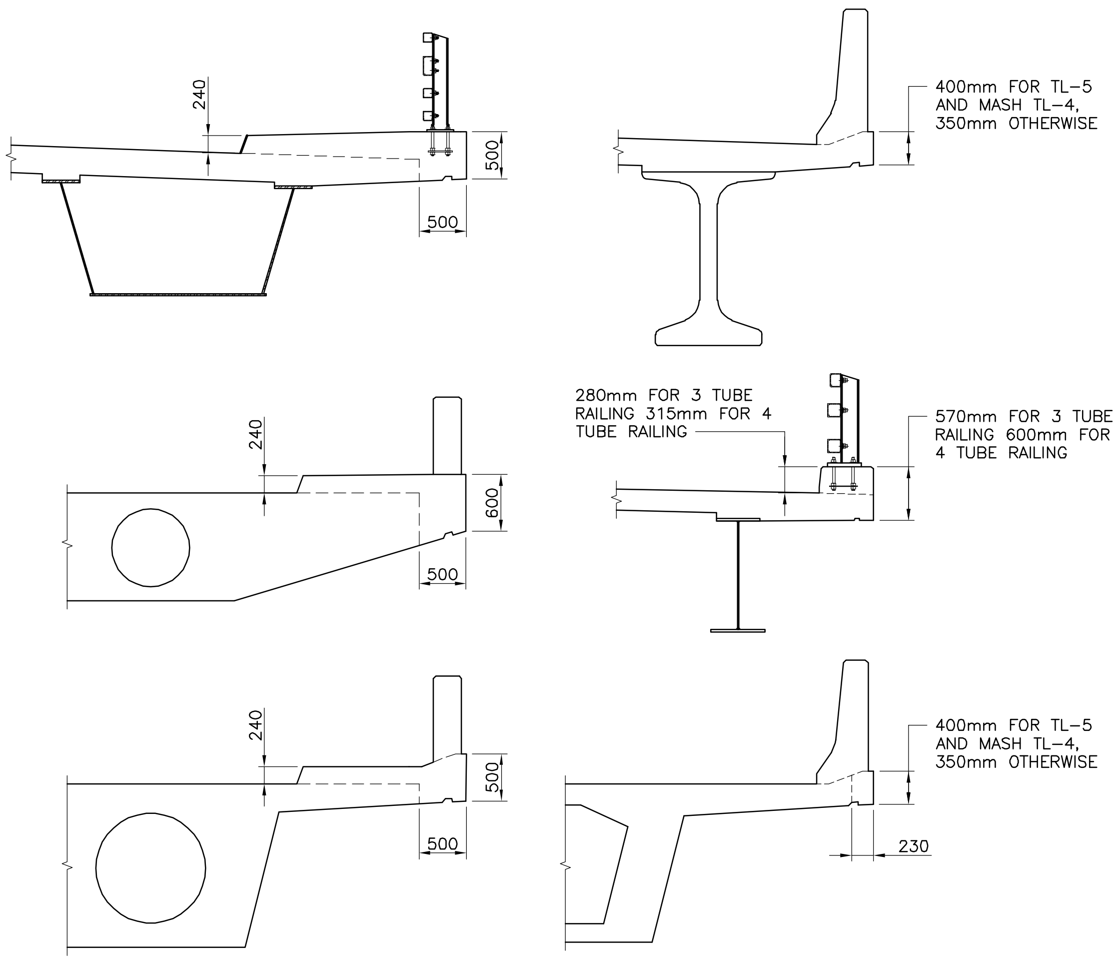


Figure 9.2-3 – Deck Fascia Details

9.3 Deck Slab Design

The Ministry developed the empirical method of bridge design in the 1970's and it has been used for deck design since the introduction of the first Ontario Highway Bridge Design Code (OHBDC) in 1979. This has been used continually since that time, with similar provisions being adopted by Section 8 of the CHBDC since the year 2000. Generally, for regular shape decks, all interior portions of decks use the empirical design method, which results in a 225 mm thick deck with a top and bottom mat of 15M reinforcement at 300 mm spacing in each layer in each direction.

9.3.1 Concrete in Deck Slabs

To ensure adequate durability, the Specified 28-day Compressive Strength for decks shall be a minimum of 30 MPa.

9.3.2 Minimum Thickness of Deck Slabs

In order to permit placing four layers of reinforcing steel with the required cover and associated tolerances, the minimum thickness of deck slabs shall be 225 mm. Such slabs should be detailed so that the bars in the positive two layers of distribution (longitudinal) reinforcement are not vertically in line with one another. If possible, size 15M bars shall be used except for negative moment regions over piers or at deck cantilevers where larger diameter reinforcing steel bars would be required.

For deck slab made with full-depth precast deck panels, the minimum thickness of the panels shall be 200 mm.

9.3.3 Longitudinal Reinforcing Steel Below Barrier Walls

Cantilever portions of deck slabs tend to develop transverse cracks at barrier wall construction joint locations.

To control the cracks, 15M at 200 mm top and bottom longitudinal reinforcement shall be provided in cantilever slabs under barrier walls starting from the outside end of the slab and extending inward 500 mm from the inside face of the barrier wall (or to the outside girder flange, if it is closer than the 500 mm). See Figure 9.3.1.

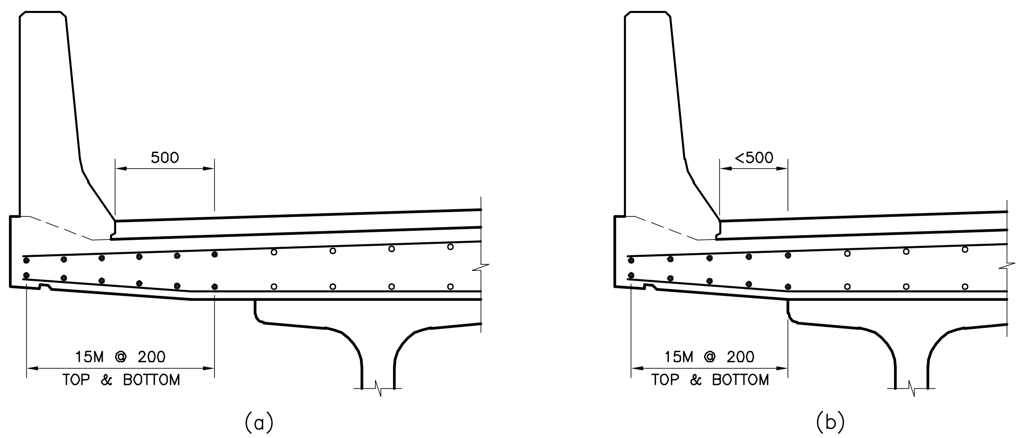


Figure 9.3.1 – Longitudinal Reinforcement Below Barrier Wall

Cantilever portions of deck slabs with sidewalks do not require this additional crack-control reinforcement in the top mat.

9.3.4 Reinforcement in Deck Cantilever Overhangs

The reinforcing arrangement shown in Figure 9.3-2 shall be adopted for all new bridges.

The size and length of the top transverse reinforcing bars shall be taken from DESIGN AID 9-1 & 9-2. These reinforcement values have been developed in accordance with the analysis and design methodologies outlined in CSA S6:25, which has increased the demand due to impact loading compared to prior versions. The design aid is valid for both solid barriers and steel railings on curbs.

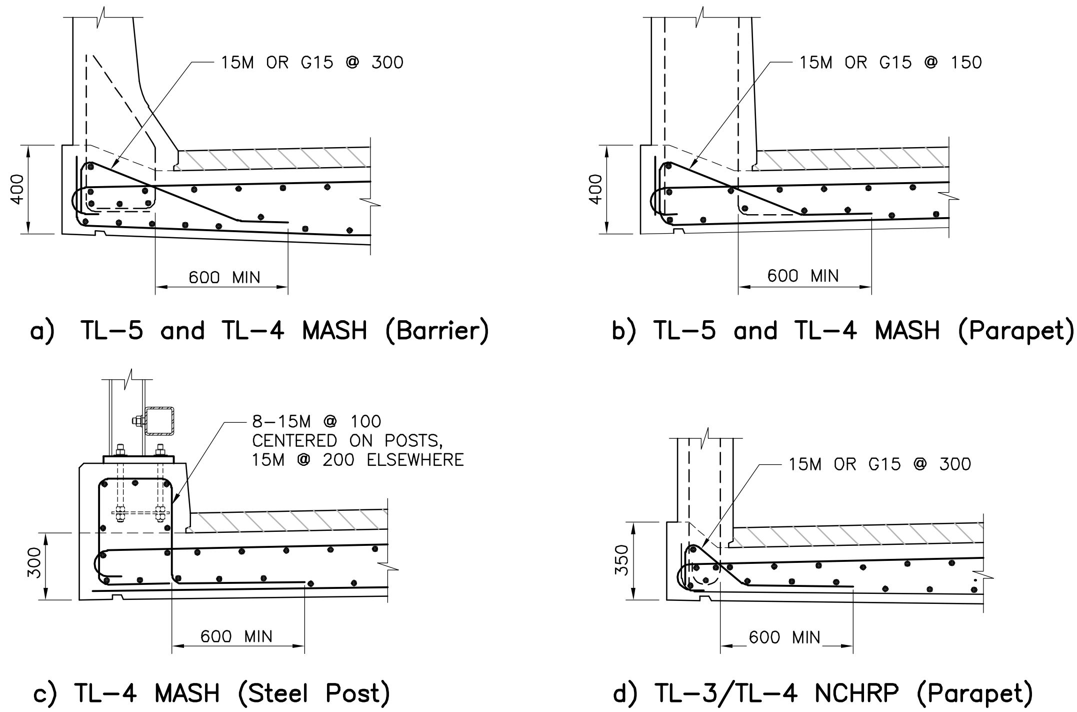
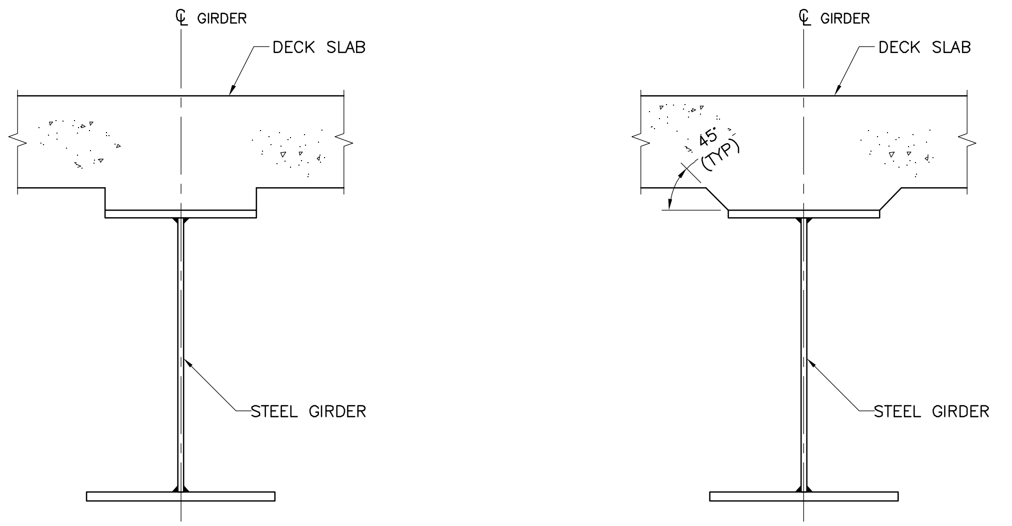


Figure 9.3-2 – Typical Reinforcing Steel Anchorage Details at Transverse Deck Slab Cantilever Overhang to Barrier Connection

9.3.5 Haunches

Concrete haunches of varying thickness may be used to fit the top of girders to a deck slab of constant thickness to achieve the proper screed elevation. The haunches shall be the same width as the top flange and shall not cover the sides of the top flange as shown in Figure 9.3-3(a). For deep haunches, the detail shown in Figure 9.3-3(b) is also acceptable but not preferred.



a) Typical Haunches b) Deep and Narrow Haunches

Figure 9.3-3 – Haunch Detailing

Shear stud height in a steel girder shall project into the deck slab above the bottom transverse bars as required by the code. Stirrup projection in a concrete girder shall extend to develop the strength of the bar across the interface between the top of the girder and the cast-in-place slab, unless a reduced strength of the bar is assumed in the calculation of interface shear. Nevertheless, the stirrups shall project a minimum of 25 mm above the bottom mat of bars. . Haunches shall not be reinforced unless the haunch depth above the flange exceeds 100 mm. Stirrup projections and shear stud height shall be designed as necessary to avoid additional flange reinforcement except when stirrup projections or shear stud length exceeds 300 mm, in which case haunches shall be reinforced to extend the bottom mat of reinforcing steel downwards into the haunch.

Typically, steel girders are fabricated to follow the roadway profile through built-in camber and a uniform haunch thickness is achieved along the girder length and in transverse direction, whereas concrete girders require a variable haunch to make up the difference between the highway profile and the deformed shape of the girder prior to casting the deck. Nevertheless, the actual haunch on site could vary from estimated haunch which may affect the stirrup projection in the deck. When stirrup projection turns out be less than required by design, the haunch is required to be reinforced with transverse bars, usually in this shape  with standard hooks, to interlock with the girder stirrups or studs. Where stirrup projections or shear studs are too long and impede cover to the top of deck, consideration can be given to bending them to achieve the cover.

Additionally, it is important to ensure that the girders do not penetrate more than 25 mm into the nominal thickness of the deck slab. This may happen if an insufficient allowance has been made at mid span for the upward prestress camber of precast girders and also if the deck has a sag vertical curve. As a general rule, to achieve all of the criteria above, it is necessary to provide haunches raising the bottom of the deck above the top of precast beams or the top of steel beam flanges, with haunch heights at the supports as shown in Table 9.3-1. For steel beams spanning 80 m and above, a thicker haunch may be necessary.

Table 9.3-1 – Typical Haunch Heights

Haunch Height (mm)	Concrete Girder	Steel Girder
50	Spaced box beams and NU 900, 1200	Steel girders with web depth < 1400 mm
75	NU 1400 or larger	Steel girders with web depth ≥ 1400 m

9.3.6 Stay-in-place Forms

Steel forms are considered to have many disadvantages and are therefore not to be used. They are or may become unsightly. They may cause maintenance problems and at the very least impede inspection. It does not appear that there is any economic advantage to their use. Stay-in-place Forms of any material shall not be used without approval of the Ministry.

9.3.7 Partial Depth Precast Deck Panels

As a result of research by MTO and others, precast, usually prestressed, concrete deck panels were introduced to Ontario in the 1990s as a means of accelerating bridge construction. The deck cantilevers are still formed conventionally, so the benefits of the panels diminish for narrow bridges with few girders, as well as for bridges constructed using staged construction where there are two additional temporary deck cantilevers. The use of the panels has been found to be advantageous for steel box girder bridges, or at least for the slab span within the steel box, to eliminate the difficult task of removing conventional formwork. The thickness of the deck panels shall be according to SS109-42 and SS109-44, and are topped with 135 mm of concrete. Tests have shown that stirrups connecting the topping slab to the panel are not required, although a few are added to the standard drawing to facilitate tying of the reinforcement for the topping slab. The design of partial depth deck panel deck is provided in the CHBDC, while details are contained in drawings SS109-42 and SS109-44.

This deck system requires continuity between the reinforcement in the deck panels. This is achieved, for the longitudinal reinforcement, by placing continuous reinforcement along the top of the panel. For bottom transverse reinforcement, strands or regular rebars must be developed over the girders. For NU girders, with their wide top flanges, lapping is easily achieved, while for steel girders the strands must be crossed over the adjacent panel.

For large spans with large negative moments in the deck, or bridges with large overhangs, adequate space between reinforcement may be difficult to achieve. For these aforementioned situations, a deck thicker than 225 mm may be required. For bridges on skew, the panels span perpendicular to the girders, and the triangular portions of deck near supports are formed conventionally with cast-in-place concrete.

9.3.8 Additional Reinforcement in Skewed Decks

The following detail (Figure 9.3.4), along with Table 9.3-2, shows the additional reinforcement required, unless analysis dictates otherwise, for the cantilevered portion of thin slab bridge decks with skews exceeding 20° but less than 45°, and with the cantilever span not exceeding 1.6 m. Skewed decks with cantilever spans less than 0.6 m do not require this additional reinforcement.

Table 9.3-2 – Additional Deck Reinforcement for Different Skews

SKEW ψ	CANTILEVER	ADDITIONAL REINFORCEMENT
20° - 35°	0.6 m - 1.2 m	6 - 20M
20° - 35°	1.2 m - 1.6 m	6 - 25M
35° - 45°	0.6 m - 1.2 m	8 - 20M
35° - 45°	1.2 m - 1.6 m	8 - 25M

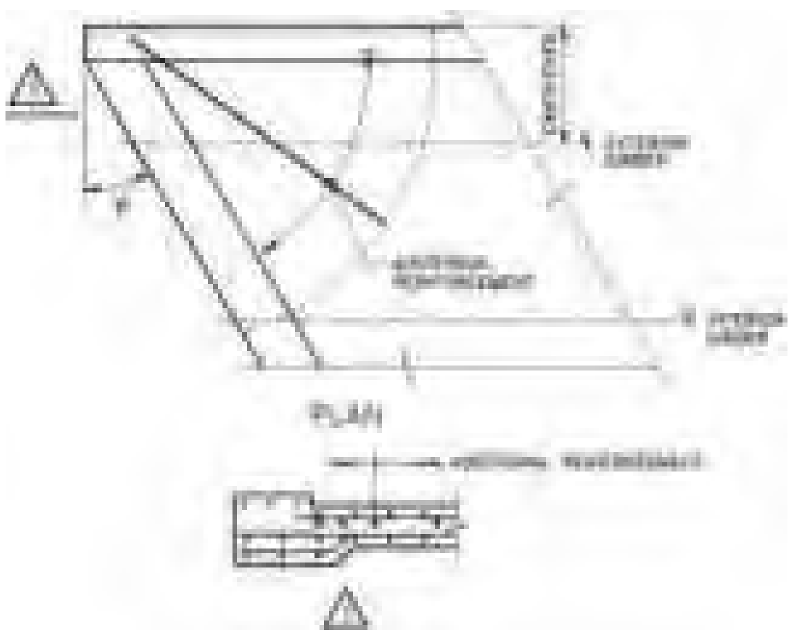


Figure 9.3.4 – Additional Reinforcement in Skewed Decks

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

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

9.4 Construction

9.4.1 Deck Construction Joints

Construction joints across which force effects are assumed to be transmitted or which must resist leakage, require special treatment. Introduction of construction joints that are not shown on contract drawings is not allowed without approval.

It should be noted that waterstops in deck construction joints shall not be used.

9.4.2 Sequence of Deck Placements for Slab-on-Girder Bridges

No deck placing sequence should be specified when the deck can be cast in one continuous operation from one abutment to the other unless there are specific reasons, such as when the deck pour could exceed 12 hours in duration or where the structure type or structural articulation dictates otherwise (i.e., arch bridges, continuous multi-span girders with short end spans that could lead to lift off the abutment bearings when concrete is placed in the adjacent span, etc.).

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

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

CONCRETE IN DECK SLABS AND DIAPHRAGMS SHALL BE RETARDED USING TYPE “B” OR “D” ADMIXTURE TO ENSURE THAT THE CONCRETE REMAINS PLASTIC FOR THE DURATION OF EACH PLACEMENT.

The deck placing sequence should be shown in numerical order.

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

9.4.3 Screed Elevations on Bridge Decks

Screed elevations are the elevation to which the deck needs to be placed to achieve the final vertical profile after all dead load deflections occur. Screed elevations are achieved by the contractor by adjusting the height of the haunches as required.

9.4.3.1 Slab on Girder Decks

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

9.4.3.2 Post Tensioned Concrete Decks

Screed elevations shall be given at break points in the deck, and on the deck at the face of curbs or barrier walls. The screed elevations shall be given at intervals not exceeding

3 m. No allowance is required normally for dead load deflections provided that prestress and dead load are more or less balanced.

9.4.4 Construction Loading from Deck Cantilever Overhangs

Girders shall be designed to account for the load effects of the deck cantilever overhang during the deck pour.

OPSS 919 requires the screed rail to be placed above the centre of the web. In cases where this is not feasible based on the need to screed the deck to the full width (i.e., staged construction of a two-lane bridge), the girder design shall account for the load of the screed machine acting on the screed rail located 0.1 m outside of the deck fascia, and an NSSP shall be included to permit this condition.

9.4.5 As-Constructed Elevations

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

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

9.5 Durability

9.5.1 Bridge Deck Waterproofing

Bridge decks detailed with an asphalt wearing surface must also be shown as being waterproofed. The drawings should state only "Asphalt and Waterproofing System, 90 mm total." (See Section **2.6.7**).

9.5.2 Future Wearing Surface

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

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

9.5.3 Waterproofing of Deck Cantilever of Steel Box-Girder Bridges

For steel box-girder bridges, the face of curb/sidewalk, if any, should preferably start beyond the exterior flange of the exterior girder. If this is not practical, hot-applied rubberized asphalt waterproofing system shall be used to prevent salt-laden deck run-off from invading the joint between the slab and the sidewalk, filtering through the deck and into the unprotected steel box-girder, causing serious internal corrosion. This damaging potential is intensified on wide-deck bridges, and especially in the vicinity of piers, where the concrete decks may have numerous flexural cracks in the negative moment region. Hot poured rubberized waterproofing system shall be according to OPSS 914 except that protection board shall not be used. The concrete sidewalk shall be placed directly on the waterproofing membrane, a minimum of 1 day after the waterproofing has been placed.

This waterproofing should start at 150 mm in front of the face of the sidewalk, continue along the top of the deck/sidewalk construction joint to the vertical joint face under the barrier wall. The waterproofing system should extend the full length of the bridge.

Whenever this requirement is used, additional dowels should be added between the deck/sidewalk interface to ensure continuity and the integrity of the deck cantilever to resist the traffic barrier loads given in Clause 3.8.8.1 of the CHBDC. The total area of dowels between the top of the deck and the sidewalk shall not **e** less than 2000 mm² per m along the length of the bridge.

9.5.4 Structure Deck Drainage

Requirements for deck drainage inlets must be established as part of the overall drainage design of the crossing. The size and number of drains shall meet the requirements of MTO's Highway Drainage Design Standards. Bridge deck drains shall be provided only where necessary. Structures having one or two lanes draining to one side, built with normal crossfall on a vertical crest curve not more than 120 m long, normally do not require deck drainage inlets.

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

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

OPSD 3340.100 deck drainage inlets do not collect a significant quantity of water and should be used only to prevent local ponding. This is sometimes necessary when flat grades are unavoidable, or structures are subject to substantial permanent deflections that cannot be accurately predicted.

Where collector drain pipes are required, they shall be hot dipped galvanized ASTM A53 steel pipe with a minimum wall thickness of 6.4 mm, supported by pipe clamps at a maximum spacing of 2 m, and carefully detailed to meet the requirements of Clause 1.7 of the CHBDC. HDPE, PE or PVC drain pipes shall not be used above ground.

To enhance long-term performance and corrosion resistance and achieve a deck drainage system with a service life commensurate with non-replaceable components, the Ministry of Transportation of Ontario (MTO) has introduced a new standard, MTOD 3340.152, which outlines the requirements for stainless steel drainage inlets. Where drain pipes are required, they shall be stainless steel ASTM A312/A312M, Type 316L, and detailed with stainless steel clamps and fasteners.
Catch basins are normally necessary just beyond the structure limits to intercept runoff from bridge decks. A continuous length of curb or gutter is provided to connect the bridge curb or barrier to the catch basin to prevent wash outs around the ends of wing walls. Detailing of the wing wall or approach slab should be such as to permit straight vertical junctions with approach curbs. The final grading drawings should be reviewed in conjunction with the structure drawings to ensure that this or an equally acceptable arrangement has been adopted.

Deck drainpipes shall be located outside of the tub or box girders and shall not cross through tub or box girders. Drainage analysis shall be performed at preliminary design stage to determine if deck drains can be avoided. If deck drains are required, the designer shall select a structural type and configure the cross-section accordingly.

9.5.5 Drip Grooves

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

The dimensions are shown in Figure 9.5-1.

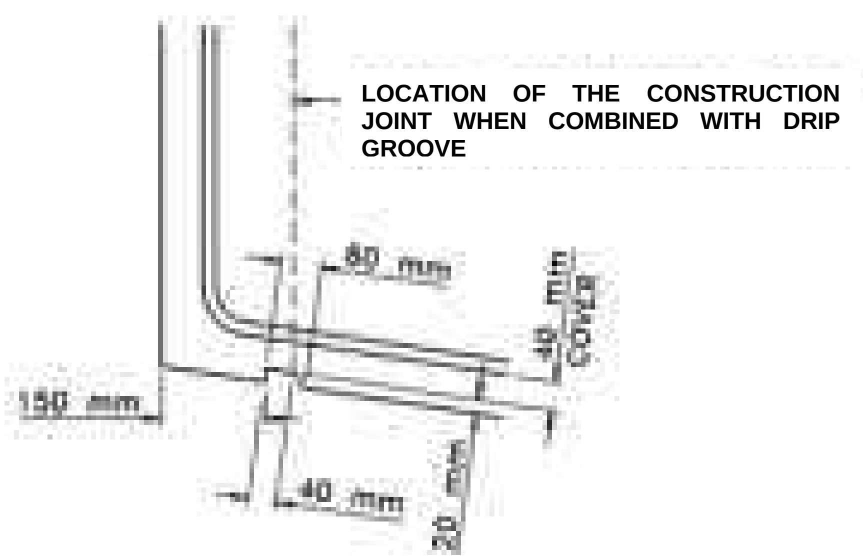


Figure 9.5-1 – Drip Grooves

For practical and aesthetic reasons, it may be desirable to combine the drip with construction joints that result from transverse prestress anchorage recesses, or decks with curbs or sidewalks. For this case the drip should not be located greater than 250 mm from the fascia (see OPSD 3390.100).

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SECTION 10 - BARRIERS AND RAILING SYSTEMS

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10 BARRIERS AND RAILING SYSTEMS

10.1 General

a) Bridge railing shall be selected in accordance with the requirements of Section 12 of the CHBDC; that is, conforming to TL-2, TL-4, or TL-5, meeting the crash test requirements specified in the NCHRP Report 350, or the AASHTO Manual for Assessing Safety Hardware (MASH).

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

All bridge barriers adopted for use by the Ministry originate from the crash-testing programs carried out in the United States. The current standards used by the Ministry are based on crash tests carried out in accordance with the crash test procedures of:

- Michie, J. D. (1981). NCHRP Report 230: Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances. Washington: Transportation Research Board.; and/or,
- Ross, H. E., Sicking, J. L., Zimmer, R. A., & Michie, J. D. (1993). NCHRP Report 350: Recommended Procedures for the Safety Performance Evaluation of Highway Features. Washington: Transportation Research Board.; and/or,
- AASHTO. (2009). Manual for Assessing Safety Hardware (1st ed.). American Association of State and Highway Transportation Officials., or deemed equivalent.

MTO is in the process of moving towards the use of MASH 2016 barriers. Further information about crash tested barrier systems can be found in AASHTO publications, NHCRP reports, Transportation Research Record, and FHWA memorandums. The reference subsection of the commentary to Section 12 of the CHBDC gives a comprehensive listing of pertinent documents, and the following FHWA web site also gives actual details of approved railing systems and memorandums:

https://safety.fhwa.dot.gov/roadway_dept/countermeasures/reduce_crash_severity/listing.cfm?code=long

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

b) Slip forming of concrete barriers on structures is not permitted.

c) When curbs are required, for sidewalks, raised medians etc., they shall be 150 mm high.

10.2 Approved Traffic Barriers

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

For bridges on low volume roads with an AADT, in both directions, of 400 or less, a Test Level lower than TL-2 is acceptable. MTO's criteria for selecting a lower test level for local low volume roads is provided in Division 1.

10.2.1 Test Level 2 (TL-2)

As none of the available crash tested TL-2 systems meet the minimum pedestrian height and other geometric requirements, none are suitable for sites where pedestrian protection is warranted. The barriers were crash tested to NCHRP 230, or AASHTO Guide Specifications for Bridge Rails, and were found to be NCHRP 350 compliant for TL-2 as per a 1997 FHWA memorandum.

The following barrier types are standards used by the Ministry for TL-2:

a) Box Beam Guide Rail – Side Mount (Structural Standard Drawing SS110-2)

The system in Figure 10.2-1 is based on the crash tested *California Type 115 Bridge Railing*. Its standard structural "W" and "HSS" shapes provide a relatively inexpensive, easy to fabricate and erect railing suitable for vehicular only warrants on low speed and/or low volume highways.

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

b) Thrie Beam Guide Rail – Side Mount (Structural Standard Drawing SS110-5)

The system in Figure 10.2-2 is based on the crash tested *Oregon Side Mounted Thrie-Beam Bridge Railing*. The Oregon side mounted system consists of standard "Thrie Beam" elements mounted on standard structural steel posts. It is ideally suited for cast in place and precast slab superstructures with at least 400 minimum slab depth. It should be noted that the "W" shape steel beam guide rail used in Ontario is not the same as the "thrie" beam section. Currently there are no crash tested bridge rail systems available incorporating the "W" shape steel beam.

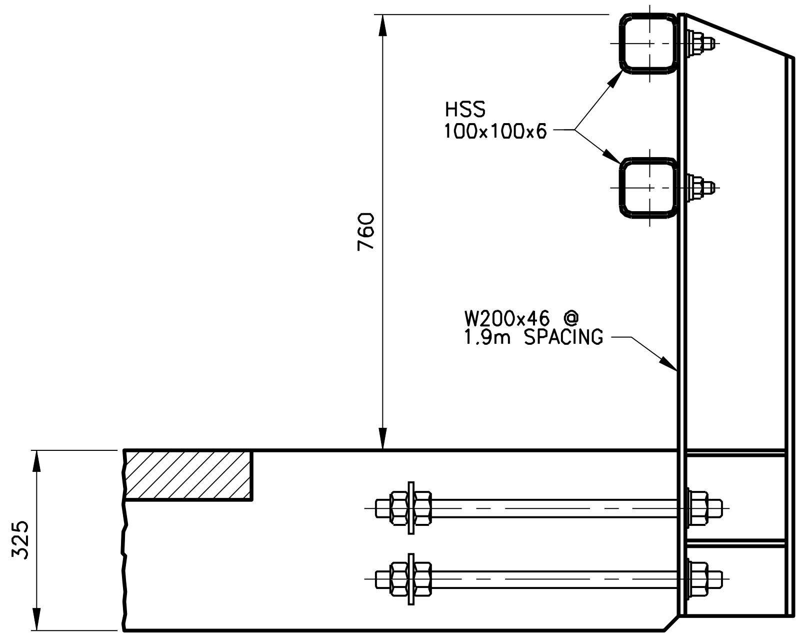

Figure 10.2-1 – Box Beam Guide Rail – Side Mount (TL-2)

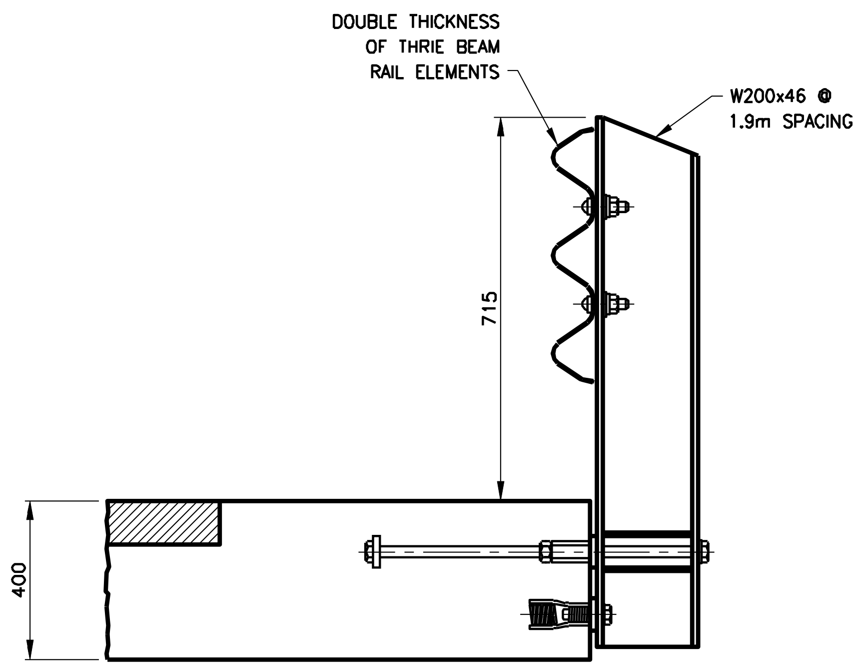

Figure 10.2-2 – Thrie Beam Guide Rail – Side Mount (TL-2)

c) Thrie Beam Guide Rail – Timber Deck (Structural Standard Drawing SS110-7)

The system in Figure 10.2-3, for longitudinal timber bridge decks, is based on the *Steel System-Thrie Beam on Steel Posts* bridge railing crash tested for the US Department of Agriculture Forest Service. This in turn is an adaptation of the *California Thrie Beam Bridge Rail* modified for timber bridge decks. It consists of a thrie beam connected to standard structural steel "W" shape posts and spacer blocks side mounted on the bridge deck. The steel system is connected to the deck with high strength bars. Material costs are more economical than longitudinal glulam and timber post alternatives.

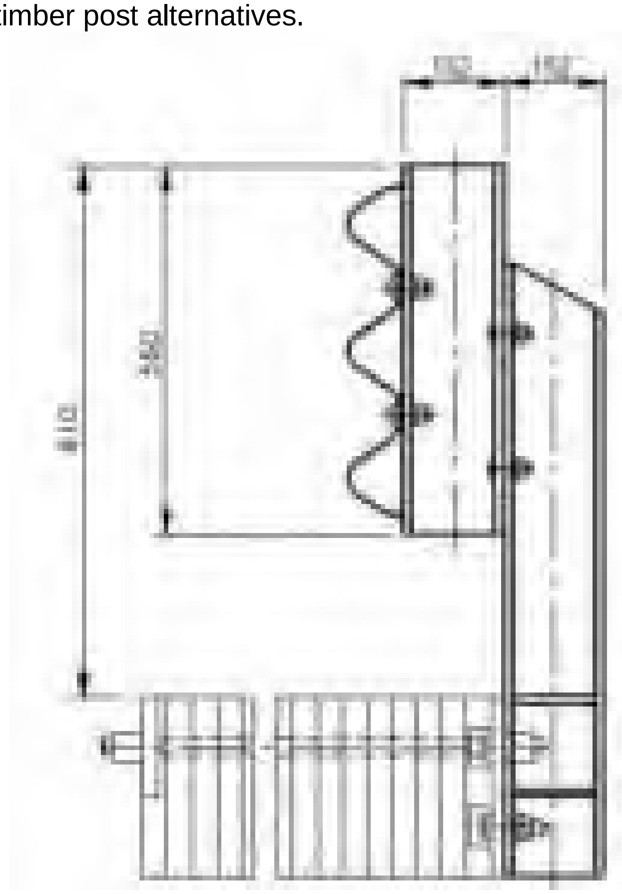

Figure 10.2-3 – Thrie Beam Guide Rail – Timber Deck (TL-2)

10.2.2 Test Level 4 (TL-4)

The following barrier types are standards used by the Ministry for TL-4, each of which is based on an NCHRP 350 crash tested barrier:

a) Barrier Wall with Railing (Structural Standard Drawings SS 110-54/58/91)

The system in Figure 10.2-4 is based on the crash tested *32-inch (813 mm) F-Shape Bridge Railing*. It is constructed of reinforced concrete and this type is the most common rigid traffic barrier in use today on both roadways and bridges. F-shape does not refer to the shape of the barrier but merely to the crash test designation. Its popularity is based on its generally effective performance as a barrier, particularly with its re-directional capabilities, and its low maintenance costs. The concrete barrier requires virtually no maintenance for most hits. The lower sloped face redirects vehicles without damage under low-impact conditions. During moderate to severe impacts, some energy is dissipated when the vehicle is lifted off the pavement. The loss of tire contact with the pavement also aids redirection. In crash tests, the F-shape has proven to be more successful than the New Jersey shape in preventing rollover for small vehicles. The barrier may redirect or contain heavy vehicles, but it was not designed for this purpose. Therefore, it is most suitable for highways carrying traffic with low heavy truck volumes.

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

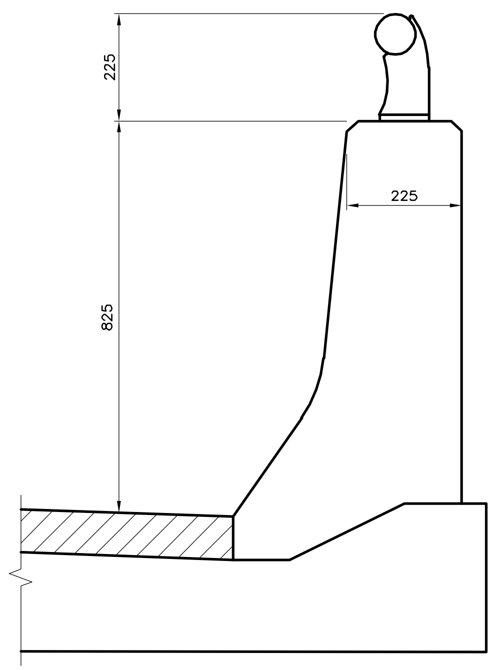

Figure 10.2-4 – Barrier Wall with Railing (TL-4)

b) Parapet Wall with Railing (Structural Standard Drawings SS 110-56/57/59/90/97/98/104/105)

The system in Figure 10.2-5 is based on the crash tested *32-in (813-mm) vertical concrete parapet*. It offers a simple to build reinforced concrete alternative to the F-shape railing. Vertical concrete walls do not have the energy management feature of the F-shape, but crash test have demonstrated that they perform acceptably as traffic barriers. Because vehicles are not lifted or tilted, all four wheels tend to stay on the ground and all the energy absorption upon impact goes into the crushing of the vehicle. Therefore, damage to a vehicle on impact with the parapet is likely to be more severe compared to the F-shape and redirection not as smooth. Potential rollover is minimised, however, with a vertical face.

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

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

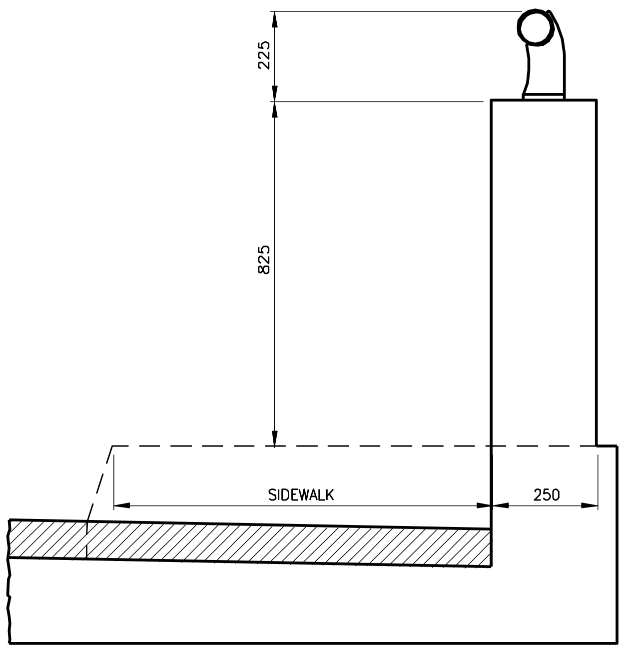

Figure 10.2-5 – Parapet Wall with Railing (TL-4)

c) Box Beam Guide Rail on Curb (Structural Standard Drawings SS 110-37/39)

The system in Figure 10.2-6 is based on the crash tested *Massachusetts Type S3 Curb Mounted Bridge Rail*. This system is constructed of standard structural steel W section posts and three HSS rails and offer a good performing lightweight and open ("see through") rail alternative to the concrete barriers outlined above. Even though these rails are acceptable in most applications they are not recommended for use on high speed and limited access highways. For these cases the F-shape is recommended, as it is better at redirecting errant vehicles and requires less maintenance. The systems are usually used with concrete end posts for connection to the approach railing system. An alternative end connection between the box beams and approach steel thrie beam guide rail has been developed to accommodate user needs.

In general, these systems are suitable for vehicular applications, however, the Massachusetts system may be used for vehicular/pedestrian applications if pickets are included.

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

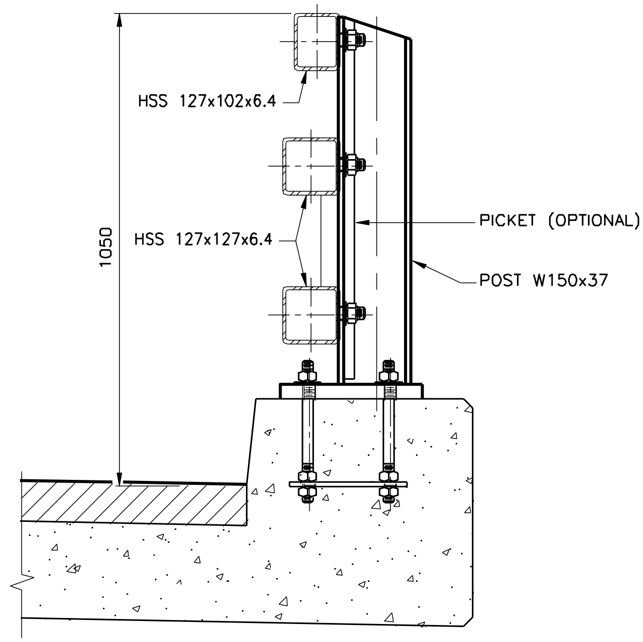

Figure 10.2-6 – Three Tube Railing on Curb (TL-4)

(d) Box beam railing on sidewalk (Structural Standard drawings SS110-46/49)

The system in Figure 10.2-7 is based on the crash tested *New England Transportation Consortium 4-Bar Sidewalk Mounted Bridge Rail*. It is constructed of standard structural steel W section posts and four HSS rails and offers a good performing lightweight and open ("see through") rail alternative to the concrete parapet wall on sidewalk with railing outlined previously.

This system is suitable for vehicular/pedestrian applications. Because it offers one of the most open railings at this test level, it is most suitable for scenic and rural sites.

Although this railing satisfies all the current requirements of the CHBDC with regard to clear spacing between the rails, the ladder like orientation of the horizontal rails make it more inviting to climb for little children. Consequently, a different system should be used when the structure is located near public schools and where the anticipated pedestrian traffic includes little children.

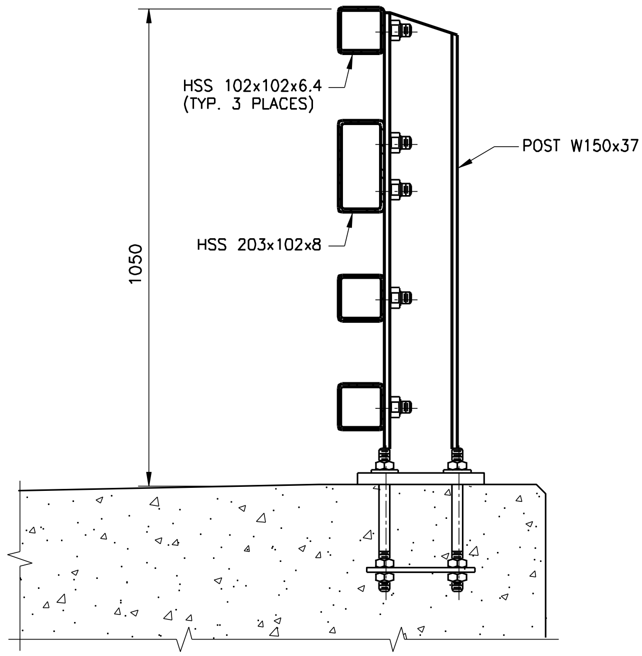

Figure 10.2-7 – Four Tube Railing on Sidewalk (TL-4)

10.2.3 Test Level 5 (TL-5)

The Ministry currently has two barrier/railing systems that satisfy the TL-5 requirements. An F-shape barrier without railing and a system consisting of box beam railing on a concrete parapet wall.

These railing systems provide the maximum level of protection, in the event of a collision, for which the Ministry has a standard. Barriers crash tested to NHCRP 350 or MASH or for special cases such as for heavy trucks are available, and details of these may be found on the FHWA web site.

a) Barrier wall without railing (Structural Standard drawings SS110-61/92/94/109)

The system in Figure 10.2-8 is based on the crash tested *42-inch (1.07 m) F-Shape Bridge Railing*. This railing system is very similar to the F-shape TL-4 concrete barrier in that the front surfaces and its construction are identical except for its height. The crash test characteristics are similar to the F-shape TL-4 barrier except that the extra height reduces potential rollover for impacting vehicles.

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

The system in Structural Standard Drawing SS110-92 is based on the crash test done by Ryerson University in 2011 at TTI on F-shape TL-5 barrier incorporating GFRP bars with anchor head. The crash test was performed in accordance with MASH TL-5.

The system in Structural Standard Drawing SS110-94 is based on the crash test done by Ryerson University in 2016 at TTI on F-shape TL-5 barrier incorporating GFRP bars with hook bar. The crash test was performed in accordance with MASH TL-5.

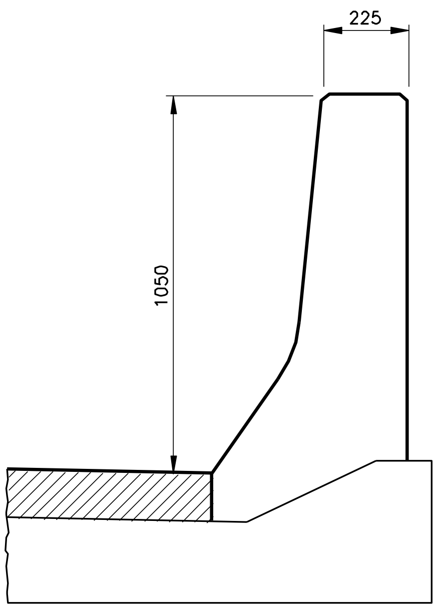

Figure 10.2-8 – Barrier Wall Without Railing (TL-5)

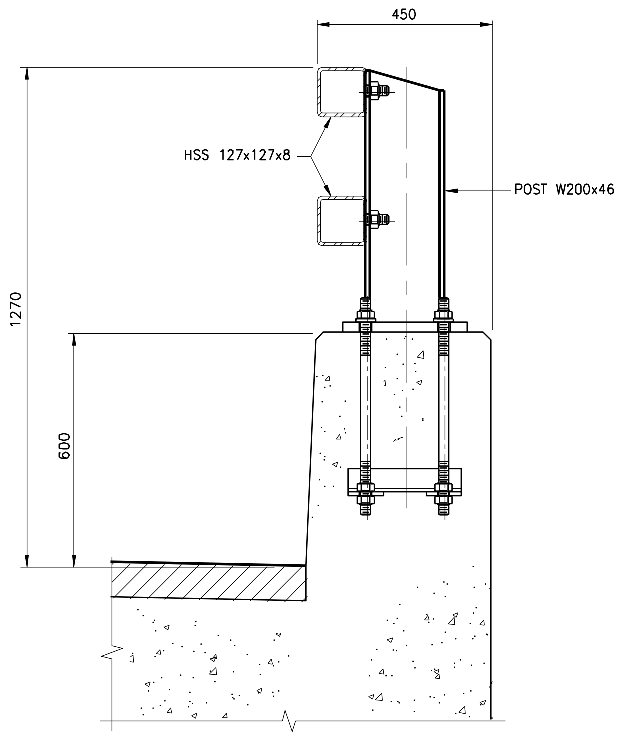

Figure 10.2-9 – Parapet Wall with Two Tube Railing (TL-5)

b) Parapet wall with two tube railing (Structural Standard drawing SS110-96)

The system in Figure 10.2-9 is based on the *PA Bridge Barrier* from Pennsylvania DOT. It was accepted by FHWA as a TL-5 barrier designation. It consists of a concrete parapet with metal railings mounted on top. The parapet facilitates transfer of post loads into the deck and the metal railings portion permit visibility through the railing. It offers a semi-open system alternative to the solid concrete barrier mentioned above.

10.3 Combination Traffic/Bicycle Rail

This railing provides protection to both bicycles and vehicles when bicycles are travelling along the roadway. It shall be located on the outside face of bridges. The system shown in Figure 10.3-1 (Structural Standard Drawings SS 110-82/83/84/85) is developed based on the flush mounted combination traffic/bicycle rail system from Oregon DOT that was accepted by FHWA as a TL-4 barrier. It consists of a concrete parapet with two metal railings mounted on top.

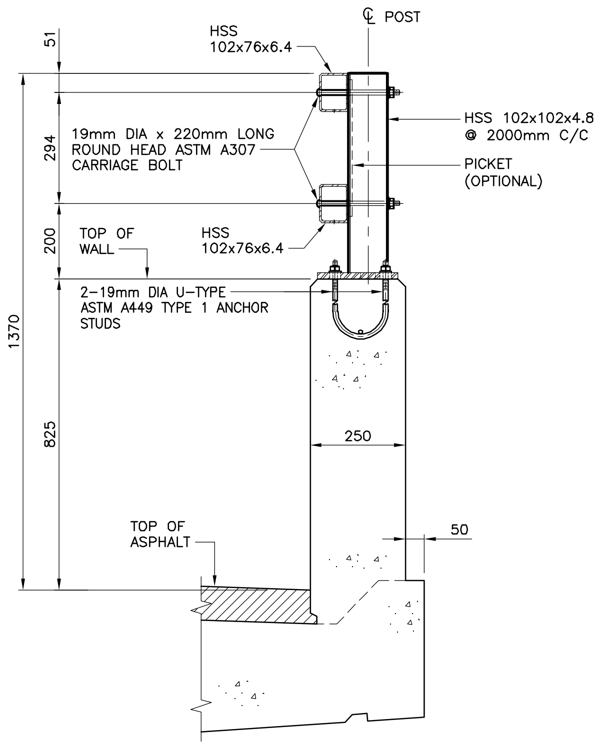

Figure 10.3-1 – Parapet Wall with Two Tube Combination Traffic/Bicycle Rail (TL-4)

Another system shown in Figure 10.3-2 (Structural Standard drawings SS110-34/36) is a TL-4 open railing which is developed based on a Maine DOT system. It consists of a steel post with 4 HSS metal railings mounted on a 225 mm curb. The overall height of both systems is 1370 mm. It meets the CHBDC requirements for bicycle barrier.

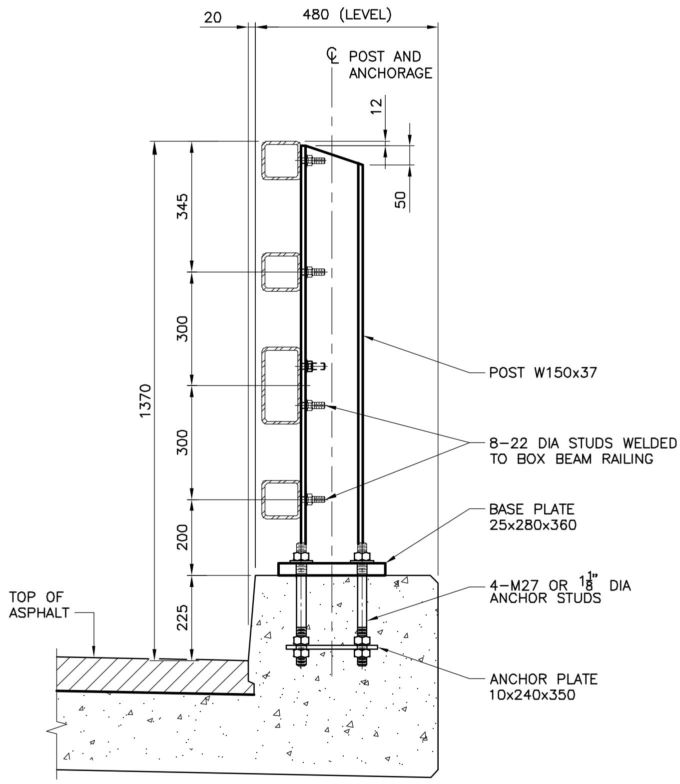

Figure 10.3-2 – Four Tube Combination Traffic/Bicycle Rail (TL-4)

10.4 Separation Barriers for Bridges with Multi use Pathways (MUPs)

A sidewalk is a raise platform for the passage of pedestrians and typically less than 2 m wide, while a Multi-Use Path (MUP) is intended for the joint use by pedestrians and bicyclists and are typically 3 m or in width or greater. When sidewalks or MUPs are used with a curb, the barrier on the outside edge non-traffic side of the sidewalk/MUP shall be a crash tested combination traffic/pedestrian or traffic/bicycle barriers respectively. These barriers should have a handrail at a minimum height of 1050 mm above the top of sidewalk and 1370 mm above the top of MUP. This handrail provides a safety “grip” for pedestrians in case they slip, makes it difficult for people to walk on top and adds some aesthetic value. The following is also noted:

- *The Barrier Wall Without Railing TL-5* railing system is not for use on sidewalks even though it meets the code requirements for pedestrians.

- *The Four Tube Railing on Sidewalk* (Structural Standard Drawings SS110-46/49) and *Parapet Wall with Railing on Sidewalk* (Structural Standard Drawings SS110-57/97/98/105) are the MTO railing systems, currently available, for use on sidewalks.

- Currently no Structural Standard drawing exists for combination barriers (without separation barrier) on MUPs meeting bicycle height requirements.

For high-speed applications, as specified in CHBDC, a separation barrier is required between traffic and pedestrians/bicyclists. In lower speed application with very high pedestrian or multi-use usage, it may also be prudent to provide a separation barrier. This results in the following configuration:

roadway – separation barrier – sidewalk/MUP – pedestrian/bicycle barrier

The separation barrier shall satisfy the crash test and geometric requirements for the traffic, with the barrier height from the pedestrian or bicyclist side satisfying the minimum 600mm requirement of CHBDC. The barrier at the outside of the bridge is protected by the traffic separation barrier and must only be meeting loading and geometric design requirements for pedestrian or bicycle barrier for sidewalks and MUPs respectively. Structural Standard Drawing SS110-110 and SS110-111 show the design and details for the TL-4 and TL-5 separation barrier respectively (see Figure 10.4-1). Both separation barriers satisfy height requirements of CHBDC on the traffic side for the protection of the MUP user. No modification should be made to the separation barrier to increase the height or change the change the geometry. A TL-5 barrier is taller than TL-4, and it could be used where enhanced pedestrian/MUP user comfort is desired.

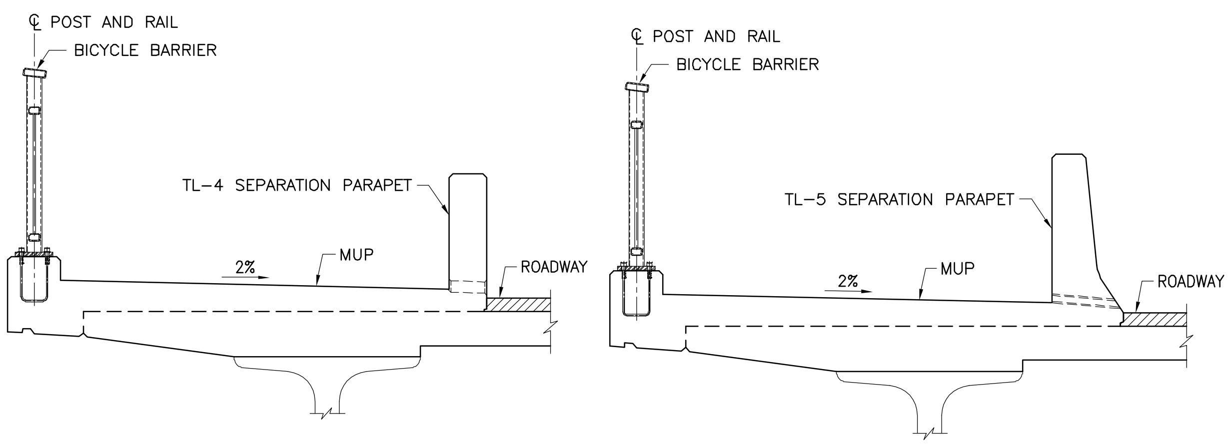

Figure 10.4-1 – Separation Barrier for TL-4 and TL-5

Drawing SS110-22 shows the design of a bicycle barrier on the outside edge of the bridge (see Figure 10.4-2). Due to the high likelihood of the presence of bicycles on an MUP or sidewalk separated from traffic by a separation barrier, only a bicycle height barrier should be used in these circumstances.

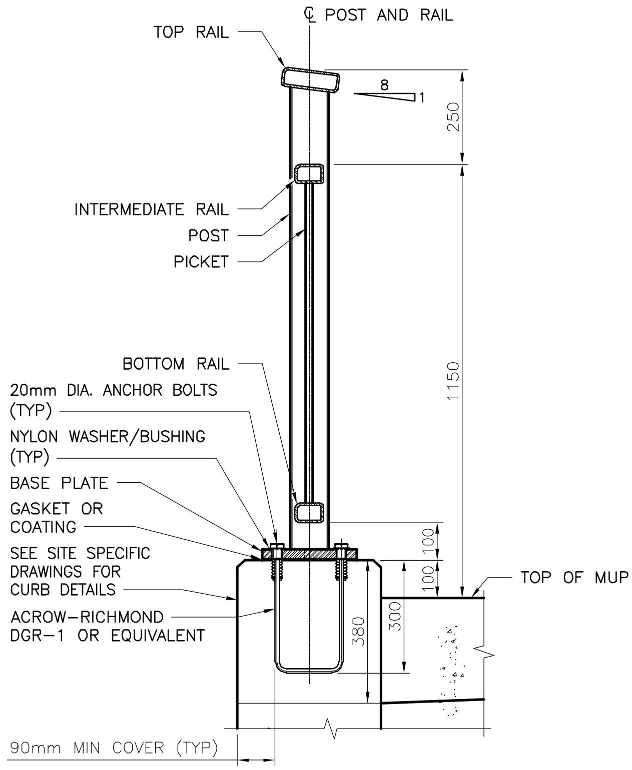

Figure 10.4-2 – Bicycle Barrier

10.5 Barrier Walls Beyond the Bridge Structure

10.5.1 Barrier Walls in Fill Piles

Because of the high cost of supporting barrier walls on piles in back fill, especially to withstand higher test level loads, the designer should investigate all possible alternatives, such as carrying the normal two-sided concrete highway barrier up to the end of the structure or retaining wall, with the standard transition, where required. If approach barriers are required to be supported on piles, the length of piles for barrier walls on fill shall be determined as follows:

a) Piles located between the structure and first pavement expansion joint from the structure: Piles 1 m into existing ground, or minimum overall length 3 m and maximum overall length 6 m;

b) Other piles under barrier walls: Piles 0.5 m into existing ground or minimum overall length 3 m, maximum overall length 5 m.

10.5.2 Barrier Walls on MSE Structure

The CHBDC does not provide design details or guidelines for traffic barriers mounted on mechanically stabilized earth (MSE) System (RSS). Furthermore, the CHBDC has increased the traffic rail impact load requirements for all test levels compared to the earlier OHBDC. Designing for these new loadings, and in particular at the TL-5 level, without knowledge of their distribution through the barrier and transfer to a structural slab and wall system can result in costly or over conservative designs using conventional design methods. This problem has been recognised in the USA, and the Transportation Research Board has carried out a NCHRP project "Design of Roadside Barrier Systems Placed on MSE Retaining Walls" in 2010 (NCHRP 663). Design guidelines for the barrier system were developed based on finite element simulation, bogie vehicle tests and full-scale Test Level TL-3 crash test.

In Ontario the design of the current barriers on MSE walls has been based on the impact loads given in OHBDC. To avoid severe wall damage during vehicle impact, top mounted traffic barriers are connected integrally to continuous footings (normally called anchor slab or moment slab) that are independent of the MSE retaining walls (see Figure 10.5-1). So far, no unsafe performance or damage, in over 30 years of use on Ministry Highways, has been reported. Given that the TL-4 loadings in CHBDC are just marginally higher than the loading given in OHBDC, it was decided to use the CHBDC TL-4 loading, on an interim basis, for the design of both TL-4 and TL-5 traffic barriers on MSE retaining walls. Furthermore, in the USA, AASHTO generally accepts TL-4 loading for the majority of its applications on highways and freeways with a normal mixture of trucks and passenger vehicles and this also seems appropriate in Ontario for the majority of its highways. Therefore, the current Structural Standard Drawings (SS110-64/65/68/69/75) may continue to be used for barriers on MSE retaining walls.

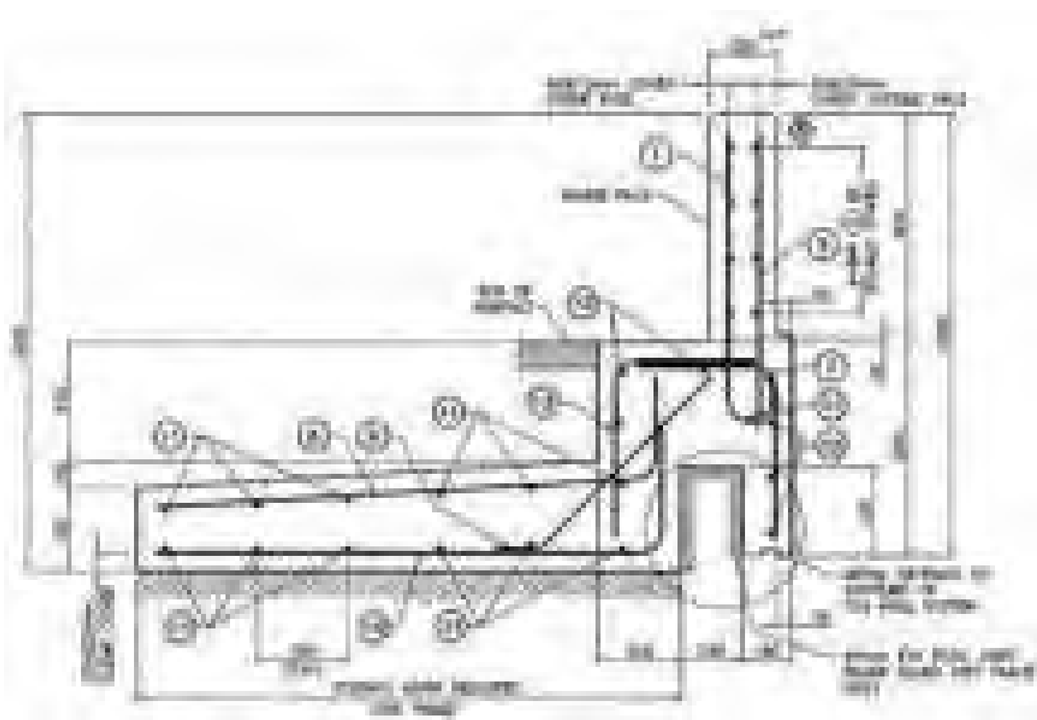

Figure 10.5-1 – Typical Barrier Wall on MSE (RSS) Wall with Moment Slab

The following requirements apply for traffic barriers on MSE retaining walls.

1. For a TL-5 traffic barrier the loading used shall be equivalent to the TL-4 loading as given in Table 3.7 of the CHBDC and it shall be applied as indicated in Figure 12.2 of the CHBDC for the TL-4 barrier.

2. For a TL-2 and TL-4 barrier the appropriate loads given in Table 3.7 of the CHBDC and applied as indicated in Figure 12.2 of the CHBDC shall be used.

3. Top mounted traffic barriers shall be connected integrally to continuous footings (i.e., anchor slab, moment slab, see Figure 10.5-1 and shall be independent of the MSE wall. The loading stipulated in (1) above shall be used for the design of the barrier footing.

4. A traffic barrier integral with the MSE crash tested to NCHRP 350 or MASH for the test level required, is acceptable as an alternative.

5. Consideration shall be given, where practical, to locating the MSE wall away from the traffic barrier. (See Section 5.4).

10.6 Other Barrier Types

10.6.1 Noise Barriers

10.6.1.1 Highway Noise Barriers

The current practice for erecting noise barriers in the Ministry's Right-of-Way is as follows:

1. When noise barriers are required on roadways and are located beyond the clear recovery zone, then the noise barriers are erected simply between steel posts.

2. When noise barriers are required on roadways and are located within the clear recovery zone, then the noise barriers are erected either on top of or behind traffic barrier walls.

Noise Barriers adjacent to roadways are approved by Highway Design Office of MTO and follow DSM 5.50 and roadside safety requirements.

10.6.1.2 Noise Barrier on Bridges

On bridges and retaining walls, the noise barriers are usually located within the clear recovery zone and are generally attached either on top of or to the back face of barrier walls. According to the CHBDC, when the noise barrier is located within the zone of intrusion of the traffic barrier, the noise barrier in combination with the traffic barrier must meet the crash test requirements of NCHRP 350 or MASH. Alternatively, in lieu of crash testing, a noise barrier system meeting the requirements of CHBDC Clauses 12.4.7.1 to 12.4.7.4 may be used. Only noise barriers specified in DSM 5.50.40 shall be used on

MTO bridges. Due to the limited number of crash-tested noise barriers available, MTO allows the use of a MASH tested TL-4 noise barrier, on TL-5 barrier wall, to be used for TL-5 conditions. For all other test levels, the noise barrier shall satisfy the crash-test requirements for the specific test level.

Recommendations

1. Before a noise barrier is considered for use on a bridge, a cost benefit analysis should be carried out. This study should consider, but not be limited to, the following:
   - That the noise reduction is significant enough to warrant the use of a noise barrier;
   - That there are a reasonable number of residents that are expected to benefit from the reduction in noise level as determined by MTO Environmental Guide for Noise Oct 2006;
   - Stopping the noise wall at the bridge and turning it obliquely (flanking) away from the highway towards the residential area. In effect avoiding noise barrier walls being erected on the bridge itself;
   - The effect of increased loading to the structure (from vertical, wind, torsion loads etc.). These effects may create the need for additional girders and cross bracing, and a refined structural analysis for verification;
   - Access for inspection of the bridge and the noise barrier components;
   - Cost of an approved traffic/noise barrier wall system and additional costs to the structure;
   - Possible snow accumulation; and,
   - Aesthetics.
2. Where it has been assessed that it will be beneficial to install a noise barrier wall on bridges and retaining walls or portions thereof, the design shall satisfy CHBDC, including requirements to contain or otherwise prevent debris that could endanger public safety, from falling.

10.6.1.3 Noise Barrier Design

Noise barriers shall be designed in accordance with the CHBDC as light slender structures not unusually sensitive to wind action, and CSA-Z107.9, Standard for Certification of Noise Barriers. Wind loads and ice accretion loads on all the noise wall elements shall be considered in the design as per the CHBDC. Reference wind pressures for a 25-year return period shall be used.

In the calculation of section properties and strength for cold formed steel members, for which the provisions of the CHBDC are not applicable, the requirements of CSA-S136 for ultimate limit state design shall apply.

In evaluating or designing a structure on which a noise barrier is to be mounted, the above criteria shall apply, except that the local reference wind pressure shall always be used and a gust factor of 2.0 is sufficient for the relatively rigid structures (e.g., retaining walls), to which noise barriers are generally attached.

10.6.2 Inspector Guards

Some areas exist where bridge inspectors and other maintenance staff are expected to approach a location where a fall hazard exists. This hazard can be mitigated by wearing appropriate PPE, or more preferably, by the use a railing. Site specific considerations shall be reviewed to assess whether there is a need for an Inspector Guard to protect against this, with the final decision made by the Head of the Structural Section. Installation of guards should be planned as part of the major capital program when work is planned on the structure. In some unique situations, guards may be provided as standalone installations.

If guards were found to be needed in the review of fall hazard risk assessment, the following policies shall apply:

1. Retaining walls within MTO's right-of-way in areas with no pedestrian access where the height of the wall exceeds 2.4 m shall be equipped with a guard or guardrail on top of the wall as shown in Figure 10.6-1. For sloped walls where the height tapers from zero to the full height of the embankment, the guard shall be placed starting from a height of 1.0 m as shown in Figure 10.6-1. Guards shall be designed to meet the requirements for guardrails of Industrial Requirements, *R.R.O. 1990, Reg. 851, s.14*. Those requirements include a top and middle rail, but do not require a kick plate. According to *R.R.O. 1990, Reg. 851, section 14. (2)* the loading and structural design shall be in accordance with *O. Reg. 332/12: Building Code*.

2. At bridges and culverts, where the fall hazard from the obvert exceeds 2.4 m, the headwall or top slab of the culvert shall be equipped with a guard on top of the wall conforming to Section 1 of this memo. Figure 10.6-2 illustrates the schematic locations of inspector guards in two typical culvert scenarios.

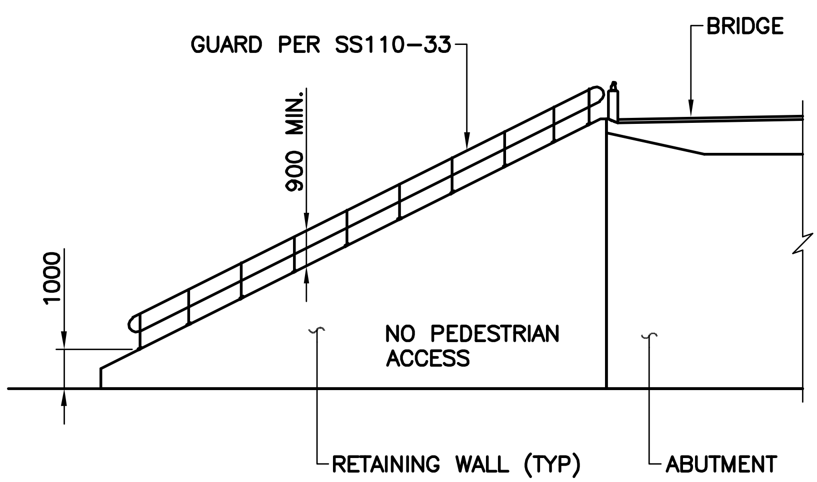

Figure 10.6-1 – Guard Requirements Within MTO's Right-of-Way Without Pedestrian Access

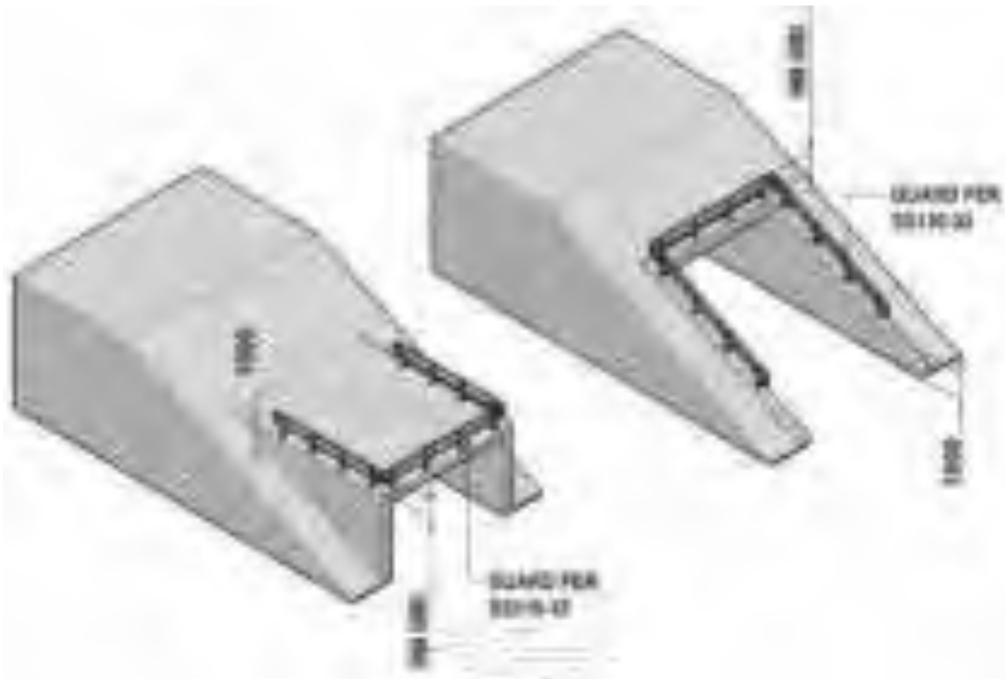

Figure 10.6-2 – Guard Requirements at Culverts Where Fall Hazard Exceeds 2.4m

The guards shall be detailed as follows:

1. The guards shall be detailed on the structural drawings within a Contract and shall include:
   a. an elevation or plan to denote the limits of the guards;
   b. details at expansion joints in the railings;
   c. mounting details, and,
   d. details of anchorage at base plates.

2. The guard shall be mounted on the wall, within 300 mm of the exterior face of the wall.

3. Posts shall be installed vertically.

4. Posts shall be mounted to the retaining wall with base plates and anchors designed to resist the loads imposed on the guard. Anchors shall be embedded into the retaining wall or anchored with epoxy. Given their history of problematic maintenance, posts shall not be embedded directly into the concrete.

5. Guards shall be designed to avoid the accumulation of standing water at posts and connections, with positive drainage to the fill side of the wall where practical.

6. Guards shall be galvanized, and pipe shall have a wall thickness no less than 3.5 mm.

7. Pickets, when required, shall be solid bar, round, or plate.

8. Connections shall be designed by the structural engineer. The structural engineer shall ensure that the underlying surface and component to which the guard is mounted, is adequate to resist the force applied to it from the post.

9. Drawings of guards shall be sealed per the requirements of the Structural Manual Division 1.

Structural Standard Drawings SS110-33, Inspector Guard Details, is designed to meet the design requirements above. Other Guards, meeting the requirements of the Industrial Regulations or Building Code, as needed, may be used for longer applications or other unique cases, where approved by the Head of the Structural Section. For Corrugated Metal Culverts the details shown on SS110-33 will require modifications to suit the placement and attachment to corrugated metal.

For locations that are more accessible to pedestrians, and with fall hazards as low as 600mm, a taller railing and/or one satisfying *Ontario Building Code, O. Reg. 332/12, s.3.3.1.17*, with loading as specified in *s.4.1.5.14* may be required.

10.6.3 Unreinforced Concrete Median Barriers and Shoulder Barriers

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

Barrier walls are not permitted to be slip-formed on structures because the horizontal reinforcement may produce voids beneath themselves during the slip-forming process. Moisture could collect in these voids and cause problems when the moisture would freeze. Concrete median barriers on structures must be cast-in-place and shall not be placed directly on waterproofing membranes and must be provided with lateral restraint to prevent movement. If the membrane is stopped at the barrier, it must be turned up the vertical face at the barrier base and into a groove. With this arrangement, the barrier must be dowelled to the deck so that small relative movement does not rupture the membrane. Median barrier details shown on Structural Standard Drawings SS 110-62/63 meet these requirements and that of the CHBDC.

10.7 Miscellaneous Details of Barriers

10.7.1 Architectural Finish on Barrier Walls

Bridge barriers are usually located where they are frequently seen by the public. An architectural finish on the wall surface provides a pleasant aesthetic while maintaining the proper structural function of the barrier.

A TL-5 concrete barrier wall with an architectural finish added to the outside face is shown in SS110-70/93/99. The surface finish can be selected from the four standard patterns given in Structural Standard Drawings SS110-71/72/73/74.

10.7.2 Detailing of Standard Steel Tube Railing

The Structural Standard Drawings permit detailing using either of two methods:

a) For structures on which the exact length of parapet or barrier railing can be readily established, e.g., individual structures on tangent, the spacing of the posts shall be shown on the drawings.

b) For other structures, e.g., structures on horizontal curves; structures or retaining walls in groups connected by railing, on which the exact rail length may not be readily established, the post spacing shall be determined by the contractor and should not be shown on the plans. A note to this effect should be given on the drawings.

Unless otherwise specified in the standards, the post spacing with single tube hand railings shall not exceed 2.5 m or 3.5 m for aluminum and steel rails respectively. For other HSS steel railings, post spacing shall not exceed values specified on the respective standard drawings. The rail shall be supplied in length to be attached to a minimum of three rail posts and the rail splice shall be located within 600 mm from the post. The rail should be ordered in standard lengths satisfying the criteria mentioned above plus pieces for each side of the bridge of such a length as to give the exact rail length required.

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

10.7.3 Reinforcement Detail of Concrete Barrier Walls on Structures

The following requirements apply:

a) Longitudinal reinforcement in concrete barrier walls must be continuous through construction joints;

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

10.7.4 Ducts in Concrete Barrier Walls

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

a) A maximum of two ducts are permitted in a barrier;

b) For all new bridges when electrical duct is required, the duct diameter shall follow electrical OPSD standards. Parapet walls shall be 300 mm wide to permit use of 75mm diameter ducts;

c) Ducts shall be no larger than 50 mm in parapet walls of 250 mm width. For existing bridges where a 300 mm width of parapet wall is not feasible, use of 50 mm diameter ducts in 250 mm wide parapet walls can be considered, with consultation with the project's electrical engineers to determine if the 50 mm diameter duct is adequate for the project-specific needs;

d) Ducts shall be positioned to be tied to the reinforcement in the back face of the parapet or barrier, with the lower duct at 200 mm above the asphalt. Duct positioning shall allow concrete to flow around tension reinforcement and shall not impede the anchorage of the tension reinforcement;

d) Junction boxes should not be located within 1 m from expansion joints, where the vertical steel is at 110 mm spacing;

e) The deflection cavity for the duct (located at bridge expansion joints) is 143 mm dia. x 150 mm deep. This would interfere with reinforcement if the cover were at the maximum tolerance of 70 ± 20 mm (90 mm). A note should be added stating:

   "ADJUST END BAR SPACING TO ACCOMMODATE DUCT DEFLECTION CAVITY FORMER.";

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

10.8 Barrier Design Aid

Dead loads of standard permanent barriers and temporary barriers are provided in Division 3 in Design Aid 10-1 and Design Aid 10-2 respectively. For permanent barriers the dead load is tabulated for typical barrier sections to be used for the design of superstructure elements.

The temporary Barrier design Aid 10-2 contains the MTO's standard Precast temporary barriers along with proprietary steel barriers. Information of these proprietary barriers can be obtained from MTO's Roadside Design Manual.

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SECTION 11 - RIGID FRAMES

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11 RIGID FRAMES

11.1 General

Rigid frame structures are ideal for short to medium span bridges. The high degree of structural indeterminacy allows redistribution of forces between the deck and the substructure contributing to resilience in the face of extreme events. The jointless nature of a rigid frame structure offers a sustainable structure with low maintenance efforts during the service life of the bridge.

The following sections provide discussion of key design parameters of rigid frame structures.

11.1.1 Backfill

Unless the abutment walls have been designed to withstand earth pressure without the deck in place, backfill should not be placed behind them until the deck is constructed. Therefore, in most cases a note stating this should be shown on the contract drawings. A standard note to be shown on the general arrangement drawing is given in Section 2.6.7.

The footings of rigid frame bridges exert a horizontal thrust upon the foundation, as well as a vertical thrust. If the footings are free to rotate the frame is said to have hinged supports. When the rotation of the footing is prevented, the supports are said to be fixed. The supports are actually rarely hinged or fixed, but the foundation conditions lie somewhere in the range between these two extremes: that is, the supports are restrained.

If the summation of the horizontal forces at the footing is not equal to zero (i.e., there is insufficient resistance against sliding) the remaining unbalanced horizontal force will cause a moment at the top of the rigid frame that should be accounted for in the design.

Normally, rigid frame structures are to be designed assuming simultaneous earth pressure on both sides. It is essential in such cases that the general arrangement should bear the standard note (given in Section 2.6.7) 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 note above, must also appear on the preliminary version of the general arrangement.

When essential due to site access problems, a structure may be designed for earth pressure on one side. This must be stated as a requirement in the Structural Design Report. When a structure is designed for earth pressure on one side, the general arrangement shall bear the following note under "Construction Notes" instead of the standard note:

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

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

11.1.2 Earth Pressure

Earth pressure distributions shall correspond to the anticipated movement at nominal temperature loads. Nevertheless, rigid frames with spans up to 30 m, and other structures where the deflection of the abutment is prevented by the propping action of the deck, may be designed for the at-rest earth pressure.

11.1.3 Drainage

The provisions of Section 5.2.1.2. (b) to (e) inclusive apply to the legs of rigid frame structures.

SECTION 12 - REINFORCING

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12 REINFORCING

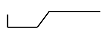
12.1 General

Reinforcing steel in structures shall be CSA G30.18 Grade 500W. Premium reinforcing steel shall be stainless steel according to OPSS 1440 or GFRP according to OPSS 1640.

For bridges with decks that are waterproofed, carbon reinforcing bars shall be used in the deck except in the locations identified in Section 12.2, where Premium Reinforcing materials (see Section 2.4.8) shall be used.

12.1.1 Reinforcing Steel Bar - Identification

Reinforcing bars shall be designated as illustrated in the following example pertaining to carbon steel reinforcing:

15M @ 300 

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

Some examples of bar designation for carbon steel reinforcing, illustrating the above, are as follows:

15M

15M @ 300

20 - 15M @ 300

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

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

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

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

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

12.1.2 Premium Reinforcing - Identification

Prefix identifiers, as specified below, shall be used with Premium Reinforcing bar designations. A prefix identifier shall be regarded as an inseparable part of the bar identification.

Stainless Reinforcing Steel:

Stainless steel reinforcing shall be prefixed by the letter S. For example, a metric, 15 mm diameter, stainless steel reinforcing bar would be denoted as S15M. The General Notes on the drawings shall indicate the Type of Stainless Steel (316LN, 2205 Duplex or 2304 Duplex) to be used.

GFRP Reinforcing:

Only Grade III GFRP reinforcement shall be used in the design. The bars shall be identified by "G" for GFRP along with the nominal diameter (for example, a metric 15 mm nominal diameter, GFRP reinforcing bar would be denoted as G15). The suffix M is omitted from GFRP bar identifiers as the bar sizes do not conform to those of Canadian, metric steel reinforcement. Bar diameter designators and nominal bar areas are defined in CSA S807. Note that the nominal area values are not based on the designated diameter. Nominal area values from CSA S807 shall be used in design.

GFRP reinforcement (bar sizes and areas as per CSA S807) is available in nominal metric bars which are equivalent in area to the imperial sizes (e.g., size 15 has area of 199 mm^2, size 25 has area of 509 mm^2). For MTO projects, nominal diameters of 13, 15, 20, and 25 shall be used. The 10 mm nominal diameter is susceptible to damage during construction, and 30 mm diameter is difficult for performing QC and QA testing, and these sizes shall only be used with approval of the Structural Section.

12.2 Premium Reinforcing – Application

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

Premium Reinforcing shall be used in exposure classes D1, D2, D3, SST5, and S2 (all exposure classes vulnerable to corrosion due to deicing salts or salt water) when bars that are within 100 mm (e.g., cover specified as 125 ± 25 mm) of the surface, unless the surface is protected by waterproofing. Figure 12.2-1 through Figure 12.2-4 illustrate the exposure classifications at typical details encountered around and below expansion joints and barriers. Bridge decks that are waterproofed and paved shall have exposure class D1, except for bridge decks carrying freeways and NHS Core and Feeder routes which shall have exposure class D3 and shall ignore the benefit of waterproofing. Even for bridge decks that are waterproofed, those on busy highways have additional wear of the waterproofing due to heavy traffic while simultaneously tending to have rehabilitation and waterproofing replacement delayed due to the desire to avoid traffic disruptions. For these bridges, there are benefits to using Premium Reinforcement in the deck.

When AADT on the bridge is between 400 and 2,000, concrete surfaces within the splash and spray zone are classified as D2 and premium reinforcement is only required in the barrier walls, curbs and sidewalks, with carbon steel bars elsewhere in the superstructure.

When AADT < 400, Premium Reinforcement is not required, concrete surfaces at the roadway level shall be classified as A5 and carbon steel bars may be used throughout.

Buried structures where the surface in contact with soil is waterproofed shall have exposure class B1.

Where premium reinforcing is required, GFRP or stainless steel may be designed subject to the following requirements:

1. In slab-on-girder bridges and in topping slabs on freeways and NHS Core and Feeder routes, GFRP shall be used unless approval to use stainless steel is obtained from the Head of Structural Section.

2. Stainless steel shall be used in the following locations due to the need to remove concrete by chipping for future replacement of replaceable components.
    a. Top surfaces of expansion joint end dams including those at sleeper slabs supporting the end of the approach slabs.
    b. Stirrups and perimeter bars within precast components (i.e., NU, CPCI, box) within 1.5 m of expansion joints.
    c. Bearing seats and pedestals exposed to roadway drainage or possible dripping.
    d. Within 1.0 m of deck drains

3. GFRP shall not be used in the following locations due to concern with constructability, congestion, or detailing limitations within the CHBDC:
    a. Pier caps, bearing seats and the surfaces of pier shafts and columns exposed to roadway drainage or possible dripping below joints (sealed or unsealed), deck drain outlets, and overhanging features from which dripping may occur.
    b. Side and end surfaces of pier shafts and pier columns, excluding pier caps, within the splash zone of an existing or future roadway. All reinforcement in the shaft or column shall be included, including dowels from the footing.

4. For surfaces of abutments, wingwalls, retaining and MSE (RSS) walls that are exposed to roadway drainage or possible dripping, the designer shall clearly delineate the areas which require premium reinforcing but shall not specify the

type of premium reinforcing on contract drawings. Premium reinforcing in MSE walls shall be supplied according to approved DSM product drawings.

Pier caps below continuous decks and exposed from the adjacent highway, are on the outer periphery of the splash zone and historically have performed well with carbon steel bars. They are not within the splash and spray envelope.

In determining the limits of deicing salt runoff below joints (sealed or unsealed), deck drain outlets, and overhanging features from which dripping may occur, runoff shall be considered to spread downwards within a vertical cone having an apex angle of 45 degrees or likely to receive run-off from a surface within such a cone.

Premium Reinforcing is not required for replaceable components nor for highway accessory structures that are covered by specific standards and that have a design service life less than 50 years, such as footings of sign support structures and footings of high mast pole structures.

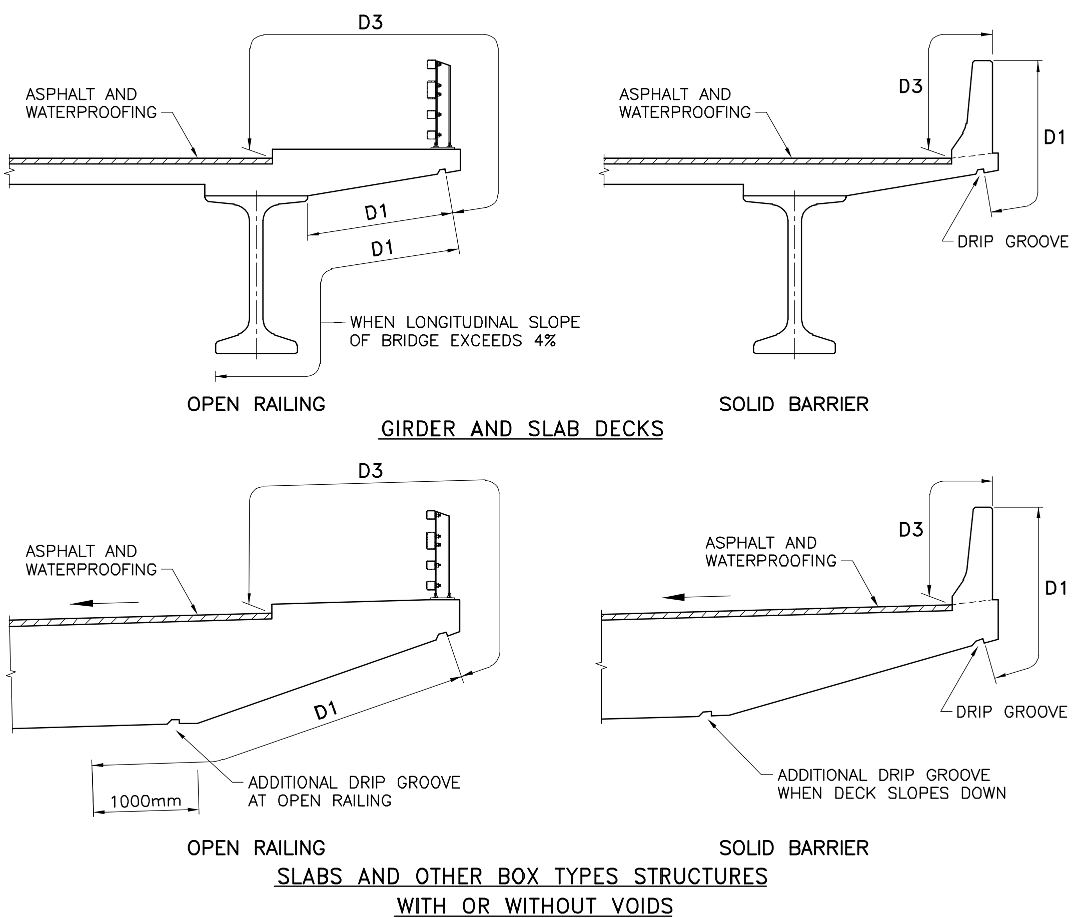


Figure 12.2-1 – Exposure Classes at Barriers

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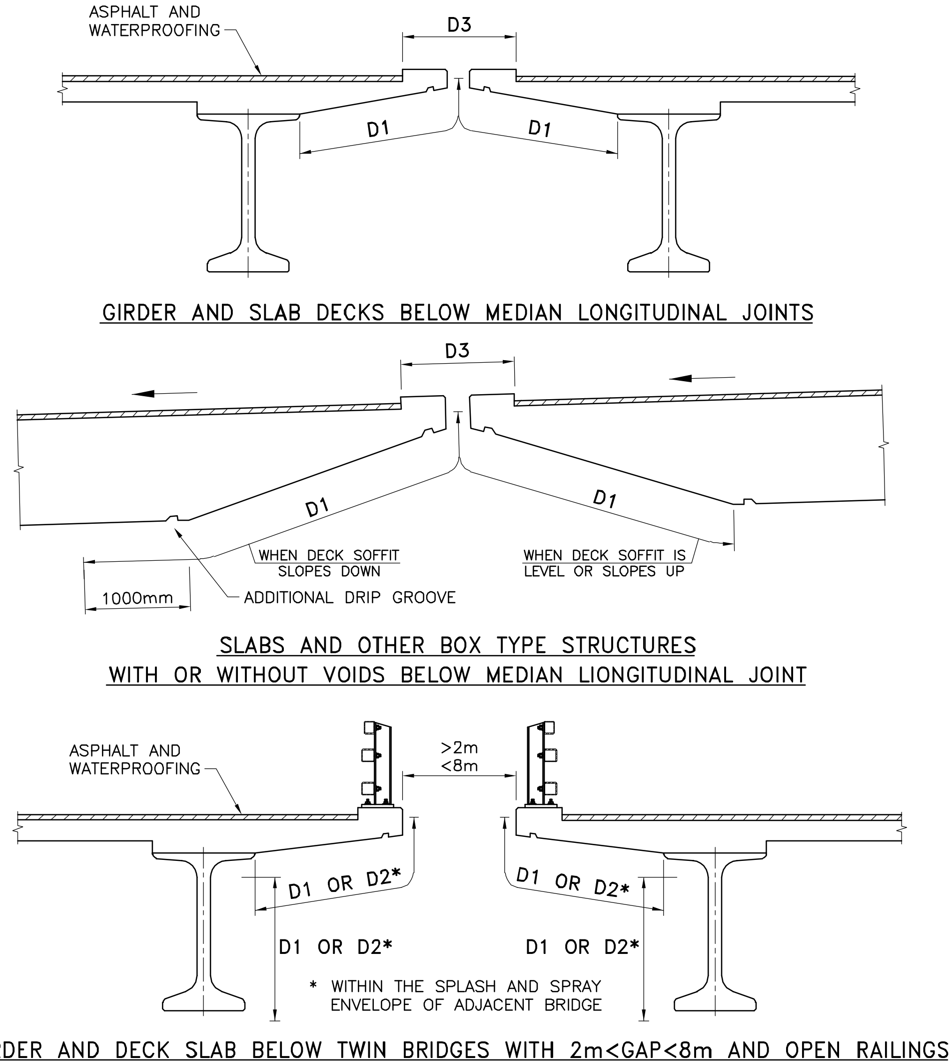

Figure 12.2-2 – Exposure Classes at Medians of Parallel Bridges

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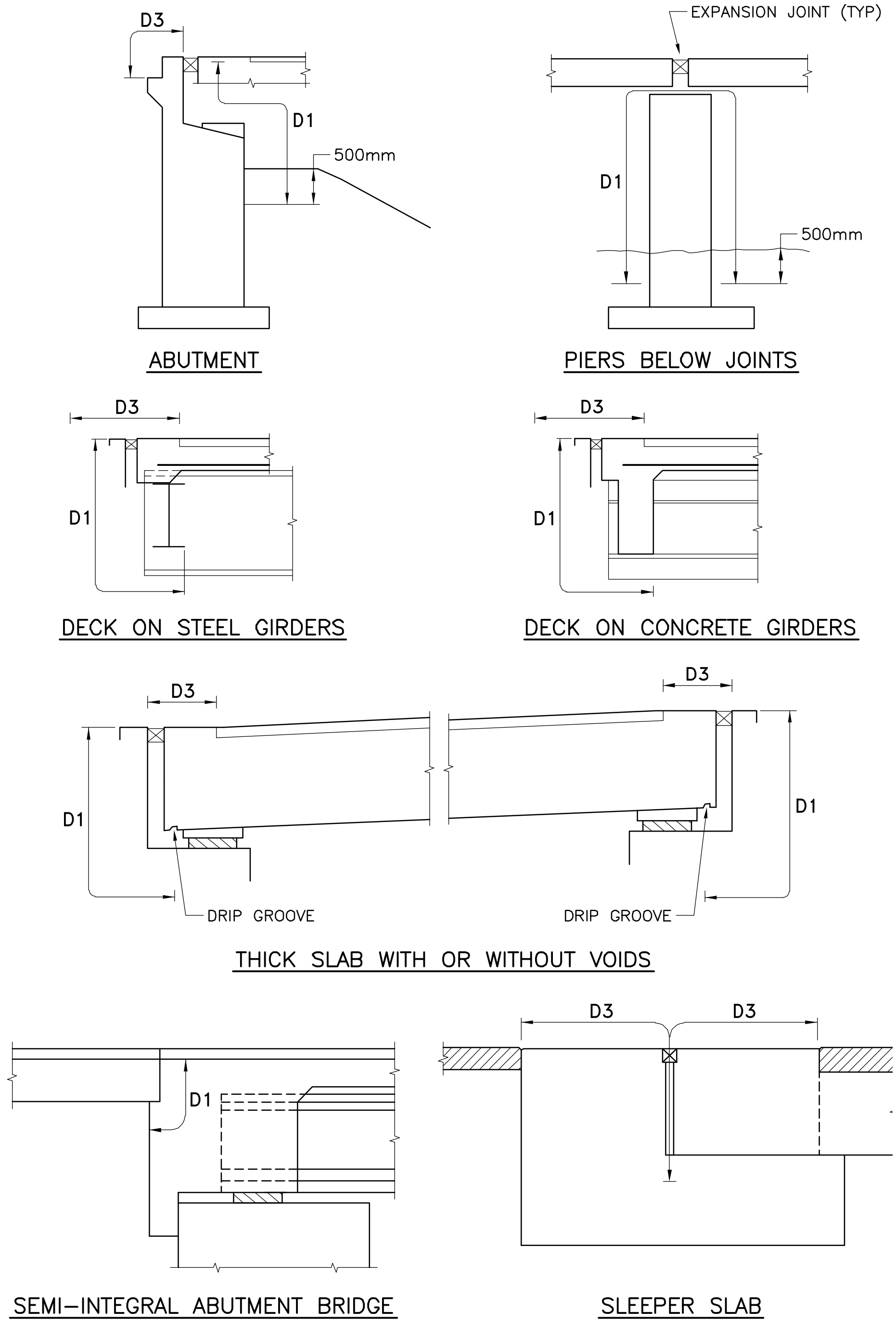

Figure 12.2-3 – Exposure Classes below Expansion Joints

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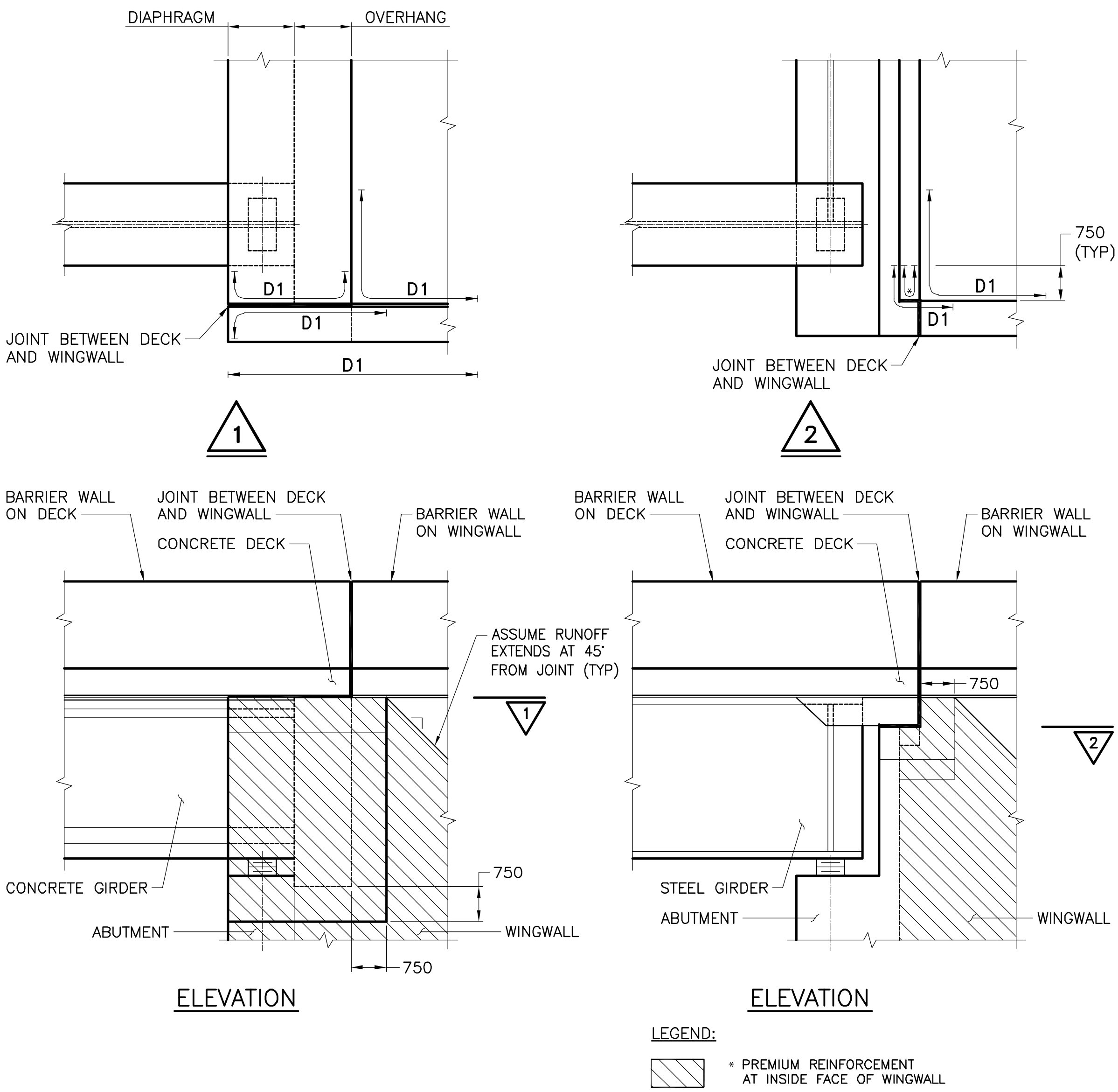


Figure 12.2-4 – Exposure Classes at Semi-Integral Abutments

12.2.1 Premium Reinforcing in Bridge Rehabilitation

Where the remaining service life of the bridge is less than 35 years, conventional concrete and carbon steel bars shall be used.

Where the remaining service life of the bridge is between 35 and 50 years, Premium Reinforcement is only required in surfaces in exposure classes D2 and D3 already being replaced as part of rehabilitation.

Where the remaining service life of the bridge is longer than 50 years, the extent of Premium Reinforcing, for components which require replacement, shall be the same as for a new bridge.

Where bridges are being widened, the reinforcement requirements shall be as specified by the Structural Section, with consideration to the remaining life of the widened portion, and whether it will survive beyond the remaining life of the existing components of the bridge.

12.3 Design

Structures shall be designed with a combination of carbon steel reinforcement and premium reinforcement, as required by Section 12.2. No other types of reinforcement shall be used unless approved by the Structures Office.

In proportioning the size and spacing of bars in all components, designs with carbon reinforcing steel shall assume a yield strength of 500 MPa and designs with stainless steel shall assume a yield strength of 520 MPa.

Steel reinforcement and GFRP shall not be used in different orthogonal directions within the same mat of reinforcement in a given area. At locations where a mat transitions from steel to GFRP reinforcement, the designer shall consider the implications for the structural behaviour (since the sectional behaviour may change from being tension controlled to compression controlled).

Examples of how to approach design with GFRP reinforcing bar are included in the following publications:

- ISIS Design Manual, No. 3, Reinforcing Concrete Structures with Fibre Reinforced Polymers, September 2007;

- ISIS Design Manual, No. 4, FRP Rehabilitation of Reinforced Concrete Structures, March 2008;

- Bank, Lawrence C. <u>Composites for Construction: Structural Design with FRP Materials.</u> 1st Ed. Hoboken: John Wiley & Sons, Inc., 2006;

- Nanni, Antonio, De Luca, Antonio, Jawaheri Zadeh, Hany. <u>Reinforced Concrete with FRP Bars: Mechanics and Design.</u> 1st Ed. London: CRC Press, 2014.

These examples shall not be considered authoritative for MTO work. MTO published documents and the CHBDC shall govern.

12.3.1 Cracking

In determining the crack width of concrete members reinforced with GFRP per Clause 16.8.2.3, the bond factor, k_b, shall be taken as 0.8.

12.3.2 Composite Construction with GFRP Reinforcement

CHBDC Sections 8.19 and 10.11 provide guidance to design a composite girder section consisting concrete deck supported and composite with precast concrete and steel girders respectfully. These code provisions cover steel reinforcement in the deck. However, MTO has built bridges with GFRP reinforcement in the concrete deck in various configuration:

- GFRP reinforcement in both top and bottom mats
- GFRP in the top mat and carbon steel in the bottom mat

In the positive moment region of the bridge where the deck is in compression (e.g., mid-span), the code provisions can be applied directly, regardless of the configuration of deck reinforcement mentioned above.

In the negative moment region of the bridge over the pier or abutment where the deck is in tension, strain compatibility analysis shall be used to determine the stress in the GFRP.

Composite Concrete Girders in Negative Moment Regions

The current code provisions can be applied in the design of concrete composite girders when GFRP is used in both mats of deck reinforcement.

In the case of using GFRP in the top mat and carbon steel in the bottom mat of deck reinforcement, the stress in the GFRP can be conservatively calculated from a strain profile where the bottom layer steel reinforcement is at the factored yield strain.

Composite Steel Girders in Negative Moment Regions

Since GFRP has lower stiffness than steel, the stress in the GFRP bars is less than their ultimate strength and needs to be calculated based on their distance from the neutral axis of the composite section.

When the SLS stress in the deck exceeds $\phi_c f_{cr}$:

a) For GFRP reinforcement used in both mats of the deck, the total longitudinal reinforcement area in a non-prestressed deck slab, shall not be less than $1.5 f_{cr} A_c / (\phi_{frp} f_{FRPu})$. At least half of the reinforcement area shall be in the top mat.

b) For GFRP reinforcement in the top mat and reinforcing steel in the bottom mat, the total longitudinal reinforcement area shall be such that the following equation is satisfied. $(\phi_{frp} f_{FRPu} A_{FRP}) + (\phi_s f_{yeff} A_S) > 1.5 f_{cr} A_c$. At least a quarter of the total area of reinforcement shall be provided in the top mat. The total area of longitudinal reinforcement is taken as $A_{FRP} + A_s$, without adjustment for the modulus of elasticity.

In Class 1 and Class 2 composite sections, the depth of plastic stress blocks shall be determined corresponding to a strain profile which limits the strain in any GFRP layer to $\phi_{frp} f_{FRPu}$. Note: You cannot assume that the plastic section is fully mobilized. There may be portions of unyielded steel in the web at the strain profile where the strain in the GFRP reaches a factored rupture strain.

In Class 3 composite sections, the stress in any GFRP layer shall be determined based on a strain profile where either reinforcing steel or structural steel are at yield in tension. For Class 3 composite sections, SLS design provisions for GFRP will be automatically satisfied.

Crack widths, in the direction of consideration, shall be controlled if the tensile stress in the longitudinal GFRP, f_{FRP}, satisfies the following:

$$f_{FRP} \leq \sqrt{\frac{3 f_{cr} E_s}{k_b d_b}}$$

Longitudinal GFRP in the top mat of the deck slab shall be no less than G15@200.

When the deck is reinforced with carbon steel reinforcement in the bottom mat, overall economy in the composite girder design can be achieved by increasing the longitudinal steel reinforcement in the bottom mat and providing nominal GFRP in the top mat.

On the other hand, when the deck is reinforced with GFRP in both top and bottom mats, an effective design is achieved by increasing the top flange of the steel girder and minimizing the GFRP reinforcement.

12.4 Reinforcing Geometry

12.4.1 Reinforcing – Dimensional and Shape Limitations

12.4.1.1 Steel Reinforcing

The normal mill length for reinforcing steel bar stock is 12 m for size 10M, and 18 m for all other bar sizes. These lengths should never be exceeded for straight bars.

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

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

12.4.1.2 GFRP Reinforcing

General Limitations and Guidance

Length and shipment limitations for straight GFRP reinforcing bars are similar to those of steel reinforcing bars.

Mechanical splices of GFRP reinforcing bar are not available, only lap splices shall be permitted.

Details for bend dimensions may be found in Section 12.4.3

General Limitations and Guidance on GFRP Bent Bars

The manufacturing process for forming and curing bent GFRP bars limits the types of bent bar shapes that can be manufactured. Not all shapes common to steel reinforcement are possible in GFRP reinforcement.

Broadly, GFRP bent shapes are cut from winding a line around a closed polygon that is without re-entrant corners (e.g. a shape with interior bend angles all rotating in the same direction). Typically, the manufacturing process winds a wet bundle of fibres around a form under tension to create the shape by rotating the form. This process does not allow the wet bundle to be threaded to the inside of the form – to the inside of the shape – nor does the tension in the wet bundle allow forming of interior corners after the winding by pressing an exterior form into the side. See Figure 12.4-1.

Due to differences in the manufacturing process between DSM listed suppliers, the longest leg length of bent bars or largest plan area of bent bar shape varies between DSM listed suppliers. The following detailing guidance should be respected in order to develop a GFRP design that all DSM suppliers are able to supply:

- For components detailed with bent bars, the bars should be the smallest practical size (G13, G15 or G20).
- For 90° bends, the length of the sum of each leg should be less than 3 m.
- Rectangular stirrups, ties, or hoops should be designed with any assembly of shapes satisfying the CHBDC, each of them designed to fit within a 2.5 m x 2.5 m rectangle.
- Spirals are available for columns up to 1.2 m in diameter.

These limitations should be discussed with manufacturers if they affect the design. Consult DSM listed supplier documentation for additional information as it may be more up-to-date.

Specific Limitations – Rectangular Ties

The following lists identify the types of rectangular tie shapes that can be manufactured and that cannot be manufactured with reference to the shape number as per Table 4 of the Reinforcing Steel Institute of Canada’s Manual of Standard Practice (2020). In general, rectangular tie shapes with 135° hook terminations that are common for steel ties are not possible with FRP ties. See Figure 12.4-1.

Table 12.4-1 - Specific Limitations on Rectangular Ties

Rectangular Tie Shapes that Can be Manufactured	Rectangular Tie Shapes that Cannot be Manufactured
17 S6, S10, S11 T2, T4, T7	S1 to S5, S7 to S9 T1, T6, T8

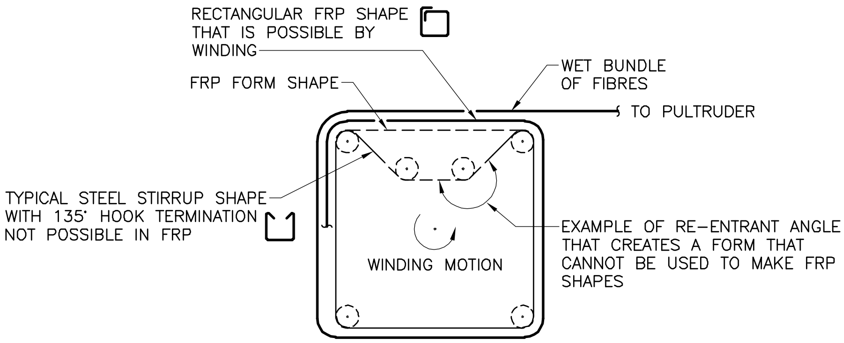


Figure 12.4-1– Example of Winding Process for Rectangular Tie

For most compressive bridge elements, the transverse tie will be made up of several tie pieces cut from rectangular windings that overlap to create the closed perimetrical tie as described in the CHBDC.

Specific Limitations – Crossties

The following lists identify the types of crosstie shapes that can be manufactured and cannot be manufactured with reference to the shape number as per Table 4 of the Reinforcing Steel Institute of Canada's Manual of Standard Practice (2020). In general, it is possible to manufacture crossties with inwardly pointing terminations because these shapes are cut from simple windings. See Figure 12.4-2.

Table 12.4-2 – Specific Limitations on Crossties

Crosstie Shapes that Can be Manufactured	Crosstie Tie Shapes that Cannot be Manufactured
1, 2 T9	3 to 8, 18 to 24 T5

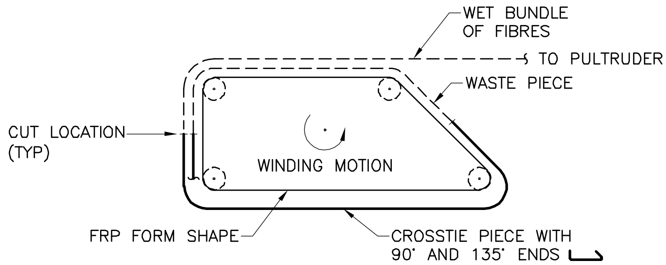


Figure 12.4-2 - Example of Winding Process for Crosstie

Specific Limitations – Circular Ties

In general, it is more economical to use a spiral whenever discrete circular ties are considered.

Circular tie shapes can be divided into two categories: i) complete circular ties conforming to CHBDC Figure 16.9.(c), and ii) partial circular D-shaped ties conforming to Figure 16.9 (a).

Complete circular tie shapes are simply rings cut from FRP spirals. However, the MTO's CHBDC Exceptions do not permit the use of this detail.

Partial circular D-shaped ties must be made in custom forms. These shapes are still made by winding a line around a closed polygon without re-entrant corners. As such, it will not be possible to strictly meet the requirement of having 90° hook terminations into the core – the terminations will not point to the centre of the circle. However, for most circular ties made up of partial circular D-shaped pieces, the achievable termination angles should reasonably approximate the requirement. See Figure 12.4-3.

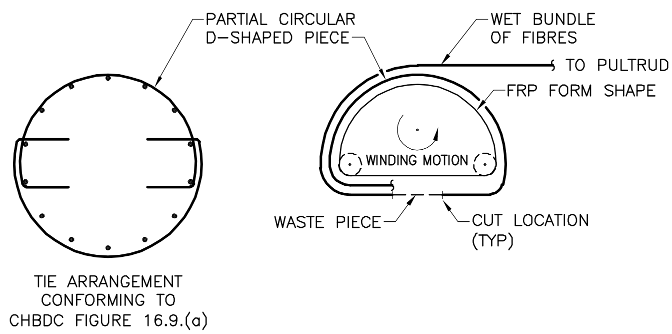


Figure 12.4-3 - Example of Winding Process for Partial Circular D-Shaped Pieces

The following lists identify the types of circular tie shapes that can be manufactured and that cannot be manufactured with reference to the shape number as per Table 4 of the Reinforcing Steel Institute of Canada's Manual of Standard Practice (2020).

Table 12.4-3 – Specific Limitations on Circular Ties

Circular Tie Shapes that Can be Manufactured	Circular Tie Shapes that Cannot be Manufactured
10 S10, S11 T3 See note below.	T3A

Note: The partial circular D-shaped piece shown in Figure 12.4-3 and CHBDC Figure 19.6.(a) is not described by one of the RSIC Table 4 shapes.

The circular tie in CHBDC Figure 19.6.(b) is hypothetical and cannot be manufactured presently because this shape represents a winding of greater than 360° around the form with re-entrant corners. Put another way, the wet bundle of fibres would have to be threaded into a hole in the cylindrical form and then threaded out of a hole somewhere else while also maintaining approximately 90° turn-in terminations.

Specific Limitations – Spirals

FRP spirals are made by winding a wet bundle of fibres around a cylindrical form. This manufacturing process does not allow the terminal ends of the spiral to turn-in to the column core.

When detailing spirals, only shape "SP" as per Table 4 of the Reinforcing Steel Institute of Canada's Manual of Standard Practice (2020) is manufacturable.

Closure

It may be possible to have made some simple shapes that are listed as those that 'cannot be manufactured' when they are made in small batch quantities using by-hand lay-up methods. Use of such shapes shall be discussed with FRP manufacturers early during the detailing process.

12.4.2 Use of Size 10M Reinforcing Bars

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

12.4.3 Availability of 45M and 55M Reinforcing Steel

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

12.5 Reinforcement Details

12.5.1 Reinforcing Schedules

Rebar schedules shall not be prepared or provided by the designer in the contract documents. Contractors are required to produce the rebar schedules and working drawings. The designer shall prepare the contract drawings so that there are sufficient rebar details and other relevant information to facilitate the preparation of rebar schedules and working drawings by the fabricator.

12.5.2 Detailing Reinforcing on Contract Drawings

1. The designer shall ensure that the contract drawings clearly show all required detailing information.

2. Reinforcing bars shall be identified according to Sections 12.1.1 and 12.1.2.

3. Reinforcement details shall include the size, shape, spacing and placing limits for individual bars, and if necessary, the number of bars. Detailing shall be adequate to eliminate ambiguity and misinterpretation. Dimensioned sketches of bars should be provided as required.

4. Where multiple layers of reinforcement exist, the bar sizes in the outer layer shall not be less than any inner layer, and the bar spacing (linear for rectangular sections and

angular for circular columns) shall not be greater than in any inner layer.

5. Lap lengths for reinforcing steel, not indicated on the contract drawings, shall be Class B as per CHBDC. Other lap lengths, where required, shall be indicated on the drawings. For GFRP, lap lengths shall be according to the requirements of CHBDC.

6. Reinforcing steel bar hooks shall be as per Structural Standard Drawing SS112-1 unless indicated otherwise.

7. Reinforcing steel stirrups shall have hooks as per Structural Standard Drawing SS112-1, unless indicated otherwise.

8. Bent or curved GFRP reinforcing shall be fabricated with radii specified by the manufacturer. Designers should be aware of manufacturers' recommendations and ensure that they are accounted for in the design, uncommon bend radii may increase the cost of bent reinforcement as manufacturer's may need to purchase components for their bend forming apparatus.

   Because of instances of poor quality GFRP reinforcing and code strength limitations at "bend" locations, straight bars should be specified where possible.

9. Reinforcing tender items are lump sum bid items and the quantity shall not be listed on the contract "Quantities-Structure" drawing.

10. The designer shall forward the quantities of reinforcing items for carbon steel reinforcing, stainless reinforcing steel and GFRP reinforcing to the Project Delivery Section for Ministry internal use.

11. The requirements of the notes 5 to 7 above should be given as part of the general notes on the general arrangement drawing of all projects.

12. Structural reinforcement shown on the contract drawings shall not be used for any electrical grounding purposes. Any electrical grounding, if required, shall be done using independent insulated grounding cables that could be buried in/through concrete, if preferred, and exiting the structural component to be connected outside to a copper grounding rod.

12.5.3 Reinforcing Steel for Concrete Culverts

Reinforcing steel quantities for all cast-in-place concrete culverts in a contract are combined and included in a separate lump sum tender item for each culvert. However, when the total quantity of reinforcing steel for all concrete culverts in a contract is less than 5 tonnes, the reinforcing steel will be included in the concrete item "Concrete in Culverts" and there will be no separate tender item for reinforcing steel.

12.5.4 Hooks and Bar Bends

Structural Standard Drawing SS112-1 shall be attached to the full-size Structural Standard Drawings in a contract. It is intended for the contractor's use, in order to produce consistently accurate hooks and bar bends in carbon and stainless reinforcing steel.

GFRP Bent Bars

Both the minimum bend-radius-to-bar-diameter ratio (r/d) and standard r/d used on MTO projects shall be 4.0. There are currently no reduced bend radii for bent bars used as stirrups or ties.

The bend strength of a GFRP reinforcing bar is strongly correlated with the r/d ratio, which means that while the CHBDC permits the use of bars with a r/d ≥ 3.0, most of the MTO's DSM listed suppliers were qualified for bends at an r/d = 4.0 as meeting bend strength requirements.

The bend radius of a GFRP bar is the same as assumed for bending of steel bars as described in the Reinforcing Steel Institute of Canada's Manual of Standard Practice: it is the radius of the bending pin.

The GFRP industry has standardized bent shapes using a bend-radius-to-bar-diameter ratio (r/d) of 4.0 and may not be able to readily manufacture bends with other r/d ratios. Bent FRP reinforcing bars are shaped, generally, by winding a wet bundle of fibres around forms or jigs that are then placed in a chamber to cure into a solid and rigid shape. Thus, non-standard bends will require the manufacturer to obtain fixturing for their forms or jigs that will increase costs to supply the material.

12.5.5 Spiral Reinforcement

Ontario reinforcement fabricators are only able to bend spirals of size 15M or smaller. If larger transverse reinforcement is required, it may be possible to use bundled spirals (2 nested 15M spirals that are touching, and then with a spiral pitch somewhat larger than what could be achieved with a single spiral. Hoops of size 20M or 25M are also possible.

12.5.6 Splicing of Reinforcing Bars

Design drawings shall clearly define splice type(s), locations, and force requirements. For example, lapped versus mechanical splices; Type 1 versus Type 2 mechanical splice.

Lap splices are preferred for splicing of reinforcing bars due to their reliability and ease of inspection/verification during construction. Mechanical splices require material testing and assembly verification during construction and should be used where geometry necessitates, or where they provide benefits of reduced congestion of reinforcement or cost savings from materials reduction, such as:

a) When the location of construction joints and provision for future construction dictates the use of mechanical splices to provide tensile continuity, e.g., staged replacement of bridge decks when projecting bars are not feasible.

b) Splicing bars greater than 35M.

c) Where spacing of rebar is insufficient to permit lapping of bars, or large amounts of reinforcement or larger bars may obstruct concrete flow. This situation is common in columns and caissons.

d) Forming openings in tub girders to facilitate formwork removal for cast-in-place decks.

OPSS 905 lists and describes the following six groups of mechanical connectors (subject to DSM lists) for reinforcing steel bars:

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

The MTO groups of mechanical connectors is descriptive and a historical practice for the DSM. The categorization of all DSM mechanical connectors is tension-compression mechanical splices that can resist both tensile and compressive forces. The type of DSM mechanical connector may be Type 1 or Type 2.

Mechanical connectors of carbon steel and stainless steel shall be supplied in accordance with the latest active DSM at the time of contract awarded. Carbon steel bars shall not be coupled with stainless steel connectors, and vice versa.

Mechanical connectors are not available or acceptable for splicing of GFRP reinforcing.

Minimum cover requirements apply to the splices and mechanical connectors. The designer shall also consider geometric and clearance requirements for couplers (e.g., adequate access for swaged connectors, space between non-staggered couplers for concrete flow, etc.).

Where required, mechanical connectors and splices shall be designed for their specified fatigue loading.

12.5.7 Anchor Headed GFRP Bars

Anchor heads are only permitted on GFRP reinforcing bars of size 15 and may be assumed to develop 100 kN of bar strength.

12.6 Anchors in Concrete

Anchors post-installed into concrete shall be adhesive bonded dowels. Mechanical, grouted bonded and screw types shall not be used.

12.6.1 Post-installed Adhesive Anchors in Concrete (Dowels)

Adhesive anchors into concrete (dowels) shall not be used in new structures, except for reinforcement installed through steel or precast concrete girders at integral abutments and piers, and for the connection of precast headwalls to precast culverts.

Dowels into concrete shall not be larger than 25M.

Installation and acceptance testing shall be according to the relevant OPSS and SSP specification.

Dowels shall be detailed with holes drilled in downward, downward inclined, and horizontal orientations. Adhesive anchors upwardly inclined, including vertically overhead shall not be allowed. DSM List # 9.30.25 does not include any adhesives for use in overhead applications; until approved products are included in the DSM and installer/inspector programs become more accessible in Ontario, to ensure public safety and long-term performance of the infrastructure, upwardly inclined dowels are not allowed.

Installation of adhesive dowels to support sustained tension loads shall be designated with “(CERT)” after the anchor call-out on the contract drawings.

SECTION 13 - EXPANSION JOINTS AND BEARING ASSEMBLIES

Table of Contents

13 EXPANSION JOINTS AND BEARING ASSEMBLIES

13.1 General

This section describes the types of deck expansion joints and bearings used on MTO's bridges. The design and detailing requirements for both components are discussed throughout the section.

13.2 Expansion Joints

13.2.1 Expansion Joints in Decks

Bridge expansion joints are designed to allow for continuous traffic across structures while accommodating movements due temperature variations of the structure, long-term effects of creep and shrinkage of concrete, and differential settlement of substructures. Expansion joints shall allow sufficient vertical movement to permit bearing replacement without the need to dismantle the bridge expansion joint.

The designer should make every effort to minimise the number of expansion joints along the length of structure and to locate expansion joints behind the abutments through integral or semi-integral abutment configurations.

In all cases, the expansion gap shall be detailed parallel to the skew of the deck, for the full width of the deck. In skewed bridges, the resulting exposed acute angled concrete edges shall be provided with 50 mm chamfers. Designers should avoid skew angles of 32 to 38 degrees, inclusive, to minimise the chances of snagging snow plough blades which are usually set at approximately 35 degrees.

In addition to longitudinal movement of the superstructure, displacements due to rotation of the structure under transient loads shall be considered.

The gap between the end of a deck and the ballast wall face must meet the requirements of the CHBDC. The gap between the underside of the deck and the bearing seat shall be large enough and accessible for cleaning. The drawings must show the gaps free of any material used to form them in construction.

When specifying an expansion joint movement rating, only those movements which can occur after the joint has been installed shall be considered. Thus, when specifying expansion joint movement ratings for rehabilitation projects, elastic shortening, hydration, shrinkage, and creep effects should be neglected. Likewise, for new prestressed concrete bridges, creep and shrinkage effects occurring prior to joint installation should be neglected. In prestressed concrete bridges, joint installation should occur as long after the completion of prestressing operations as the schedule permits and shall be indicated on the expansion joint standard.

Proper installation is critical in achieving the expected long-term performance of expansion joints. Important elements of the installation include proper alignment, installation after paving, and epoxy injection under the armouring. Proper installation of expansion joint assemblies accounts for compaction of the asphalt wearing surface, by traffic, adjacent to the concrete end dams.

Expansion joints may be installed prior to or after paving. The Ministry's preference has historically been to install deck joint assemblies after paving because this method provides a superior ride for vehicles, resulting in lower impact on the joint and its anchorage, and superior durability. Installing deck joint assemblies after paving often places the operation on the critical path of the construction schedule. Projects have the potential to be delayed and incur cost overruns if the designer and/or contractor does not fully account for this. It has been found that there are some instances where installation prior to paving is acceptable, such as when a reduced durability is acceptable over the remaining service life of the bridge.

The designer shall specify the method of deck joint assembly installation in all new bridge construction or bridge rehabilitation projects which require new deck joint assemblies as follows:

1. The designer may choose to specify installation prior to paving when all the following criteria are met:

   a) The bridge is being rehabilitated;

   b) The bridge is not skewed;

   c) Average Annual Daily Traffic (AADT) is less than 2000;

   d) The estimated working days for structure work, with installation prior to paving, is at least 10% less than with installation after paving.

   The designer shall specify installation after paving for all other scenarios, unless written Approval is obtained by the designer from the Head, Structural Section.

2. The designer shall include Structural Standard Drawing SS113-38 in the Contract Documents and modify it to reflect the actual bridge and to show the specified method of installation.

3. The designer shall include fill-in standard special provision SP 920F03 in the Contract Documents, amending OPSS 920 to clearly specify the method of deck joint assembly installation for each bridge.

The designer shall consider that installation prior to paving reduces the durability and longevity of the expansion joint. A service life of 20 years shall be assumed for financial analysis.

The guidelines included above may also be used to assist the designer in evaluating a contractor's request to install deck joint assemblies prior to paving.

Concrete end dams detailed between an asphalt wearing surface and an expansion joint seal shall be wide enough to permit the placing of concrete through the gap between joint armouring and concrete dam armouring. Concrete end dams should not be less than 500 mm in width, either side of the joint seal. Weep holes through the deck are required at low points at these dams. Refer to OPSD 3349.100.

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

a) for modular joints: two shorter bars lapped between the support boxes; and,

b) for all joints: at least two shorter lapped bars which are to be weaved through anchor loops and through other reinforcing steel.

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

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

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

13.2.2 Approved Expansion Joint Assemblies

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

Table 13.2-1 – Approved Expansion Joint Systems

DSM NO.	DIVISION/APPLICATION/PRODUCT
DSM 9.40.18	Expansion, Injection Systems for Armouring
DSM 9.40.20	Expansion, Modular
DSM 9.40.24	Expansion, Strip Seals Anchored in Concrete, Type A
DSM 9.40.27	Expansion, Strip Seals Anchored in Concrete, Type C
DSM 9.40.33	Expansion, Strip Seals in Preformed Retainer

Guidelines for the application of these lists are given in 13.2.1 to 13.2.5. The selection of the expansion joints for bridges shall be based on the total design movements at SLS and other requirements of this section. During the design process, the designer shall select which type should be used for each joint location on the bridge (see Table 13.2-2).

Table 13.2-2 – Expansion Joint Selection

Design Movement at SLS (mm)	Integral and Semi-integral Abutment Bridges	Conventional Abutment Bridges	Note
0 to 10	-	Type A or C	Section 16.6 or Section 13.2.2
11 to 40	Type C*	Type A or C	* at Sleeper Slab
41 to 80	Type C*	Type A or C	* at Sleeper Slab
76 to 120	-	Sliding Plate	-
81 to 400	-	Modular	Field splicing of modular joints is not permitted

13.2.3 Expansion Joint Assemblies in Skewed Bridges

A plan and profile must be provided on the drawings for skewed expansion joints.

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

Where this is not possible the horizontal change in direction should be affected at a location not less than 100 mm and not greater than 600 mm from the edge of the asphalt at the barrier wall. The change in direction shall be affected by means of a 600 mm radius bend. The exact location of the radius bend should appear on the plan view of the structure on the contract drawings and on the contractor's shop drawings. Locating the horizontal change as close as is practicable to the barrier wall reduces the modifications necessary to the substructure.

A horizontal change in direction shall NOT be located within 100 mm of a vertical change in direction since simultaneous bending of the seal in two planes is either physically impossible or likely to lead to leakage between the seal and the clamping device.

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

13.2.4 Expansion Joint Drainage System

Strip seal expansion joints and the sliding plate expansion joint assembly are available with a drainage system which allows runoff from the seal to be channeled through the barrier and down to the ground through a scupper and downpipe according SS113-14. The drainage system prevents local ponding where the joint seal is otherwise upturned into the barrier and flushes the joint seal of debris. The flushing is particularly beneficial for sliding plate joints where the strip seal is covered with steel plates. The use of the drainage system is recommended when freezing and/or ponding of water is a concern, specifically for bridges on flat profiles or close to a flat section of a vertical curve. The drainage system shall not be used in aesthetically sensitive locations and underpasses.

13.2.5 Expansion Joints at the End of the Approach Slab of Integral and Semi-Integral Bridges

Integral abutment bridges are single or multi-span bridges with the superstructure integrally connected to the abutments. Both integral and semi-Integral bridge approach slabs are connected to and move together with the superstructure.

The movement demand is proportional to the expansion length of the superstructure. The approach slabs configuration and specified treatment at the end of the approach slab varies based on the expected movement, refer to Figure 13.2-1

Regardless of the movement calculated, the following detail shall be used on integral and semi-integral bridges of up to 30 m in total length.

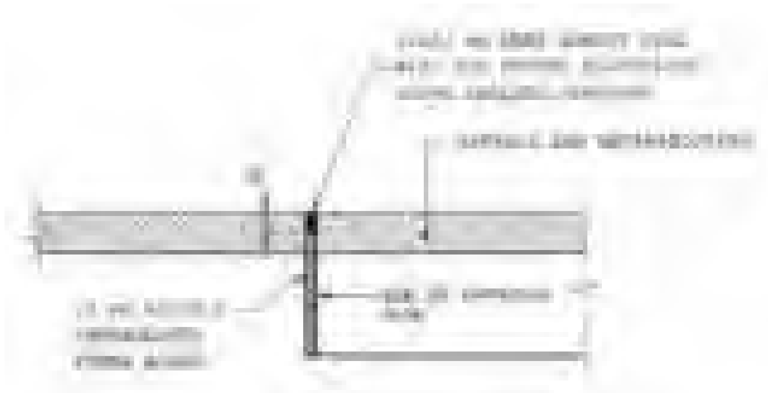


Figure 13.2-1 – Expansion Joint Detail at the End of Approach Slab for Span Length < 40m

Buried approach slabs shall be used for all new integral and semi-integral abutment bridges without sidewalks, within the following limits:

a) In concrete bridges with lengths more than 30 m but not exceeding 125 m;

b) In steel bridges with lengths more than 30 m but not exceeding 100 m;

For all other integral and semi-integral abutment bridges, Structural Standard Drawing SS113-36 shall be used. The approach end of the approach slab shall be supported on a sleeper slab and a Type C expansion joint shall be provided between the approach slab and the sleeper slab.

13.2.6 Longitudinal Joints and Gaps

Longitudinal joints exposed to traffic shall be avoided unless approved by the Head of the Structural Section. When used, they shall be Type C expansion joints to accommodate movement demands from differential deflection and movement and have adequate durability to withstand the traffic loads placed upon them.

Twin bridges separated by a narrow longitudinal gap shall be sealed to prevent the accumulation and passage of debris, snow, water, and chlorides. These longitudinal gaps (typically between raised medians or parallel barrier walls) shall be detailed with a proprietary expansion joint seal according to MTO Policy Memo #SCB-SO-2021-02.

13.2.7 Completion of Structural Standard Drawings

The following note on the expansion joint Structural Standard Drawings must be completed by the designer:

"EXPANSION JOINT SHALL BE IN ACCORDANCE WITH THE DESIGNATED SOURCES FOR MATERIALS LIST DSM 9.40.[Note 1] AND SHALL HAVE A RATED TOTAL MOVEMENT CAPACITY OF … mm." [Note 2]

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

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

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

In completing the table of design requirements, the “J” dimension shall be determined by setting the joint seal to be closed to the minimum opening at the hottest effective temperature, increased by 5 mm where possible (i.e., when the full rated movement of the joint is not fully utilised). This will have the effect of providing slightly more opening for future replacement of the seal.

13.2.8 Force Due to Movement of Modular Joints

The force associated with the movement of modular joints shall be considered when designing abutments.

When a bridge contracts in the longitudinal direction due to a decrease in Temperature (as well as due to Creep and Shrinkage in prestressed concrete bridges), the modular joint responds by opening further to accommodate this movement. As the joint opens, the control springs which maintain equidistant openings across the seals are compressed, causing a corresponding resistance to the opening movement, and thereby transferring this resisting force as a horizontal load to the top of the ballast wall.

Assuming a control spring stiffness of 0.15 kN/mm, and supports beams spaced at 800 mm centre-to-centre, the resulting unfactored horizontal force would be approximately 8 kN per metre length of ballast wall.

13.2.9 Strip Seal Joints Anchored in Concrete

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

DSM 9.40.24 and 9.40.27 is subdivided into several types based on the method of seal retention used. General guidelines showing the characteristics of each type and the applications for which it should be considered, are given on the following pages.

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

Structural Standard Drawing SS113-11 covers anchorages and armouring for all types of expansion joints listed in DSM 9.40.24 and 9.40.27.

<u>Strip Seals - Type A Steel Plate Clamping Device</u>

Clamping Plate Subject to Direct Wheel Load

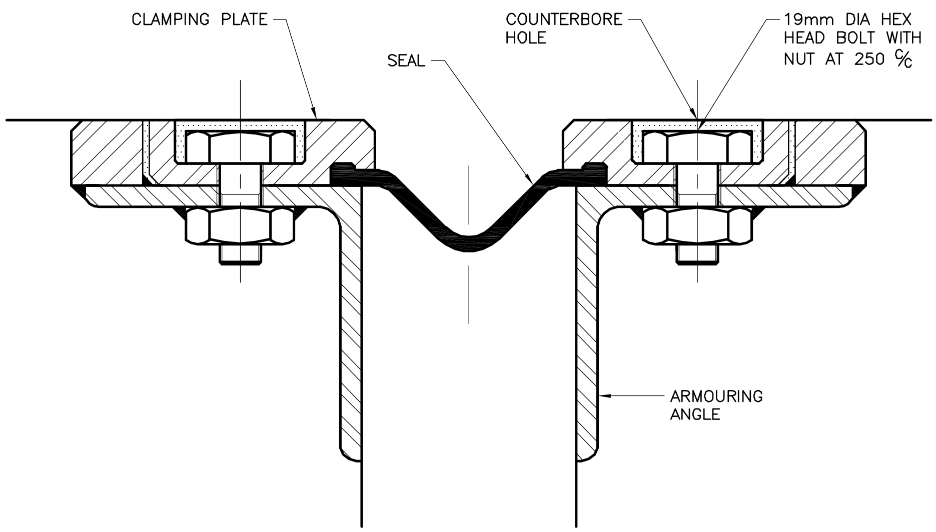


Figure 13.2-2 – Strip Seals – Type A Steel Plate Clamping Device

<u>Characteristics</u>

- Clamping plate directly supported.
- Few components.
- Excellent seal retention.

- Easy to install and replace seal.
- Direct access to clamping bolts and seal.
- Seal can be inspected completely and reused, if undamaged.
- Easy to modify for changes in elevation.
- Higher initial cost.

Strip Seals - Type C Retainers and Stop Bars

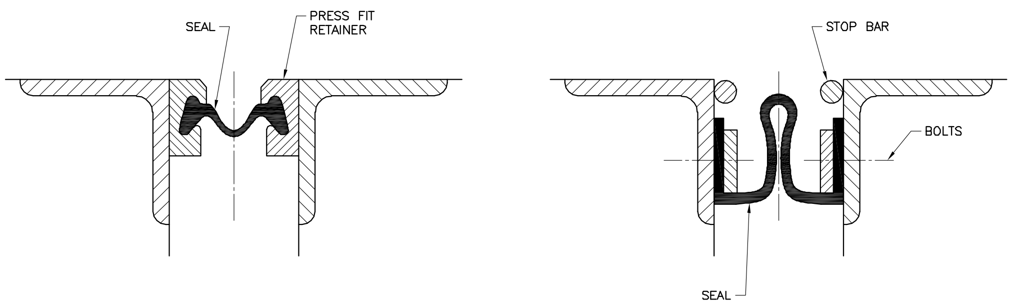


Figure 13.2-3 – Strip Seals – Type C Retainers and Stop Bars

Characteristics

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

Application

Type A joints as per DSM 9.40.24 should be considered for all bridges and particularly for bridges which carry high volume freeway/arterial traffic (and especially when there is a high proportion of heavy vehicles). For bridges which, as determined by the Head, Structural Section, it is essential to minimise the frequency and duration of future lane closures, only Type A joints shall be specified.

Type C joints as per DSM 9.40.27 shall be specified for all bridges except those where only Type A joints are allowed. Expansion joints between sleeper slabs and approach slabs shall be Type C.

13.2.10 Sliding Plate Joints

Sliding plate expansion joints (see Figure 13.2-4) and Structural Standard Drawings SS113-30 to SS113-35) shall be used for joints where the SLS design movement is greater than 75 mm and less than or equal to 120 mm. The sliding plate is designed to behave as propped cantilever. It is essential that the sliding plate be installed with the correct tolerance to the stop plate. During service, the joint is designed to function with the sliding plate either directly bearing on the sliding plate or up to a maximum of 3 mm above the sliding plate. At installation, the gap between the sliding plate and the stop plate should be no more than 1 mm.

Sliding plate expansion joints shall only be used at abutments of post-tensioned deck bridges and at abutments of girder type bridges. They shall not be used at pier locations. Sliding plate joints shall not be installed in bridges when the slope of the roadway at the joint exceeds 2% unless the bearings at that location are inclined to the same slope as the highway. When the deck is sloped and the bearings are installed level, translation due to expansion and contraction results in a change in the vertical gap between the sliding plate and the stop plate.

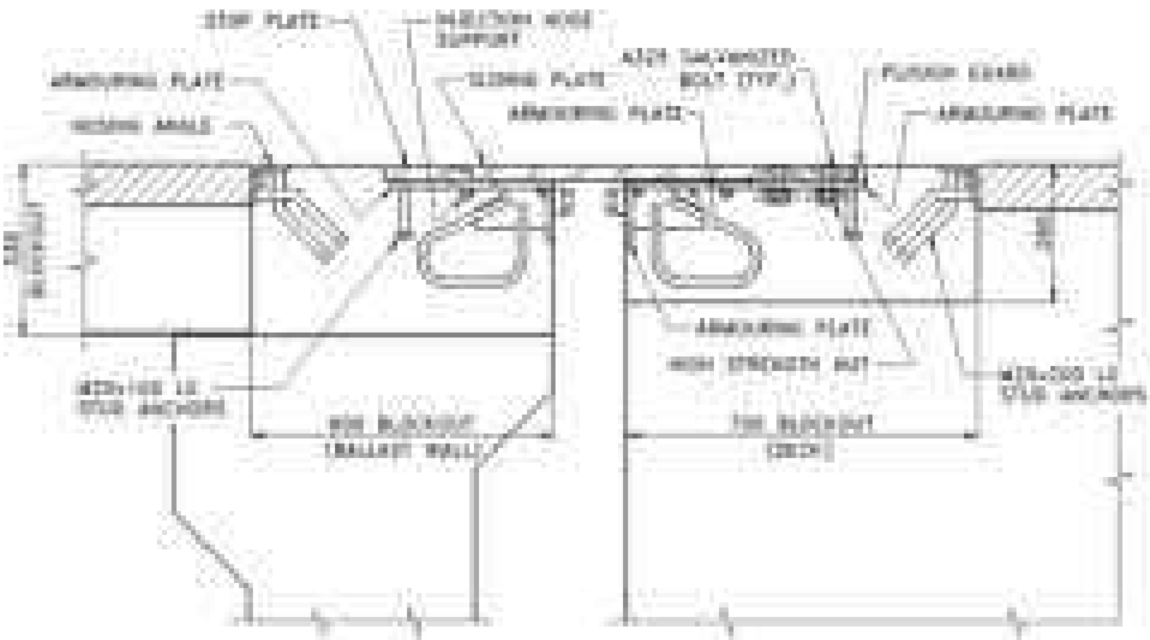


Figure 13.2-4 – Sliding Plate Expansion Joint

13.2.11 Modular Joints

13.2.11.1 Modular Joint Use

Modular joints shall be used when total deck joint movements, after joint assembly installation and measured parallel to the centreline of the highway, are greater than 120 mm in one direction. With this type of expansion joint, wheels bear directly on the separation beams between the seals. Seals and separation beams form a watertight joint at the riding surface. Separation beams are supported by steel support bars each having its own support box at both ends. One end of the support bar is fixed in a short support box in the end dam, and the other end slides on engineered bearings inside a longer support box. Modular joints are classified by the number of longitudinal support bars and how they are connected to the transverse separation beams. In a multiple support bar system, each separation beam is supported on an individual support bar. The number of support bars therefore equals the number of separation beams and in a large movement joint, the number of support bars leads to a joint that is not easily accessed from below for maintenance. A single support bar system has all the separation beams supported on one large support beam, and the separation beams slide on bearings on the top face of the support beam. A single support bar system is therefore more accessible for maintenance. Due to past performance, the Ministry has shifted towards single support bar systems.

Modular joints shall not be field spliced.

In order to preserve the ride quality across a modular joint, the bearings supporting the girder end shall be installed at the same slope as the highway vertical profile, to allow the support bars to remain parallel to the highway at all range of movement.

Available Modular joints can have a minimum of 3 to a maximum of 8 seals. The maximum movement of any one seal, measured parallel to the centreline of the highway, must not exceed 60 mm.

The end dam over the ballast wall, the gap between the ballast wall and the deck end, and the end dam at the deck end shall be detailed to permit installation of any modular joint listed on DSM No. 9.40.20.

The absolute minimum distance between end dams, denoted **F**, is 250 mm at 15°C, to permit access to the joint components for maintenance. The distance between the ballast wall and the deck end, denoted **L1** shall be as specified in Table 13.2-3 to permit inspection and maintenance of the expansion joint from beneath. A corbel shall be introduced on the ballast wall side with length **D** to support the end dam. The corbel shall be detailed with a dimension D of 250 mm for 3 seals, 200 mm for 4 seals, and 150 mm for 5 or more seals. The corbel shall be designed and detailed to be increased by up to 150 mm to accommodate the specific joint supplier. Dimensions **F** and **D** are shown Figure 13.2-5.

For post-tensioned concrete bridges, **L1** should be calculated at 15°C for the required joint size and movements due to creep, shrinkage and temperature fall occurring after time of joint installation. In lieu of more accurate data, the designer may assume joint installation occurs at 90 days after stressing of prestressed concrete bridges.

The width of blockout **B1 and B2** depend on the number of seals in the joint, while the depth of blockout **H** depends on the number of seals and on the supplier of the expansion joint. Dimensions **B1**, **B2** and **H** in Figure 13.2-6 shall be detailed according to Table 13.2-3.

The following note shall be added to the ABUTMENT drawing:

"THE CONTRACTOR SHALL ADJUST CORBEL DIMENSION 'D' AND REINFORCING STEEL TO SUIT THE ACTUAL MODULAR EXPANSION JOINT FOR THE BRIDGE, UP TO A MAXIMUM OF 150 mm."

13.2.11.2 Modular Joint Selection

1. Calculate the Design Movement, due to the sum of Creep + Shrinkage + Temperature Range.
   Creep and Shrinkage movements are from time of joint installation (e.g., t = 90 days) to joint replacement, assumed at a service life of 40 years.
   Movements due to Temperature Range are from the sum of temperature fall to temperature rise.

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

   If the SLS Design Movement < 120 mm, use strip seal (see Section 13.2.2 and Section 13.2.2 to Section 13.2.5) or sliding plate joints (see Section 13.2.6) as appropriate.

3. For Modular Joints, select the joint size (number of seals required) based on the SLS Design Movement as provided in DSM 9.40.20.

4. Detail the abutment ballast wall and deck ends to accommodate dimensions "**B1**", “**B2**” and "**H**".

Table 13.2-3 – Modular Joint Detailing Table

No. OF SEALS	2	3	4	5	6	7	8
B1 (mm)	590	675	760	835	915	995	1075
B2 (mm)	600	600	600	650	650	700	700
H (mm) MINIMUM	355	355	355	355	380	380	380
F @ 15°C (mm)	200	330	460	600	730	850	980
L1 @ 15°C (mm)	600	600	660	750	880	1000	1130

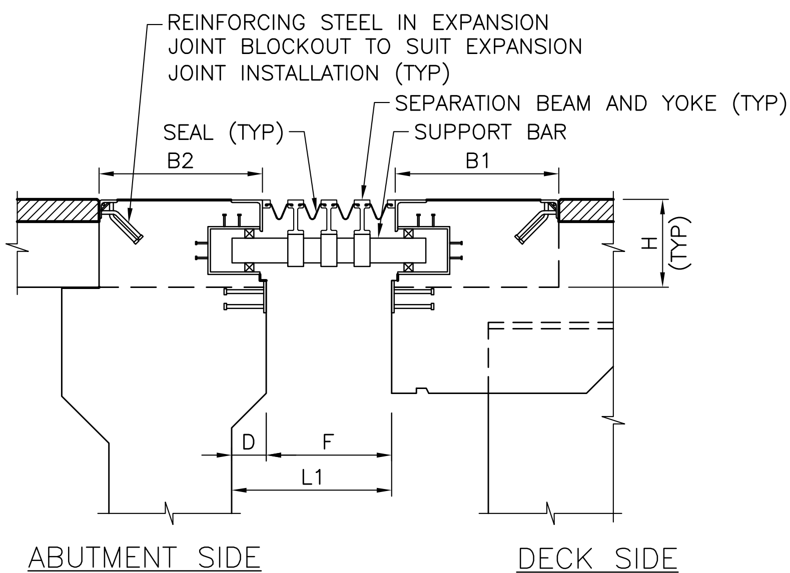


Figure 13.2-5 – Modular Joint Detail – Section at Boxes (Single Support Bar System)

13.2.11.3 Modular Joint Setting

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

For prestressed concrete structures, "J" at 15ºC equals 0.8 (CR + SH + T_{fall}), where T_{fall} is the movement due to a temperature drop from 15°C to the Minimum Effective Temperature used in design, and CR and SH correspond to the anticipated creep and shrinkage of the bridge for 40 years after joint installation.

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

13.2.11.4 Abutment Width

Since large movements are expected when modular joints are used, the abutment should be proportioned based on the distances shown in Figure 13.2-6. The effect of skew and minimum seating requirements for earthquake should also be considered. L1 shall be a minimum of 600 mm for modular joints, to permit inspection and maintenance of the joint. For post-tensioned bridges, the minimum lengths of L1 and L2 shall be satisfied at 90 days after placing concrete.

L1 = Gap between ballast wall and end of deck

L2 = Distance between centreline of bearings and end of deck, as required to satisfy minimum seating length requirements,

L3 = Distance between centreline of bearings and face of abutment

t = Thickness of ballast wall (see Section 5.2.3.1 for minimum thickness)

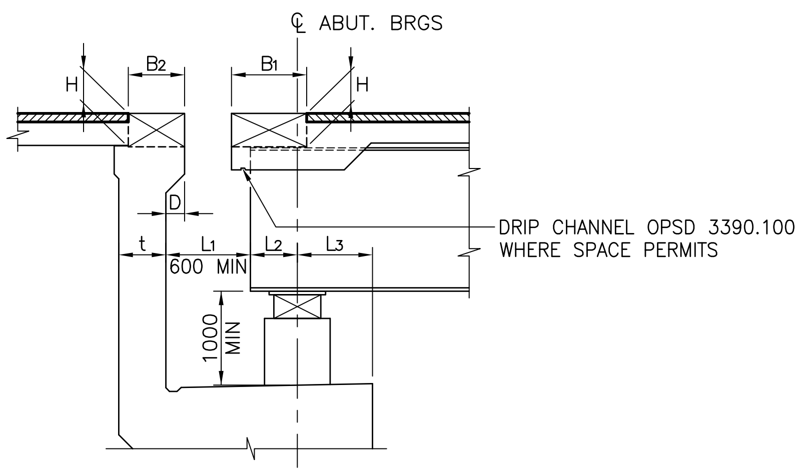


Figure 13.2-6 – Minimum Width of Abutment

13.3 Bridge Bearings

13.3.1 Bearing Selection

MTO maintains standards for rotational bearings and for elastomeric bearings. The selection of the bearings should be based on the articulation requirements of the structure. The requirements of the bearings such as translations, rotations, vertical and lateral load capacities and any special requirements must be established during the design process. Design values provided in the tables on the drawings shall be the actual demands on the bearings, from the structure, and shall not include additional tolerances. Tolerances are added to the values specified in the design, as required by OPSS 1202 and 1203, which include the tolerances required by the CHBDC.

For reactions of up to 3000 kN at SLS1, the designer shall specify elastomeric bearings because of their long-term durability and cost-effectiveness, unless movement or rotation demands preclude their use.

For high loads and/or long movements, rotational bearings shall be selected. For the majority of bridges, all three types of rotational bearings will satisfy all the structural requirements. However, there will be some bridges where, for reasons such as rotational capacity or rotational stiffness, only one or two classes of bearings will be acceptable.

For bridges with more than one span, except for integral bridges where only plain elastomeric bearings are used and for prestressed girder bridges with elastomeric bearings, a bearing drawing shall be included in the design drawings for the bridge. The bearing drawing shall contain a plan layout of the bearing locations at every support, along with a symbol to indicate the bearing type and fixity. Figure 13.3-1 shows an example of a bearing layout, along with the standard symbols to be used for rotational and elastomeric bearings, and their associated restraints.

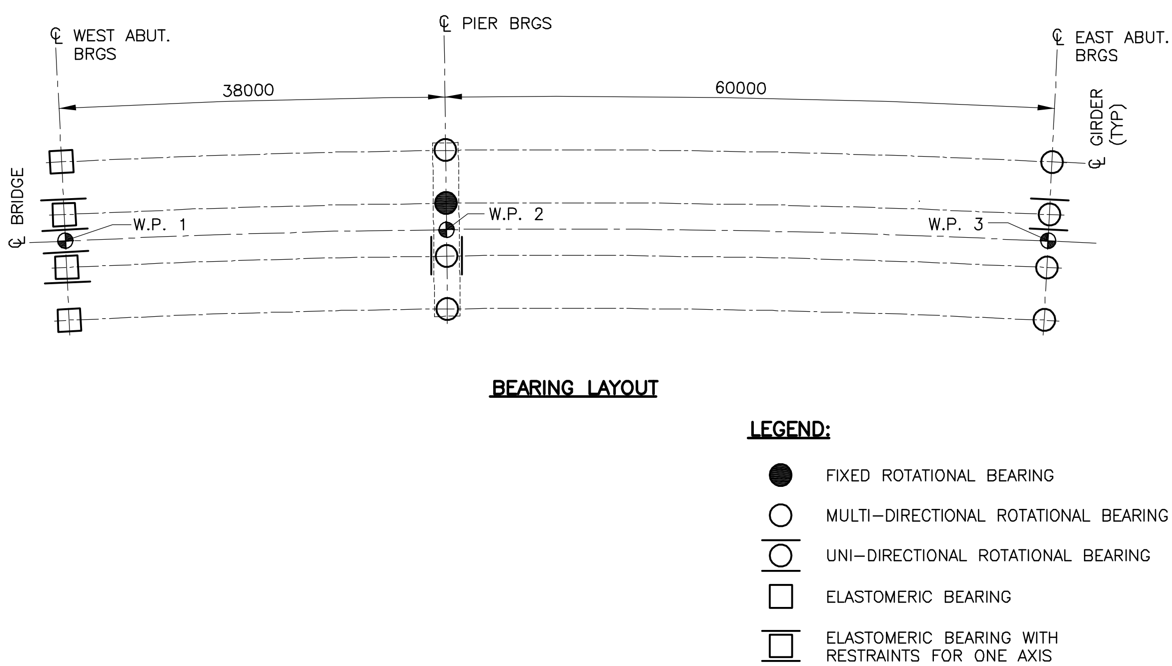


Figure 13.3-1 – Bearing Layout and Legend

Since the final bearing design is completed by the bearing supplier, the designer shall provide a schematic section on the bearing drawing to convey the main components of the bearing and the outer dimensions of the bearing assumed in the design. For elastomeric bearings, the height of the bearing shall be shown along with any details of positive attachment or plates above and below the bearing. For rotational bearings, a typical section, as shown in Figure 13.3-5 (bearing section), shall be included on the bearings drawing. For most cases, a section through the bearing transverse to the bridge profile is sufficient but a longitudinal section may be required for bearings with longer movements or highways with steeper grades.

13.3.1.1 Bearings Supporting Steel Tub Girders and Concrete Box Girders on Skew

Where the skew of girders relative to the substructure exceeds 20 degrees, single bearings shall be designed to support each girder at each substructure location. The overall stability of the bridge is maintained by the diaphragms between adjacent girders.

A single bearing below each girder, at each support, allows unrestrained rotation during deck placement. The deflection of the girders during concrete deck placement causes a rotation at the end of the girder which is about the axis normal to the girder. When pairs of bearings are used to support a tub or concrete box girder, the girder is forced to twist to accommodate a rotation about the axis of the supports. Usually the girder cannot accommodate the twist and rotation at the supports, and in construction, the bearings frequently end up with unequal support reactions and a lack of contact at the acute corners of the each girder.

13.3.2 Rotational Bridge Bearings

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

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

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

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

Uplift restraint devices should be avoided and their use requires the approval of the Structures Office Manager. Uplift-restraint devices shall not restrict the function of a bearing and shall not limit the rotation capacity nor the displacement of the bearings under any load combination. OPSS 1203 and 922 do not include provisions for design of uplift-restraint bearings. When they are used, the contract shall include special provisions to cover their design.

Columns supporting expansion bearings made of lubricated, unfilled PTFE surfaces shall be proportioned based on a design coefficient of friction as given in the CHBDC.

13.3.2.1 Effect of Rotation on Eccentricity of Axial Load

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

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

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

$$e = \mu R$$

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

a) 4% of the diameter of the confined elastomer for pot bearings;

b) 10% of the diameter of the polyether urethane polymer compound for disc bearings;

c) 10% of the plan diameter of the curved PTFE surface for spherical bearings.

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

13.3.2.2 Translation and Rotation Capacity

Translational and rotational requirements about any horizontal axis and about the vertical axis through the centre of the bearing provided in the bearing design data table shall be as required by design.

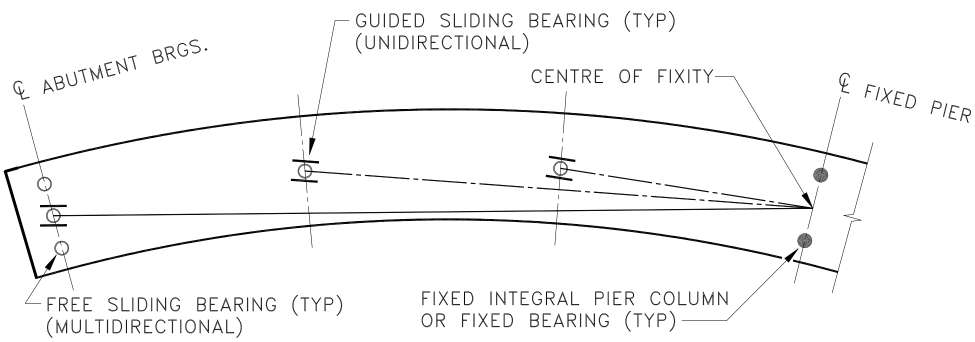
13.3.2.3 Orientation of Guided Bearings on Curved Structures

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

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

a) The structure is guided to a Centre of Fixity by aligning guided sliding bearings parallel to the chord drawn from the bearing to the theoretical point of fixity (see Figure 13.3-2). The Centre of Fixity is the point of zero movement of the superstructure for internally induced forces or movements due to temperature change, as well as creep and shrinkage if applicable. It is calculated by taking into consideration the combined stiffness of all supports including shear stiffness of bearings, flexural stiffness of supports (abutment or pier) and rotation of footings due to strain of piles or subsoil.

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

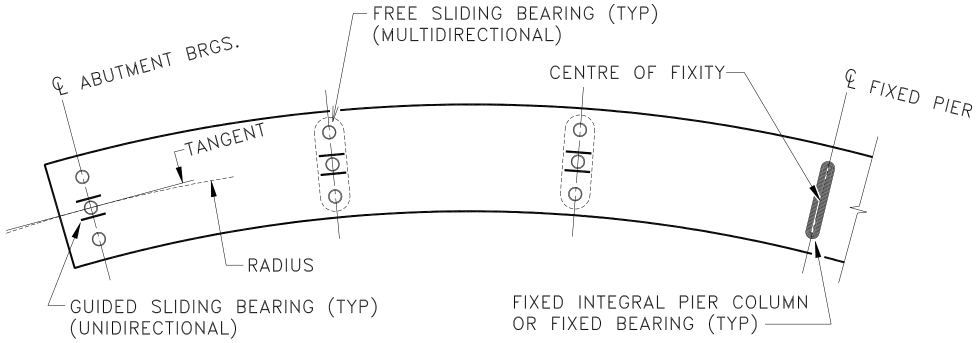


- Guided bearings aligned parallel to chord.

Figure 13.3-2 – 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 13.3-3).

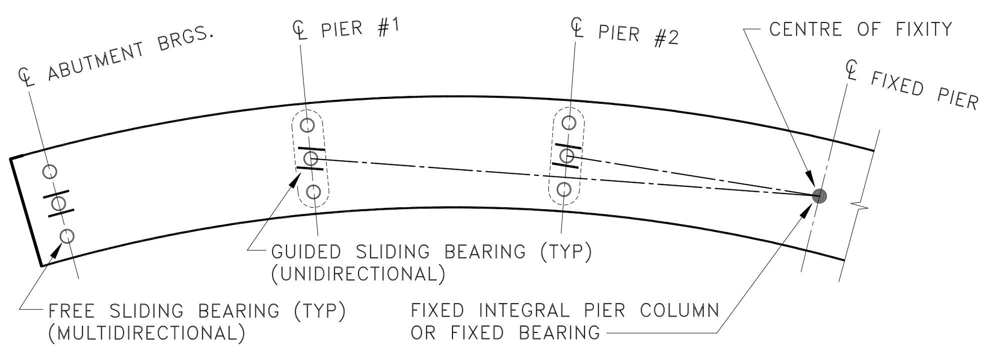
   Bearing orientation B relies on the guided bearings to force the horizontal movement in the direction of the longitudinal curved axis of the superstructure, resulting in horizontal forces being transferred to the substructure through these bearings.



- Guided bearings aligned tangent to structure radius.

Figure 13.3-3 – 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 13.3-4). As a result, because expansion joints at abutments are placed radially, they will expand and contract parallel to the abutment guided bearings without distortion and the abutments are generally better able to handle the lateral loads.



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

Figure 13.3-4 – Bearing Orientation C

Whichever method of bearing orientation is used in design, the designer is encouraged to use a refined method of analysis to determine the resulting horizontal movements and corresponding forces transferred to the substructure.

13.3.2.4 Lateral Load Capacity

Fixed and guided bearings shall be capable of resisting lateral loads in the restrained direction as required by the design and as given in the CHBDC. All lateral loads shall be resisted by a single bearing at a support line (pier or abutment line) for ULS-1 though ULS-4. For ULS-5 through ULS-8 (i.e., for Exceptional Loads), when more than one identical bearing with guides for lateral restraint are provided at a pier or abutment, one bearing shall be assumed to achieve its full lateral resistance while a second bearing shall be assumed to resist 50% of its full lateral capacity and any subsequent guided bearings shall not contribute to the lateral resistance. Other means of lateral restraint (i.e., a shear key) should be used to provide lateral resistance to the bridge superstructure when the factored lateral force exceeds 50% of the permanent reaction at the bearing.

13.3.2.5 Concrete Bearing Pressure

The average concrete bearing pressure used by the bearing manufacturer to proportion the bearing shall be based on a Specified 28-day Compressive Strength of 30 MPa.

The concrete bearing surfaces above and below the bearing should be proportioned and reinforced when necessary to withstand the combination of vertical bearing reactions and lateral loads.

13.3.2.6 Installation

As the elevation at the top of bearings is critical especially for bolting diaphragms of steel box girders, the tolerances for the elevation at the top of bearings as specified in OPSS 922 are as follows:

Concrete deck or girders	±5 mm
Steel plate girders	0 to +3.0 mm
Steel box girders	0 to 2.0 mm

The Specification requires all bearings to be bedded over their entire area on an approved grout to achieve the theoretical elevations within these tolerances. The thickness of the bedding grout shall be 12 mm +/- 3 mm.

The height of bearing, ‘D’ (as shown in Figure 13.3-5) assumed in establishing the bearing seat elevation should include 12 mm for grout bedding and the drawings should include the following note:

"HEIGHTS OF BEARINGS INCLUDE 12 MM BEDDING GROUT AND EXCLUDE THE SHOE PLATE THICKNESS. HEIGHTS OF BEARINGS ASSUMED IN ESTABLISHING BEARING SEAT ELEVATIONS ARE AS FOLLOWS. THE CONTRACTOR SHALL ADJUST BEARING SEAT ELEVATIONS AND REINFORCING STEEL TO SUIT ACTUAL HEIGHTS OF BEARINGS.

ABUTMENTS … mm
PIERS … mm "

Precautions must be taken by the Designer, when there is the possibility of bearings being installed under extreme cold or hot temperatures and not at 15ºC. 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.

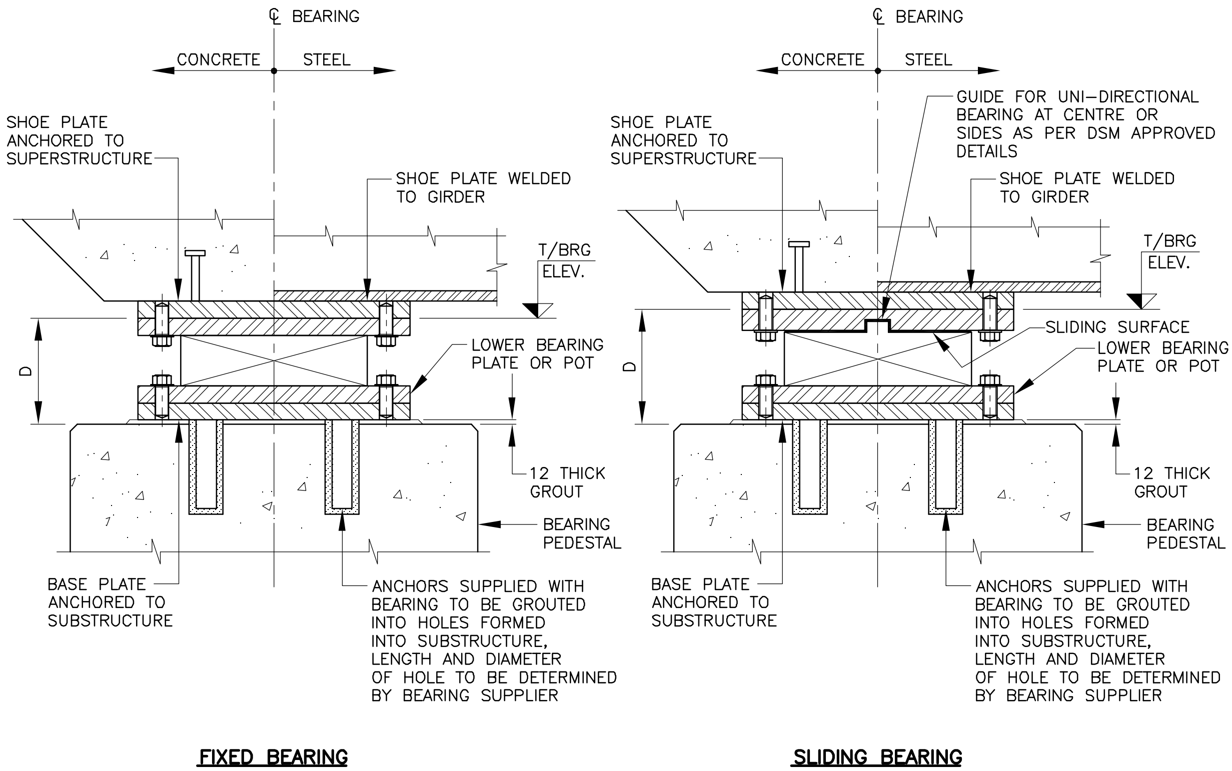


Figure 13.3-5 – Rotational Bearing Typical Section

13.3.2.7 Bearing Design Data

The bearing design data provided on the plans shall be as indicated below. Applied horizontal loads shall be consistent with applied axial loads. Additional rows may be added if necessary for atypical bridges where one of many ULS combinations may govern. Any special requirements must also be specified.

Table 13.3-1 – ROTATIONAL BEARING DESIGN DATA

LOCATION	TYPE	LIMIT STATE	COMBINATION	AXIAL LOAD (kN)	MAX. HORIZ. LOAD (kN)		MAX. ROTATION (radians)		MAX. TRANSLATION (mm)	
					LONGIT	TRANS	ABOUT HOR. AXIS	ABOUT VERT. AXIS	LONGIT	TRANS
	[Fixed or uni-direction	Serviceability	PERMANENT							
			PERMANENT + TRANSITORY MAX.							

			PERMANENT + TRANSITORY MIN.							
		Ultimate	PERMANENT							
			PERMANENT + TRANSITORY MAX.							
			PERMANENT + TRANSITORY MIN.							
			PERMANENT + EXCEPTIONAL MAX.							
			PERMANENT + EXCEPTIONAL MIN.							

The contract drawings shall include the following notes:

1. ROTATIONAL BEARINGS SHALL BE CLASS/CLASSES LISTED IN DSM 9.15.71, 9.15.75, and 9.15.80 UNDER THE HEADING "BEARINGS, BRIDGE (ROTATIONAL)”.

2. BEARING SUPPLIERS ARE REQUIRED TO PROVIDE ADDITIONAL ROTATIONAL CAPACITY OF 0.02 RADIANS ABOUT THE HORIZONTAL AXIS AND 0.02 RADIANS ABOUT THE VERTICAL AXIS IN ACCORDANCE WITH OPSS 1203 AND AS REQUIRED BY CHBDC.

3. THE CONTRACTOR SHALL COORDINATE WITH THE BEARING SUPPLIER THE LOCATION OF THE BEARING ANCHORAGES IN THE PIERS AND/OR ABUTMENTS. HOLES TO RECEIVE ANCHORS SHALL BE FORMED INTO THE SUBSTRUCTURE. CORING IS NOT PERMITTED.

The additional rotations mentioned in note 3 shall not be included in the bearing design table.

If the live load at SLS on a sliding bearing is less than 80% of the permanent load at SLS the following note shall be added to the notes in the contract drawings, “THE CONTRACTOR SHALL ESTABLISH THE BEARING SIZE SUCH THAT THE CONTACT PRESSURE ON THE PTFE FOR THE HORIZONTAL SLIDING SURFACE FOR UNI-DIRECTIONAL AND MULTI-DIRECTIONAL BEARINGS UNDER PERMANENT LOADS AT SLS IS NOT LESS THAN 25 MPa.”. Higher pressure on sliding and rotational bearings from permanent loads reduces the friction between the PTFE and mating surface and reduces the rotational restraint of the bearing.

For bearings subject to uplift, the maximum permitted separation of the bearing components must be specified.

The maximum required translation must be based on the assumption that, at the time of installation, the longitudinal and transverse centrelines of the bearing sliding plate will be set to coincide with the longitudinal and transverse centrelines of the bearing.

The coefficient of friction for PTFE sliding surfaces given in the CHBDC depends on whether the PTFE resin is “filled” or “unfilled”. The type of PTFE resin assumed for design must be included with the bearing design data given on the contract drawings, specifically giving the following note: “UNFILLED PTFE, LUBRICATED DIMPLED SHEET HAS BEEN USED IN THE DESIGN FOR ESTABLISHING THE BEARING COEFFICIENT OF FRICTION.” This produces the minimum coefficient of friction. If for some reason the designer wants to transfer a greater than minimum horizontal force to a particular sub-structure, other sliding surface treatments given in the CHBDC may be considered.

13.3.3 Elastomeric Bearings

13.3.3.1 General

Elastomeric bearings, plain and laminated, shall be natural rubber supplied in accordance with OPSS 1202. The following bearing sizes are recommended but other sizes may be available.

Table 13.3-2 – Standard Laminated Elastomeric Bearing Sizes

Size (mm x mm)	Minimum Thickness (mm)	Target SLS1 Load (kN)
300 x 200	50	420
350 x 250	55	610
400 x 300	55	840
450 x 350	60	1100
500 x 400	65	1400
550 x 450	70	1730
600 x 500	80	2100
600 x 600	90	2520
600 x 700	90	2940
600 x 800	100	3360

CSA S6:25 limits the total shear strain on the bearing but does not limit the bearing pressure. OPSS 1202 further limits the compressive deformation of the bearing to approximately 0.07 of the effective elastomer thickness. Target SLS1 values correspond to a total pressure of 7.0 MPa. The target SLS1 load values included in the table have proven performance but may be exceeded provided the bearing is designed to satisfy CSA S6:25 and OPSS 1202.

Higher aspect ratio (width to length ratio) bearings may be preferable when thinner bearings are required and rotation limitations are governing; they may be used when approval is obtained by the designer from the Head, Structural Section.

The rotational capacity of any bearing is a function of the effective elastomer thickness and is determined as per CHBDC.

13.3.3.2 Elastomeric Bearings Thickness

The thickness of plain elastomeric bearings specified on the contract drawings shall be between 15 mm and 25 mm. Plain elastomeric bearings shall be used only for temporary applications, for bedding of larger bearings, or in cases where they are encased in concrete.

Bearing thickness shall be chosen to limit shear strain due to shear displacement to 0.5.

The minimum thickness of laminated elastomeric bearings shall be according to Table 13.3-2. The minimum thicknesses in the table were established to accommodate a rotation of 0.002 rad due to transient loads in addition to the rotation tolerances in CSA S6:25. As long as bearing is specified with a thickness equal to or greater than the value in the table, and loaded with at least 3 MPa under permanent loads, the bearing will meet the strain requirements (CSA S6:25 clauses 11.6.6.3.2 to 11.6.6.3.4), lift-off requirements (Clause 11.6.6.8), positive attachment (11.6.6.6) for service temperature T_s > -44ºC and elastomer-to-concrete interfaces, as well as deformation limits required by OPSS 1202.

For post-tensioned concrete structures, the use of a thicker elastomeric bearing may be a more economical choice than jacking the structure several months after stressing to release the shear deformation in the bearing.

Thicker bearings do not add significant cost. A taller bearing may be preferable to reduce the force transferred to the substructure, to avoid the need for positive attachment, or to accommodate larger rotations.

13.3.3.3 Strip Bearings for Precast Side-by-side Box Girders and Precast Slabs

Strip plain elastomeric bearings may be used for precast side-by-side box girders or precast slab bridges. The thickness of the strip bearing shall not exceed 25 mm.

13.3.3.4 Laminated Elastomeric Bearing with Slider Plate Assembly

For durability reasons, a slider plate shall not be used when a thicker bearing without a slider plate is feasible and shall not be used for a movement range of less than 50 mm. This type of bearing may be a reasonable choice for a long structure where the only other solution would be a rotational bearing (pot, disc, or spherical bearing). OPSS 922 does not cover the design provisions for laminated elastomeric bearings with sliding elements, and if used, details of the sliding surfaces should be provided in the Contract Drawings with appropriate NSSP's to cover their design and fabrication.

13.3.3.5 Supply of Additional Sample Bearings for Testing

To ensure the quality of the bearings supplied, standard special provision 922F01 requires that additional elastomeric bearings be supplied by the contractor for testing purposes.

The number of bearings requiring testing is based on the type of bearing, and the number of bearings of each size. The fill-in special provision for bearings provides guidance to the designer on the exact number of bearings to test. This special provision shall be included in the contract. Each structure requires at least one bearing of each size for testing. No samples for testing are required for bearings that are temporary and/or subsequently encased in concrete (e.g., bearings located at the integral supports of integral abutment type structures).

Additional bearings are not required to be supplied for all other approved bearings (i.e., disc, pot, spherical).

Twin structures shall be considered as separate structures.

13.3.3.6 Shear Rate of Elastomeric Bearings

When an elastomeric bearing undergoes shear deformation due to horizontal loading, a horizontal force develops which is transferred to the substructure. The magnitude of this force is a function of the shear stiffness (shear rate) of the bearing at the time of displacement and the amount of bearing deformation.

The amount of deformation (due to superstructure movement) depends on the combined effects of the following internally induced forces:

a) Heat of hydration (of concrete);

b) Elastic shortening (of concrete due to prestressing);

c) Shrinkage (of concrete);

d) Creep (of concrete under prestress and permanent loads);

e) Temperature effects.

Note: a) to d) are applicable to concrete structures only, and e) is applicable to all structures.

The shear stiffness of any elastomeric bearing depends on the plan area and total thickness of the rubber, and the shear modulus of the elastomer. The shear stiffness is nearly constant for all temperatures above freezing but increases rapidly with decreasing temperatures due to crystallization of the rubber. At -40°C, the shear rate can be two times the stiffness at construction temperature.

In light of this, and since the various structure movements occur not only at its different ages (important when considering creep and shrinkage) but also at a variety of temperatures, the designer must use the bearing shear stiffnesses that correspond to each condition in order to design the substructure.

For bearing design data given on the contract drawings, the designer should give the shear stiffness rate that is assumed in the design of the substructure, at 20°C, denoted

K_{20}. Bearing catalogues provide the shear rates for standard laminated elastomeric bearings sizes.

The force that is transferred to the substructure for which it must be designed shall be determined as follows.

For temperature drop, even though the bearing will experience a sustained shear deformation over an extended period of time during cold temperatures, the bearing stiffness starts at near K_{20} and increases at the lower temperatures after much of the deformation has already occurred. The arithmetic mean of the full range of K values should be used. For temperature drop, $K = 1.35 * K_{20}$ should be used.

For temperature rise, since such bridge movements occur at a temperature above 15°C, K_{20} should be used.

A) Shear Rate for Concrete Structures

Because shrinkage and creep occur throughout the year and gradually over many seasons, it is expected that the bearing will undergo cycles of cooling and warming. Any force that is induced during colder periods will be relieved to that calculated with K_{20} when the temperature rises. Therefore, the horizontal force due to creep and shrinkage is calculated assuming K_{20}.

For post-tensioned bridge, the superstructure may be jacked up (at approximately three months after construction) to relieve all horizontal shear deformation that has taken place in the bearing pad until that time, returning the bearing to its originally undeformed configuration. This relieves bearing movement due to heat of hydration, elastic shortening and a portion of the total shrinkage and creep. After jacking, the bearing returns to vertical at the assumed construction temperature and must be designed for the remaining portion of shrinkage and creep, and all displacement due to temperature.

13.3.3.7 Elastomeric Bearing Design Data

For elastomeric bearings, the minimum design data provided on the contract drawings shall be as indicated below. Any special requirements must also be specified.

Table 13.3-3 – Design Data to be Provided in Contract Drawings
ELASTOMERIC BEARING DATA

LOCATION	ABUTMENTS	PIERS
TYPE	---	---
SIZE (mm)	__x__x__	__x__x__
NUMBER REQUIRED	---	---
SERVICEABILITY LIMIT STATES		
PERMANENT LOAD (kN)	---	---
TOTAL LOAD (kN)	---	---
MOVEMENT (mm)	± ___	± ___

ROTATION (radian)	---	---
SHEAR RATE at 20ºC (kN/mm)	---	---

The rotation value in the table shall be the imposed rotation required by the design, plus the tolerances for rotations due to manufacturing and construction as required by the CHBDC but shall be not less than ± 0.009 radians. Rotation in the bearing data table corresponds to total load.

13.3.3.8 Installation of Elastomeric Bearings

Elastomeric bearings are to be installed directly on the bearing seats. No filler or grout is to be shown on the drawings.

Precautions must be taken by the designer when there is the possibility of bearings being installed below 0ºC or above 20 ºC. The translational capacity of the elastomeric bearing shall be greater than the anticipated maximum movement.

Positive attachment shall be detailed on the contract drawings where required by the CHBDC.

Laminated elastomeric bearings supporting steel bridges shall be detailed with two pintles between the shoe plate and the top lamina of the bearing, with the detail shown in Figure 13.3-6. This prevents the structural steel from unseating from the bearing during construction. This is especially important for longer bridges where laminated elastomeric bearings cannot accommodate any movements through shear deformation until a greater axial load form the deck concrete is applied across the bearing to initiate frictional resistance to shear.

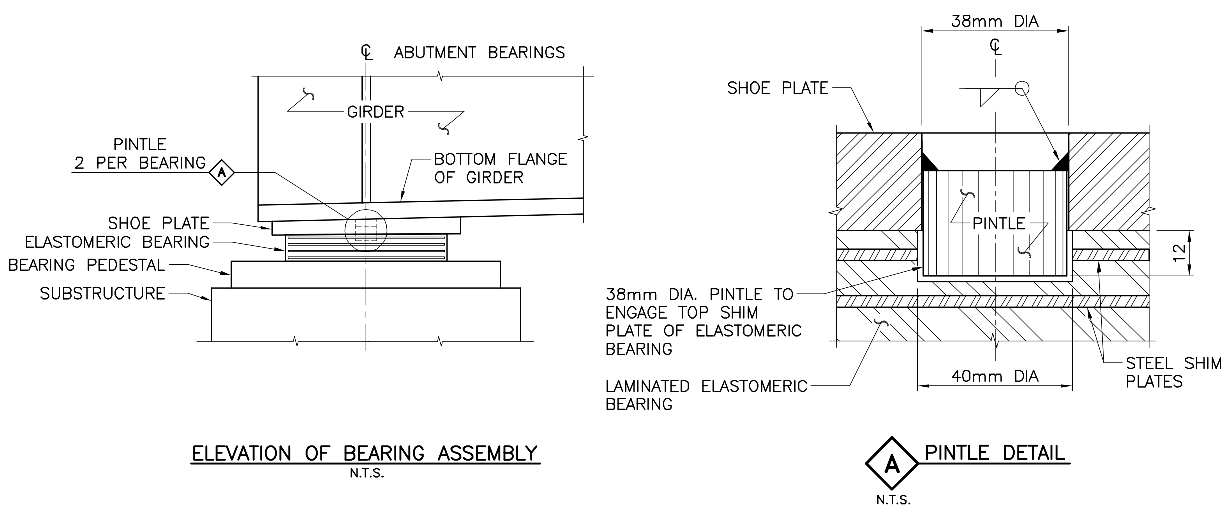


Figure 13.3-6 – Typical Pintle Detail for Laminated Elastomeric Bearings

13.3.4 Provision for Future Bearing Replacement

Enough space, both vertically and horizontally, must be provided between the superstructure and substructure to accommodate the required jacks for replacing the bearings. The vertical clearance for jacks shall be no less than 200 mm. The designer shall detail the jacking locations in the contract drawings based on the MTO Bearing Replacement and Jacking Guidelines.

Connections (e.g., between bearings and shoe plates) must be bolted or make use of accessible screws and should only be welded when future access and removability can be guaranteed.

Requirements and guidance on bearing replacement procedures and bridge jacking methods are provided in MTO document (BRO-062) – Bearing Replacement and Jacking Guidelines.

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SECTION 14 - CULVERTS

Table of Contents

14 CULVERTS

14.1 General

This section shall apply to:

a) all culverts (buried structures) with spans 3m and greater; and,

b) all cast-in-place culverts detailed in the MTO Concrete Culvert Design and Detailing Manual.

c) All culvert sizes specified in OPSS 1821,

d) all culverts contained in the future MTO Precast Concrete Box Culvert Manual.

On contract and working drawings the Culvert shall be identified as SPAN X RISE.

Where fill conditions permit, regardless of culverts shall be detailed with a minimum height of 2.4 m to facilitate access for construction, inspection and maintenance.

14.2 Concrete Culverts

14.2.1 Cast in Place Concrete Culverts

All cast in place concrete culverts should be designed and detailed in accordance with the latest version of the Concrete Culvert Design and Detailing Manual. In this manual the designs meet the requirements of the CHBDC and contains information necessary to complete the standard drawings and quantities for concrete box and open footing culverts. The following culvert types are covered:

a) Rigid frame open footing culverts with fill heights of 0.6 m to 6.0 m and sizes 2.0 m span x 1.25 m height to 6.0 m span x 4.0 m height;

b) Rigid frame box culverts with fill heights of 0.6 m to 6.0 m and sizes 2.0 m span x 1.25 m height to 6.0 m span x 4.0 m height; and,

c) Non-rigid frame box culverts with fill heights of 0.6 m to 5.0 m and sizes 1.25 m span x 1.25 m height to 1.5 m span x 1.5 m height.

Reduced scale copies of the standard drawings for all of the above culvert types are available in the “Concrete Culvert Design and Detailing Manual.”

14.2.2 Precast Concrete Culverts

Currently in MTO’s inventory, two categories of precast concrete culverts exist:

1. Culverts spanning 3m or less, which are fabricated and built according to OPSS 1821 “Material Specifications for Precast Reinforced Box Culverts and Box Sewers”, and typically use Dry-Cast concrete. These culverts are generally used in drainage and

sewerage applications. The design of these culverts, in specific span and height combinations to achieve maximum efficiency, is specified in the design tables which are included in OPSS 1821.
The culverts of this category are reinforced with welded wire fabric (WWF). The material requirements are contained in OPSS 1340 for Concrete and OPSS 1440 for WWF steel reinforcement. The construction specifications of these culverts are covered in OPSS 422.

2. Culverts spanning greater than 3m are typically designed for larger scale application for example high volume of stream flow or to act as a small bridge for multi-use (e.g., pedestrian, bicycle) traffic. Current practice is that the designer is required to perform project specific analysis and design of the large culverts. These culverts are to be reinforced either with black steel reinforcement or with WWF. Construction and material requirements are specified in OPSS 912.

MTO has recently published construction specification OPSS 912 for Culverts with Spans MTO has also issued design tables for this category of culverts in Memo # SCB-SO-2021-01 which will enable designers to use the design information directly from the tables on the contract drawings. Precast Concrete Box Culvert Manual containing design guidelines will be published in the near future.

Irrespective of the categories, all precast box culverts shall be designed in accordance with the CHBDC.

14.2.3 Waterproofing of Culverts

Concrete culverts with spans 3.0 m and greater shall be waterproofed irrespective of fill height. Hot applied rubberized asphalt waterproofing system is the standard treatment for the top surface of culverts with spans 3.0 m or greater. For precast concrete culverts, self-adhering waterproofing membrane shall be applied at the joints prior to application of the hot applied rubberized asphalt waterproofing on the top surface of the culvert. General waterproofing and joint waterproofing shall extend a distance down from the top as specified in SSP 599S30. Waterproofing need not be placed on areas beyond the limits of embankment fill or under a distribution slab, as specified in the SSP. Protection board is also added to protect the waterproofing from damage from backfill.

Waterproofing shall be specified for concrete culverts with a span 3.0 m or less for following situations:

a) where a 75-year service life is required;

b) where additional durability is required;

c) under high volume roads where culvert rehabilitation or replacement would be disruptive; or,

d) where use by pedestrians or snowmobilers during the winter is expected (where icicles can pose a danger to the public).

Hot-applied rubberized asphalt (HRA) waterproofing system is used by MTO for top surface waterproofing of all types of culverts. However, the self-adhering waterproofing membrane may also be used in special situations, such as in remote areas where labour and equipment to install hot-applied rubberized asphalt waterproofing membrane system becomes unavailable or deemed uneconomical. Use of self-adhering waterproofing system shall require the approval from the Structural Section in advance, prior to the start of waterproofing operation on site.

14.2.4 Earth Pressure

Culverts and other structures where the deflection of the side walls is prevented by the propping action of slabs shall be designed for the earth pressures specified in CHBDC.

14.2.5 Design of Concrete Culverts

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

Cast-in-place culverts and precast culverts with fill height less than 600 mm shall require reinforced concrete distribution slab above the culvert for distribution of live loads. The distribution slab shall be designed as per the CHBDC.

Design of culvert appurtenances such as headwalls, retaining walls or wingwalls are to be designed as per CHBDC. In the design, consideration should be given if the culvert appurtenances are cast-in-place or precast. For precast elements, design should consider ease of site installation of the element on the precast culvert with integral connection. Culvert cut-off wall or apron wall is considered a non-structural component.

14.3 Metal Culverts

14.3.1 Corrugated Metal Culverts

Corrugated Steel Culvert (CSP) and Structural Plate Corrugated Steel Pipe (SPCSP) terms are often used interchangeably. When used separately, the former is used typically for the smaller sized made of a continuous helical corrugated plate, while the latter is used for typically larger diameters made up of several plates around the circumference. Steel is the most common material, although aluminum is also available, typically for more severe environments.

Structural Plate Corrugated Steel Pipe Culverts (SPCSP) are fabricated by using hot-rolled sheets or plates that are corrugated, curved to radius, custom hot-dip galvanized or has a thermoplastic copolymer coating, assembled, and bolted together to form steel culverts, pipes, arches, pipe-arches, and other shapes.

Environmental factors shall be considered while checking the suitability of site for using SPCSP. Water and soil testing, as required by Clause 2.3.2.2 of the CHBDC, should be conducted in the spring, summer, and fall, at both the upstream and downstream pools.

The design shall account for environmental conditions that exist at the site or are likely to exist during the design service life of the structure. The anticipated steel material loss during the design service life, according to Table 2.5 of the CHBDC for the environmental conditions at the site, shall be added to the thickness required for structural load-carrying requirements. The MTO Gravity Pipe Design Guidelines may also be used as a referenced for guidance. Notwithstanding the requirements above, the thickness of the SPCSP shall not be less than 5 mm. The thickness of SPCSP and other steel components shall be determined according to CHBDC to ensure the culvert is structurally sound until the end of its service life.

All metal culverts irrespective of their span length must have adequate site preparation and compacted backfill material around them.

The use of pipe arches is restricted to sites at which round pipes cannot be accommodated. Other alternatives should also be considered. It is necessary that the installation of pipe arches make provision for the necessary backfill volume of engineered soils (granular 'A') on each side of the structure and that each installation have cut off walls or headwalls on each end.

MTO is currently working to create standards for metal culverts material and construction specifications. Until these standards are published designer must create non-standard specifications and seek for Structural Office's endorsement. For NSP preparation, MTO's Special Provision No. DBSP3271 could be referred to.

14.3.2 Corrugated Steel Pipe (CSP) Products Dimensioning

All dimensions of corrugated steel pipe products must be given in metric units, which are generally conversions of dimensions set in imperial.

14.3.3 Length of Structural Plate Pipes and Pipe Arches

The overall length of structural plate pipe and pipe arches shall always be detailed on the drawings and the dimension shown shall always be multiples of 610 mm, +100 mm.

e.g., 61,100 mm is acceptable.

This permits the use of an integral number of standard plate sizes, without cutting, and ensures that the hook bolts in the cut off walls will be aligned with the standard circumferential holes in the plates.

The spacing of hook bolts, if required, should be a multiple of 244 mm, to match the spacing of standard holes in the plates.

14.3.4 Durability

Joints, including seams, shall be constructed and treated to prevent leakage and infiltration. To prevent leakage through the culvert and associated passage of chloride contaminated water leading to corrosion, the culvert shall be covered by a waterproofing membrane. The membrane has a minimum thickness of 0.5 mm and is made of PVC, HDPE, or some other polymer materials. It is draped over the culvert to take water away from the culvert, with granular backfill between the liner and culvert to prevent punctures from bolts or other edges.

Structural Plate Corrugated Steel Pipe (SPCSP) and Corrugated Steel Pipe (Culverts) made of galvanized steel have shown signs of early corrosion in many environments in Ontario. Polymer-coated pipe and metallic coated bolts have been used in newer culverts. Structural Section and Structures Office shall be contacted to assist in determining the suitability of the metal pipe for the given soil and water conditions, as well as for requirements for the pipe system.

SECTION 15 - WOOD STRUCTURES

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15 WOOD STRUCTURES

15.1 Material Properties of Softwood

Properties of individual species are available, but for the purposes of design, some similar species and varieties of sub-species are grouped together such as Southern Yellow Pine (*Pinus spp.*), or the species combinations of CSA O86.

Hardwood is not commonly used for structural lumber but is very similar to softwood.

15.1.1 Structure of a Tree

Wood is an anisotropic, cellulosic, semicrystalline, cellular material. The material complexity may be simplified by understanding that wood is a biological structure that serves three functions in living plants:

1. To conduct water and nutrients from the roots to the leaves;

2. Mechanical support of the plant body;

3. Storage of biochemicals.

These functions have resulted in cells that are structured and interconnected in ways to meet the needs of the plant, which has implications on the properties and design of wood. The trunk is the useable portion of a tree for wood used in structures which may be divided generally into heartwood and sapwood (Figure 15.1-1).

Figure 15.1-1 – Macrostructure of Trunk (Bole)

Figure 15.1-2 – Principal Axes of Wood

Sapwood is the active conduction portion of the stem with live, metabolically active cells. The pores are more open and there is a higher moisture content. The sapwood is more prone to shrinkage and cracking, and is more susceptible to decay, but is easier to treat with preservatives.

The heartwood functions as long-term storage of biochemicals (extractives) which occludes pores and makes it more difficult to treat. The cells are dead, but it is more naturally decay resistant than sapwood and is denser, dryer, and stronger than sapwood (acts as the central, supporting pillar of the tree).

A review of the microstructure of the wood shows that discrete cells, typically many times longer than they are wide, are interconnected in two separate systems of cells, i.e., an axial system and a radial system. The axial system acts primarily as long-distance water movement, so it has the most free water in the cell lumina and provides the bulk of mechanical strength to the tree. The radial system provides lateral transport for biochemicals and in many cases performs a large fraction of the storage function in wood. More detail on the structure and properties of wood may be found elsewhere, such as Forest Products Laboratory, *Wood Handbook, Wood as an Engineering Material* (FPL-GTR-282).

The cell structure directly affects the properties of wood as a building material. As such, the mechanical properties (elastic, strength, and rheologic) exhibit strong orientation effects (axial and radial system) and are complicated by the addition of growth irregularities (e.g., branches are seen as knots in sawn lumber). Mechanically, clear wood obeys the laws of elastic orthotropic materials, and its failure characteristics are well described by strain energy of distortion-type theory. Because cell structure is predictable, wood may generally be described as orthotropic. The material properties vary according to the longitudinal, radial, and tangential axes with respect to fibre direction (

Figure 15.1-2).

15.1.2 Strength Properties

Common mechanical properties are measured and represented as strength properties for species grades in CSA O86 and are represented as parallel, at an angle to, or perpendicular to fibre direction (grain).

The reported properties reflect test mode and the gradation, shape, and size of test specimen. Remanufacture (ripping, resawing or dressing) of graded dimension lumber and timber should be avoided. Remanufacture can affect the rated properties of the member, voids the NLGA grade stamp and contractors are generally not in a position to re-grade the pieces. Specifically desired properties or shapes may be fabricated with engineered wood products.

15.1.3 Moisture

Wood exchanges moisture with the surrounding environment. The water content depends on the relative humidity and temperature of air and the current amount of water in the wood.

Shrinkage occurs when the water content drops below the fibre saturation point of the wood species. Average values for shrinkage vary for different wood species when drying from fibre saturation to service dry conditions, but there may also be a different percentage of moisture movement in each fibre direction for the same species (longitudinal direction of the stem / axial), radial direction (across annular rings) and tangential direction (along annular rings).

The ratio of tangential to radial shrinkage generally represents the uniformity of shrinkage of a species. The volumetric shrinkage indicates the magnitude of shrinkage. Longitudinal shrinkage is effectively zero. Shrinkage is roughly twice as fast in the end grain as the side grain.

At equilibrium moisture content, the wood is neither gaining nor losing moisture (or swelling or shrinking). Service dry wood (19%) is about the typical moisture content expected of in-service open decks (16-17%) and should shrink in a linear relationship to moisture content. Higher moisture contents exhibit a non-linear relationship.

To accommodate movement, connections need to consider orientation of wood with respect to adjoining wood, and the in-service moisture content expected at the site.

15.2 Design

Under suitable conditions, wood will give centuries of service. Unsuitable conditions include abiotic and biotic factors and result in degradation. Wood must be protected during processing, merchandising and in use.

In an unprotected environment, wood is susceptible to nonliving or physical agents such as heat, abrasion, ultraviolet light, and strong chemicals, which generally act slowly to decrease wood strength. However, the greatest hazard to timber bridges results from living or biotic agents such as decay fungi, bacteria, insects, and marine borers. Most biotic agents that enter and decay wood require four basic conditions for survival:

1. Free oxygen;

2. Suitable temperature (generally a range of 10-30 degrees Celsius. Decay effectively stops at less than 2 and greater than 38 degrees);

3. Sufficient water (a moisture level in the wood above the fibre saturation point which varies by wood species. On average, fibre saturation may be considered 25-30% and 28% for Canadian softwood species); and,

4. A food source.

Elimination of any one condition above can prevent wood decay, however oxygen supply and temperature cannot reasonably be controlled in the environment of a bridge structure. The food source may be poisoned with wood preservative treatments. Preservative treatment is the addition of a registered product to the wood and treatments come in two primary varieties based upon their composition. Preservative treatments can be applied through pressure-treatment, injection and surface coatings and affects a relatively shallow depth of wood members. Since preservative treatment could fail during the life of a structure, it should be combined with the elimination of as much water exposure as is practical.

Durability and longevity of outdoor treated wood articles is dramatically impacted by exposure of bright untreated wood by machining post treatment. Durability of wood is dependent upon limiting exposure of bright untreated wood to the elements. A durable wood design uses treated wood, deflects water from components, provides good drainage, and facilitates member drying.

15.2.1 Species

The availability of wood species which may be used in design is practically limited by what is available to post-treat wood after fabrication.

15.2.1.1 Permitted Species and Species Combinations for Sawn Wood

Only individual species and species combinations specified in CHBDC Table 9.15 are permitted for sawn wood.

Table 15.2-1 – Treatment Use Category for Wood Species

Species Combination	**Treatable Species[1]**	**Treatment[2]**	
		UC 4.1	**UC 4.2**
Douglas Fir-Larch	**Douglas Fir[4]**	ACQ, ACZA, CA-B, CCA, CR, CR-PS. CR-S, PCP-C	ACZA, CCA, CR, CR-S, PCP-C
	Western Larch[3]	ACZA, CCA, CR, CR-PS, CR-S, PCP-C	CR, CR-S, PCP-C
Hem-Fir	**Western Hemlock[3]**	ACQ, ACZA, CA-B, CCA, CR, CR-PS, CR-S, MCA, MCQ, PCP-C	ACQ-C, ACQ-D, ACZA, CA-B, CCA, CR, CR-S, MCA, PCP-C
	Amabilis Fir[3]		
Spruce-Pine-Fir (SPF)	**Lodgepole Pine**		
	Jack Pine		
	White Spruce	CR, CR-PS, CR-S, PCP-C	No
	Engelmann Spruce	ACQ-C, ACQ-D, CA-B, CR, CR-PS, CR-S, PCP-C	No
	Black Spruce	CR, CR-PS, CR-S, PCP-C	No
	Red Spruce		No
	Alpine Fir	ACQ-C, ACQ-D, CA-B, CR, CR-PS, CR-S, PCP-C	No
	Balsam Fir	CR, CR-PS, CR-S, PCP-C	No
Northern Species	**Ponderosa Pine**	ACQ, ACZA, CA-B, CCA, CR, CR-PS, CR-S, MCA, MCQ, PCP-C	ACQ-C, ACQ-D, ACZA, CA-B, CCA, CR, CR-S, MCA, PCP-C
	Red Pine		
	Western Red Cedar	No	No
	White Cedar	No	No
	Yellow Cedar	No	No
	Grand Fir	ACQ, ACZA, CA-B, CCA, CR, CR-PS, CR-S, MCA, MCQ, PCP-C	ACQ-C, ACQ-D, ACZA, CA-B, CCA, CR, CR-S, MCA, PCP-C
	Eastern Hemlock		
	Eastern White Pine		
	Western White Pine	ACQ, CA-B, CR, CR-PS, CR-S, MCA, MCQ, PCP-C	ACQ-C, ACQ-D, ACZA, CA-B, CCA, MCA
	Western White Spruce	ACQ-C, ACQ-D, CA-B, CR, CR-PS, CR-S, PCP-C	No

1 Bold species are specifically identified in CSA S6-19, Table 9.15 as treatable species for sawn wood. Western Hemlock and Lodgepole Pine may be treated with some difficulty.
2 CSA O80 Series Sawn Product treatment options for the Use Category specified in OPSS 1601.
3 Coastal species are identified separately under NLGA Grading Rules but are not a separate species combination for strength properties according to CSA O86. The differences between interior and coastal species affect the ability to treat wood with preservative.
4 Preferred wood species for structural characteristics but interior species are difficult to treat. Consider interior not treatable and only specify coastal Douglas Fir.

15.2.1.2 Permitted Species and Species Combinations for Glulam

Glulam is primarily produced in Canada as D.Fir-L or Spruce-Pine. Hem-Fir GLT may be available in certain areas and local fabricators should be consulted for product availability.

The post-gluing treatment options with oil-based preservatives limits availability of GLT products to Coastal Douglas Fir and Western Hemlock (Section 15.2.3.1 and Table 15.2-2).

15.2.2 Dimensions

Structural members can generally be fabricated to any size or shape. However, treatment with preservatives practically limits members to that which can be handled and will fit in the pressure treatment vessel, which varies by treatment plant. Weight of the treated member may be a limiting factor for some treatment plants.

To be generally treatable, individual members should be:

- Less than 12m in length;
- Up to 1m in depth;
- Less than 8000 kg.

The designer shall contact treatment plants for treatment ability for all curved shapes or when the dimensions or weight above are exceeded. (Note: the largest known member dimensions which may be treated in Ontario are 15m length and ~1.5m depth).

15.2.2.1 Sawn Timber

There are many different nominal and actual dimensions of sawn timber depending on the machining processes and moisture conditions. To avoid any ambiguity, all contract drawing dimensions shall be actual dimensions, in millimetres (e.g., 191x191), and standard sizes for sawn lumber or sawn timber.

Sawn timber shall not be specified as dressed.

Sawn timber is generally available up to 4.85 m length in sections up to 241 x 241 mm. For any CSA O86 sizes above these dimensions, the timber may be a special order and the designer should first contact a mill or consider glulam. Custom sizes not listed in CSA O86 should not be used.

15.2.2.2 Glulam

Beams are typically fabricated in standard finished widths of 80, 130, 175, 215, 265, 315 and 365 mm, corresponding with CSA O86 selection tables. Additional widths up to 515 mm are available according to CSA O122 but members wider than 365 mm are by special order.

Glulam member depth shall be specified in 38 mm increments. Glulam member depth may be specified in 19 mm increments for curved elements when permitted in CSA O86.

15.2.3 Pressure Preservative Treatment for Wood

All wood shall be pressure treated with preservatives according to CSA O80 Series and the accompanying American Wood Protection Association (AWPA) preservation standards in OPSS 1601.

Preservatives in CSA O80 are current at the time of publication and are subject to the product label. The AWPA specification U1 is updated annually and may have more up-to-date preservative treatments available than CSA O80. However, the production and use of wood preservatives in Canada is regulated by the Pest Management Regulatory Agency (PMRA) under the Pest Control Products Act (PCPA). Specific preservatives listed in CSA O80 or AWPA U1 may be restricted or may not be available during design. The designer shall check with preservative treatment suppliers about product availability before specifying treatments in the contract documents.

Inorganic Boron (SBX), also known as Disodium Octaborate Tetrahydrate (DOT) requires dual treatment where the SBX treated product is treated separately with an oil-borne preservative (typically Creosote). The oil-borne treatment is required to seal in the SBX which depletes and vapourizes when the moisture content exceeds about 22%.

Elements shall be specified in the drawing notes under the use category system. Each element shall identify the wood species and size, use category, and type of preservative (see Section 15.4).

15.2.3.1 Preservative Treatment of Glulam

Glulam shall be treated with an oil-based preservative (Table 15.2-2). The glulam laminations are individually kiln-dried and are effectively sealed during gluing to a service-dry moisture content. Adequately protected glulam can generally be considered to be in service-dry condition. Incising and treatment with water-born preservatives increases the moisture content of the product to a wet condition and may distort element dimensions and any machined details, impacting fit-up.

Table 15.2-2 – GLT Post-Gluing Preservative Treatments

Treatable Species	Stamp Identification	Treatment	
		UC 4.1	UC 4.2
Coastal Douglas Fir	D FIR(N)	CR, CR-PS, PCP-A, PCP-C	CR, CR-PS, PCP-A, PCP-C
Western Hemlock	W Hem(N)		

There is currently an issue with procuring oil-based preservative treated glulam:

- The sole North American supplier of Pentachlorophenol (PCP) shut down production in December 2021 and the PRMA product registration has ended, effectively banning the use of PCP to treat any new products.

- There are three Creosote preservative treatment plants in Canada, limiting availability. Wood may also be creosote treated in the USA.

- Creosote Petroleum Solution is not available in Canada. Wood may be treated in the USA.

- Copper Naphthenate (CuN) in hydrocarbon solvent is currently specified to UC 4.1 only. The CuN supplier has submitted information to the CSA O80 technical committee for review and may become a future alternative.

- 4,5-Dichloro-2n-Octyl-4-Isothiazolin-3-One (DCOI) is under review by PMRA as a potential future alternative to PCP.

15.2.3.2 Component Treatment Use Category Designation

Components shall be specified to the Use Category in Table 15.2-3.

Table 15.2-3 – Wood Component Use Category

Component	CSA O80 Use Category
Foundation elements	UC-4.2
Structure elements (except barrier)	UC-4.2
Structure barrier	UC-4.1
Guide Rail Posts	UC-4.1
Guide Rail Blocks	UC-4.1
Ground Mounted Sign Posts	UC-4.1
Fence Posts	UC-4.1

15.2.3.3 Fire Retardant Treated Wood

Pressure preservative treated processes are available in CSA O80 Series for fire-retardant-treated lumber, timber, and plywood. It is not expected that fire retardance of wood through chemical treatment will be required because:

- The occurrence of fires near or under bridges is relatively rare; and,

- Bridge members are generally large timbers that will have an inherent fire resistance because of the slow burning rate of large timbers and the insulating effects of the char layer.

Information about calculating fire resistance of wood members and assemblies may be found in American Wood Council Technical Report No. 10.

15.2.4 Decks

Decks on wood girders shall be glulam unless approved by the Head of Structural Section.

Wood-concrete composite decks shall not be used unless approved by the Head of Structural Section. MTO performance experience with wood-concrete decks has varied but poorly performing ones all experience rot at the wood/concrete interface and can only be ‘repaired’ by deck replacement. The detail in Figure 9.4 of CSA S6-19 shall not be used; it contains angled shear key reinforcing nails installed top down at the wood/concrete interface facilitating water penetration deeper into the wood laminates.

15.2.5 Traffic Barrier and Railing

TL-4 barrier crash tested to NCHRP Report 350 may be designed. References for design of this timber TL-4 barrier are:

Faller, R. K., Ritter, M. A., Rosson, B. T., Fowler, M. D., & Duwadi, S. R. (2000, April 3-5). Two Test Level 4 Bridge Railing and Transition Systems for Transverse Timber Deck Bridges. Transportation Research Record, 1(1696), 19.

Duren, J. T., Yosef, T. Y., Rosenbaugh, S. K., Faller, R. K., Bielenberg, R. W., & Steelman, J. S. (2023). Crash-Tested Bridge Railings and Transitions for Wood Bridges – Phase IIA. Forest Products Laboratory. U.S. Department of Agriculture.

15.3 Fabrication

All structural components shall be designed to be prefabricated and post-treated with preservatives. Machining and fastening post-treatment into bright wood is a significant contributor to deterioration of wood bridges and reduces service life below a 75-year target.

15.3.1 Shop Trial Assembly

The designer shall specify in the contract documents, girders and other main components requiring shop trial assemblies before and, if required, after preservative treatment.

A typical single span simply supported girder bridge may not require a shop trial assembly of fabricated elements prior to preservative treatment. However, wood bridges are not regularly constructed on MTO highways so pre-assembly should be considered for all glulam and dowel laminated (DLT) structures until there is general familiarity with them in design and construction.

Oil-based preservative treatments are more dimensionally stable and will typically not need a second trial assembly after preservative treatment.

15.3.2 Details

15.3.2.1 Changes in Cross Section

Always provide taper cuts at notches to reduce abrupt differential stiffness change. Taper cuts shall be slope cut not less than 1:12 to a maximum length of 300 mm or a maximum depth of D/4, where D is the depth of the member being cut. Taper cuts in glulam shall not be permitted in compression zones of the member.

Notches or abrupt changes in cross section introduce stress raisers which can promote cracking to bright wood. Notches and abrupt changes in cross section shall not be permitted unless a detailed assessment of the stress concentration effect has been made. Notches on the tension side shall also be verified for fracture shear resistance at the notch in accordance with CSA O86.

15.3.2.2 Accommodating Shrinkage and Expansion

Before fabrication of connector joints, members should be seasoned to a moisture content corresponding as nearly as practical to that which they will attain in service. The moisture content shall be indicated on the contract drawings.

15.3.2.3 Connections

The designer shall design all connections.

Connections of members and steel plates connecting members with their grain perpendicular to each other shall be provided with slotted holes.

Connections should consider shrinkage and swelling, notching effects, decay prevention and lateral restraint at supports. Examples of unacceptable connections that promote cracking, and preferred alternatives, are provided in CSA O86 and American Institute of Timber Construction (AITC) 104-2003, *Typical Construction Details*.

All holes should be pre-bored during fabrication so that they are effectively protected by post-treatment with preservatives. Through holes shall be horizontal. Fastener and hole information may be found in OPSS 907.

Vertical fasteners installed top down in structural elements or that penetrate or expose bright wood shall not be permitted. Figure 15.3-1 shows an example of bottom-up deck fastening that accommodates shrinkage. All through vertical fasteners should be avoided, but where used shall be in pre-machined, treated holes.

Flat washers shall be installed between the timber surface and the nuts of bolts, or between the timber surface and spring washers where spring washers are used.

Gauge lines for horizontal nailing shall be provided.
Installation torque of lag screws shall be based on the specific fastener, diameter and depth, and wood species specific gravity.

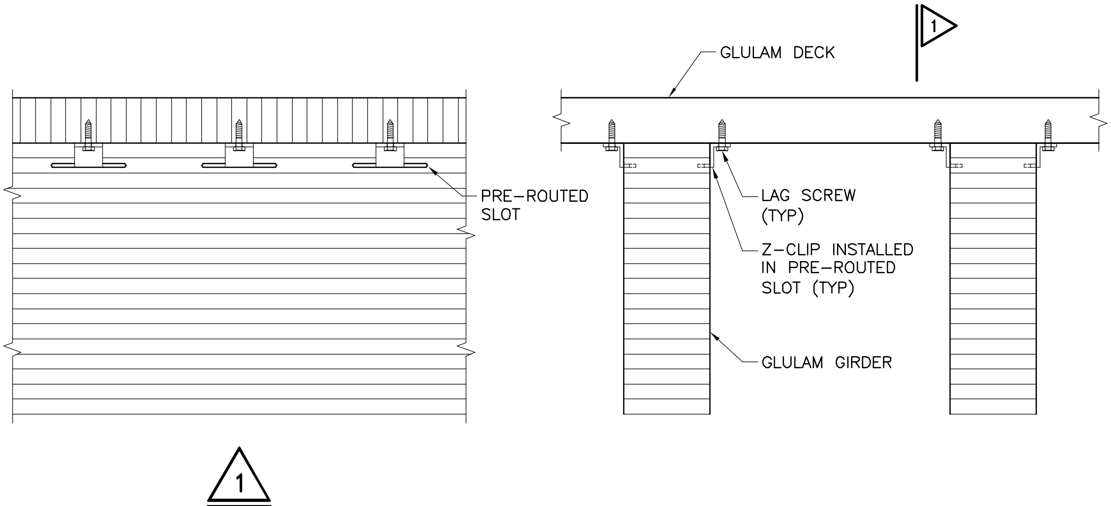

Figure 15.3-1 – Glulam Deck Fastening

Fasteners shall be the same material as the piece they are connecting and compatible with the preservative treatment used. E.g., copper nails with copper flashing; stainless steel nails with stainless steel flashing. Stainless steel type 304 or type 316 shall be used when fasteners are driven into wood treated with ACQ, CA, CCA or MCA.

Electrolytic corrosion is most often considered where dissimilar metals are in direct contact. However, the metals do not need to be in direct contact, they only need to be electrically connected. While wood is generally a good electrical insulator, under certain conditions wet wood may form an electrical connection between two dissimilar fasteners, leading to an anode/cathode relationship. If the fasteners are embedded, acidic and basic chemicals may accumulate at the node and cathode respectively. These chemicals may degrade the wood surrounding the fasteners. Fasteners should be the same material as embedded materials nearby.

15.3.3 Appearance

Appearance grade of glulam shall be industrial. Commercial grade glulam may be considered for elements within 5 m of a pedestrian's view and shall be clearly identified on the contract drawings when used.

Note: CWC appearance grade designations are different from APA classifications. If suppliers from the United States are anticipated because of species/treatment/size requirements, the appearance grade should be industrial or framing. Architectural grade may be considered for elements exposed to view.

15.3.4 Coatings

Wood with a minimum nominal dimension greater than 51 mm shall not be painted. Coatings shall not contain more than 29% solids. Stains are preferred to paint.
Coatings with greater than 29% solids have been historically used to protect rail elements with added benefit of visual appeal. These high-solid paints tend to crack over time as the wood expands and contracts, allowing moisture to infiltrate the member. The moisture becomes trapped under the paint causing areas of high moisture content, and often causing the paint to delaminate from the member. Thus, high solids coatings can often promote rather than prevent decay.

15.4 Wood Notes

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

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

A. General Notes

1. ALL WOOD SHALL CONFORM TO CSA O86.

2. ALL SAWN LUMBER AND TIMBER SHALL CONFORM TO CSA O141. SAWN LUMBER AND TIMBER SHALL CONFORM TO THE NLGA “STANDARD GRADING RULES FOR CANADIAN LUMBER”.

3. If the bridge uses glued-laminated timber, the following note shall be added:

   ALL GLUED-LAMINATED TIMBER (GLT OR GLULAM) SHALL CONFORM TO CSA O122. GLUED-LAMINATED TIMBERS SHALL BE INDUSTRIAL APPEARANCE GRADE UNLESS INDICATED OTHERWISE ON THE CONTRACT DRAWINGS.

4. If the bridge uses mechanically laminated timber, the following note shall be added:

   ALL MECHANICALLY LAMINATED TIMBER (MLT) SHALL BE DOWEL LAMINATED TIMBER (DLT) AND SHALL CONFORM TO CSA O125.

5. MOISTURE CONTENT (BY WEIGHT) OF LUMBER AND TIMBER SHALL NOT EXCEED 19% AND GLULAM SHALL NOT EXCEED 15% AT THE TIME OF INSTALLATION. ALL WOOD SHALL BE ADEQUATELY PROTECTED DURING ALL STAGES OF CONSTRUCTION TO ENSURE MOISTURE CONTENT REMAINS BELOW THE SPECIFIED LIMIT.

   (This note is about the maximum moisture content permissible of the wood product, this does not represent the expected moisture content of the site.)

6. NO CHAINS, HOOKS OR PEAVIES SHALL BE USED IN HANDLING TREATED WOOD.

7. FIELD DRILLING OR CUTTING NOT SHOWN IN THE CONTRACT DRAWINGS SHALL NOT BE PERMITTED. FIELD MODIFICATIONS SHALL NOT BE PERMITTED WITHOUT PRIOR APPROVAL FROM THE OWNER.

   (Field drilling and cutting into bright wood is a significant contributor to deterioration of a wood structure. If field drilling and cutting cannot be avoided, notes 8 and 9 shall always be included with note 7.)

8. ALL FIELD-DRILLED HOLES SHALL BE TREATED TO REFUSAL/SATURATION WITH COPPER NAPHTHENATE PRESERVATIVE ACCORDING TO OPSS 1601. TREAT HOLES AS SOON AS POSSIBLE AFTER DRILLING. ENSURE HOLES ARE CLEAN AND FREE OF SAWDUST OR DEBRIS PRIOR TO TREATING.

9. FIELD-CUT SURFACES SHALL BE TREATED TO REFUSAL/SATURATION WITH COPPER NAPHTHENATE PRESERVATIVE ACCORDING TO OPSS 1601. END GRAIN SHALL BE COATED WITH ANCHORSEAL® PARAFFIN SEALANT. TREAT BEAMS AS SOON AS POSSIBLE AFTER CUTTING. ENSURE CUTS ARE CLEAN AND FREE OF SAWDUST OR DEBRIS PRIOR TO TREATING.

10. If the bridge uses prefabricated deck panels, the following note shall be added:

    DECK PANELS MAY SHRINK OR EXPAND AFTER FABRICATION. DURING INSTALLATION, PERIODICALLY CHECK THE CUMULATIVE DISTANCE FROM THE END AND ADJUST GAP SIZE AS NEEDED. ORIENT ANY PANEL CURVATURE IN THE SAME DIRECTION TO MINIMIZE GAPS.

B. Connection Notes

1. ALL STEEL CONNECTION HARDWARE SHALL BE HOT DIP GALVANIZED ACCORDING TO ASTM A123. CONNECTOR PLATES SHALL BE FASTENED ACCORDING TO THE MANUFACTURER’S RECOMMENDATIONS TO DEVELOP THE FULL CAPACITY OF THE CONNECTOR.

   (Fasteners used with hot-dip galvanized plates shall also be hot-dip galvanized. If connector plates are stainless steel, fasteners shall also be stainless steel and the note above shall be modified.)

2. NO CHECKING OR SPLITTING SHALL BE PERMITTED AT AREAS TO BE BOLTED OR LAGGED.

3. A STANDARD FLAT WASHER SHALL BE PROVIDED WHERE A BOLT OR LAG SCREW HEAD OR NUT BEARS DIRECTLY ON WOOD. WASHERS SHALL BE HOT DIP GALVANIZED ACCORDING TO ASTM A123, OR STAINLESS STEEL ALLOY 304 OR 316 ACCORDING TO ASTM F594.

4. BOLT HOLES SHALL BE ACCURATELY ALIGNED AND DRILLED 1-2mm LARGER THAN THE BOLT DIAMETER. OVERSIZED HOLES SHALL NOT BE PERMITTED UNLESS INDICATED ON THE CONTRACT DRAWINGS.

5. If the bridge uses bolted connections, the following note shall be added:

   BOLTS FOR TIMBER CONNECTIONS SHALL BE TIGHTENED ONLY TO A SNUG-TIGHT FIT. LOCK WASHERS SHOULD BE FLATTENED AND THE NUT SHALL BE TURNED AN ADDITIONAL ¼ TO ½ TURN.

   WOOD SHALL NOT BE DEFORMED UNDER WASHERS OR STEEL PLATES DUE TO OVERTIGHTENING. UNLESS SPECIFIED OTHERWISE IN THE CONTRACT DOCUMENTS, BOLTED CONNECTIONS SHALL HAVE A MAXIMUM TORQUE OF 40 N-m.

   AFTER BOLTS ARE TIGHTENED, BURR OR EPOXY 3-5 THREADS ON THE CLEAN END OF THREADS. THE MINIMUM NUMBER OF CLEAR THREADS PAST THE NUT SHALL BE 3.

6. If the bridge uses lag screw connections, the following note as well as note 7 and 8 below shall be added:

   DRILL A PILOT HOLE TO A DEPTH EQUAL TO THE DISTANCE FROM THE INSIDE OF THE LAG SCREW HEAD TO THE SHOULDER OF THREADS (DOES NOT INCLUDE THE TIP).

   FOR REDUCED-BODY DIAMETER LAG SCREWS, THE DIAMETER OF PILOT HOLE ACCOMMODATING THE THREADED PORTION OF THE LAG SCREW (E.G. PARENT WOOD SEGMENT) SHALL BE 90% OF SHANK DIAMETER IN HARDWOODS OR 65% OF SHANK DIAMETER IN SOFTWOODS. THE DIAMETER OF PILOT HOLE NOT ACCOMMODATING THE THREADED PORTION (E.G. LAG SEGMENT) SHALL BE OVERSIZED BY 1.6mm OVER THE SHANK DIAMETER.

   FOR FULL DIAMETER LAG SCREWS, THE DIAMETER OF PILOT HOLE ACCOMMODATING THE THREADED PORTION OF THE LAG SCREW SHALL BE 90% OF THREAD OUTER DIAMETER IN HARDWOODS OR 65% OF THREAD OUTER DIAMETER IN SOFTWOODS. THE DIAMETER OF PILOT HOLE NOT ACCOMMODATING THE THREADED PORTION SHALL BE OVERSIZED BY 1.6mm OVER THE THREAD OUTER DIAMETER.

7. SUITABILITY OF PILOT HOLE DIAMETER FOR LAG SCREWS SHALL BE CONFIRMED ON SITE WITH A TRIAL HOLE. INSTALLATION SHALL NOT RESULT IN CLEAVING.

8. LAG SCREWS SHALL BE INSTALLED BY TURNING WITH A WRENCH. DRIVING WITH A HAMMER SHALL NOT BE PERMITTED. PETROLEUM-BASED LUBRICANT SHALL BE USED TO FACILITATE INSERTION OF LAG SCERWS. POWER-ASSISTED WRENCHES MAY BE USED FOR PRIMARY INSERTION OF

THE LAG SCREW, OR IF A CALIBRATED WRENCH IS SET TO THE TORQUE VALUE SPECIFIED IN THE CONTRACT DOCUMENTS.

FINAL TORQUING OF LAG SCREWS SHALL BE TO … N-m, COMPLETED WITH A TORQUE WRENCH OR SUITABLE TORQUE MEASURING DEVICE.

(Torque level is to be based on thread diameter, pitch, length, and species of wood. Insert the required torque value for each fastener specified).

9. WHEN USING NAIL GUNS, INCREASE THE NAIL SIZE TO ACHIEVE THE REQUIRED DIAMETER.

C. Wood Preservatives Notes

1. ALL STRUCTURAL WOOD ELEMENTS SHALL BE PRESSURE TREATED WITH PRESERVATIVES ACCORDING TO OPSS 1601 AND CSA O80 SERIES TO THE USE CATEGORY SPECIFIED IN OPSS 1601.

   (If elements will be incised, modify the note as below)

   ALL STRUCTURAL WOOD ELEMENTS SHALL BE INCISED AND PRESSURE TREATED WITH PRESERVATIVES ACCORDING TO OPSS 1601 AND CSA O80 SERIES TO THE USE CATEGORY SPECIFIED IN OPSS 1601.

   (If elements will be treated with products anticipated to only be available in the United States, include references to AWPA standards, e.g.)

   STRUCTURAL WOOD ELEMENTS PRESSURE TREATED WITH CREOSOTE SHALL BE TREATED IN ACCORDANCE WITH CSA O80 OR AWPA U1 AND AWPA T1 TO USE CATEGORY UC4.2 OR UC4B.

2. ALL MACHINING (E.G., CUTTING AND DRILLING) SHALL BE COMPLETED PRIOR TO PRESERVATIVE TREATMENT.

3. WOOD COMPONENTS SHALL BE:

   (List the unique wood components and specify their parameters. E.g., component, element size, timber species, Use Category and treatment preservative)

   DECKING – 38x140, LPP (LODGEPOLE PINE), UC4.2, PCP-C
   GIRDERS – 315x1712 GLT, D FIR (COASTAL DOUGLAS FIR), UC4.2, PCP-C
   POSTS – 140x140, RED PINE, UC4.1, CA-B

15.5 Maintenance Schedule

The designer shall prepare and submit a maintenance schedule that identifies:

- Required periodic maintenance;
- Inspection requirements;
- Component replacement.

Required periodic maintenance may include, but is not limited to the following:

- Time of first maintenance of galvanized components;
- Fastener tightening intervals. There should be at least one interval during the time units are coming to moisture equilibrium (normally during the first 2-3 years of service life);
- Post-tensioning re-tightening intervals;
- Cleaning moisture-trapping dirt and debris;
- Component replacement.

15.6 Sources of Information

15.6.1 Extra Design Information

Extra design information may be found in the AASHTO LRFDBDS-9, *LRFD Bridge Design Specifications*, 9th Edition and the American Wood Council ANSI/AWC *National Design Specification for Wood Construction*.

15.6.2 Historical MTO Research References

The Ministry directed its continuous bridge testing program towards wood structures in 1973 and produced research publications about wood structures until 1992. References are provided for convenience below.

Bakht, B. 1986. *Testing of Two Pin-Connected Truss Bridges.* Report, Structures Research Office, Ontario Ministry of Transportation, Downsview: Queen's Printer for Ontario, 53.

Csagoly, P F, and R J Taylor. 1979. *A Development Program for Wood Highway Bridges.* 1979 RTAC Annual Conference, Structures Research Office, Ministry of Transportation and Communications, Queen's Printer for Ontario, 65.

Csagoly, P F, and R J Taylor. 1986. *A Structural Wood System for Highway Bridges.* Paper, Structures Research Office, Ontario Ministry of Transportation, Downsview: Queen's Printer for Ontario, 31.

Doyle-Forintek, E, W Mortimer, H Walthert, R J Taylor, and J Wear. 1992. *Pressure Treated Wood in Canada: Task Force Report.* Jurisdictional Review, Research and Development Branch, Ontario Ministry of Transportation, Downsview: Queen's Printer for Ontario, 125.

Jaeger, L G, and B Bakht. 1986. *Analysis of Failure Loads of Timber Bridges.* Paper, Structures Research Office, Ontario Ministry of Transportation, Downsview: Queen's Printer for Ontario, 48.

Jaeger, L G, and Baidar Bakht. 1985. *Probabilistic Assessment of The Failure of Timber Bridges.* Presentation for 65th Annual Meeting of Transportation Research Board, Research and Development Branch, Ministry of Transportation and Communications, Downsview: Queen's Printer for Ontario, 21.

Maghsoudi, Akbar, and Baidar Bakht. 1982. *Simulation of Timber Bridges for Statistical Analysis.* Paper, Research Development Branch, Ministry of Transportation and Communications, Downsview: Queen's Printer for Ontario, 60.

Taylor, R J. 1987. *Applications in Prestressed Wood.* Report, Research and Development Branch, Ontario Ministry of Transportation, Downsview: Queen's Printer for Ontario, 29.

Taylor, R J. 1983. *Design of Prestressed Wood Bridges Using the Ontario Highway Bridge Design Code.* Guide, Structures Research Office, Ministry of Transportation and Communications, Downsview: Queen's Printer for Ontario, 27.

Taylor, R J. 1984. *Design of Prestressed Wood Bridges Using the Ontario Highway Bridge Design Code.* Guide, Structures Research Office, Ontario Ministry of Transportation, Downsview: Queen's Printer for Ontario, 35.

Taylor, R J. 1983. *Design of Wood Bridges Using the Ontario Highway Bridge Design Code.* Paper, Structures Research Office, Ontario Ministry of Transportation, Downsview: Queen's Printer for Ontario, 30.

Taylor, R J. 1983. *Draft of OHBDC Section 13, Wood, Code and Commentary.* Appendix to SRR-83-02, Structures Research Office, Ontario Ministry of Transportation, Downsview: Queen's Printer for Ontario, 123.

Taylor, R J. 1991. *Testing & Analysis of Short Span Wood Decks.* Paper, Structures Research Office, Ontario Ministry of Transportation, Downsview: Queen's Printer for Ontario, 60.

Taylor, R J. 1984. *Wood Bridge Calibration Study for the OHBDC.* Paper, Research and Development Branch, Ministry of Transportation and Communications, Downsview: Queen's Printer for Ontario, 44.

Taylor, R J, and H Walsh. 1983. *A Prototype Prestressed Wood Bridge.* Paper, Structures Research Office, Ontario Ministry of Transportation, Downsview: Queen's Printer for Ontario, 85.

Taylor, R J, and N Patel. 1986. *Load Testing and Analysis of Wood Bridges Rehabilitated by Transverse Post-Tensioning.* Paper, Structures Research Office, Ontario Ministry of Transportation, Downsview: Queen's Printer for Ontario, 37.

Taylor, R J, B deV Batchelor, and K van Dalen. 1983. *Prestressed Wood Bridges.* Paper, Structures Research Office, Ontario Ministry of Transportation, Downsview: Queen's Printer for Ontario, 23.

Tharmabala, T, and B Bakht. 1986. *Steel-Wood Composite Bridges.* Paper, Structures Research Office, Ontario Ministry of Transportation, Downsview: Queen's Printer for Ontario, 108.

15.6.3 Other Information

Other publications which have not been referenced above and may be informative, but shall not be taken as authoritative for the purposes of design of wood for MTO contracts may be found from the following sources:

- Ontario Wood Bridge Reference Guide (2017);
- The Canadian Wood Council;
- WoodWorks;
- The Engineered Wood Association;
- The American Wood Protection Association;
- The American Institute of Timber Construction;
- USDA Forest Products Laboratory.

SECTION 16 - MISCELLANEOUS

Table of Contents

16 MISCELLANEOUS

16.1 Utilities

16.1.1 Services Carried Through or Under Bridges

Fluid carrying pipelines are not normally allowed to be carried through or under bridges, unless approved. This includes oil and gas pipelines, sanitary and storm sewers, and water mains.

Electrical power lines may be carried through, under, or over bridges provided the voltage does not exceed 44,000 volts.

The following details show approved details for the accommodation of non MTO utilities on bridges. Bridges shall be used for this purpose only as a last resort and after the proponent has carried out a cost benefit analysis. Schemes other than those shown below may be considered, subject to the approval of MTO.

Utility plant is not allowed in sidewalks or to be directly suspended from thin deck slabs. Also, utility hardware must not be placed in a location which prohibits routine inspection of structural components. In any case, it must not be located below the underside of girders for slab on girder bridges, or the underside of deck for slab type bridges.

Steel components that make up the duct support system for bridges having an all-concrete superstructure or for bridges having painted steel girders, must be stainless steel, galvanised, metallised, or painted. For bridges having ACR steel girders, all non-embedded components may be fabricated of the same steel. Attachments shall not be welded to flanges of superstructure girders.

Where necessary, the designer must also provide special details to allow the utility ducts to pass through the abutment ballast walls and superstructure diaphragms. These details must ensure:

1. That the ducts will be able to accommodate all prescribed structure movements including deck jacking for bearing repairs;

2. That fill material behind the ballast walls is not washed out; and,

3. Any settlement that may occur in the fill material behind the ballast wall does not impose undue stresses on the ducts. When the utilities are run through a steel box girder, the designer must consider the location of intermediate cross braces, vertical stiffeners in pier diaphragms, access openings in pier diaphragms, etc. All utilities shall be designed with provisions to allow for future bridge maintenance.

Examples of how ducts could be accommodated through or carried by a structure may include the following cases:

A. Duct location for structures with accessible cavities or spaces located within the limits of the of the superstructure.

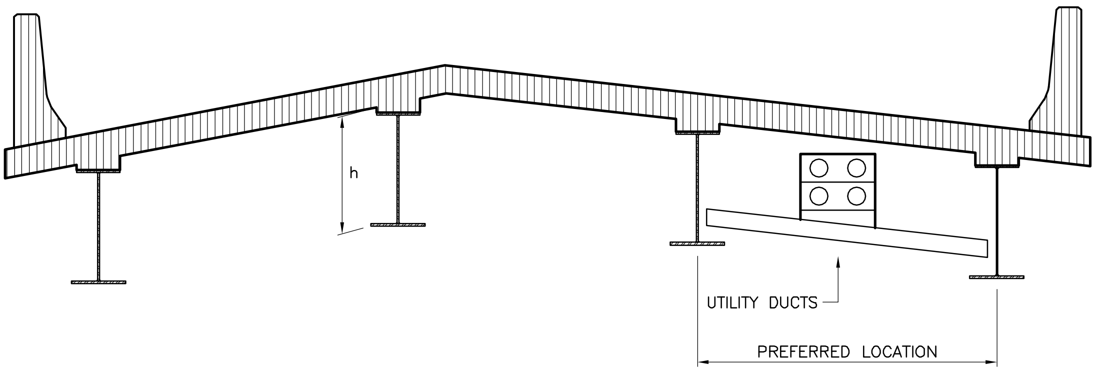

Figure 16.1-1 – Slab on Concrete/Steel I Type Girders

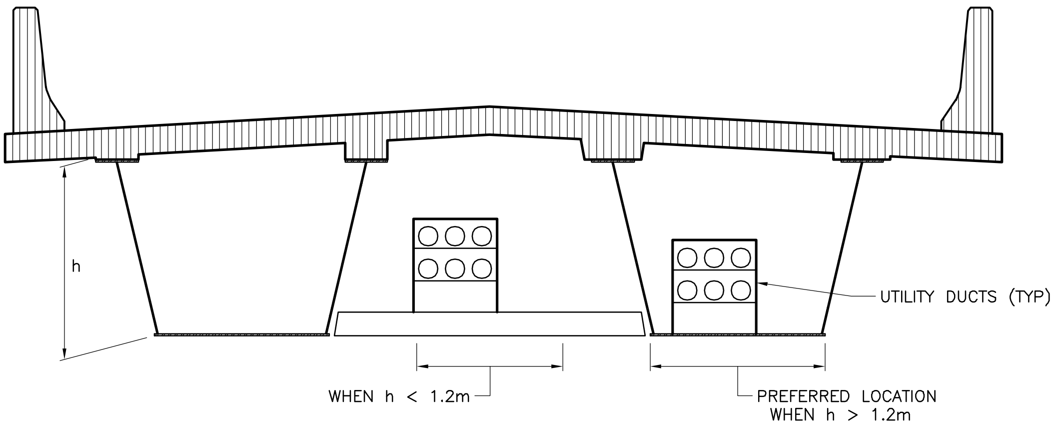

Figure 16.1-2 – Slab on Concrete/Steel Box Girders

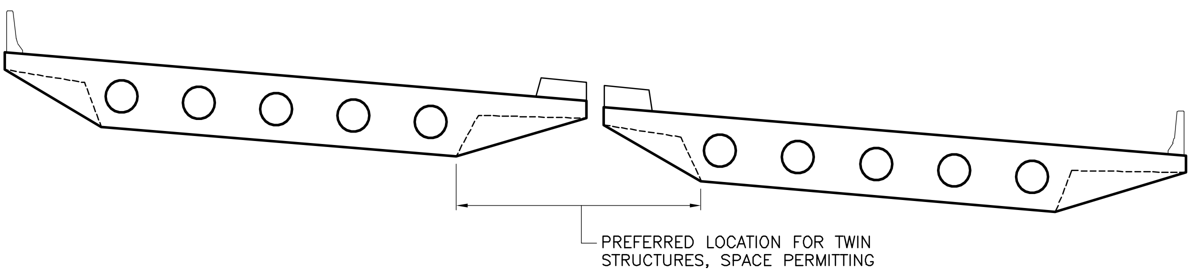

Figure 16.1-3 – Twinned Round Voided Post Tensioned Decks

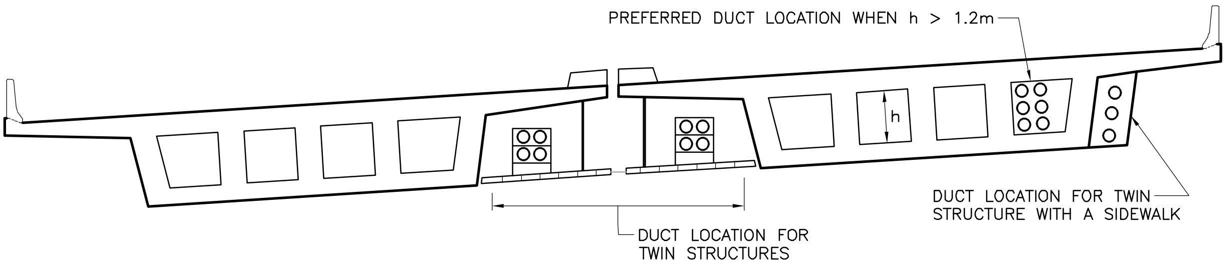

Figure 16.1-4 Post Tensioned Decks with Trapezoidal Voids

B. Duct location in slab type structures, (superstructures without accessible cavities).

Projects covered by this criterion require the construction of a non-structural utility corridor outside of and adjacent to the barrier wall. Structure types included in this category include rigid frames, culverts, and solid or round voided post tensioned concrete structures. This scheme is best suited for structures without sidewalks or situated in non-urban locations because of the possibility of pedestrians climbing up onto the utility corridor.

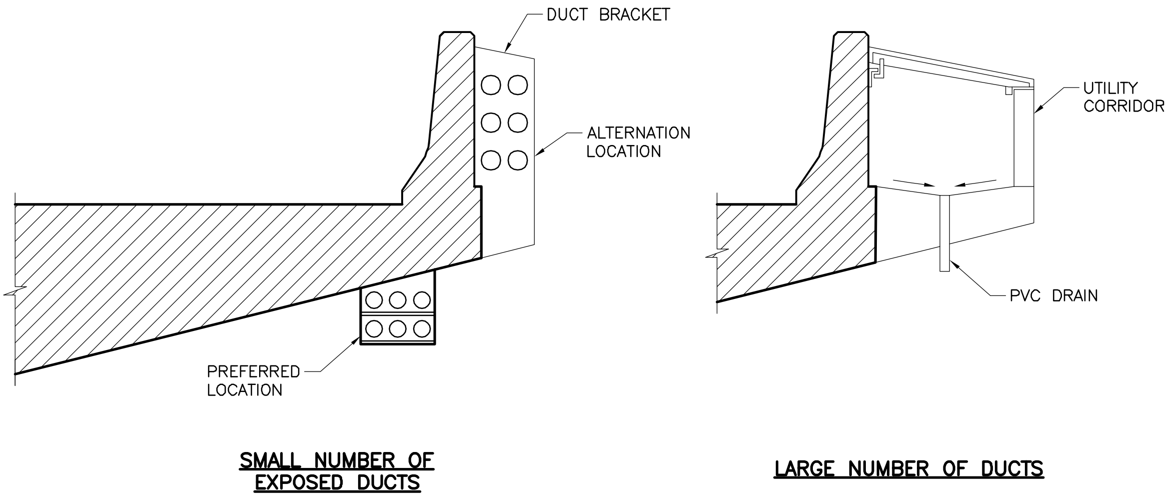

Figure 16.1-5 – Cantilevered Utility Corridor

Electrical lighting ducts (including MTO utilities) shall not exceed 50 mm diameter, except when installed in barrier walls, and shall be placed in accordance with the following criteria:

1. In abutments, ballast walls, slab piers and rigid frames, members must be greater than 500 mm in thickness and spacing of adjacent ducts shall be not less than 6 m.

2. In round columns:
   - Maximum of one electrical duct, outside diameter not exceeding 50 mm, may be placed at centre of columns having a diameter of 1000 mm or more.
   - Junction boxes are not permitted in columns.
   - Entrance/exit of duct at the bottom of columns to be through the footing.
   - Entrance/exit of duct at the top of columns:
     - for fixed-fixed columns (no bearings at top) the duct shall pass straight through the deck column interface.
     - for columns with bearings at top, duct may pass through side of column, provided it does not interfere with reinforcement or bearing assembly and is kept clear of the zone of high stress near the bearings.

3. In post tensioned decks:
   - Ducts and junction boxes may be placed in post tensioned decks provided they are located in solid portions of the deck adjacent to supports; they must be placed above the bottom mat of reinforcement (and transverse post tensioning, when present).

- Longitudinal ducts placed in post tensioned decks may be located inside deck voids, but in any case, shall not be located in web portions of the deck.

When ducts are to be placed in such elements, positive drainage of the embedded duct work must be assured, and the ducts must be located behind the steel reinforcement. The structural Engineer shall also give consideration to the location of junction boxes necessary in such installations and to the potential effects of duct bursting on the structural integrity of the component(s). Electrical ducts left unused would need to be grouted as is required with unused post tensioning ducts.

Other services may be provided for only as a result of specific approvals and agreements. These, and approval for exemption from above restrictions, should be referred to the Assistant Deputy Minister of Operations.

16.1.2 Grounding for Electrical Safety of Structures

16.1.2.1 General

Where applicable, and in accordance with the guidance provided below, all grounding systems shall be designed and installed in full compliance with the Ontario Electrical Safety Code (OESC).

16.1.2.2 Lightening Protection

Electrical grounding for lightening protection is only required for:

1. High level bridges such as Skyway Bridges or
2. Bridges located in lightning-prone regions

16.1.2.3 Protection Against Electrical Faults

Grounding shall be provided on bridge structures in the following situations to ensure protection against electrical faults and compliance with the Ontario Electrical Safety Code:

- Bridges carrying live electrical wiring
- Bridges with provisions for future electrical installations, especially in staged construction projects, in accordance with the Ministry Policy for Highway Illumination.
- Bridges crossed by or adjacent to overhead high-voltage transmission lines, where grounding is required to mitigate induced voltages and ensure public safety.

Traffic Management and Engineering Office, Electrical Engineering Section may be consulted for questions regarding electrical safety of bridge structures.

16.2 Transportation and Fabricator Handling of Structural Components

Prefabricated, indivisible structural components that exceed any of the following limitations typically require special oversize/overweight hauling permit(s) in order to be transported by highway carrier over King’s highways.

Length .... 19.0 m

Width .... 3.5 m

Height .... 2.6 m

Weight .... 30,000 kg

For the following two categories: (A) Routine oversize/overweight loads, and (B) Non routine oversize/overweight loads, pre-approval for King's highways transportation is not required. The length/width/height/weight limitations for both categories are detailed in SSP No. 109F16.

Components exceeding the limitations of categories (A) or (B) require that the designer of the load obtain transportation approvals from the MTO Structural Section and the Weight and Load Engineer, Carrier safety and Enforcement Branch, MTO and other authorities, all as required by SSP No. 109F16. As such, designers should attempt to design component size to fit into one of the categories (A) or (B) when possible, by the positioning of girder and component field splices. Loads over 42 m in length shall only be used when approved by the MTO Structural Section. Approximate load lengths are provided in Table 16.2-1, using typical transport equipment. The values may vary depending on the magnitude, width, height, and stability of the load.

In terms of width, the load should be kept to a width of less than 3.8 m, or up to 4.3 m if required. Loads wider than 5.0 m require a Superload permit, while loads between 4.3 m and 5.0 m fall into category (B). Design of structural components necessitating loads wider than 4.3 m requires approval of the MTO Structural Section.

In terms of height, a long load (girder) is typically placed on the tractor about 1.5 m above the ground. Thus, girders above 2.65 m in depth (including projecting shear connectors and upwards camber) likely require a permit. Highway clearances are typically limited to between 4.5 m and 5.0 m, depending on the route. Design of girders with depth (including projecting shear connectors and upwards camber) greater than 3.0 m requires approval of the MTO Structural Section.

In terms of weight, Permit (A) typically allow for a load of up to 50 tonnes, while permit (B) allows for permits up to 80 tonnes – although the values may vary somewhat because the total gross vehicle and load weight is permitted, and the exact truck configuration depends on the magnitude, width, height, and stability of the load. Superloads, with total gross weight over 120 tonnes, require evaluation of all bridges and culverts crossed along the route of travel, and shall only be used with approval of the MTO Structural Section.

Loads requiring no permit, or the shorter ones requiring permit, are typically carried on flatbed or extendable flatbed trailers. As the loads get longer, either a steerable pole trailer is used (where the back axles are connected to the tractor), or separate steerable rear dolly (relying exclusively on the load (girder) for connection. Shorter and heavier loads typically increase the length to spread the load out over more axles and a greater distance. Some possible truck combinations are shown in Figure 16.2-1.

Table 16.2-1 – Approximate Maximum Load Lengths by Permit Category

Permit Category	Condition	Max. Overall Length (m) **	Max. Load Length (m)
No Permit	No Condition *	23.00 m	~ 18 m
(A) Routine Oversize/Overweight Permit	No Escort *	36.75 m	~ 31 m
(B) Non-Routine Oversize/Overweight Permit	1 Escorts *	46.75 m	~ 40 m
(B) Non-Routine Oversize/Overweight Permit SuperLoad	2 Escorts *	55.00 m	~ 50 m

*   - Additional escort may be required if load overhand exceeds 4.65 m.
** - Overall length from front bumper to end of trailer or load.

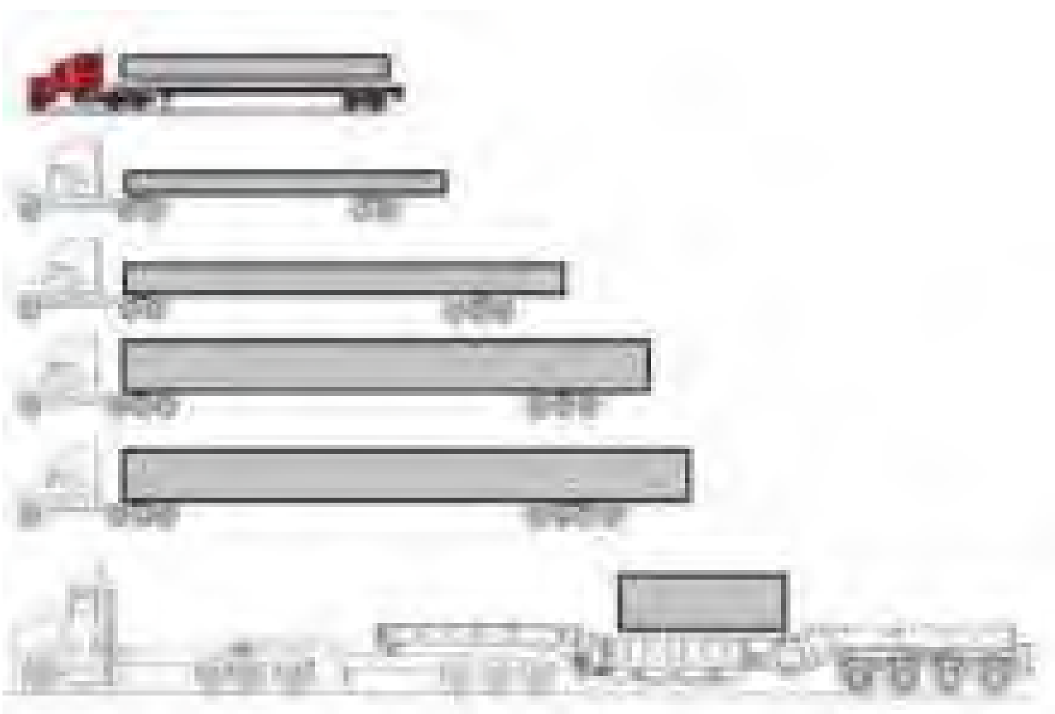

Figure 16.2-1 – Vehicle Configurations for Carrying Loads of Various Weights and Lengths

16.3 Modular Bridges

16.3.1 Temporary Modular Bridge (TMB)

The use of temporary modular bridge (TMB) (such as Acrow, Bailey, Mabey, AIL, etc.) is primarily for installations as detours during new bridge construction or for emergency situations with existing bridges. The use of a TMB as a permanent structure shall be subject to the following:

- Consideration of the volume of traffic at the bridge site Fatigue design as per the CHBDC; and,
- The required traffic barrier crash test level (e.g., TL-3, TL-4, etc.).

The responsibility and control of the supply, maintenance, and inventory of all TMB components under the jurisdiction and ownership of MTO is governed by the following Policy, Planning & Standards Directive PHM-B-22:

"Temporary Modular Bridges": *(A) Issuing Priorities*
(B) Supply, Maintenance and Inventory Control
(C) Retrieval, Rental and Disposal

A summary of the contents of this document is given in this subsection.

16.3.2 MTO's TMB Policy for Municipalities and other Agencies

The Ministry will only supply TMB to municipalities when there is a real emergency. The Ministry will not supply TMB to municipalities for detour purposes when new bridges are being constructed or for bridge rehabilitations.

Municipalities that need TMB components in an emergency shall initially attempt to obtain their requirements from the private sector. If the private sector is unable to supply the components within one week from the date of the emergency, the municipality may request them from the Ministry. When submitting a request to the Ministry, the municipality must provide evidence that they were unable to obtain the TMB components in a timely manner. The procedure to follow when making an emergency request and details of a legal TMB rental/purchase agreement that must be entered into is given in Directive PLNG–B-007.

The Ministry will not supply TMB to any other jurisdictions.

16.3.3 Policy for Issuing Priorities of TMBs

The Regional Director for the region in which the TMB is required is responsible for approving the release of TMB. The Manager of Operations, Northern Region, North Bay is responsible for issuing TMB. Issuing priorities will be in the following order of precedence:

1. Emergency needs on the Ministry's Provincial Highway System.

2. Emergency needs in municipalities.

3. Detour needs on the Ministry's Provincial Highway System.

16.3.4 Emergency Needs Policy for TMB's

To ensure a responsible supply of TMB components sufficient to meet most anticipated emergencies, the Ministry maintains and services an emergency stock of TMB components at a stockyard in Northern Region.

An emergency may exist due to failure of any bridge component or collapse of an existing bridge that leads to a permanent closure of the bridge. An emergency requiring the issue of TMB components is deemed to exist when one or more of the following conditions are satisfied:

a) An existing structure has been damaged beyond immediate repair;

b) The road is completely closed to traffic due to damage to the bridge;

c) The shortest detour that exists exceeds what would normally be considered reasonable for purposes of emergency response (i.e., fire, police, and ambulance);

d) The time required to make repairs to the existing structure is considered excessive;

e) Alternative solutions, other than the supply of TMB components are not viable.

The approval procedure for emergency requests at Ministry and municipality sites, prior to the release of TMB components, shall be as detailed in Directive PLNG-B-007.

16.3.5 Policy for Detours and Non-Emergency Needs

At present, the Ministry maintains and services a portion of the existing inventory of TMB components for use as detours at Ministry construction sites. This stock is kept separate from that intended for emergency use. However, this detour stock may be depleted by attrition with time and the supply of TMB components for operational needs will eventually be obtained from the private sector.

The need for a TMB and sufficient information for design must be clearly stated in a Structural Planning Report. This will be the Structural Planning Report for a structural project for which a TMB detour is required or, if the TMB is not part of such a project, a separate report.

TMB drawings, quantities, and appropriate tender items are to be prepared by the Regional Structural Section responsible for the project. Site numbers should be assigned according to Section 2.5.1, e.g., 34E-2221/B0 in the title block.

During the preliminary stages of design, the Regional Structural Section shall obtain confirmation from the Manager of Operations, Northern Region that the required quantities of TMB components will be available when needed. If this is not possible then consideration shall be given to alternative designs from the private sector.

The approval procedure, bills of materials and requests for the release of TMB components shall be as detailed in Directive PLNG-B-007.

16.3.6 Use of Bailey Bridges as TMB

Bailey Bridges have been used in Ontario for over a half century. Due to their modular components, Bailey bridges can be assembled in numerous ways, with multiple truss panels either adjacent or on top of each other. Similarly, the floor system is made up of a number of adjacent stringer sets (depending on the width of bridge) and a number of transverse floor transoms, usually 2 or 4 per 10 ft (3 m) bay.

For Bailey Bridges with two floor transoms per bay, an evaluation according to the CHBDC found that the load carrying capacity of the floor system was deficient. Subsequent load testing by the MTO Structures Office confirmed that the floor system was incapable of carrying full CHBDC CL-625-Ont loading.

Consequently, all new Bailey Bridge installations shall be specified with four transverse transoms per bay when load posting is not desirable and when two transverse transoms are specified, the bridge shall have a triple load posting of 25 tonnes, 40 tonnes, and 55 tonnes for a single, two unit and multi-vehicle train respectively.

16.4 High Mast Lighting Poles

16.4.1 Design of High Mast Lighting Pole

The design of high mast lighting poles (HMLP) and their foundations shall be based on the requirements of the CHBDC. A reference wind pressure for a 50-year return period shall be used. AASHTO's "Standard Specifications for Structural Supports for Highway Signs, Luminaries and Traffic Signals" shall also be used for design requirements, for example fatigue design, not specified by the CHBDC. However, where there is a conflict the CHBDC shall govern.

On Ministry projects pole heights of 25, 30, 35, 40 and 45 m are currently used and the design for these high mast lighting supporting structures has already been carried out. MTO issued standard drawings showing details of these structures. General details are shown on the following MTO standards:

- OPSD 2450.011 HMLP 25, 30 & 35 m 8-Sided Pole;
- OPSD 2450.021 HMLP 40 & 45 m 12-Sided Pole.

The installation of high mast poles is covered by OPSS 630 "Construction Specification for the Installation of High Mast Poles" and related special provisions.

High mast lighting supports have fixed based support systems that do not yield or break away upon impact. The large mass of the poles and the potential safety consequences of them falling to the ground necessitate a fixed base design. Since fixed base systems are rigid obstacles, they should not be used in the roadside clear zone unless protected by a barrier or approved by the Ministry.

16.4.2 Foundations of High Mast Lighting Pole

An anchorage assembly detailed on the following standards connects the high mast lighting support to a concrete caisson foundation:

- OPSD 2218.010 HMP Anchorage Assembly Placement;
- OPSD 2456.011 HMLP Anchorage Assembly Details.

The design of foundations shall be based on the requirements stated in Section 16.4.1.

Foundations for high mast poles may be designed according to the Structures Office publication BRO-009, "Guidelines for the Design of High Mast Pole Foundations, third edition," which is based on the requirements of Section 16.4.1. This guide presents the design of concrete caisson type foundations in a variety of soil conditions including rock and layered soils. The designs and design methods given in this document are modeled on short piles with a rotational limit of 0.005 radians at the ground surface, and the theoretical analyses given in papers by Bengt, Broms and others.

The following Structural Standard Drawings for high mast pole foundations that are ground or median mounted are available for use:

- SS116-50 High Mast Lighting Pole Footing Ground Mounted;
- SS116-51 High Mast Lighting Pole Footing Median Tall Mounted (Symmetrical);
- SS116-52 High Mast Lighting Pole Footing Median Tall Mounted (Asymmetrical).

The construction of foundations for high mast poles is covered by OPSS 631 "Construction Specification for Concrete Footings and Maintenance Platforms for High Mast Lighting Poles" and related special provisions.

16.5 Pedestrian, Bicycle and MUP Bridges

This section provides guidelines for the design of pedestrian and bicycle bridges. Pedestrian, Bicycle and MUP bridges shall be designed following CSA S7-23 – Pedestrian, cycling, and multiuse bridge design guideline. CSA S7-23 relies heavily on CSA S6 (CHBDC). In addition to those requirements, the requirements of Section 16.5.1 shall be followed.

16.5.1 Loads

Where the clear width of the bridge is ≥2.4 m and <3 m, the bridge shall be designed for a load equal to 50% of the maintenance vehicle load specified in CSA S7, while for bridges with clear width ≥3.0 m, they shall be designed for 100% of the maintenance vehicle load.

16.6 Ethylene Vinyl Acetate (EVA) Foam

16.6.1 Overview

EVA foam is a flexible cellular expanded rubber foam product, usually composed of Ethylene-Vinyl Acetate/Polyethylene. It is typically used for providing a separation between adjacent concrete elements.

OPSS 920 has historically classified EVA foam only as a joint seal in accordance with

ASTM D1056. It has been used as a joint seal between twin structures, but its use has recently been more common in other bridge details such as semi-integral conversions. ASTM D1056 allows for many different types, classes, and grades of EVA foam. There is a need to better define the EVA foam material on MTO bridge projects to ensure it is supplied and installed to meet the performance requirements of the design.

Designers shall include Standard Special Provision SSP 109S18 in MTO bridge projects. The SSP amends OPSS 904 and provides material and construction requirements for EVA foam.

Designers shall designate the type and grade of EVA foam according to guidance described below.

16.6.2 Material Properties and Classifications

ASTM D1056-14 gives the requirements for EVA foam, and classifies the material based on Type, Class, and Grade. There are two main Types of EVA foam which can be specified. These include:

1. **Type 1: Open-Cell Rubber** - A product whose cells are not totally enclosed by its walls and open to the surface, either directly or by interconnecting with other cells;

2. **Type 2: Closed-Cell Rubber** - A product whose cells are totally enclosed by its walls and hence not interconnecting with other cells.

Both types are further classified into four different Classes of material. All Classes are made from either natural, synthetic, reclaimed, or rubber-like materials, but each have various oil and/or temperature interaction requirements. These Classes according to ASTM D1056-14 are:

- Class A: Specific resistance to the action of petroleum base oils is not required.
- Class B: Specific requirements for oil resistance with low mass change.
- Class C: Specific requirements for oil resistance with medium mass change.
- Class D: Specific requirements for extreme temperature resistance (-75 to 175°C), but specific requirements for petroleum-based oils is not required.

Types and Classes of EVA foam are then further divided into different Grades, which specify the range of stiffness for the material based on compression of 25% of the thickness. This is an important property of the material which the designer must consider when calculating forces acting on the ballast wall due to thermal movements of a semi-integral downturn. These Grades (according to ASTM D1056-14), as well as the calculated unfactored elastic modulus values are:

Table 16.6-1 – Properties of EVA Foam

Grade	Compression-Deflection Range (kPa)	Avg. Modulus of Elasticity, E_{avg} (kPa)	Max. Modulus of Elasticity, E_{max} (kPa)
0	0 - 13.8	27.6	55.2
1	13.8 - 34.5	96.6	138
2	34.5 - 62.1	193	248
3	62.1 - 89.6	303	358
4	89.6 - 117.2	414	469
5	117.2 - 172.4	579	690

The average modulus of elasticity is based on the midrange value of the compression deflection range, which is used for estimating deflections across the material. The maximum modulus of elasticity is based on the upper bound value in the range, which is used for determining design forces transferred to an adjacent element across the EVA foam. Both values are based on 25% compression, since they are calculated from the specified compression deflection ranges given in ASTM D105614.

16.6.3 MTO's Best Practices and Preferences

MTO preference is to use Type 2 (Closed Cell) and Class A (no specific oil or temperature requirements) EVA foam on bridge projects. When EVA foam is not explicitly classified, Contract special provisions will require the contractor to supply Type 2, Class A, and either Grade 2 or 3 EVA foam by default. The designer may specify a different grade, but this should be limited only to areas where a specific grade is required.

It is not economical to specify numerous types, classes, and grades of EVA foam on bridge projects. Since Type 2 or 3 and Class A is preferred, designers should make an attempt to limit the number of grades required.

It is preferred to use a single thickness of EVA foam rather than layering different sizes together. There are many thicknesses available, but the most common are 20, 25, 30, 40, 50, 75, and 100 mm. If greater than 100 mm is required, the designer should specify a total thickness that can be built up using multiple layers of one of the most common thicknesses. The width and height are usually cut from sheets, so these dimensions can be prescribed as required.

16.6.4 Design Considerations for EVA Foam in MTO's Projects

EVA Foam Supporting Dead Loads

The designer should consider the stiffness of EVA foam when it is used to support dead loads such as approach slabs resting on wingwalls.

The compressive deflection of EVA foam supporting dead loads should not exceed 10% of its specified thickness as a best practice. The following table gives the maximum amount of dead load that each grade of EVA foam can support, based on the calculated average modulus of elasticity (E_{avg}) and a maximum strain of 0.1:

Table 16.6-2 – EVA Foam Maximum Dead Load Capacity

Grade	Maximum Dead Load (kPa)
0	2.8
1	9.7
2	19.0
3	30.0
4	41.0
5	58.0

EVA Foam Accommodating Movements

Design forces transmitted to structural components through EVA foam need to be considered in design. The maximum modulus of elasticity should be used in calculating the applied forces as follows:

$$\sigma = E_{max} \times (\Delta_c/t)$$

Where:

Δ_c = Imposed displacement across the EVA foam
t = Thickness of EVA Foam

A sufficient thickness of EVA foam shall be provided such that the applied strain (Δ_c/t) does not exceed 0.25 (25% of the specified thickness).

Design Example #1

A semi-integral abutment detail experiences 13.4 mm of thermal contraction. A total of 60 mm thickness of EVA Foam Grade 2A2 is provided with a height of 300 mm. Calculate the unfactored applied force on the ballast wall as a result of this movement.

Solution:

$$\sigma = E_{max} \times (\Delta_c/t);\ \sigma = (248\ \text{kPa}) \times (13.4\ \text{mm} / 60\ \text{mm});\ \sigma = 55.4\ \text{kPa}$$

For a unit length of ballast wall: P = (55.4 kPa) x (300 mm); P = 16.6 kN/m

Design Example #2:

Re-calculate the result in Example #1 with 80 mm thickness of EVA Foam grade 2A1 provided instead.

Solution:

$$\sigma = E_{max} \times (\Delta_c/t);\ \sigma = (138\ \text{kPa}) \times (13.4\ \text{mm} / 80\ \text{mm});\ \sigma = 23.1\ \text{kPa}$$

For a unit length of ballast wall: P = (23.1 kPa) x (300 mm); P = 6.9 kN/m

16.6.5 Presentation of EVA Foam on Contract Drawings

When the default classification of EVA foam is appropriate (Type 2, Class A, Grade 2 or 3), the designer shall use the following notation in the contract drawings:

“X” mm EVA FOAM

Where:
X = Thickness of EVA Foam

The designer may specify a different classification of EVA foam where necessary. Type 2 and Class A EVA foam should always be specified, and only the Grade should be changed. The designer shall use the following notation in the contract drawings when a specific grade is required:

“X” mm EVA FOAM, GRADE “TCG”

Where:
X = Thickness of EVA Foam,
T = Type (2),
C = Class (A), and
G = Grade (0 through 5)

For example, for 40 mm thick of Grade 4 EVA Foam, the designer would show it on the contract drawings as “40 mm EVA FOAM, GRADE 2A4”.

SECTION 17 - COMPUTER APPLICATIONS

17 COMPUTER APPLICATIONS - NOT USED

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SECTION 18 - SIGN SUPPORTS

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18 SIGN SUPPORTS

18.1 General

The design and detailing of standard overhead sign support structures is to be carried out in accordance with the latest version of the Sign Support Manual (SSM). Roadside sign supports are covered in Ontario Traffic Manual – Book 3, Ground-Mounted Sign Support and Installation, 2022.

The SSM contains information needed to prepare the contract drawings, tender quantities and special provisions for standard sign supports. The design of the cantilever, tri-chord, monotube and variable message sign (VMS) support structures meets the requirements of the CHBDC.

All non-standard sign support structures that do not meet the assumptions stated in the SSM must be custom designed, sealed and signed by two engineers.

The standard sign support structures covered by the SSM are:

18.2 Overhead Sign Supports

18.2.1 Cantilever Static Sign Supports

A sign support that cantilevers from the roadside over the closest driving lane and shoulder for static signs. Two types of cantilever sign supports, single cantilever and butterfly, are currently available in the SSM. The butterfly sign support is suitable for total signboard sizes up to and including 48 m^2 and for single cantilever sign support signboard size including and up to 32m^2 is suitable, depending on the sign type, class, reference wind pressure and the eccentricity of the centreline of the signboard. The static signs range in depth from 1525 mm to 2745 mm inclusive. The structure is constructed of galvanised structural steel and consists of an overhead truss supported on a single leg column with a concrete caisson type foundation.

18.2.2 Tri-Chord Static Sign Supports

a) <u>Simply Supported Type:</u>

This is a sign support structure that crosses the highway with spans ranging from 14 m to 36 m inclusive. These sign supports are suitable for carrying total signboard areas of up to and including 45 m^2. The static signs range in depth from 1525 mm to 3965 mm inclusive, depending on the structure type. The structure is constructed of galvanised structural steel and consists of a three-chord overhead truss supported on a single leg column on each side with concrete caisson type foundations.

b) <u>Cantilever Type:</u>

A sign support which cantilevers from the roadside over the closest driving lane and shoulder. This sign support is suitable for total signboard sizes up to and including 26.7 m^2. The static signs range in depth from 1525 mm to 2745 mm inclusive. The structure is constructed of galvanised structural steel and consists of a three-chord overhead truss supported on a single leg column with concrete caisson type foundation.

18.2.3 Monotube Sign Supports

A sign support that crosses the highway with spans ranging from 13.5 m to 24 m inclusive for lane designation signs. This sign support is suitable for signboard sizes up to and including 1200 mm by 1200 mm. The structure is constructed of galvanised structural steel and consists of four tapered, octagonal monotubes with concrete caisson type foundations.

18.2.4 VMS Sign Supports

a) Overhead truss:

A sign support that crosses the highway with spans ranging from 17.6 m to 34 m inclusive for variable message sign systems. This sign support is suitable for sign systems up to and including 40 m^2. The structure is constructed of an overhead aluminum truss supported on galvanised structural steel columns with concrete caisson type foundations.

b) Pole mounted:

This VMS sign support is fabricated from a galvanised structural steel pole with concrete caisson type foundation. The pole is carrying the VMS digital panel. The maximum VMS size is 3000 mm in depth by 5000 mm in width.

18.2.5 Bridge Mounted Sign Supports

A sign support mounted on bridges for static signs ranging in depth from 1525 mm to 2745 mm inclusive, and width as required. The structure is constructed of aluminium struts and arms connected to the bridge by stainless steel anchors.

18.3 Ground Mounted Sign Supports

Ground mounted sign supports are available with break-away supports, or rigid supports when located beyond the clear recovery zone or adequately behind a roadside barrier. These sign supports are no longer contained in the MTO’s Sign Support Manual and are in Ontario Traffic Manual - Book 3 (Ground-Mounted Sign Support and Installation).

SECTION 19 - BRIDGE REHABILITATION

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SECTION 19 - BRIDGE REHABILITATION

19 BRIDGE REHABILITATION

19.1 Rehabilitation of Bridge Structures

The overall structural design and detailing for the rehabilitation and repair of bridge structures and components shall be according to the Structural Manual and CHBDC. Methods, procedures, and detailed technical information related to bridge rehabilitation are covered in the MTO Structure Rehabilitation Manual (SRM).

Additionally, other MTO manuals, guidelines, policies, and standards that are applicable to bridge rehabilitation projects should also be referred to.

19.1.1 MTO's Structure Rehabilitation Manual (SRM)

The SRM contains information on performing detailed site investigations, selecting the appropriate methods and scope of rehabilitation, information on contract preparation, and other guidance as it relates to bridge (and other structure) rehabilitation.

19.1.2 Seismic Analysis and Retrofit Requirements for Bridge Rehabilitation Projects

The Seismic performance Category (SPC) shall be determined per this section prior to starting a rehabilitation design assignment.

Seismic analysis requirements for bridge rehabilitations shall be determined as follows:

1. Isolated minor repairs to specific components and holding strategies with an expected remaining service life of 15 years or less do not require any seismic considerations.

2. For all other instances, the Earthquakes with 10%, 5%, and 2% probabilities of exceedance in 50 years described in CHBDC Section 4.4.3.1 shall be replaced by X, Y, and Z respectively depending on the expected remaining service life, as given in Table 19.1-1 .

Table 19.1-1 – Earthquake Probabilities for use in CHBDC Section 4 (% occurrence in 50 years)

Remaining Service Life	X%	Y%	Z%
>60-75+	10%	5%	2%
>40-60	15%	7%	3.5%
>30-40	20%	10%	5%
>15-30	30%	20%	5%

In lieu of a more refined analysis, seismic parameters for intermediate seismic events may be taken by linear interpolation (using percentages) of the parameters provided by Geological Survey of Canada. (Approximate Return Period in brackets)

3. The procedures of CHBDC Section 4, with S(1.0) for the 2% in 50 years shall be followed to determine the Seismic Performance Category (SPC). All other requirements use the modified seismic demands per Table 19.1-1.

SECTION 19 - BRIDGE REHABILITATION

~~4.~~ Seismic evaluation and retrofit design shall be done according to CHBDC requirements and the associated calculations shall clearly demonstrate the fulfillment of the code requirements. The theoretical validity and compatibility with CHBDC must be established and reported in sufficient detail where the evaluators/designers will use guidelines or manuals other than the CHBDC.

5. When the minimum performance levels of CHBDC Table 4.15 cannot be met due to cost, low remaining expected service life, or other factors, the Structural Section, in consultation with Bridge Office, may specify a higher probability of exceedance seismic hazard (i.e. lower magnitude earthquake) from Table 19.1-1 above.

6. When the required scope of rehabilitation is excessive based on CHBDC Section 4 using seismic events outlined in this memo, the Structural Section, in consultation with Bridge Office, may waive or alter certain rehabilitation needs depending on the degree of deficiency, the remaining service life, and other site-specific conditions.

DIVISION 3 - DESIGN AIDS

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SECTION 1 - INDEX OF DESIGN AIDS

1 INDEX OF DESIGN AIDS

2 GENERAL DESIGN AND DRAFTING

3 PILES

None.

4 FOOTINGS

None.

5 ABUTMENTS, WINGWALLS AND RETAINING WALLS

6 PIERS

None.

7 PRESTRESSED CONCRETE

8 STRUCTURAL STEEL

9 DECK AND CURBS

None.

10 BARRIERS AND RAILING SYSTEMS

SECTION 1 - INDEX OF DESIGN AIDS

11 RIGID FRAMES

None.

12 REINFORCING STEEL

13 BEARING ASSEMBLIES AND EXPANSION JOINTS

None.

14 CULVERTS

None.

15 WOOD STRUCTURES

None.

16 MISCELLANEOUS

17 COMPUTER APPLICATIONS

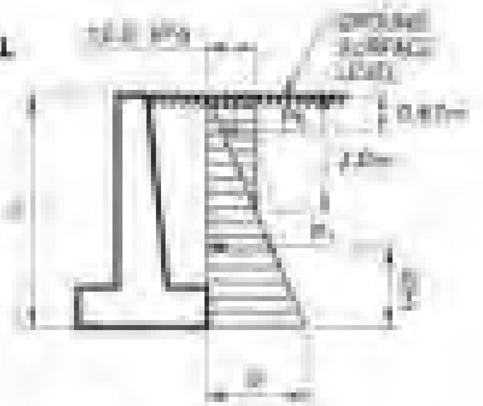
None.

18 SIGN SUPPORTS

None.

DESIGN AID 2-10
ACTIVE EARTH PRESSURE WITHOUT SURCHARGE

q = EQUIVALENT FLUID PRESSURE (kPa)
p = UNIT PRESSURE AT DEPTH H METRES
P = TOTAL FORCE PER LINEAR METRE OF WALL
M = OVERTURNING MOMENT (kN-m)



AT SERVICEABILITY LIMIT STATES

$\alpha_E = 1.0$
$q = 7.0 \times (H)$

AT ULTIMATE LIMIT STATES

$\alpha_E = 1.25$
$q = 1.25 \times (7.0) \times (H)$

$\phi = 30°$
$P = P_1 + P_2$

– RESULTS INCLUDE THE EFFECT OF COMPACTION SURCHARGE

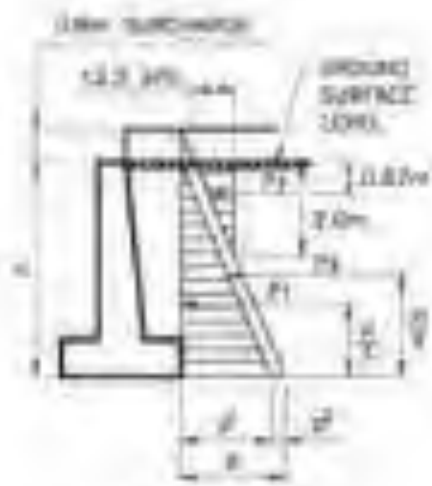
SLS			
H (m)	p (kPa)	P (kN)	M (kN-m)
1.0	7	13	6
1.5	11	19	14
2.0	14	26	25
2.5	18	33	40
3.0	21	44	59
3.5	25	56	84
4.0	28	68	114
4.5	32	83	152
5.0	35	100	197
5.5	39	118	251
6.0	42	138	315

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	248
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.

DESIGN AID 2-11
ACTIVE EARTH PRESSURE WITH 800 mm SURCHARGE

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

$\alpha_E = 1.25$
$q = 1.25 \times (7.0) \times (H)$

$\phi = 30°$
$P = P_1 + P_2 + P_3$
$p = p^1 + p^2$

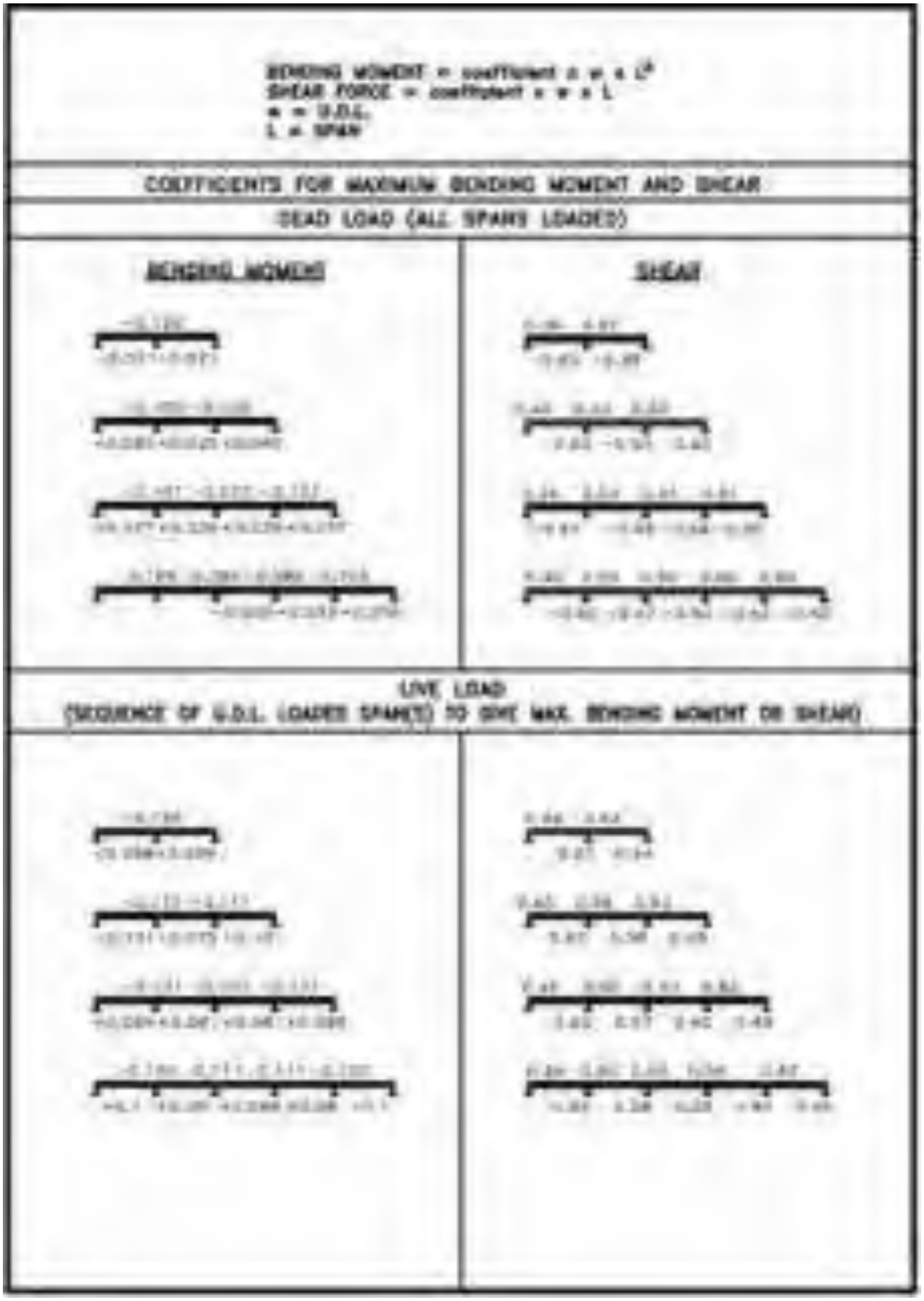
– RESULTS INCLUDE THE EFFECT OF COMPACTION SURCHARGE

SLS			
H (m)	p (kPa)	P (kN)	M (kN-m)
1.0	13	18	9
1.5	16	28	20
2.0	20	37	37
2.5	23	48	58
3.0	27	60	85
3.5	30	75	118
4.0	34	90	156
4.5	37	108	209
5.0	41	128	268
5.5	44	149	337
6.0	48	172	417

ULS			
H (m)	p (kPa)	P (kN)	M (kN-m)
1.0	16	23	11
1.5	20	35	25
2.0	25	46	46
2.5	29	60	73
3.0	34	75	106
3.5	38	94	148
4.0	43	113	196
4.5	46	135	261
5.0	51	160	335
5.5	55	186	421
6.0	60	215	521

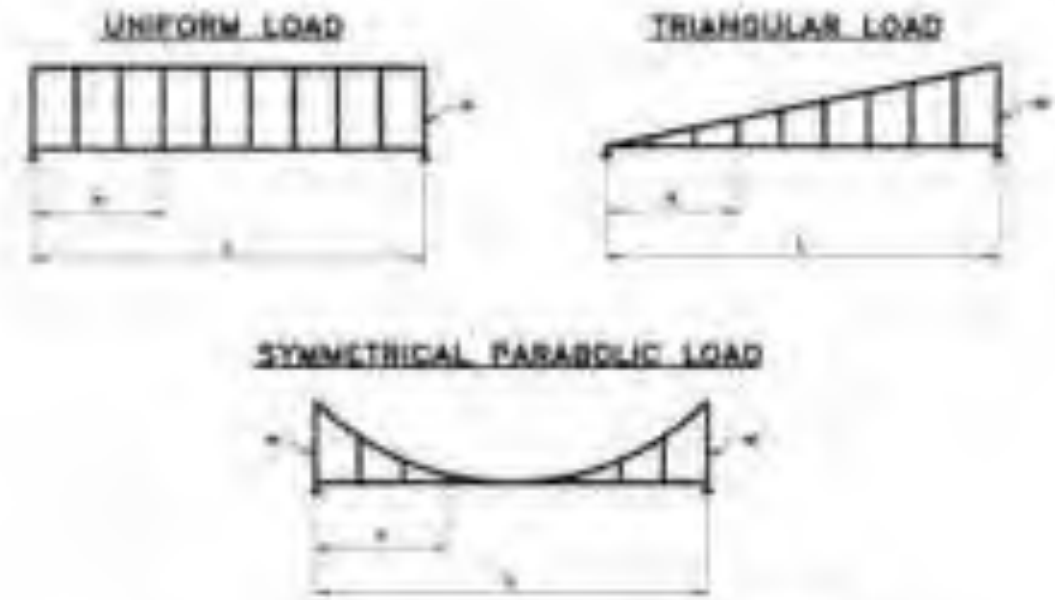
THE METHOD OF EQUIVALENT FLUID PRESSURES IS LIMITED TO A MAXIMUM HEIGHT OF 6.0m. FOR RETAINING WALLS WITH HEIGHTS > 6.0m, THE EARTH PRESSURE DISTRIBUTION SHALL BE ESTABLISHED BY A GEOTECHNICAL ENGINEER.

DESIGN AID 2-12
CONTINUOUS BEAMS (EQUAL SPANS)



DESIGN AID 2-13

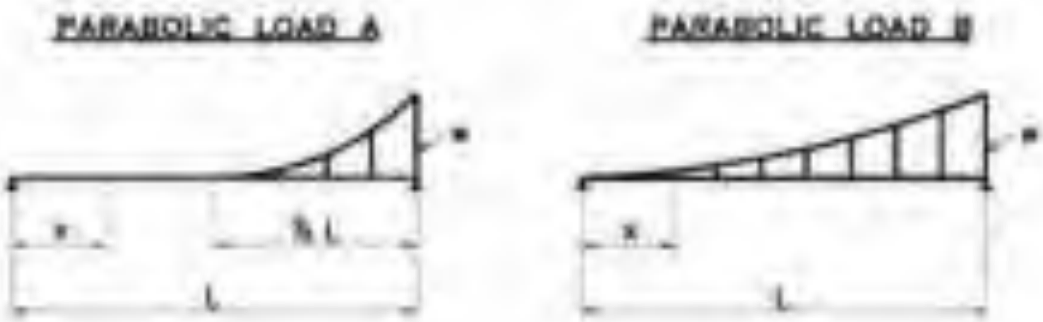
SIMPLE BEAM MOMENT COEFFICIENTS (Uniform, Triangular, and Symmetrical Parabolic Loads)



	C		
$\frac{x}{L}$	UNIFORM LOAD	TRIANGULAR LOAD	SYMMETRICAL PARABOLIC LOAD
0.05	0.02375	0.00831	0.007165
0.10	0.04500	0.01650	0.012500
0.15	0.06375	0.02444	0.015881
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	0.020683
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	0.06254	0.020683
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	0.6375	0.03931	0.015881
0.90	0.04500	0.02850	0.012500
0.95	0.02375	0.01544	0.007165

MOMENT = cwL^2

DESIGN AID 2-14
SIMPLE BEAM MOMENT COEFFICIENTS (Parabolic Load A and B)

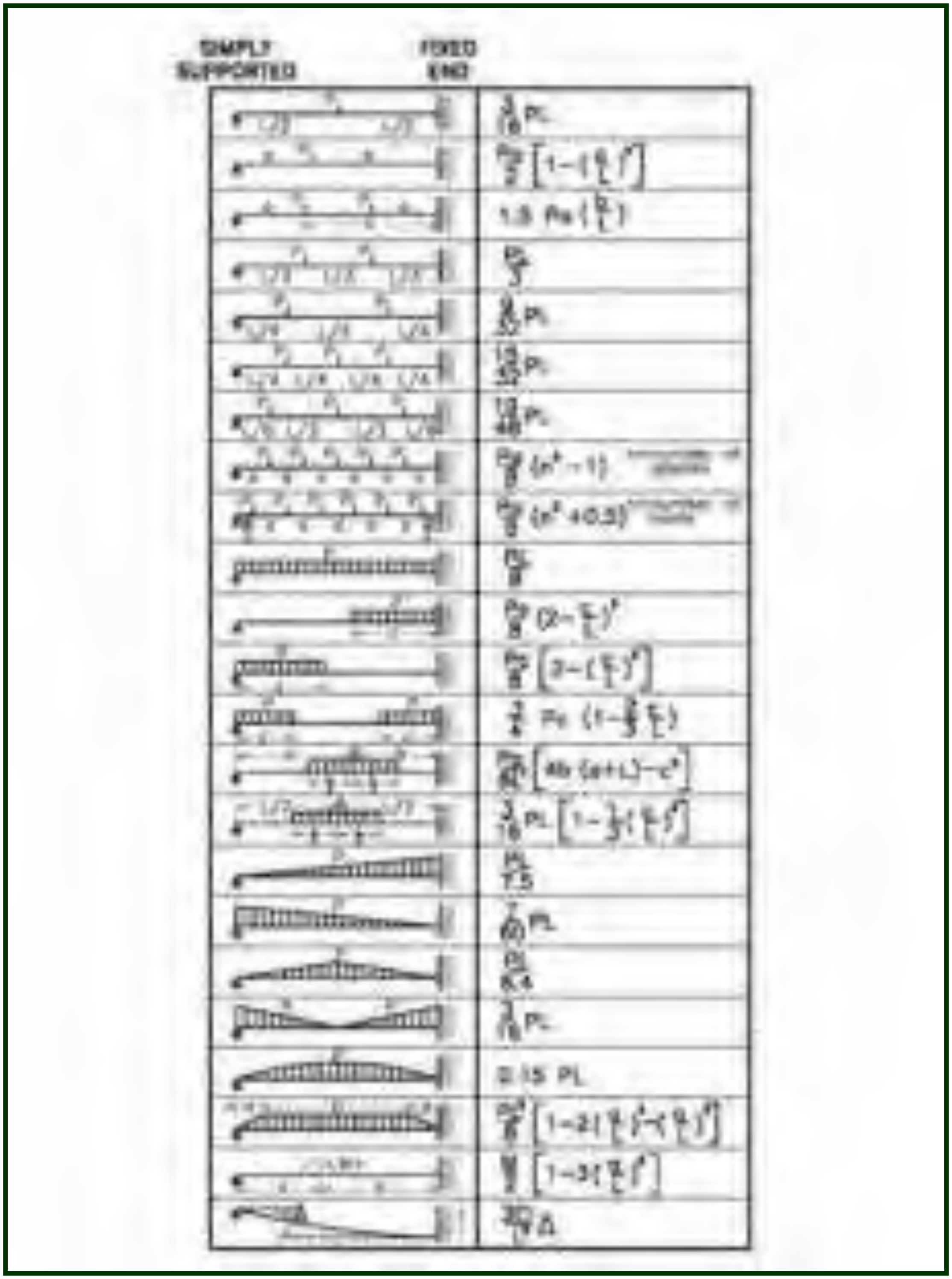


	α	
x/L	PARABOLIC LOAD A	PARABOLIC LOAD B
0.05	0.001042	0.004185
0.10	0.002083	0.008329
0.15	0.003125	0.012457
0.20	0.004167	0.016533
0.25	0.005208	0.020507
0.30	0.006250	0.024335
0.35	0.007292	0.027913
0.40	0.008333	0.031250
0.45	0.009375	0.034062
0.50	0.010417	0.036458
0.55	0.011458	0.038211
0.60	0.012467	0.039600
0.65	0.013573	0.039780
0.70	0.014080	0.038835
0.75	0.014229	0.036152
0.80	0.013867	0.032533
0.85	0.012708	0.027552
0.90	0.010017	0.020329
0.95	0.006123	0.011290

MOMENT = $\alpha w L^2$

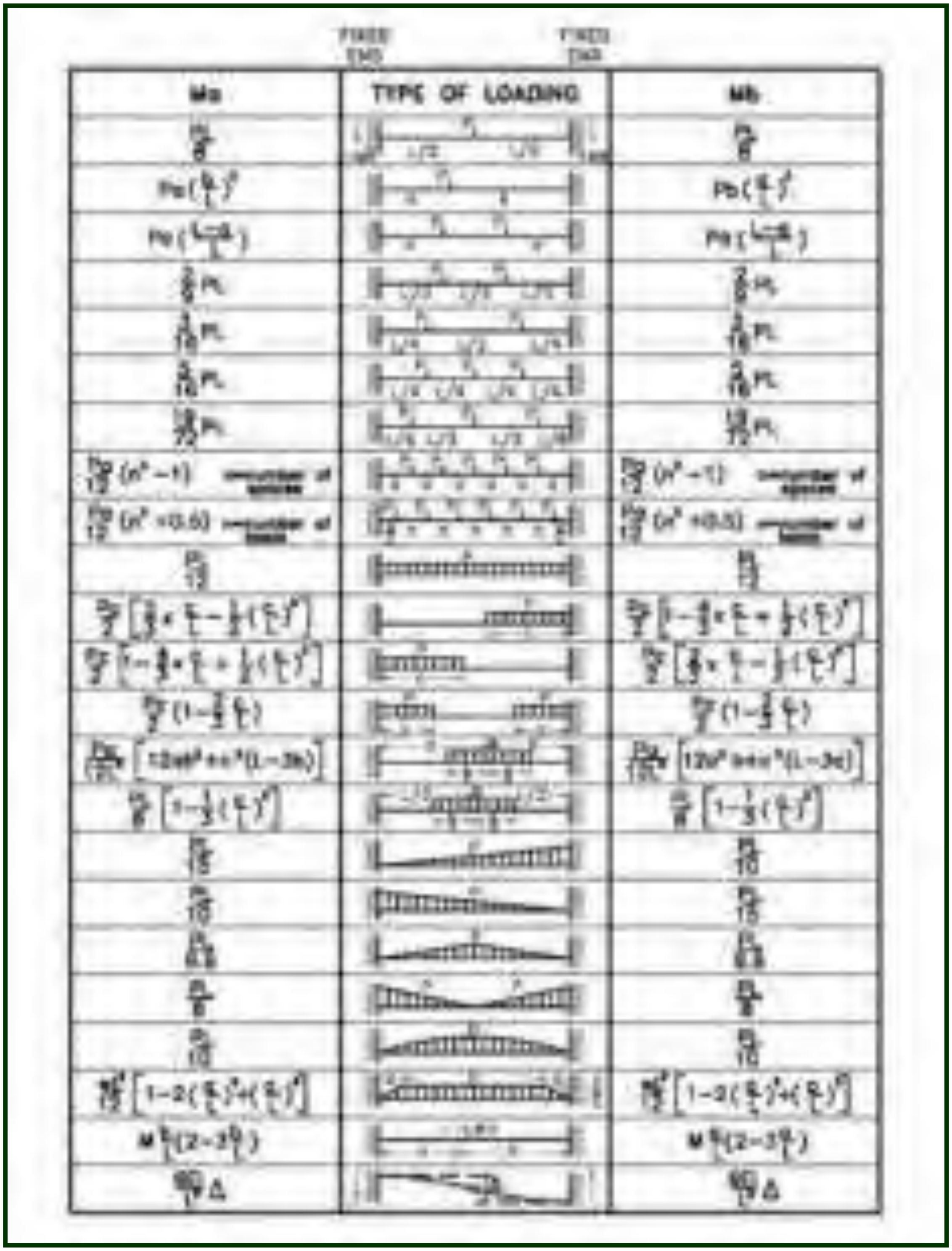
DESIGN AID 2-15

FIXED END MOMENTS FOR PRISMATIC BEAMS(Simply Supported / Fixed End)



DESIGN AID 2-16

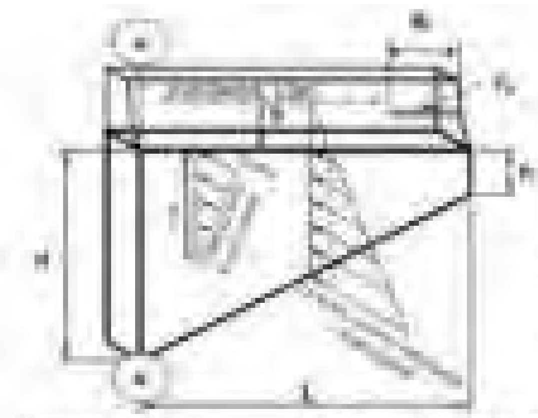
FIXED END MOMENTS FOR PRISMATIC BEAMS(Fixed End / Fixed End)



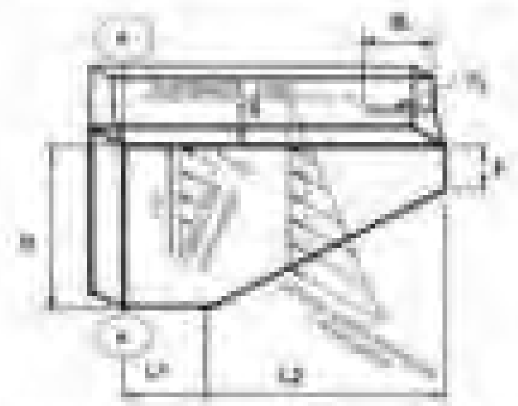
SECTION 5 - ABUTMENTS, WINGWALLS AND RETAINING WALLS

WINGWALL DESIGN AID

Moments and Lateral Thrust for Cantilever Wingwalls



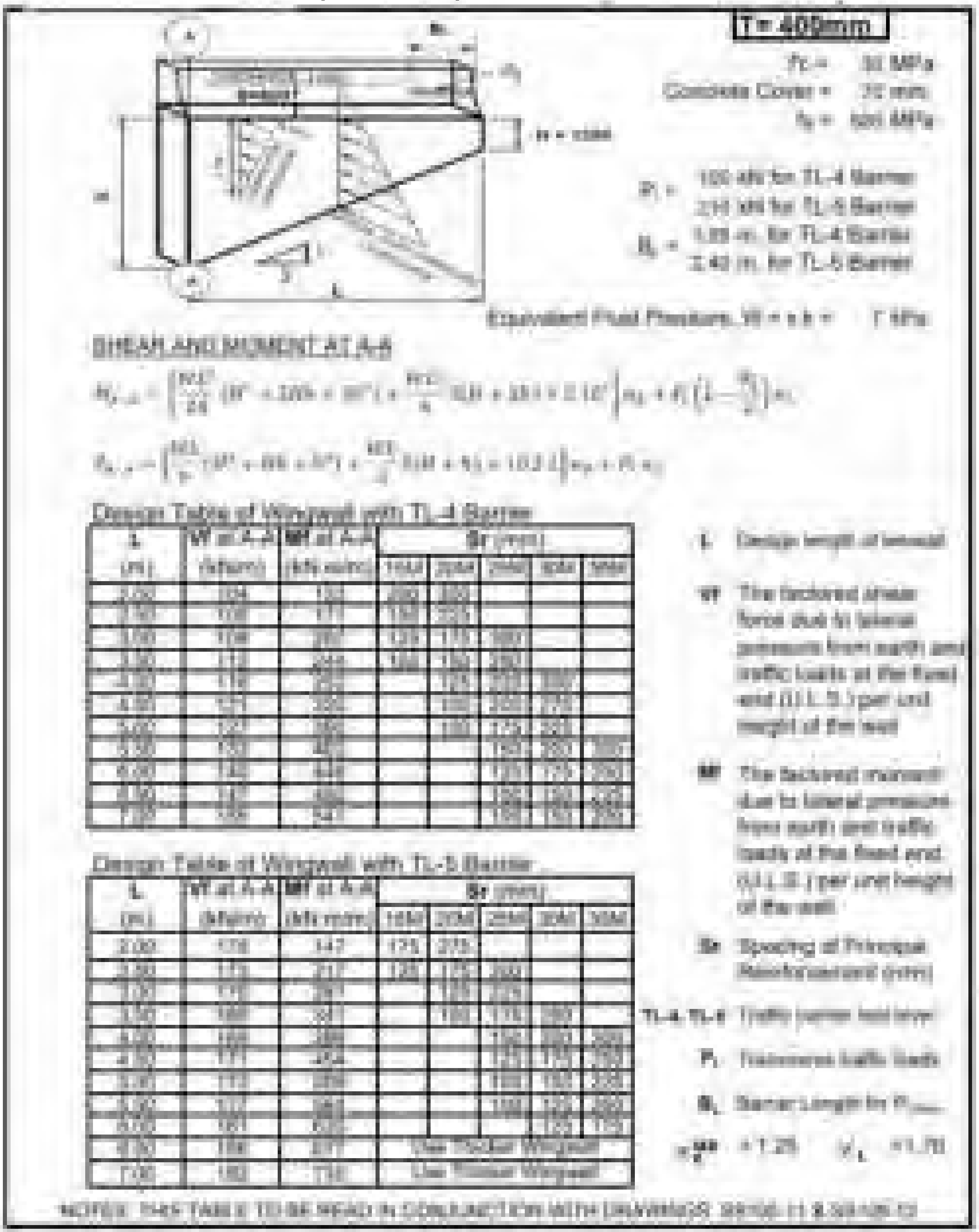
Loadings	Total Moment at A-A	Total Lateral Thrust
Lateral Earth Pressure	[illegible]	[illegible]
Surcharge	[illegible]	[illegible]
Compaction Surcharge	[illegible]	[illegible]
Live Load on Barrier	[illegible]	[illegible]



Loads	Total Moment at A-A	Total Lateral Thrust
Lateral Earth Pressure	[illegible]	[illegible]
Surcharge	[illegible]	[illegible]
Compaction Surcharge	[illegible]	[illegible]
Live Load on Barrier	[illegible]	[illegible]

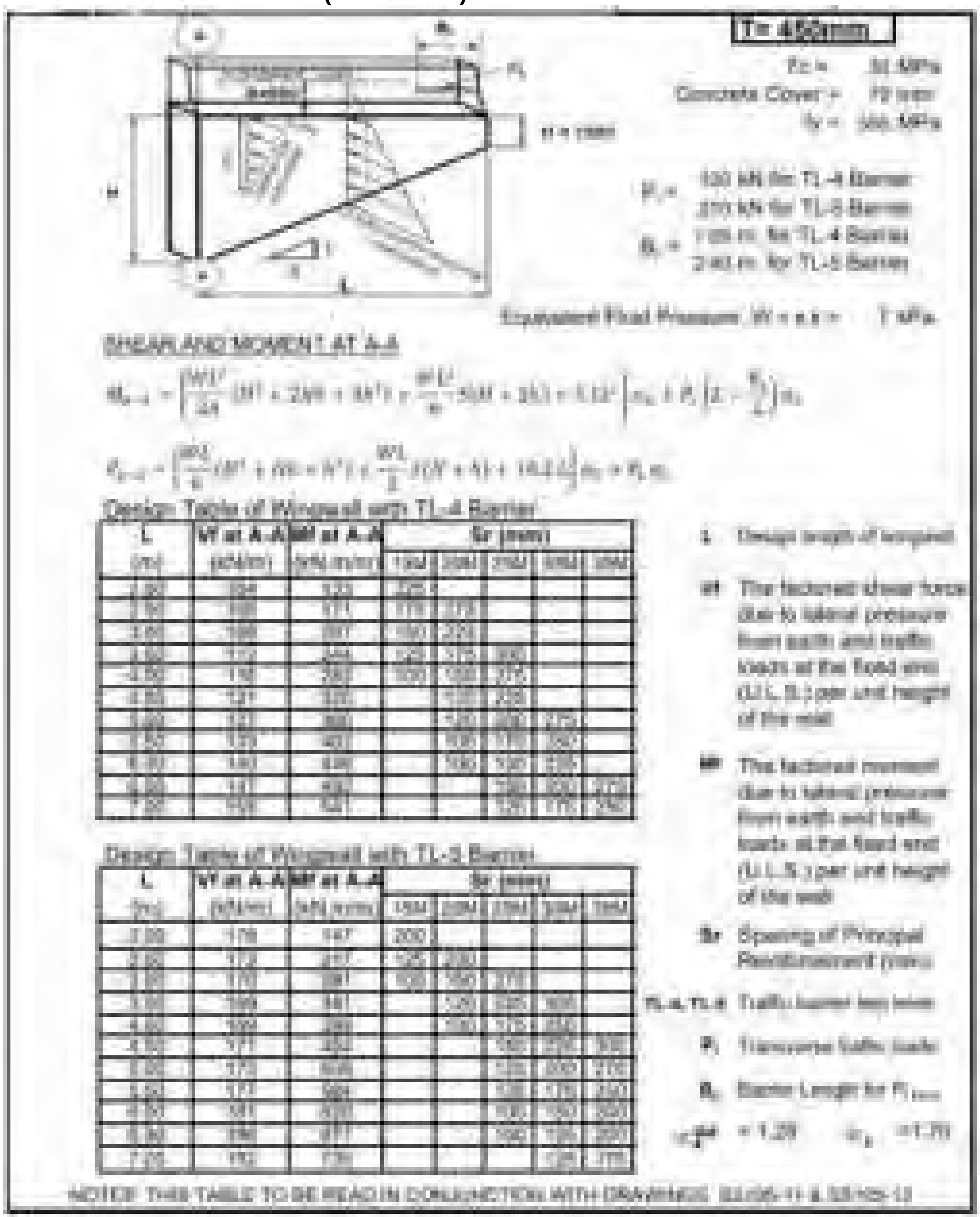
SECTION 5 - ABUTMENTS, WINGWALLS AND RETAINING WALLS

DESIGN AID 5-1
WINGWALL DESIGN TABLE (T = 400 mm)



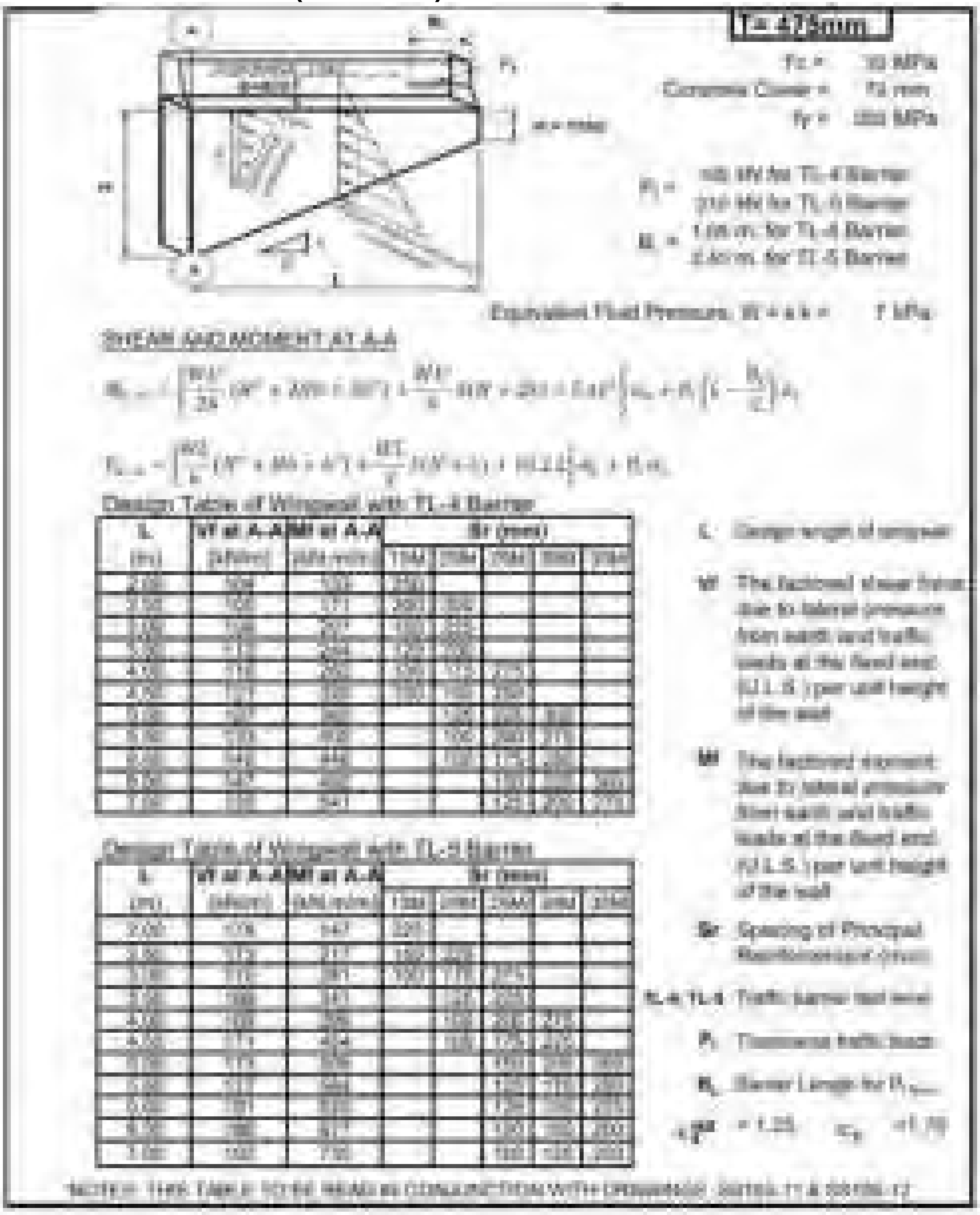
SECTION 5 - ABUTMENTS, WINGWALLS AND RETAINING WALLS

DESIGN AID 5-2
WINGWALL DESIGN TABLE (T = 450 mm)

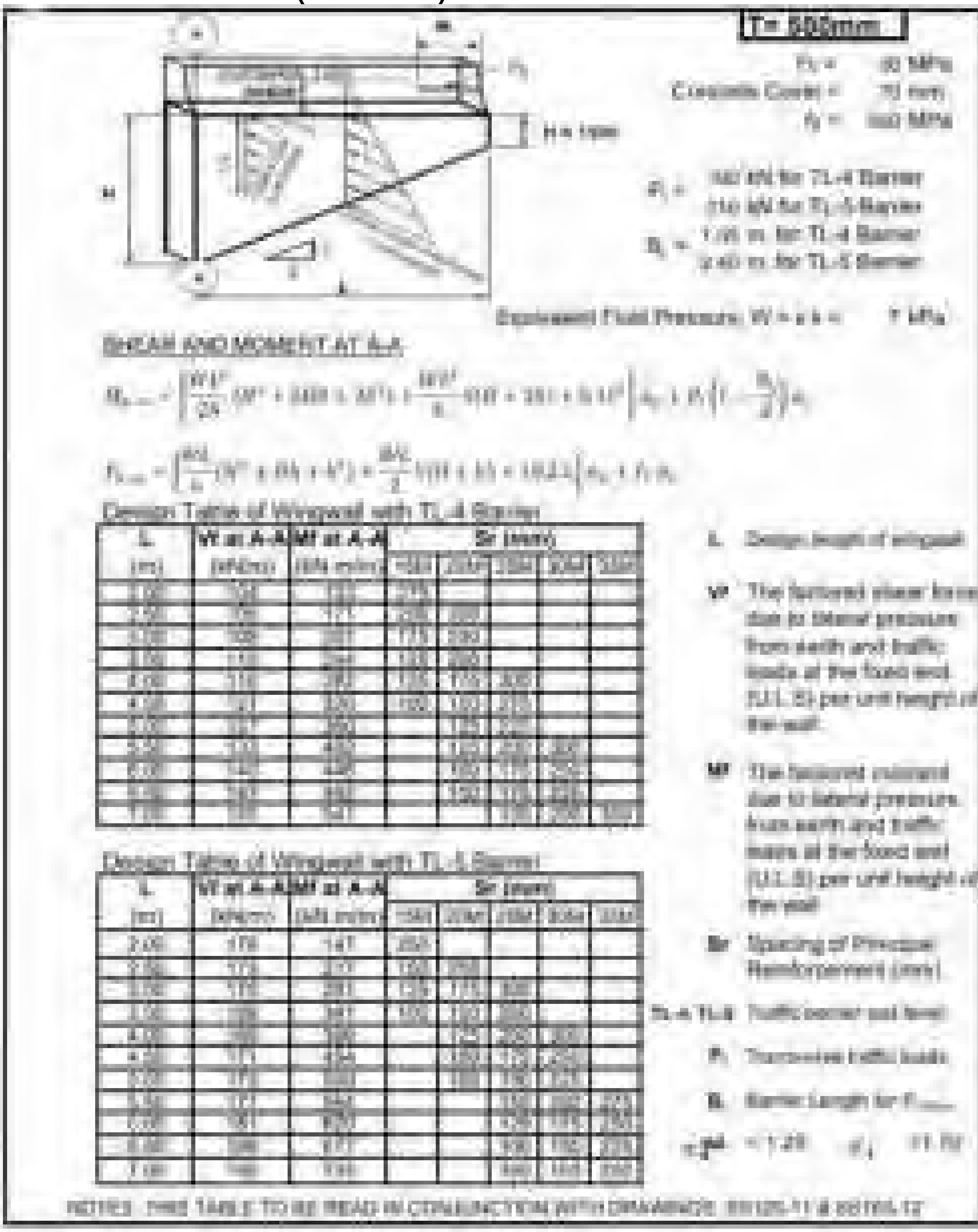


SECTION 5 - ABUTMENTS, WINGWALLS AND RETAINING WALLS

DESIGN AID 5-3
WINGWALL DESIGN TABLE (T = 475 mm)

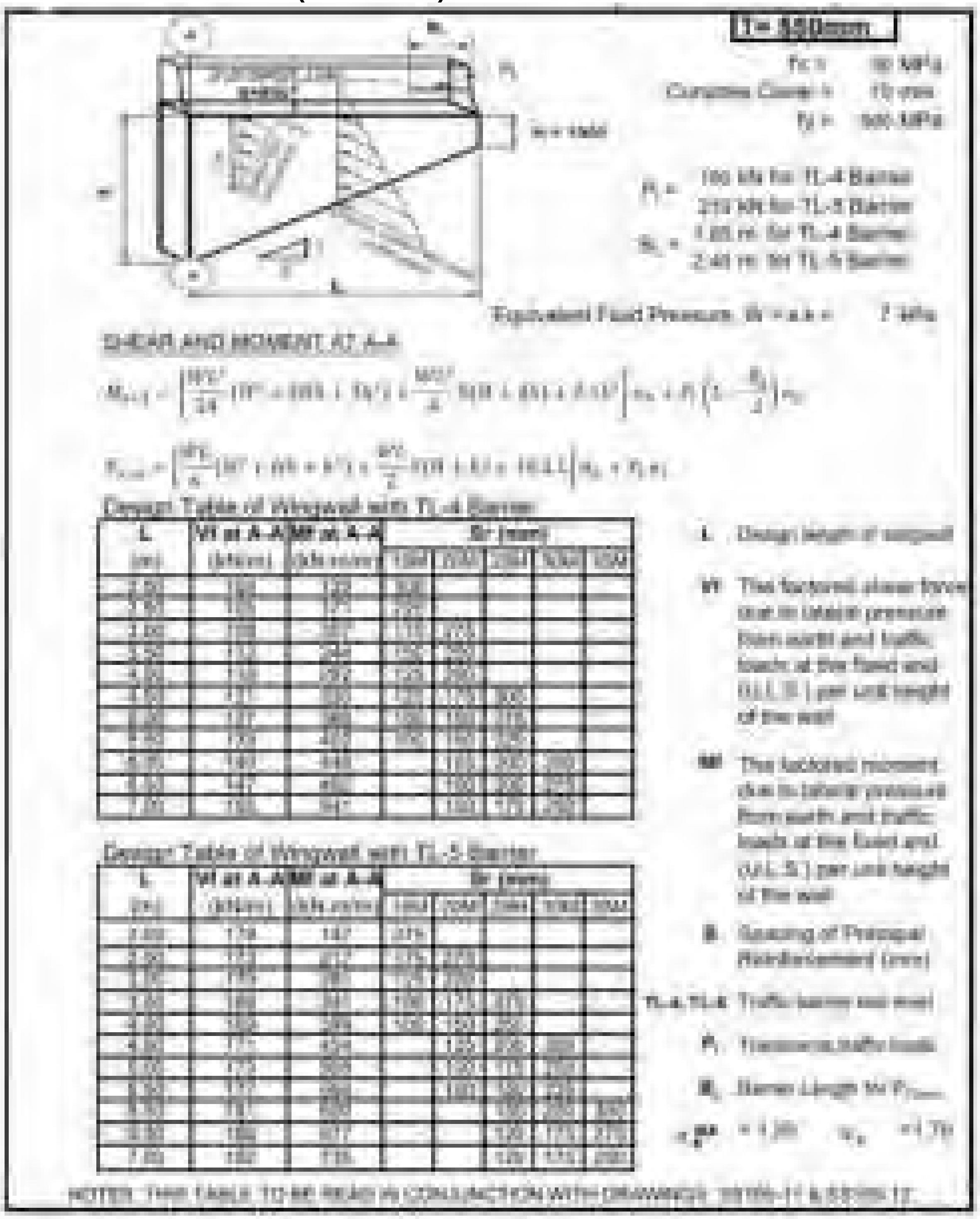


DESIGN AID 5-4
WINGWALL DESIGN TABLE (T = 500 mm)

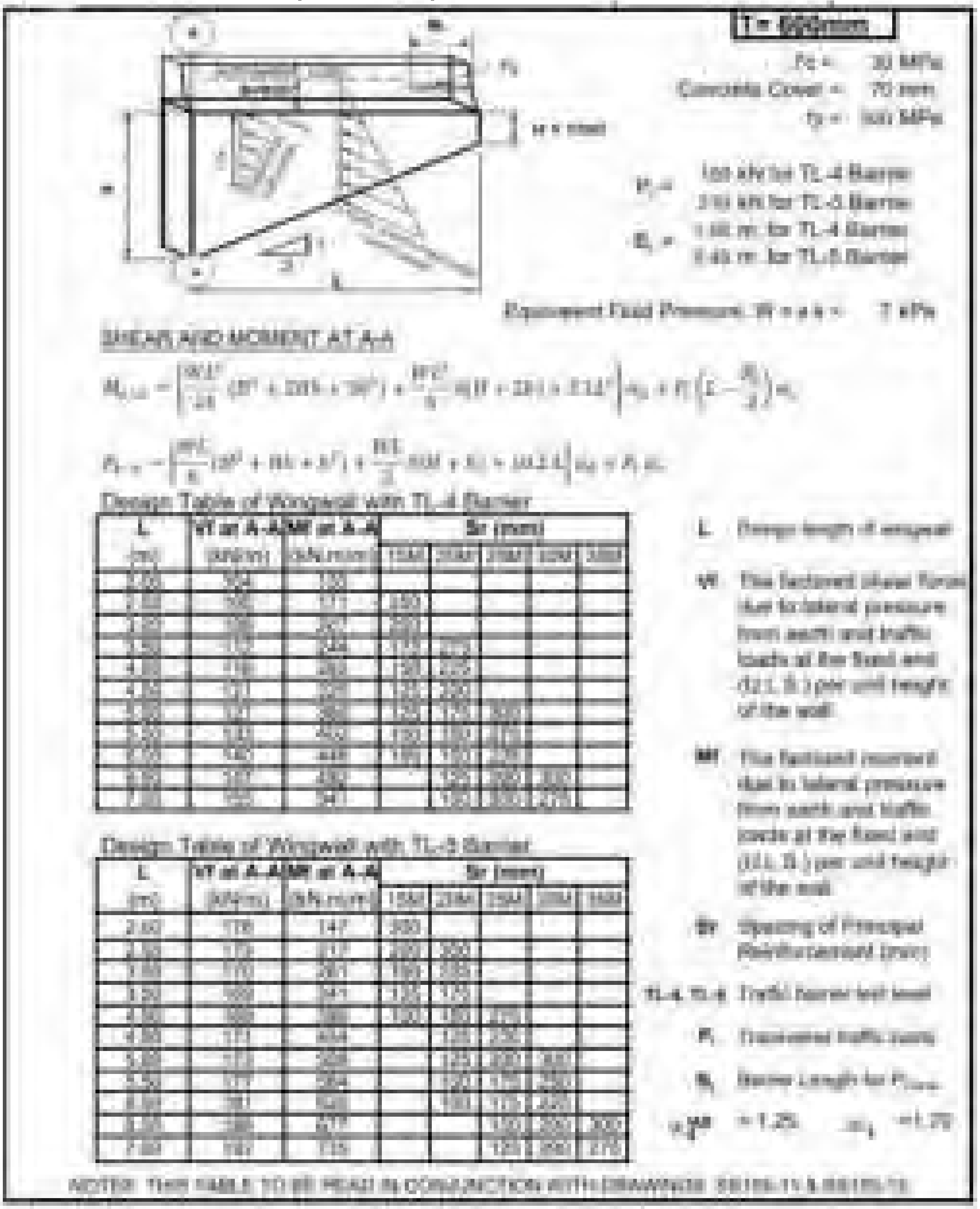


SECTION 5 - ABUTMENTS, WINGWALLS AND RETAINING WALLS

DESIGN AID 5-5
WINGWALL DESIGN TABLE (T = 550 mm)



DESIGN AID 5-6
WINGWALL DESIGN TABLE (T = 600 mm)



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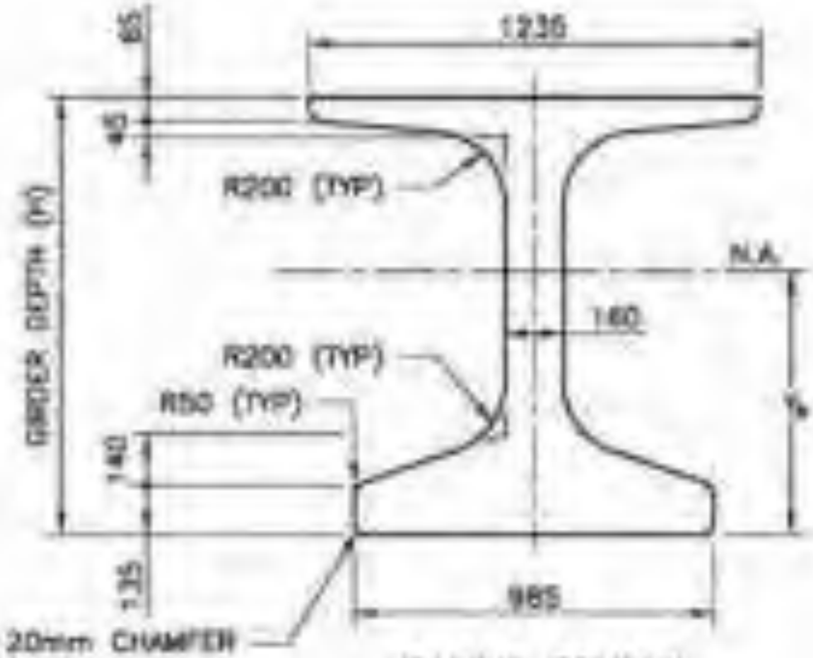
SECTION 7 - PRESTRESSED CONCRETE

DESIGN AID 7-1
PRESTRESSED NU GIRDERS



TYPICAL SECTION

SECTION PROPERTIES

GIRDER DEPTH H (mm)	AREA (mm^2)	Y_b (mm)	MOMENT OF INERTIA ABOUT N.A. (mm^4)
900	427747	410.2	4.651 E+10
1200	475747	542.4	9.565 E+10
1400	507747	632.4	1.400 E+11
1600	539747	723.6	1.945 E+11
1800	571747	815.7	2.598 E+11
1900	587747	862.2	2.964 E+11
2000	603747	908.8	3.362 E+11
2400	667747	1096.8	5.260 E+11

SECTION 8 - STRUCTURAL STEEL

DESIGN AID 8-1
STANDARD WELDING SYMBOLS

SECTION 9 - DECKS AND CURBS

DESIGN AID 9-1 & 9-2

Design of Reinforcement of Transverse Deck Slab Cantilever Overhangs of Slab on Girder and Slab Bridges

This design aid applies to the deck slab cantilever overhangs of slab-on-girder bridges, with up to 2.5 m length, where the girders run parallel to the direction of travel, and to the design of thin slab deck cantilever overhangs on thick solid slab and voided slab bridges.

The transverse top reinforcement in deck slab cantilever overhangs serves as the primary structural reinforcement to resist negative bending moments induced by applied loads. For the design of a thin slab deck cantilever overhang, the reinforcement configurations provided in the tables below should be directly adopted.

This design is developed for a concrete deck with a compressive strength of 30 MPa and a clear cover to top transverse reinforcement of 60 mm. Carbon and stainless reinforcing steel are assumed to have a yield strength of 500 MPa, and GFRP is Grade III. For TL-5 and TL-4 MASH impact loading, the fascia depth is taken as 400 mm which corresponds to a deck thickness of 285 mm at the interior face of the barrier. The slab thickness is assumed to be at least 275 mm at the exterior girder. The reinforcement design accounts for moment due to dead loads and truck live loading, as well and moment and tensile demand due to barrier impact. The reinforcement design accounts for all limit states.

As shown in figure below for slab-on-girder bridges, top transverse bar in cantilever overhang shall be extended beyond exterior girder(Steel or Concrete) by minimum of 0.75 times spacing of girders or development length from design table below.

For solid slab and voided slab bridges, the top transverse bars shall be extended into the thicker portion of the cross-section sufficiently to fully develop their capacity at the cantilever support.

Extents of Transverse Deck Cantilever Overhang Top Transverse Reinforcement

SECTION 9 - DECKS AND CURBS

DESIGN AID 9-1 Bar Size and Spacing for Stainless or Carbon Reinforcing Steel in the Top Layer of the Transverse Deck Cantilever Overhang

		Cantilever Length (Centre of Exterior Girder to Deck Fascia)	
Type of Barrier	***Location**	**Span ≤ 2.0m**	**2.0m < Span ≤ 2.5m**
TL-5 and TL-4 MASH (400 mm fascia)	Typical Interior/middle Region	20M @ 150	25M @ 300 + 20M @ 300
	End Region (Within 3.7 m of transverse free edge)	25M @ 300 + 20M @ 300	25M @ 150
TL-3 (350 mm fascia)	Typical Interior/Middle	20M at 150	25M @ 300 + 20M @ 300
	End Region (Within 2.5 m of transverse free edge)	25M @ 300 +20M @ 300	25M @ 150

*Refer to Figure 8.13 of CHBDC S6-25 for transverse free edge

DESIGN AID 9-2 Bar Size and Spacing for GFRP Reinforcement in the Top Layer of the Transverse Deck Cantilever Overhang

		Cantilever Span (Centre of exterior girder to end of deck)			
Type of Barrier	**Location**	0 - 1.4m	1.4 - 1.7m	1.7m - 2.1m	2.1m - 2.5m
TL-5, TL-4 MASH (400 mm fascia)	Typical Interior/Middle Region	G15 @ 150	G20 @150	G20 @ 300 + G25 @ 300	G25 @ 150
	End Region(Within 2.6 m of transverse free edge)	G20 @ 300 + G25 @ 300	G20 @ 300 + G25 @ 300	G20 @ 300 + G25 @ 300	G25 @ 150
TL-3 (350 mm fascia)	Typical Inner	G15 @ 150 mm	G20 @ 150 mm	G20 @ 300 + G25 @ 300	G25 @ 150
	End(Within 2.0 m of transverse free edge)	G20 @ 300 + G25 @ 300	G20 @ 300 + G25 @ 300	G20 @ 300 + G25 @ 300	G25 @ 150

*Refer to Figure 8.13 of CHBDC S6-25 for transverse free edge

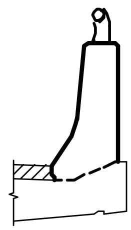
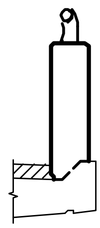
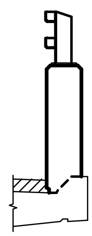
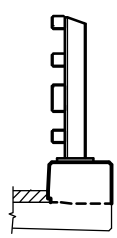
DESIGN AID 9-3
Development Length of Transverse Top Reinforcement of Cantilever Slab

	Material	
Bar Size	**GFRP**	**Steel or Stainless Steel**
15M/G15	1000 mm	500 mm
20M/G20	1200 mm	600 mm
25M/G25	1200 mm (governed by SLS, reduced development)	900 mm

SECTION 10 – BARRIERS AND RAILING SYSTEMS

DESIGN AID 10-1
PERMANENT BARRIERS DEAD LOAD

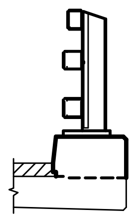
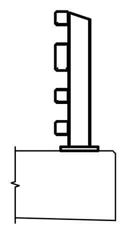
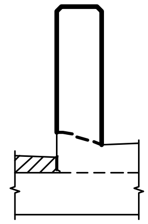
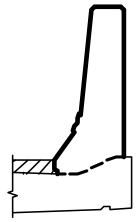
Note: *Barrier Dead Load is based on barrier/curb concrete above the dashed line as marked*

Barrier Type	Related Structural Standard Drawing	Barrier Shape	DL of Concrete Wall/curb (kN/m)	DL of Steel (kN/m)	Total Weight of Barrier (kN/m)
Barrier Wall With Railing, TL-4	SS110-21, SS110-54,58,68,69,91		6.1	0.3	6.4
Parapet Wall With Railing, TL-4	SS110-21, SS110-56,57,59,65,75,90,97,98,104,105		5.2	0.3	5.4
Parapet Wall for Combination Traffic/Bicycle Rail, TL-4	SS110-85, SS110-82,83,84		5.2	0.3	5.5
Four Tube Combination Traffic/Bicycle Railing, TL-4	SS110-34,36		3.4	0.9	4.3

SECTION 10 – BARRIERS AND RAILING SYSTEMS

DESIGN AID 10-1
PERMANENT BARRIERS DEAD LOAD

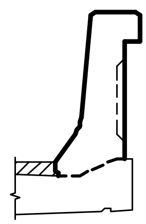
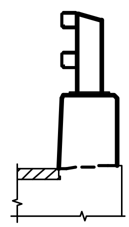
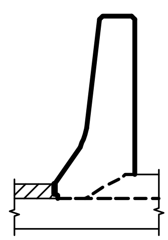
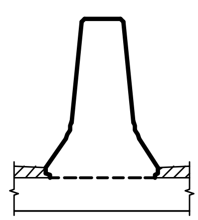
Note: *Barrier Dead Load is based on barrier/curb concrete above the dashed line as marked*

Barrier Type	Related Structural Standard Drawing	Barrier Shape	DL of Concrete Wall/curb (kN/m)	DL of Steel (kN/m)	Total Weight of Barrier (kN/m)
Three Tube Railing on Curb, TL-4	SS110-37,39		3.4	0.7	4.0
Four Tube Railing on Sidewalk, TL-4	SS110-46,49		N/A	0.9	0.9
Multi Use Path (MUP) Separation Barrier (Parapet), TL-4	SS110-110		5.5	N/A	5.5
Barrier Wall Without Railing, TL-5	SS110-92,94,109		7.9	N/A	7.9

SECTION 10 – BARRIERS AND RAILING SYSTEMS

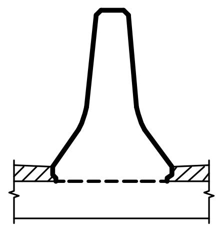
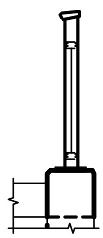
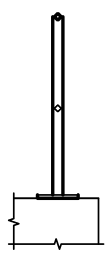
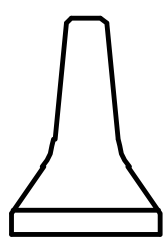
DESIGN AID 10-1
PERMANENT BARRIERS DEAD LOAD

Note: *Barrier Dead Load is based on barrier/curb concrete above the dashed line as marked*

Barrier Type	Related Structural Standard Drawing	Barrier Shape	DL of Concrete Wall/curb (kN/m)	DL of Steel (kN/m)	Total Weight of Barrier (kN/m)
Barrier Wall With Architectural Finish, TL-5	SS110-70,71,72, 73 and 74,93,99		8.7	N/A	8.7
Parapet Wall for Two Tube Railing, TL-5	S110-95,96		7.0	0.6	7.6
Multi-Use Path (MUP), Separation Barrier, TL-5	SS110-111		7.8	N/A	7.8
Reinforced Concrete Median Barrier Wall on Structures - Type I	SS110-62		12.2	N/A	12.2

DESIGN AID 10-1
PERMANENT BARRIERS DEAD LOAD

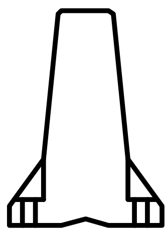
Note: *Barrier Dead Load is based on barrier/curb concrete above the dashed line as marked*

Barrier Type	Related Structural Standard Drawing	Barrier Shape	DL of Concrete Wall/curb (kN/m)	DL of Steel (kN/m)	Total Weight of Barrier (kN/m)
Reinforced Concrete Median Barrier Wall on Structures - Type II	SS110-63		6.9	N/A	6.9
Multi Use Path (MUP) Bicycle Barrier	SS0110.0022		3.5	0.8	4.2
Inspector Guard Details	SS110-33		N/A	0.4	0.4
Guide Rail System, Concrete Barrier, Type J Connection, 4 m Length Installation, Temporary	OPSD 911.150		6.9	N/A	6.9

SECTION 10 – BARRIERS AND RAILING SYSTEMS

DESIGN AID 10-1
PERMANENT BARRIERS DEAD LOAD

Note: *Barrier Dead Load is based on barrier/curb concrete above the dashed line as marked*

Barrier Type	Related Structural Standard Drawing	Barrier Shape	DL of Concrete Wall/curb (kN/m)	DL of Steel (kN/m)	Total Weight of Barrier (kN/m)
Guide Rail System, Concrete Barrier, Cast-In-Place, Type M, Installation	OPSD 911.16		6.8	N/A	6.8

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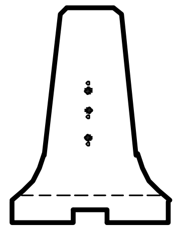
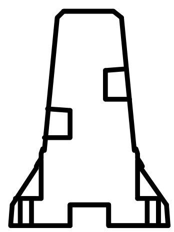
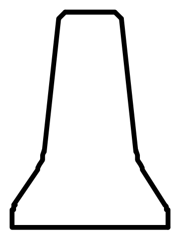
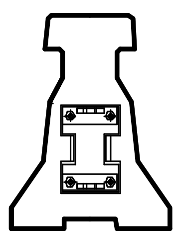
DESIGN AID 10-2
TEMPORARY BARRIERS DEAD LOAD

Barrier Type	Related Structural Standard Drawing	Barrier Shape/Photo	DL of Barrier (kN/m)
Guide Rail System, Concrete Barrier Type T Connection, 4.0m Length Installation - Temporary	OPSD 0911.1800		6.8
Guide Rail System, Concrete Barrier, Type X Connection, 4.0 m Length, Installation, Temporary	MTOD 911.194		6.8
Guide Rail System, Concrete Barrier, Type Z, 12.0 m Length, Installation, Temporary	MTOD 911.201		5.5
Guide Rail System, Concrete Barrier, QuickChange Moveable Barrier, Installation, Temporary	OPSD 911.610		6.7

DESIGN AID 10-2
TEMPORARY BARRIERS DEAD LOAD

Barrier Type	Related Structural Standard Drawing	Barrier Shape/Photo	DL of Barrier (kN/m)
BarrierGuard 800	**RDM 5.1.4.1**		0.9
ZoneGuard	**RDM 5.1.4.2**		0.9
Defender	**RDM 5.1.4.3**		0.8
Safezone	**RDM 5.1.4.4**		0.8

DESIGN AID 10-2
TEMPORARY BARRIERS DEAD LOAD

HV2	**RDM 5.1.4.5**		3.5
Highway Guard	**RDM 5.1.4.6**		0.9

SECTION 12 - REINFORCING STEEL

DESIGN AID 12-1
REINFORCING STEEL BARS

PROPERTIES OF REINFORCING STEEL BARS

BAR SIZE	MASS (kg/m)	NOMINAL DIMENSIONS		
		DIAMETER (mm)	CROSS SECTIONAL AREA (mm^2)	PERIMETER (mm)
10M	0.785	11.3	100	35.5
15M	1.570	16.0	200	50.1
20M	2.355	19.5	300	61.3
25M	3.925	25.2	500	79.2
30M	5.495	29.9	700	93.9
35M	7.850	35.7	1000	112.2
45M	11.775	43.7	1500	137.3
55M	19.625	56.4	2500	177.2

STEEL AREA PER METRE WIDTH (mm^2)

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

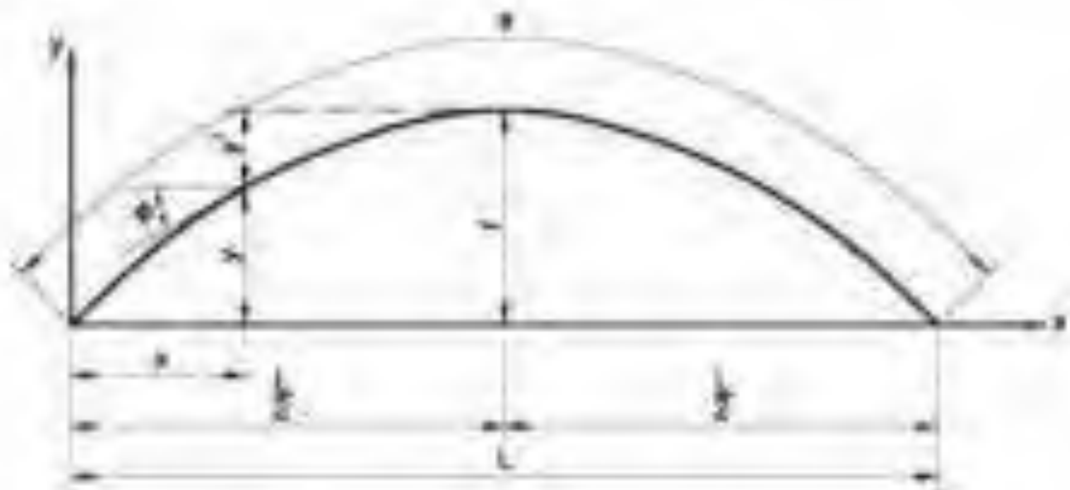
DESIGN AID 16-5
IMPERIAL - S.I. CONVERSION FACTORS

SUBJECT	CONVERSION FACTORS
LINEAR	1 in. = 25.4 mm 1 ft. = 304.8 mm = 0.3048 m 1 mile = 1.609 km
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 DENSITY	1 lb./ft^3 = 16.018 kg/m^3 1 lb./in^3 = 27.682 kg/m^3
SPEED	1 mi./hr = 1.609 km/hr 1 ft./s = 0.305 m/s
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
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

SELECTED SI PREFIXES		
Multiplying Factor	Prefix	Symbol
10^9	giga	G
10^6	mega	M
10^3	kilo	k
10^2	hecto	h
10^{-1}	deci	d
10^{-3}	milli	m
10^{-6}	micro	μ

SUBJECT	CONVERSION FACTORS
AREA	1 in^2 = 645.16 mm^2 1 ft^2 = 0.0929 m^2
MASS	1 lb. = 0.4536 kg 1 kip = 453.6 kg 1 ton = 907.2 kg
FORCE	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.
BENDING MOMENT OR TORSION	1 kip in. = 112.985 N.m 1 kip ft. = 1.35582 kN.m
MOMENT OF INERTIA	1 in^4 = 416231.42 mm^4
SECTION MODULUS	1 in^3 = 16387.064 mm^3
TEMPERATURE	°C = 5/9 (°F − 32)
THERMAL EXPANSION	in./in. °F = 0.556 x 10^{-6} 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	

DESIGN AID 16-10
PROPERTIES OF A PARABOLIC CURVE



$$y = \frac{4f}{L^2}(L-x)x$$

$$\tan\phi = \frac{dy}{dx} = \frac{4f}{L^2}(L-2x)$$

$$s = \left\{\sqrt{1+\left(\frac{4f}{L}\right)^2} + \frac{L}{4f}\log_e\left[\frac{4f}{L}+\sqrt{1+\left(\frac{4f}{L}\right)^2}\right]\right\}\times\frac{L}{2}$$

$\frac{x}{L}$	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	1.0983L	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	2.1448L
0.45	1.4047L	0.95	2.2338L
0.50	1.4789L	1.00	2.3234L