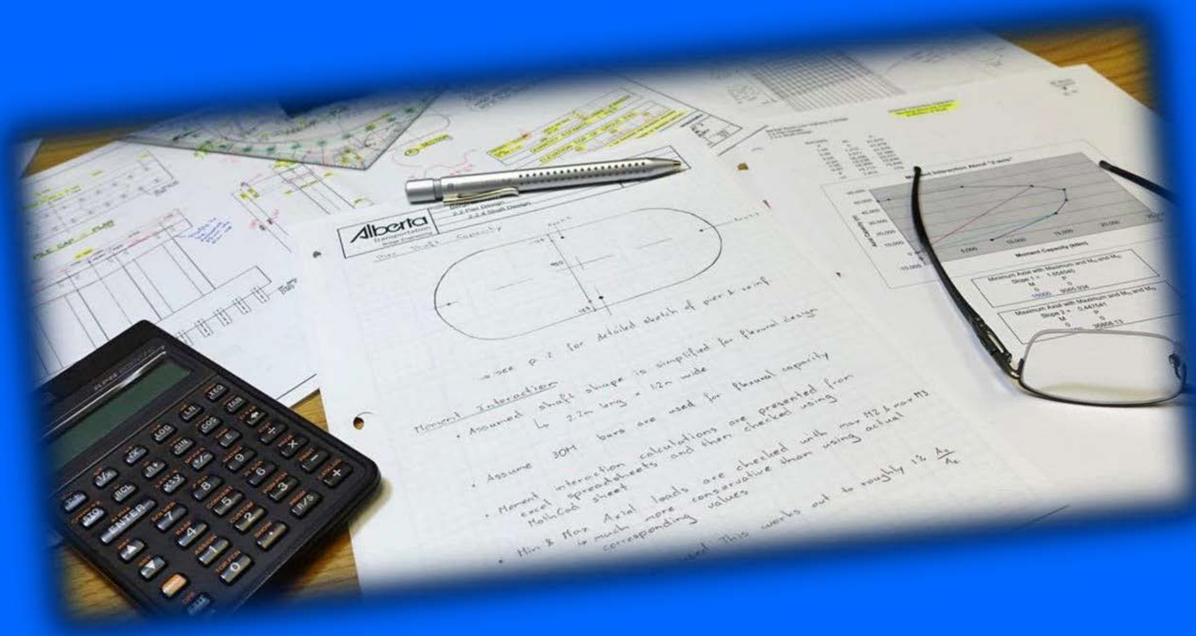

Bridge Structures Design Criteria

Version 8.0

Alberta Government

SUPERSEDED BY VERSION 8.1 SEPTEMBER 30, 2018

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Bridge Structures Design Criteria

Version 8.0

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BRIDGE STRUCTURES DESIGN CRITERIA

Technical Standards Branch

Alberta Transportation

April 2017

PREFACE

The Alberta Transportation Bridge Structures Design Criteria (BSDC) documents Alberta Transportation's bridge design practices and policies and is intended to supplement the requirements of the Canadian Highway Bridge Design Code CSA S6-14 (*CHBDC*).

While the document is primarily focused on structural engineering issues, it also touches upon conceptual design issues, drafting standards and material specifications. As a result, certain components in the document overlap with other Department documents. The Department strives to provide consistency between these documents, however occasionally changes to one document may not be immediately reflected in other documents. If a discrepancy is found, the Consultant should request clarification from the Department.

This document includes exceptions, modifications and clarifications of requirements in the *CHBDC.* Additionally, items pertaining to geometry, detailing, and materials are included to produce a reasonably uniform design product. In the Department's experience, these items have been found to reduce design, construction, inspection and maintenance problems, while providing a reasonable balance between safety, quality and cost.

It is not the intent of the document to limit progress or discourage innovation. Consultants are encouraged to explore all engineering options they deem appropriate for a specific site. The Department's Design Exception Process may be used to propose an engineering option that is not in accordance with this document.

Approved:

Date: 2017.4.10

for: Des Williamson
Director, Bridge Engineering Section
Technical Standards Branch
Alberta Transportation

D Williamson

Date: 2017.4.10

Des Williamson
A/ Executive Director, Technical Standards Branch
Alberta Transportation

LIST OF CHANGES

The following page is reserved for documenting changes to this version of the Bridge Structures Design Criteria. When changes are completed to the document, the following actions will be completed:

- The version of the document will be updated;
- A revision triangle will be placed next to the change in the document;
- A basic description and the date of the change will be summarized below.

Document Revision	Date	Description

Table of Contents

1. INTRODUCTION

This "Bridge Structures Design Criteria" (BSDC) applies to the design of highway and pedestrian bridge structures, bridge-sized culverts, overhead sign structures and retaining wall structures.

Unless noted otherwise in this BSDC, all structures shall be designed in accordance with "CSA-S6-14 Canadian Highway Bridge Design Code" (*CHBDC*).

Exceptions to the design requirements set out in this BSDC may be justified under special circumstances. In accordance with the Department's "Design Exception" process, such design exceptions shall be fully documented by the Consultant and must be submitted in writing for review and Approval by the Bridge Engineering Section.

1.1 Department Documents and Drawings

The Department has a number of other design documents that are relevant and shall be used in conjunction with this BSDC, including but not limited to:

- *Bridge Concept Design Guidelines (BCDG)*;
- *Design Guidelines for Bridge Size Culverts*;
- *Standard Specifications for Bridge Construction (SSBC)*;
- *Various Bridge Best Practice Guidelines*;
- *Roadside Design Guide (RDG)*;
- *Engineering Drafting Guidelines for Highway and Bridge Projects (Drafting Guidelines)*; and
- *Engineering Consultant Guidelines for Highway, Bridge and Water Projects Volume 1 - Design and Tender*.

In addition, the Department has a number of Standard Drawings and Typical Detail Drawings which are referred to within this BSDC. Standard Drawings have a prefix "S" in front of the drawing number. These are engineered documents. The Consultant shall refer directly to these documents on the project detailed design drawings and shall include them in the tender drawing set. These Standard Drawings often require additional project specific engineering and detailing to be included on the detailed design drawings. Standard Drawings are occasionally updated and Consultants shall ensure they are including the latest version of the drawings in their tender set. Typical Detail Drawings have a prefix "T" in front of the drawing number. These are not engineered documents, rather are documents provided to demonstrate the Department's preferred details. Consultants shall utilize the preferred details unless otherwise permitted by the Department. Consultants are fully responsible to properly design and draft all details on the project detailed design drawings. Typical Detail Drawings shall not be included in the drawing tender set.

At the time of publishing, this BSDC is consistent with the design and construction requirements presented in the above mentioned documents and the Standard Drawings and Typical Detail Drawings. However, these documents will be updated from time to time and the changes may not be immediately reflected in the BSDC. It is the Consultant's responsibility to resolve any differences. Typically, the latest version of the documents should govern, but if any doubt exists, the Consultant should discuss with the Bridge Engineering Section.

The following Department documents can be found on the Department's website. The latest version of these documents shall always be used.

Bridges and Structures: www.transportation.alberta.ca/565.htm

Bridge Conceptual Design Guidelines: www.transportation.alberta.ca/4865.htm

Design Guidelines for Bridge Size Culverts: www.transportation.alberta.ca/4871.htm

Standard Specifications for Bridge Construction (SSBC): www.transportation.alberta.ca/2653.htm

Bridge Best Practice Guidelines (BPG): www.transportation.alberta.ca/2649.htm

Bridge Standard Drawings and Typical Detail Drawings: www.transportation.alberta.ca/4738.htm

Engineering Drafting Guidelines for Highway and Bridge Projects: www.transportation.alberta.ca/2651.htm

Engineering Consultant Guidelines for Highway and Bridge Projects Volume 1 – Design and Tender: http://www.transportation.alberta.ca/915.htm

Roadside Design Guide (RDG): www.transportation.alberta.ca/3451.htm

Design Bulletins: www.transportation.alberta.ca/649.htm

The following Department standard drawings and typical detail drawings are referred to in this BSDC and can be found on the Department's website (www.transportation.alberta.ca/4738.htm). The latest version of these documents shall always be used.

S-1409-17 (Standard Concrete Slope Protection)

S-1638-17 (Standard Finger Plate Deck Joint Assembly - General Layout)
S-1639-17 (Standard Finger Plate Deck Joint Assembly - Typical Cross-sections)
S-1640-17 (Standard Finger Plate Deck Joint Assembly - Typical Drain Trough Details)

S-1642-17 (TL-4 Double Tube Type Bridgerail - Bridgerail Details)
S-1643-17 (TL-4 Double Tube Type Bridgerail - Approach Rail Transition Details)

S-1648-17 (TL-4 Thrie Beam On Curb Bridgerail - Bridgerail Details)
S-1649-17 (TL-4 Thrie Beam On Curb Bridgerail - Approach Rail Transition Details)

S-1650-17 (TL-4 Single Slope Concrete Bridge Barrier Details)
S-1651-17 (TL-4 Single Slope Concrete Bridge Barrier - Approach Rail Transition Details)

S-1652-17 (TL-2 Thrie Beam Bridgerail)
S-1653-17 (TL-2 Low Height Thrie Beam Bridgerail)

S-1681-17 (Bridgerail to Modified Thrie Beam Transition Details)

S-1700-17 (TL-4 Combination Barrier - Bridgerail Details)
S-1701-17 (TL-4 Combination Barrier - Barrier End Details)

S-1702-17 (TL-5 Double Tube Type Bridgerail - Bridgerail Details)

S-1703-17 (TL-5 Double Tube Type Bridgerail - Barrier End Details)

S-1704-17 (TL-5 Double Tube Type Bridgerail - Concrete Barrier Wall Details)
S-1705-17 (TL-5 Double Tube Type Bridgerail - Approach Rail Transition Details)

S-1797-17 (TL-2 Thrie Beam Bridgerail on 75mm High Curb)

S-1798-17 (TL-4 Single Slope Concrete and Double Tube Type Barriers along Top of MSE Wall)

S-1800-17 (Cover Plated V-Seal Deck Joint - Sheet 1)

S-1801-17 (Cover Plated V-Seal Deck Joint - Sheet 2)
S-1802-17 (Cover Plated V-Seal Deck Joint - Sheet 3)

S-1810-17 (Type I Strip Seal Deck Joint - Sheet 1)

S-1811-17 (Type I Strip Seal Deck Joint - Sheet 2)
S-1812-17 (Type I Strip Seal Deck Joint - Sheet 3)

S-1815-17 (TL-2 W-Beam Guardrail Approach Rail at Intersection Details)

S-1838-17 (Standard Waterproofing System For Deck and Abutments - Sheet 1)

S-1839-17 (Standard Waterproofing System For Deck and Abutments - Sheet 2)
S-1840-17 (Standard Waterproofing System For Deck and Abutments - Sheet 3)

S-1841-17 (Standard Drain Trough For Conventional Abutments)

S-1842-17 (Standard Drain Trough For Grade Separation Bridges With Integral Abutments)
S-1843-17 (Standard Drain Trough For Water Crossing Bridges with Integral Abutments)

S-1845-17 (Standard Pedestrian and Bicycle Barrier - Sheet 1)
S-1846-17 (Standard Pedestrian and Bicycle Barrier - Sheet 2)

S-1847-17 (Standard Identification Plaques and Benchmark Tablets)
S-1848-17 (Standard Identification Tags)

S-1851-17 (Standard Concrete Sealer Surface Treatment For Major Bridges)

S-1852-17 (Standard Concrete Sealer Surface Treatment For Standard Bridges)

T-1680-17 (Typical Curb and Wingwall Details)

T-1721-17 (Typical Overhead Sign Structure General Layout)

T-1757-17 (NU Girder Bridges Typical Details - Sheet 1)

T-1758-17 (NU Girder Bridges Typical Details - Sheet 2)

T-1759-17 (Steel Plate Girder Bridges Typical Details - Sheet 1)

T-1760-17 (Steel Plate Girder Bridges Typical Details - Sheet 2)

T-1761-17 (Typical Expansion Bearing Details)

2. EXCEPTIONS TO CSA-S6-14

Exceptions to "CSA-S6-14 Canadian Highway Bridge Design Code" (*CHBDC*) requirements are noted in this Section 2. Other minor exceptions may be noted throughout the rest of this document.

1. Notwithstanding *Clause 1.3.2 (General administrative definitions)* of the *CHBDC*, Regulatory Authority shall be defined as "the Department's Director of Bridge Engineering".

2. Notwithstanding *Clause 1.4.2.5 (Single-load-path structures)* of the *CHBDC*, approval shall not be given for the use of single load path structures. Exceptions to this are piers with three columns or less, and straddle bents, which are permitted as long as the requirements of Section 10 (Substructure and Foundations) are met. Slab and girder bridge structures shall have a minimum of four girder lines.

3. *CHBDC Clauses 1.3.4 (Hydraulic definitions)* and *1.9 (Hydraulic design)* shall not apply. Hydrotechnical design shall be in accordance with the Department's *Bridge Conceptual Design Guidelines*.

4. Notwithstanding *Clause 1.4.3. (Evaluation and rehabilitation of existing bridges)* of the *CHBDC*, bridge load evaluations and rehabilitations shall be in accordance Section 3.4 (Load Evaluation and Bridge Rehabilitation).

5. Notwithstanding *Clause 1.8.2.3.1 (General)* of the *CHBDC,* the spacing and capacity of bridge deck drains shall be determined in accordance with Section 13 (Bridge Drainage).

6. Notwithstanding *Clause 3.5 (Load factors and load combinations)* of the *CHBDC*, load factors for special permit vehicles shall be determined in accordance with Section 3.1 (Highway Bridges).

7. Notwithstanding *Clause 4.4.5.3.1 (Analysis requirements and design approach)* of the *CHBDC*, seismic design approach shall be determined in accordance with Section 3.1.2 (Seismic Design).

8. Notwithstanding *Clause 8.4.4.5.2 (Size)* of the *CHBDC*, for precast prestressed concrete I-shaped girders, the maximum size of post-tensioning ducts may be determined in accordance with Section 9.2 (Precast Prestressed Concrete Girder Bridges).

9. Notwithstanding *Clause 8.16.3.2 (Reinforcement)* of the *CHBDC*, for precast prestressed concrete I-shaped girders, the additional stirrups for end control cracking shall be determined in accordance with Section 9.2 (Precast Prestressed Concrete Girder Bridges).

10. Notwithstanding *Clause 10.7.4 (Camber)* of the *CHBDC*, welded steel girders spanning less than 25 m shall be cambered to compensate for dead load deflection and highway grade profile.

3. DESIGN LOADS

3.1 Highway Bridges

Highway bridges are defined as all bridges carrying vehicular traffic with or without pedestrian traffic.

For highway bridges, the design vehicle shall be a CL-800 Truck as defined in the *CHBDC*. No adjustments are required for the 9 kN/m uniformly distributed load for lane load.

For specific projects, the Department may require that new bridges are designed to accommodate special permit vehicles. For these situations, notwithstanding *Clause 3.5 (Load factors and load combinations)* of the *CHBDC*, load factors for special permit vehicles shall be determined in accordance with the Department's *Bridge Load Evaluation Manual*.

Live load distribution factors used for girder design shall not be less than the empirical factors determined in accordance with the *CHBDC*, unless otherwise Approved. If a bridge does not satisfy the criteria that allow the empirical factors to be used, the live load distribution factors used for girder design shall not be less than the empirical factors that would have been used if the bridge had met these criteria. The distribution factors used shall be shown on the detailed design drawings.

In *Clause 5.6.8* of the *CHBDC*, the width (B) of the bridge may be assumed to be reduced to a width that provides a value of B ≤ 10 m. The number of design lanes (n) shall be reduced as required and shall be consistent with the assumed bridge width (B).

As it relates to *Clause 3.4.4* of the *CHBDC*, the anticipated degree of pedestrian use for all highway bridges with sidewalks will normally be "occasional pedestrian use". When a higher anticipated degree of pedestrian/cyclist use is identified, a request to use "frequent pedestrian use" shall be submitted for Approval.

3.1.1 Fatigue Design

All new highway bridges shall be designed to comply with Class A Highway requirements (*Clause 1.4.2.2 of the CHBDC*). Fatigue stress ranges shall be determined using the CL-800 Truck. In addition, in *Clause 10.17.2.2* of the *CHBDC*, the C_L factor shall always be taken equal to 1.0. These requirements shall apply to all bridge components for considerations of structural fatigue.

All light poles mounted on bridges shall be designed to the requirements of AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals Fatigue Category I.

3.1.2 Seismic Design

Further to *Clause 4.4.4 (Seismic performance category)* of the *CHBDC*, all bridges located on or above a Level 1 or Level 2 highway (as determined in accordance with the Department's *Design Bulletin #27 - Provincial Highway Service Classification System*) shall be designated as "Major-route bridges". All remaining highway bridges shall be designated as "Other bridges".

Notwithstanding *Clause 4.4.5.3.1 (Analysis requirements and design approach)* and *Table 4.11* of the *CHBDC*, the "force based design (FBD)" approach is permitted for all major-route bridges that fall into "Seismic Performance Category 2".

3.2 Pedestrian Bridges

Pedestrian bridges are defined as all bridges that do not carry motorized vehicular traffic, but instead carry pedestrian traffic and/or bicycle traffic. The clear deck width of pedestrian bridges shall at a minimum meet that of the approach pathway, plus any additional width required for barrier shy offset as defined by the TAC Geometric Design Guide.

The minimum pedestrian bridge live loads shall be in accordance with *Clauses 3.8.9 (Pedestrian Load) and 3.8.11 (Maintenance Vehicle Load)* of the *CHBDC*. For flexible structures, dynamic response including vertical and/or side sway motions that could cause discomfort to pedestrians shall be considered.

For bridges that are expected to have high numbers of pedestrians or bridges on which pedestrians might reasonably be expected to stop and congregate, it may be more appropriate to design the pedestrian barrier in accordance with the loads and geometry requirements specified in the *Alberta Building Code (2014)*.

3.3 Vehicle Collision Load on Bridge Supports

Any bridge structure support located less than 10 m from the edge of the ultimate stage pavement shall be designed for a collision load at ULS Combination 8 loading.

For roadways with a design speed < 80 km/hr, the collision load shall be equivalent to an unfactored horizontal static force of 1400 kN and shall be applied in accordance with the *CHBDC Clause 3.15*.

For roadways with a design speed ≥ 80 km/hr, the collision load shall be equivalent to an unfactored horizontal static force of 1800 kN and shall be applied in any direction in a horizontal plane located 1.2 m above ground level at the pier.

3.4 Load Evaluation and Bridge Rehabilitation

Load evaluations of existing bridges shall be in accordance with the Department's *Bridge Load Evaluation Manual.*

Design for bridge rehabilitation is normally based on meeting the *CHBDC Section 15 (Rehabilitation and repair)*, except that the vehicles used for rehabilitation design shall be the Department's current rating vehicles (CS1 28t, CS2 49t and CS3 63.5t) as specified in the Department's *Bridge Load Evaluation Manual*. It should be recognized that when a bridge strengthening is required to meet the CS1, CS2 and CS3 vehicles, it may be possible to increase the strengthening design to meet the CL800 design vehicle with minimal impact on the overall rehabilitation cost. Considerations should include the type and age of structure, expected remaining life, and expected use of the highway.

4. BRIDGE GEOMETRY

The overall bridge and headslope geometry (eg. roadway profile, roadway plan, clear road width, streambed width, etc.) should be determined in accordance with the Department's Bridge Conceptual Design Guidelines and these BSDC.

4.1 Bridge Alignment

Where practical, bridges shall be on tangent horizontal alignments. Curved bridges require extra design and detailing, and cost more for construction and maintenance. If required, curve effects on operation safety, deck drainage, maintenance, etc., shall be fully considered during the preliminary engineering phase.

Where practical, skews on bridges should be minimized. Skewed bridges require extra design and detailing, and cost more for construction and maintenance.

4.2 Grades and Crossfall

For deck drainage purposes, the Department considers a minimum longitudinal grade of 1% to be desirable. However, the Department recognizes that gradeline constraints for grade separation structures may require crest curves that result in portions of the bridge deck having a grade less than 1%. Wherever possible, the tops of crest curves shall not be located on the bridge and shall be located beyond the end of the bridge approach slabs. It should be noted that reduced grades can result in standing pools of water on the deck, and may potentially cause operational and safety issues on the structure due to the presence of ice patches, and may require the coring of retro-fit drains through the deck or curb. In addition, reduced grades affect the sub-surface drainage (below the ACP) which will result in reduced life of the ACP overlay and deck and will result in accelerated structure deterioration.

Bridges that have a resultant slope of 4% or greater anywhere due to the combined effects of roadway grade and crossfall, and bridges that are located in areas where changes in traffic speed are required, shall be designed with considerations for an appropriate preferential icing mitigation strategy.

Bridge decks shall have a 2% crossfall away from the crown line unless the bridge structure is super-elevated.

4.3 Headslopes, Sideslopes and Approaches

The finished ground top of fill headslope width shall be equal to the "outside of bridge structure to outside of bridge structure" width plus at least 2.0 m (at least 1.0 m on each side of the bridge, as measured perpendicular to the faces of the wingwalls).

The top of fill headslope width shall be extended down the slope along and parallel to each of the wingwalls and abutment seats in an alignment that intersects with the top of the headslope or the backside of any MSE wall coping/swale, on both sides of the abutment (refer to sketch SK-B9 in Appendix B for illustrations of this requirement). The wingwall design shall ensure that it penetrates below the top surface of the headslope fill a minimum of 600mm depth of bury at all locations (refer to sketches SK-B1 to SK-B4 in Appendix B for illustrations of this requirement).

The width of fills beyond the bridge (ie. on the approaches) shall be sufficient to meet approach rail / guardrail standard requirements and shall transition to meet the roadway design sideslope requirements (refer to sketch SK-B9 in Appendix B for illustrations of this requirement). In cases where transition barriers are not required beyond the bridge end, the width transition from the top of headslope to the top of roadway sideslope shall taper at a rate of 30:1.

Corner transitions between the bridge headslope and the sideslope shall use an elliptical curve at the elevation of the toe of the headslope. A design tool is available at the AT Bridge Planning Tools FTP site (www.transportation.alberta.ca/PlanningTools/Tools/Planning/).

The approach roadways to bridges shall be designed for roadside safety requirements such as appropriate approach guardrail length of need, curved corner guardrails for intersecting ramps or roadways, barrier end cushions, and the safety of pedestrians and cyclists (*RDG*).

A new crash tested design for a TL-2 curved corner guardrail is provided on standard drawing *S-1815-17 (TL-2 W-Beam Guardrail Approach Rail at Intersection Details), S-1815-17 (TL-2 W-Beam Guardrail Approach Rail at Intersection Details)*. This can be used where intersecting roadways or crossing ramps are located near the end of the bridge and where the roadway design speed is ≤ 80 km/hr. For higher speeds, guardrail shall satisfy the length of need requirements in the *RDG*.

A design for terminating approach guardrail within a curbed roadway section is provided on standard roadway drawing *RDG-B1.11*.

4.4 Deck Width

Deck widths shall be determined in accordance with the *Bridge Conceptual Design* Guidelines.

In general, bridge deck clear roadway width shall match the width of the approaching roadway (existing or future planned). For long bridges, shoulders on bridges may be reduced for economic reasons, but this should be reviewed at the preliminary engineering stage and must be submitted for Approval.

Urban roadways often incorporate curb and gutter, and do not have shoulders. For bridges located on such urban roadways, the clear bridge deck width shall include shoulders which provide the appropriate shy distances to the bridge barriers. The transition between the bridge clear deck width (with shoulders) to the narrower approach roadway width (with curb and gutter) shall occur off the bridge. Additional widening may be required to meet minimum sight lines or to meet drainage requirements during peak storm flows.

Bridge decks shall not incorporate longitudinal joints. The clear clearance between two nominally parallel bridges shall not be less than 3.0 m in order to discourage pedestrians from attempting to jump from one bridge to the other.

4.5 Span Lengths, Substructure Stationing and Bearing Setting

4.5.1 Steel Girder and Cast-in-Place or Segmental Concrete Superstructures

Span lengths established from preliminary engineering requirements shall be rounded up to the nearest whole metre. The span lengths shown on 'General Layout' detailed design drawing shall be measured at a fabrication temperature of +20˚ C, from centreline bearing to centreline bearing along the bottom flange for uniform depth girders, and along the top flange for tapered or haunched girders.

Ground stationing for locating the centreline bearing of substructure elements shall be adjusted to account for the following:

- length difference between gradeline profile and horizontal surveyed distances;
- length difference due to thermal change between +20˚ C and -5˚ C;
- longitudinal shift due to off-plumb tilting of bearing stiffeners or control sections set perpendicular to the top flange, when span lengths are measured along the top flange;
- differences between grid and ground distances or other surveying systems (for more information on ground and grid coordinates refer to *Design Bulletin #34 – Grid-to-Ground Survey Application*).

For sliding expansion bearings, a bearing temperature setting chart shall be provided for positioning bearing components according to the girder temperature at the time of bearing setting.

For fixed bearings for continuous steel girder bridges, the design shall be based on bearings centred on girder bearing stiffeners at -5˚ C. However, girder erection can happen over a wide range of temperatures, which cannot be determined at the time of design. Since the bearings should stay centred on the bearing stiffeners, the piers should be designed for any eccentricity due to the shift of the bearings. The size of voids for grouting anchor rods should have sufficient room to accommodate girder length changes at the time of erection, in addition to normal construction tolerances.

4.5.2 Precast Concrete Girder Superstructures

Span lengths established from preliminary engineering requirements shall be rounded up to the nearest whole metre. Length of precast concrete girders is to be shown on the 'General Layout' detailed design drawings together with pier diaphragm thicknesses between girder ends, and distance from abutment girder end to centreline abutment bearing. Precast girder lengths shall be measured along the bottom flange at a fabrication temperature of +20˚C. The precast supplier shall make appropriate allowance for prestress shortening, shrinkage and creep up to the time of girder erection.

Ground stationing for locating the centreline bearing of substructure elements shall be adjusted to account for the following:

- length difference between gradeline profile and horizontal surveyed distances;
- length difference due to thermal change between +20˚ C and -5˚ C;
- differences between grid and ground distances, or other surveying systems (for more information on ground and grid coordinates refer to *Design Bulletin #34 – Grid-to-Ground Survey Application*).

Bridge bearings shall be centred on centreline bearing at -5˚ C. For expansion bearings, a bearing temperature setting chart shall be provided for positioning bearing components according to the girder temperature at the time of setting the bearing. Bearing design and setting chart shall make allowance for girder shortening due to post-tensioning and long term shrinkage and creep.

4.5.3 Curved Superstructures

For curved structures with equal girder lengths (parallel chords) within each span, measure span length along girder lines as defined above.

For curved or flared bridges with variable girder lengths (either curved or chords) within a span, measure span length along a selected girder line near the centre on the 'General Layout' detailed design drawing, with a cross-reference to the detailed 'Girder Layout' detailed design drawing showing complete geometry of all girders.

4.6 Horizontal Clearances

Horizontal clearances to bridge abutments, piers, retaining walls and overhead sign structure columns shall be in accordance with the *Bridge Conceptual Design Guidelines*.

For high speed under-passing roadways designated to have a barrier free roadside, the over-passing bridge structure supports, including abutments, piers, retaining walls and overhead sign structure columns, shall not be located within the under-passing roadway's clear recovery zone and shall allow all required sight distances to be met.

Clear recovery zone requirements and the location of the calculated critical clear zone shall be shown on the 'General Layout' detailed design drawings for all grade separations.

4.7 Vertical Clearance

Vertical clearance signs (WA-27) shall be provided on all bridge structures with under-passing roadways. The signs shall be mounted on the lower half of the "upstream" fascia girders at the location of the under-passing roadway where it is clearly visible to oncoming traffic. The sign should be centered over all the travel lanes in the direction of travel (one sign per direction). On divided roads with a separate collector-distributor road running parallel to the main road, an additional WA-27 sign should be placed over the center of the travel lanes of the collector-distributor road. The location of the critical vertical clearance(s) and the location of the WA-27 signs shall be shown on the 'General Layout' detailed design drawings and the sign attachment details shall be shown on the appropriate girder detailed design drawings.

The Department has a commitment to the trucking industry to not change vertical clearance and posting after the opening of a new bridge. At the time of any future works for the under-passing roadway, the finished road elevation under a bridge shall not be raised. The pavement structure under a bridge shall be provided with extra built-in thickness to accommodate any future milling and filling, or other roadway modifications. Repaving operations shall incorporate appropriate transition paving to tie in with the approach roadway away from the bridge. Transitioning profiles shall meet AT geometric standards.

When considering vertical clearances for bridges, two values must be considered: the clearance value to be used while designing the bridge; and the minimum allowable value that will be posted once the bridge is built. For example, for a conventional roadway grade separation, the Consultant shall use a vertical clearance of 5.40 m during the design phase. However, the actual minimum allowable posted clearance is 5.20 m.

For a roadway bridge over a railway, the minimum "as-constructed" vertical clearance shall be 7.01 m (for cost-share projects), as measured from the underside of the roadway bridge superstructure to the top of rail (refer to *RDG Figure H7.3*). When designing the bridge, Consultants shall include some additional vertical clearance for construction tolerance. For non cost-share projects, a smaller vertical clearance may be negotiated with the railway.

For an overhead sign structure over a roadway, the minimum "as-constructed" vertical clearance (as specified in the Standard Specifications for Bridge Construction) shall be 6.0 m, as measured from the bottom edge of the sign panel to the top of roadway. In addition, the bottom edge of the structural framing for the sign panel shall be at least 0.6 m higher than the bottom edge of the sign panels. When designing the overhead sign structure, some additional vertical clearance should be included for construction tolerance.

4.7.1 Design Vertical Clearance

The following design clearance values shall apply at the most critical location (refer to *RDG Figures H7.1 to H7.3*) and shall include allowance for any planned future ultimate stages (eg. bridge widening, etc.).

For bridges over roadways, the design vertical clearance shall be as follows:

- From the underside of roadway bridge superstructure to top of roadway 5.40 m
- From the underside of railway bridge superstructure to top of roadway 5.40 m
- From the underside of pedestrian bridge superstructure to top of roadway 5.70 m

The above design clearance values already include an allowance for construction tolerance.

4.7.2 Posted Vertical Clearances

For bridges, the minimum allowable posted vertical clearance shall be as follows:

- Underside of roadway bridge superstructure to top of roadway 5.20 m
- Underside of railway bridge superstructure to top of roadway 5.20 m
- Underside of pedestrian bridge superstructure to top of roadway 5.50 m

The Department's process for determining the vertical clearance posting for bridge structures is as follows:

i. Measure the minimum as-constructed vertical clearance between the roadway surface and lower bottom edge of the girder/structure within the roadway width (including shoulders) to the nearest centimetre (e.g. 5.37 m); then

ii. Subtract one decimetre for tolerance (ie. 5.27 m); then

iii. Round down to the nearest decimetre (i.e. Post vertical clearance as 5.20 m)

Vertical clearance measurements shall be made in accordance with *Chapter 7 - Vertical Measurements (VCL2)* in the Department's *Bridge Inspection and Maintenance System – Level 2 Inspection Manual*.

5. DURABILITY AND MATERIALS

Durability of bridge structures is essential to the overall good management of Alberta's bridge inventory. Durability relies on good design, detailing, selection of appropriate materials and good construction practices. This Section 5 addresses the material requirements as well as a number of design considerations that are related directly to durability. Other design and detailing considerations are integrated into other sections of this BSDC and construction requirements are addressed in the *SSBC*.

5.1 Design Life

Minimum design life of bridge structures shall be:

- Bridges including bridge size culverts** 75 years
- MSE walls 100 years
- Overhead sign structures 50 years

**Metal culvert liners may be designed with a service life of 50 years providing that they are oversized to allow for future lining. Cathodic protection is not permitted.

5.2 Bridge Decks, Roof Slabs and Approach Slabs

All bridge decks, roof slabs and approach slabs shall consist of Class HPC concrete and Corrosion Resistant Reinforcing (CCR) bars.

All bridge decks, roof slabs and approach slabs shall be protected with a waterproofing system (ie. membrane, ACP, etc.) as shown on standard drawings *S-1838-17*, *S-1839-17* and *S-1840-17 (Standard Waterproofing System for Deck and Abutments – Sheet 1, Sheet 2 and Sheet 3).*

The standard deck protection and wearing surface system has a total thickness of 90 mm consisting of a nominal 5 mm thick rubberized asphalt waterproofing membrane, plus 3 mm protective board, plus two 40 mm lifts of asphaltic concrete pavement. On Highways 1 - 216 and on high traffic local roads, a Type H2 Asphalt Mix using PG 58-28 asphalt cement grade shall be specified. On Highways 500 - 986 and low volume local roads, a Type M1 Asphalt Mix using a PG 52-34 asphalt cement grade shall be specified. Roof slab waterproofing membranes shall include wick drains along the gutter lines to allow for the controlled drainage of sub-surface water that penetrates the asphaltic wearing surface, and discharge of the sub-surface discharge at the bridge deck or roof slab ends only. For integral abutments, wick drains shall be continued to the ends of the approach slabs and discharge of the sub-surface water must be accommodated at the Control Joints at the ends of the approach slab.

Further discussion is also available in *BPG 3 - Protection Systems for New Concrete Bridge Decks*

5.3 Control of Bridge Deck Drainage

The number of deck joints shall be kept to a minimum. Bridge superstructures shall be continuous for live load over the piers and deck joints shall not be used in these locations. All deck joints shall include provision to capture and manage deck drainage such that it does not come into contact with other concrete and steel surfaces of other bridge elements other than concrete slope protection or drain troughs.

Bridge deck drainage shall be controlled and shall not be allowed to discharge onto any exposed substructure concrete surfaces, nor to discharge within 4 m of piers and abutments or pedestrian pathways, pedestrian bridges or multi-use trails, or to be directed onto the roadway pavement beneath.

Joints around abutments and approach slabs shall be sealed at the surface and kept sealed throughout the life of the structure with proper maintenance. Any steel elements (including buried elements) that may potentially be exposed to leakage of salt contaminated moisture shall be protected by an Approved impervious waterproofing membrane.

5.4 Splash Zone Surfaces

Splash Zone Surfaces are surfaces subject to or potentially subject to roadway splash or spray, and as a minimum shall include the following:

- Top surfaces of all pier and abutment concrete that project beyond the footprint of the bridge deck or bridge abutment, to a horizontal distance of 6 m from inside edge of the bridge barrier/curb. This includes the affected members of trellis structures and straddle bents.
- Vertical or near vertical faces of substructure elements, monolithic concrete protection barriers, or MSE wall precast concrete fascia panels that fall within a horizontal distance of 6 m of edge of lane of under-passing roadway.

Concrete slope protection and drain troughs are not treated as splash zone surfaces.

5.5 Sealer

Concrete sealers shall be specified in accordance with standard drawings *S-1851-17 (Standard Concrete Sealer Surface Treatment For Major Bridges)* and *S-1852-17 (Standard Concrete Sealer Surface Treatment For Standard Bridges)* and the *SSBC*.

5.6 Materials

All materials selected for a design shall be specified in the notes of the appropriate detailed design drawings, complete with the appropriate material properties used for design (ie. 28 day strength for concrete, yield or ultimate strength for steel, etc.).

5.6.1 Concrete Classes

Detailed information regarding concrete class material properties can be found in *SSBC Section 4: Cast-In-Place Concrete.* The following table gives concrete classes that shall be used in the specified locations on bridges.

DESCRIPTION	CLASS	f'c @ 28 days

DESCRIPTION	CLASS	f'c @ 28 days
Precast bridge girders (refer to *SSBC Section 7: Precast Concrete Units*).		50 to 70 MPa
Cast-in-place decks, curbs, barriers, sidewalks and medians; Cast-in-place concrete deck overlays; Abutment & pier diaphragms; Deck joint blockouts; Upper portions of abutment backwalls (300 mm minimum vertical dimension); Abutment roof slabs, approach slabs and sleeper slabs; Precast concrete partial depth deck panels; MSE wall precast concrete fascia panels and MSE wall cast-in-place copings; All Splash Zone Surfaces(1); If any portion of the following component is a Splash Zone Surface(1), the entire component shall be included (this applies to trellis beams, pier caps, or pier shaft/columns) Girder shear keys (14 mm max. aggregates), except SLW standard design.	HPC	45 MPa
Pile caps; Substructure elements and monolithic concrete protection barriers other than those specified as Class HPC; Overhead sign structure foundations (with the exception that cement shall be High Sulphate Resistant Type HS or HSb); Drilled caissons above the frost line; Concrete slope protection; Concrete drain troughs; MSE wall levelling pads.	C	35 MPa
Pipe pile infill concrete; Drilled caisson concrete below frost line.	Pile	30 MPa

(1) As defined in Section 5.4 (Splash Zone Surfaces).

5.6.2 Reinforcing Steel

Reinforcing steel shall conform to the grades outlined in the following table. In addition, the yield strengths shown in the following table shall be used for design with the applicable reinforcing steel grades:

REINFORCING TYPE	SPECIFICATION / DESIGN YIELD
Stainless steel bars	ASTM A276/A276M and ASTM A955/A955M including Annex 1.2 or 1.3 with a minimum yield strength of 420 MPa. The design and proportioning of the stainless reinforcing steel, including hooks, development lengths and bar splices, shall be based on a yield strength of 420 MPa.
Low Carbon/Chromium bars	ASTM A1035/A1035M alloy type CS with a minimum yield strength of 690 MPa based on the 0.2% offset method. The design and proportioning of the low carbon/chromium steel reinforcing bar, including hooks, development lengths and bar splices, shall be based on a yield strength of 500 MPa.
Carbon steel bars	CSA G30.18M Grade 400W with a minimum yield strength of 400 MPa.
Epoxy Coated bars	Shall meet the requirements for carbon steel reinforcing bars and shall be prepared and coated according to the requirements of the Ontario Provincial Standard Specification ("OPSS") 1442, *Material Specification for Epoxy-coated Steel Reinforcement for Concrete*, unless specified otherwise.
Deformed welded wire reinforcement	ASTM A1064/A1064M Grade 485 with a minimum yield strength of 485 MPa based on the 0.2% offset method.

The following gives steel reinforcing types that shall be used in the specified locations on bridges. For selection and detailing of Corrosion Resistant Reinforcing (CRR) types see Appendix C:

- Corrosion Resistant Reinforcing Bars (Low Carbon/Chromium or Stainless Steel)
  - full depth cast-in-place decks and partial depth cast-in-place decks composite with and cast on precast concrete partial depth deck panels;
  - reinforcing bars projecting from precast concrete partial depth deck panels;

  - curbs and barriers above the deck/wingwall construction joint, including dowels projecting through the construction joint;
  - sidewalks and medians;
  - abutment roof slabs, approach slabs and sleeper slabs;
  - all reinforcing bars in a trellis beam, straddle bent or pier cap where any portion of the component is a Splash Zone Surface;
  - located within 300 mm of the upper portions of abutment backwalls, diaphragms and wingwalls; and
  - located within 300 mm of Splash Zone Surfaces, unless otherwise specified.
- Low Carbon/Chromium Reinforcing Bars
  - stirrups projecting from precast or cast-in-place concrete girders into deck slabs.
- Stainless Steel Reinforcing Bars
  - deck joint blockouts;
  - corbels and dowels connecting approach slabs to corbels.
- Epoxy Coated Reinforcing Bars
  - MSE wall precast concrete fascia panels.
- Carbon Steel Reinforcing Bars
  - precast and cast-in-place concrete girders, except for low carbon/chromium stirrups projecting from girders into deck slabs;
  - all locations not otherwise specified.
- Deformed Welded Wire Reinforcement
  - precast concrete girders, except for low carbon/chromium stirrups projecting from girders into deck slabs.

Welding of structural reinforcing shall be prohibited. Welding of additional non-structural reinforcing shall be reviewed and shall be submitted for Approval. If approved, Grade 400W steel shall be used.

The minimum size of reinforcing bars (excluding welded wire mesh) in all bridge elements shall be 15M, except in drain troughs and concrete slope protection where 10M reinforcing bars may be used.

5.6.3 Prestressing Steel

For use in pretensioned and post-tensioned concrete, prestressing strands shall conform to ASTM A416/A416M Grade 1860 for low relaxation strand with a minimum tensile strength of 1860 MPa.

Prestressing rods shall conform to ASTM A722/A722M with a minimum tensile strength of 1035 MPa.

For soil nail wall systems, the soil nails shall be galvanized, centered in a protective sheath consisting of corrugated high-density polyethylene (HDPE) or polyvinyl chloride (PVC) pipe, and the annulus filled with an Approved grout.

5.6.4 Structural Steel

The following structural steel grades and material properties shall be used in the specified locations on bridges. Where more than one grade is provided, both are considered acceptable and the Consultant shall select as required to meet the design requirements. Occasionally, contractors will request material substitutions that are essentially equivalent to those used for design. The more common rough equivalent grades are included in brackets below. The strengths of these equivalent grades may not be exactly the same and it is the Consultant's responsibility to ensure that the design intent is satisfied when approving the substitution.

DESCRIPTION	GRADES
For girders and all material welded to steel girders; and For ungalvanized steel girder bracing members that are considered a Primary component (even if bolted to the girders). This typically applies to bracing members in heavily skewed bridges, curved girders or kinked girders.	CSA G40.21M Grade 350AT CAT 3 (ASTM A709 Grade 345WT Type B with Charpy value of 27 J @ -30˚ C)
For ungalvanized bearing materials; and For ungalvanized steel girder bracing members considered a Secondary component and bolted to the girders.	CSA G40.21M Grade 350A (ASTM A709 Grade 345 Type B)
For galvanized bearing sole plates and galvanized rocker plates, which are bolted to steel girders; For galvanized shoe plates cast into precast concrete girders and galvanized sole plates welded to shoe plates; For galvanized bearing base plates; and For galvanized precast concrete girder bracing members and bolted to the girders.	CSA G40.21M Grade 300W or Grade 350W and galvanized in accordance with ASTM A123/A123M
For galvanized miscellaneous steel components (eg. strip seal deck joint components, cover plated V-seal components, and cover plates for curb, barrier and raised median, but not including finger plate deck joint components, see below).	CSA G40.21M Grade 300W or ASTM A36, galvanized in accordance with ASTM A123/A123M and ASTM F2329 as applicable.

DESCRIPTION	GRADES
Finger plate deck joint components (except for galvanized cover plates for curb, barrier and raised median).	CSA G40.21M Grade 350A (ASTM A709 Grade 345W Type B)
For galvanized steel bridge rail components.	Plate steel and structural shapes shall conform to CSA G40.21M Grade 350 or ASTM A36, except that structural tubing shall conform to ASTM A500 Grade B (Fy = 315 MPa and Fu = 400 MPa) or ASTM A500 Grade C (Fy = 345 MPa and Fu = 425 MPa). All plate steel and structural shapes shall be galvanized in accordance with ASTM A123/A123M.
For structural steel bolts for weathering steel structural applications.	22 mm diameter ASTM F3125 Grade A325M Type 3 weathering steel
For structural bolts for galvanized steel applications.	Galvanized 22 mm diameter ASTM F3125 Grade A325M Type 1, galvanized in accordance with F2329

5.6.5 Anchor Rods Projecting from Concrete Base

The following anchor rod grades material properties shall be used in the specified locations on bridges.

DESCRIPTION	GRADES
For stainless steel bearing anchor rods in contact with black steel.	Stainless steel AISI Type 316 (Fy = 290 MPa based on 0.2% offset)
For galvanized steel bearing anchor rods in contact with galvanized steel bearing plates.	CSA G40.21M Grade 300W or ASTM A307, galvanized in accordance with ASTM F2329
For galvanized high strength steel anchor rods in contact with galvanized steel base plates (eg. galvanized bridge rail post base plates).	ASTM A193 Grade B7 (Fy = 725 MPa and Fu = 860 MPa). Note that galvanizing of high strength steel anchor rods requires a special procedure in accordance with *SSBC Section 6: Structural Steel* and *Section 12: Bridgerail*.

5.6.6 Steel "H" Piling

Steel "H" piling shall meet the requirements of CSA G40.21M Grade 350W or ASTM A36. Steel "H" piles designed to be exposed (e.g. pier columns) shall be galvanized to 1000 mm below ground or stream bed, in accordance with ASTM A123/A123M.

5.6.7 Steel Pipe Piling

Steel pipe piling shall meet the requirements of ASTM A252 Grade 2, except that hydrostatic testing is not required. Steel pipe piles designed to be exposed (e.g. pier columns) shall be galvanized to 1000 mm below ground or stream bed, in accordance with ASTM A123/A123M.

5.7 Clear Concrete Cover

The clear concrete cover for reinforcing steel, pretensioning strands, and post-tensioning ducts shall be in accordance with the following two tables. The clear concrete cover values shall be the basis for design and shall be specified on the detailed design drawings, unless noted otherwise on Department standard drawings. Where more than one concrete cover reference is inferable for a given situation from the following tables, the greater clear concrete cover value shall govern. The clear cover for anchorages and mechanical connections shall be those specified for reinforcing steel. Clear cover tolerances shall be in accordance with the *SSBC*.

Clear Concrete Cover for Cast-in-place Concrete Components

LOCATION	CLEAR CONCRETE COVER
Clear concrete cover unless noted otherwise.	60 mm
The bottom layer of approach slab on clean granular fill and polyethylene sheeting.	50 mm
The top layer of cast-in-place decks and slabs protected with waterproofing membrane and ACP wearing surface.	60 mm
The bottom layer of suspended decks and slabs.	40 mm
Concrete components which are not protected by a waterproofing membrane and ACP wearing surface, but will come into contact with de-icing salts, including Splash Zone Surfaces (excluding the near vertical traffic faces of curbs, medians and barriers).	70 mm
Near vertical traffic faces of curbs, medians and barriers.	100 mm
Concrete cast in contact with soil (no form).	100 mm

Note: Concrete covers in the above table measured to reinforcing steel.

Clear Concrete Cover for Precast Concrete Components

LOCATION	CLEAR CONCRETE COVER
Clear concrete cover to reinforcing steel unless noted otherwise.	50 mm
Precast concrete girder with 28 day compressive strength greater than or equal to 65 MPa:	
– measured to reinforcing steel	30 mm
– measured to pretensioning steel	45 mm
– measured to post-tensioning ducts	45 mm
Precast concrete girder with 28 day compressive strength less than 65 MPa:	
– measured to reinforcing steel	30 mm
– measured to pretensioning steel	50 mm
– measured to post-tensioning ducts	50 mm
Precast concrete partial depth deck panels – measured to pretensioning steel.	40 mm
MSE wall precast concrete fascia panels – measured to reinforcing steel.	50 mm

5.8 Attachments to Bridges

Other than vertical clearance signs (as described in Section 9 (Girders / Superstructures)) and ducts and conduits (as described in Section 14 (Ducts and Conduit Systems)), no objects (including but not limited to signs and utilities) shall be attached to any part of a highway bridge, pedestrian bridge or retaining wall.

6. Substructures, Foundations and Embankments

6.1 General Requirements

Bridge substructure components provide support to the superstructure. Substructure elements experience differential and global settlement, rotation or tilt, and other stressors. When these become excessive, they may cause premature deterioration of substructure and superstructure components which can result in costly repairs or reduced bridge service life. The Department has experienced the following problems with substructures:

- Abutment or pier settlement;
- Scour exposing pier foundations, and in extreme cases scour at abutment locations;
- Progressive headslope movement or approach fill failure due to geotechnical issues that push and tilt substructure elements;
- Bridge or highway drainage around abutments that undermine approach slabs, abutment seats, , drain troughs, concrete slope protection, and general slope erosion;
- Abutment, pier retaining wall translation or rotation; and
- Pile foundation construction problems.

New designs shall fully address the above-mentioned issues to avoid their occurrence.

Bridge substructures, foundations and embankments shall be designed in accordance with the *CHBDC* and supplemented as required by the *Canadian Foundation Engineering Manual, 4th Edition*. In accordance with the *CHBDC Clause 6.8 (Design liaison, contract documentation, and support during construction)*, it is expected that the geotechnical Consultant will be fully involved during the detailed bridge design, including review of the final embankment and foundation design drawings, and during bridge construction.

Further to *Clause 6.5.1 (Consequence classification)* of the *CHBDC*, all bridge foundations, embankments and retaining walls shall be classified as having a “Typical consequence” level unless otherwise Approved.

Geotechnical investigations shall be carried out in accordance with *Chapter 7* of the *Engineering Consultant Guidelines for Highway, Bridge and Water Projects Volume 1 - Design and Tender* and this BSDC. The selection of representative or “characteristic” geotechnical parameters used to determine foundation capacity shall be based on the results of appropriate field and laboratory investigations and shall represent the geotechnical Consultant’s “best estimate” of the likely values of the parameters, taking into account all the factors that may have influence on the soil properties. The key geotechnical parameters shall be presented in a table on the ‘Information Sheet’ detailed design drawing in accordance with the *Engineering Drafting Guidelines for Highway and Bridge Projects Appendix F – Standard Drawings Notes for Bridge Projects*. This includes test hole information as well as geotechnical foundation design parameters (eg. skin friction, end bearing, modulus of subgrade reaction, downdrag, group effects, estimated settlements, etc.).

Spread footing foundations, including those for open-bottomed culverts, shall not be used for foundations that are at risk of scour, including any foundations at the outside edge of river bends, on the banks of highly mobile streams or directly within stream beds. For more background information, see *BPG 7 - Spread Footings*.

For information on designing rock protection for stream related substructure elements refer to the *Bridge Conceptual Design Guidelines*.

6.2 Piling

The design pile tip elevations shall terminate a minimum of 3.0 m above the available geotechnical test hole tip elevation. This condition is required to reduce risk and uncertainty in the contractor's bidding process and to reduce the occurrence of claims based on differing soils conditions. If the pile design cannot satisfy this condition, additional geotechnical test holes information must be obtained to confirm the geotechnical parameters used for the pile design.

Permissible pile types are: concrete cast-in-place piles (friction and rock socket); driven steel pipe piles and driven steel H-piles (series HP 310 or larger piles shall be used for better driveability). Designs incorporating other pile types will require Approval.

Dynamically cast-in place piles (Compacto piles) are not permitted.

The following design pile load information for abutment and/or pier piles shall be shown in the 'General Notes' on the appropriate 'Information Sheet' detailed design drawing:

- SLS permanent loads only
- SLS extreme loads (combination # _)
- ULS permanent loads only
- ULS extreme loads (combination # _)

The "SLS extreme loads" shown on the 'Information Sheet' is the appropriate value to use with the 'Bearing Formula' prescribed in *SSBC Section 3: Bearing Piles*.

Welded pile splices are not desirable where tensile or flexural capacity is critical to the structural integrity of the bridge (eg. within the flexing length of integral abutment piles). However, as a result of the variability of pile driving, it is difficult to completely avoid splices in tension zones. As a result, the *SSBC* allow piles splices to occur in tension zones, but require 100% of these splices to be tested to confirm the quality of the splice. The detailed design drawings shall identify the extent of the tension zones (direct or flexural) and include a note stating that all welded splices within the tension zone shall require testing. The following note is an example:

> "ALL OF THE PILE SPLICE WELDS THAT ARE REQUIRED WITHIN THE TOP "X" METRES OF THE PILE ARE TENSION SPLICE WELDS"

6.2.1 <u>Pile Capacity</u>

As presented in the *CHBDC*, the ultimate geotechnical resistance for a pile may be estimated by static analysis, static test, dynamic analysis or dynamic test. The factored geotechnical resistance at ULS is then obtained by multiplying the ultimate geotechnical resistance by the relevant ultimate geotechnical resistance factor in accordance with the *CHBDC Table 6.2*.

The Consultant shall also assess the structural capacity of the pile. The pile must be sufficiently sized to accommodate pile driving loads as well as in-service loads. The in-service loads are particularly important for bridges with integral abutments.

Historically, the 'Bearing Formula' (otherwise known as the Hiley Formula) was most often used to determine the in-field geotechnical capacity of driven piles. The 'Bearing Formula' is a form of dynamic analysis, however it has been determined to be less reliable for certain type of soils. As a result, the geotechnical community has shifted towards the use of WEAP analysis, which is also a form of dynamic analysis. In accordance with *SSBC Section 3: Foundation Piles*, a WEAP analysis is the Department's preferred approach for determining the geotechnical capacity of driven piles. This approach requires close coordination between the bridge engineer and the geotechnical engineer during design and construction. The "Bearing Formula' shall only be used for standard bridges as approved by the Department and requires that the Consultant develop a site specific Special Provision indicating that the "Bearing Formula" is to be used.

Static analysis is typically used to determine the geotechnical capacity of cast-in-place concrete piles.

Depending on the cost of dynamic and static load tests, the economics of pile design may be improved by increasing the relevant ultimate geotechnical resistance factor in accordance with the *CHBDC Table 6.2*. Consultants shall consider the cost of carrying out more detailed geotechnical investigation and/or testing in order to reduce the pile costs. A discussion of the rationale used to balance the quality and quantity of geotechnical information collected with the foundation costs shall be included in the Structure Alternatives Report.

When determining the appropriate method for determining pile capacities, it is assumed that the geotechnical investigation has been completed in full accordance with *Chapter 7* of the *Engineering Consultant Guidelines for Highway, Bridge and Water Projects Volume 1 - Design and Tender* and that any pile drilling will be supervised by a competent geotechnical engineer. Pile capacity may be determined as follows:

6.2.1.1 Static Analysis

A static analysis uses empirical equations and correlations to estimate the pile capacity. This is typically used where the pile capacity is determined through correlation with the Standard Penetration Test (SPT). The pile design may be based on a geotechnical resistance factor using the "Typical" "Degree of understanding" from *Table 6.2* of the *CHBDC.* In order to utilize the "Higher" "Degree of understanding", geotechnical parameters should be based on the Cone Penetration Test (CPT). If for whatever reason it is not possible to obtain geotechnical boreholes at each foundation element, the pile design should be based on a geotechnical resistance factor using the "Low" "Degree of understanding".

6.2.1.2 Static Test

Static tests are typically used for larger diameter concrete piles or at sites with geotechnical complexity, in order to obtain a greater understanding of the predicted pile capacity and distribution of load between skin friction and end bearing. There are many site specific conditions that should be assessed prior to undertaking a static load test. If a minimum of one pile for each group of 100 piles is tested, the pile design may be based on a geotechnical resistance factor using the "Typical" "Degree of understanding" from *Table 6.2* of the *CHBDC.* The frequency of testing shall be increased to account for changing soil conditions, pile sections and types, distances between foundation elements, and construction methods. In order to utilize the "Higher" "Degree of understanding", tests would be required at each substructure element. Single load tests that are extrapolated to distant locations with soil conditions correlated by SPT methods would utilize the "Low" "Degree of understanding".

6.2.1.3 Dynamic Analysis

Dynamic analysis is typically used for driven piles using energy equations to estimate the pile capacity. The 'Bearing Formula' and WEAP analysis are both examples of dynamic analyses. In accordance with *SSBC Section 3: Foundation Piles*, a WEAP analysis is the Department's preferred approach and the "Bearing Formula' shall only be used for standard bridges as approved by the Department.

The following options exist for selecting the appropriate geotechnical resistance factor:

- If the 'Bearing Formula' will be utilized during construction (ie. for standard bridges), the pile design shall be based on a geotechnical resistance factor using the "Low" "Degree of understanding" from *Table 6.2* of the *CHBDC*;
- If a WEAP analysis will be utilized during construction, the pile design may be based on a geotechnical resistance factor using the "Typical" "Degree of understanding" from *Table 6.2* of the *CHBDC*; and
- If a WEAP analysis will be utilized during construction and the hammer efficiency is known (ie. will be measured at the beginning of the pile driving), the pile design may be based on a geotechnical resistance factor using the "High" "Degree of understanding" from *Table 6.2* of the *CHBDC.*

6.2.1.4 Dynamic Test

Dynamic tests are typically used for driven piles or smaller concrete piles in order to obtain a greater understanding of the pile capacity. If a minimum of 10% of the piles and one at each substructure element are tested, the pile design may be based on a geotechnical resistance factor using the "Typical" "Degree of understanding" from *Table 6.2* of the *CHBDC.* The frequency of testing shall be increased to account for changing soil conditions, pile sections and types, distances between foundation elements, and construction methods. In order to utilize the "Higher" "Degree of understanding", a minimum of 20% of the piles and at least two at each substructure element must be tested. Designs based on dynamic test results that are extrapolated and correlated to adjacent structures would utilize the "Low" "Degree of understanding".

6.3 Bridge Piers

Piers that are at risk of scour, including any piers at the outside edge of river bends, on the banks of highly mobile streams or directly within stream beds (refer to *BPG 7 - Spread Footings*), shall not be founded on spread footing foundations, but shall be founded on driven piles or drilled cast-in-place concrete piles with a minimum penetration of 5.0 m into competent material.

Land-based piers are typically founded on piles but they may be founded on spread footings as long as the spread footings are founded directly on competent bedrock, are well below the anticipated frost line, and there is no reasonable risk of loss of soil support adjacent to the pier footing. The geotechnical investigation for land-based piers with spread footings must confirm the slope stability of the terrain that the pier will rest upon. Typically, it is only economical to use spread footings if the competent bedrock is close to the ground surface. Construction risks due to size and depth of excavations, and potential for ground softening due to water exposure during construction need to be considered.

For roadway grade separation bridges, piers shall be designed to avoid collapse if the pier is struck by an errant vehicle. As a minimum: piers with one column or two columns shall have columns with a minimum cross-sectional area of 2.8 m^2 for each column; and piers with three or more columns shall have columns with a minimum cross-sectional area of 2.8 m^2 for each column, or alternatively may have smaller columns that are linked together with a strut extending from 1 m below ground to 1.4 m above the ground adjacent to the pier. Vehicle collision loads on pier columns/shafts shall be as per Section 3.3 (Vehicle Collision Load on Bridge Supports).

The tops of pier caps shall have a wash slope of 3% from the middle of the cap to the outside of the cap.

Ends of pier caps and pier shafts facing oncoming traffic shall be either circular or chamfered (minimum 300 x 300 mm).

Each end of pier cap cantilevers shall have a cast-in stainless steel drip strip across the full underside width of the pier cap, located 300 mm from the end of the pier cap to prevent staining of the substructure concrete.

The upstream face of river piers shall be protected by an embedded galvanized nose plate.

For monolithic pier diaphragms which are cast around girder ends, the girders shall be erected on a minimum 150 mm high plinth to provide sufficient clear space between the girder underside and previously cast supporting concrete, to facilitate flow and consolidation of concrete under the ends of the girders.

Contractors often have difficulty achieving formwork alignment and surface tolerances for large flat vertical concrete surfaces. For highly visible bridges (eg. roadway grade separation bridges), poor finishing on large visible surfaces (eg. pier shaft faces) can impact the overall aesthetics of the bridge. In order to prevent this concern (and reduce formwork challenges), if the pier shaft has a flat face area greater than 5 m^2, the Consultant shall incorporate an aesthetically pleasing reveal finish on the shaft face and provide details on the detailed design drawings.

6.4 Bridge Embankments / Abutments

6.4.1 General

The Department's preferred abutment configuration for major highway bridges is a pile supported cast-in-place concrete stub abutment seat with cast-in-place concrete wingwalls parallel to the over-passing roadway, located near the top of an open headslope. For smaller standard bridges, prefabricated abutment seats with wingwalls perpendicular to the over-passing roadway are often utilized.

The Department utilizes conventional abutments and integral abutments (both semi- and fully-integral). Integral abutments are the Department's preferred abutment type as long as the bridge conforms to the requirements in Section 6.4.3 (Layout of Retaining Walls at Abutments) and Section 6.4.4.6 (Integral Abutments).

Conventional abutments are those where superstructure movements and rotations are completely isolated from the abutment through the use of bearings and expansion joints. The Department uses simple conventional abutments (with abutment seats on piles, cantilever wingwalls, approach slabs, bearings and deck joints) and more complex conventional abutments (with roof slabs and grade beams on piles). Semi-integral and fully-integral abutments are discussed in Section 6.4.4.6 (Integral Abutments).

6.4.2 Embankments

Embankments shall include bridge headslopes, approach fills within 5m from the end of the bridge approach slab, transitions between the approach fill sideslopes to the headslopes and any retaining walls within this area.

The design of the bridge embankments shall account for stability, long term settlements and wall deformations. Stability analyses shall be carried out to determine that the embankments have acceptable short-term and long term stability and will satisfy ultimate and serviceability limit states design criteria. The global stability of bridge embankments shall be designed for a minimum factor of safety against failure of 1.5.

Deformations of the embankment (including settlement and lateral movements) shall be determined using appropriate deformation analyses, with representative soil parameters derived from site specific geotechnical investigations and local experience. The expected range of embankment displacements including settlement and lateral movements shall be taken into account in the design of the bridge and shall provide for acceptable structural and aesthetics performance of the embankments as well as the bridge. Embankments at bridge abutments shall be designed such that approach slabs, deck joints, bearings, barriers, and integral abutment piles in casings will operate as intended by the design without imposing excessive stresses on the structure, or requiring premature replacement of any bridge superstructure or substructure components due to long term movements of abutment seats. The structural design shall include soil structure interaction analysis where appropriate. Mitigating measures such as early fill placement, temporary surcharges, excavation and replacement of inadequate base material, wick drains, stone columns, lightweight fill, or soil reinforcement shall be carried out where necessary to limit long term movements. In addition to *CHBDC* requirements, differential settlement between the bridge structure and the approach fills shall not result in a deviation of more than 0.5% from the roadway design grade.

Conservative estimates of the long term vertical, longitudinal and lateral movements of headslopes and retaining walls that will follow after completion of construction shall be made. These movements shall be estimated at the elevations of deck joints, bearings and tops of piles as applicable. Joints, bearings and piles shall be designed to accommodate these long term movements over and above cyclical thermal movements and permanent prestressed girder creep shortening, in addition to an allowance for construction tolerances. The long term movements incorporated into the design shall be identified on the detailed design drawings.

Silt material specified as “ML” or “MH” material (in accordance with the “Modified Unified Soil Classification System”) and tire-derived aggregate material shall not be used in the design and construction of the bridge embankment.

Cellular concrete and expanded polystyrene (EPS) blocks may be considered for lightweight abutment fill, but will require Approval. Proposals for use of these materials must be submitted to the Department with well thought out details to ensure long term performance. Important considerations would include protection of ESP blocks from exposure to hydrocarbons; erosion due to leakage of highway drainage around abutments and wing walls; effects of differential settlement and other movements; buoyancy of EPS blocks when submerged; compatibility of material properties such as long term creep and short term elastic characteristics; etc.).

The geometry of embankments and retaining walls shall be sloped so that all drainage is directed away from bridge abutments. The drainage path shall be protected appropriately to prevent erosion.

6.4.2.1 Headslopes

The Department's preference is to specify open headslopes where ever possible. Open headslopes maintain the openness of the highway cross-section and visual consistency with the approaching roadway, which reduces driver workload and improves the operational quality of the under-passing roadway. Additionally, open headslopes facilitate access for inspection and maintenance at abutments and provides flexibility for cost effective future expansion of the under-passing roadway by cutting back the toe of the headslope.

The use of high abutments or retaining walls along the edge of the clearzone instead of open headslopes shall only be considered under constrained site conditions, such as restricted right of way in the vicinity of developed areas, and shall require Approval. Use of high abutments or retaining walls shall require Approval. Options showing high abutments or retaining walls must be presented for review to the Department during the choose design process with full documentation of supporting rationale. It should be noted that high retaining walls are often associated with long term movements and rotations that may increase future bridge maintenance. Further information on the layout of retaining walls at abutments is provided in Section 6.4.3 (Layout of Retaining Walls at Abutments). Additional design information for retaining walls in general is provided in Section 7 (Retaining Wall Structures).

For bridges over railways, a single span with MSE abutments and continuous retaining walls parallel to the railway is generally the most efficient for providing the required clearance box over railways.

Headslopes shall be protected with concrete slope protection as shown on standard drawing *S-1409-17 (Standard Concrete Slope Protection)*.

6.4.2.2 Inspection Access

Headslopes and retaining walls at abutments shall be configured to facilitate inspection of the girder ends, bearings, expansion joints and abutment seat. This inspection access shall not require the use of ladders or any specialized equipment, such as snooper trucks or fall protection.

For bridges with open headslopes, include the following:

- Provide a minimum 0.6 m wide bench at grade in front of abutment seats; and
- Limit the abutment seat height (maximum heights are illustrated on sketches SK-B1 to SK-B4 in Appendix B).

For bridges with abutments behind independent retaining walls, inspection access shall include the following:

- Provide a concrete walkway suitable for inspection access not less than 1.0 m wide (minimum dimension clear between abutment face and safety rail) in front of the abutment seat. This walkway shall be a concrete slab, monolithic with the abutment retaining wall coping;
- Provide a continuous safety railing at the outside edge of the walkway, designed as a "guard" in accordance with Part 9 of the Alberta Building Code (2014), having a minimum height of 1070 mm and consisting of vertical posts and not less than two horizontal rails. Chain link fence is not permitted;

- Limit the abutment seat height (maximum heights are illustrated on sketches SK-B1 to SK-B4 in Appendix B).

Further to the above, for side-by-side girder or solid slab bridges used in combination with integral or semi-integral abutments, very short abutment seats can be problematic for inspectors. For these bridges, the height from top of grade or top of walkway to underside of girders/slab shall be nominally 1.5 m.

6.4.3 Layout of Retaining Walls at Abutments

This Section 6.4.3 applies to the layout of independent high retaining walls at abutments adjacent to roadways and railways, and shall be read together with sketches SK-B5 and SK-B6 in Appendix B. All retaining walls must also meet the requirements of Section 7 (Retaining Wall Structures).

In this Section 6.4.3, bridge skew at an abutment shall be considered to be the skew angle between centreline of the over-passing roadway and the edge of shoulder of the under-passing roadway, or the centreline of track of the under-passing railway adjacent to the abutment.

Retaining walls at abutments shall preferably be placed parallel to the under-passing roadway except that the retaining wall wingwall on the approaching traffic side for the under-passing roadway shall be flared away from traffic at a flare rate of 20:1 and the end of the retaining wall wingwall shall be buried into the ground. For walls parallel to under-passing railways, the retaining wall wingwalls shall be flared away from the track at both sides of the abutments at a flare rate of 20:1.

Two retaining wall layout examples have been developed are included in Appendix B. These two layouts are assumed to include MSE walls with steel straps and therefore include some design features such as an impermeable geomembrane and drain troughs that are located away the ends of the bridge, as discussed in Section 7.2 (Mechanically Stabilized Earth Walls). This can complicate the design but is done to ensure that the steel soil reinforcement does not get exposed to de-icing salts. For bridges where MSE walls are being included in the design, the design features described below (and shown on the drawings in Appendix B) shall be incorporated onto the detailed design drawings. For retaining walls that do not include steel soil reinforcement, the same layouts shall be followed, but the impermeable geomembrane is not required and the drain troughs do not need to be located away from the end of the bridge to avoid draining over the steel soil reinforcement. Retaining walls at abutments shall have one of the following two layouts:

- Layout 1 (illustrated on sketch SK-B5 (Wall Layout and Site Drainage for Bridge Skew Angles ≤45 degrees) in Appendix B). For all bridge skews up to and including 45 degrees, retaining wall wingwalls shall be placed parallel to the under-passing roadway at all locations, except that the retaining wall wingwall on the approaching traffic side for the under-passing roadway shall be flared away from traffic at a flare rate of 20:1 and the end of the retaining wall wingwall shall be buried into the ground. For walls parallel to under-passing railways, the retaining wall wingwalls shall be flared away from the track at both sides of the abutments at a flare rate of 20:1. For this layout:
    - For MSE retaining walls, the roadway drainage at the end of the bridge shall be controlled and discharged into the drain troughs away from the ends of the bridge. This will require the use of a barrier system to direct the water. Sketch SK-B5 shows a mountable curb with a modified

thrie beam as one option. Other barrier options may be used to fit the site specific design considerations.

- Layout 2 (illustrated on sketch SK-B6 (Wall Layout and Site Drainage for Bridge Skew Angles > 45 degrees) in Appendix B). This layout shall only be used for skews greater than 45 degrees. For this layout:
  - At the acute angled corner of the bridge only, the retaining wall shall be turned back parallel to the over-passing roadway, and the exterior face of the retaining wall wingwall shall be set-back behind the exterior face of the traffic barrier along the top of the wall. Notwithstanding Section 7 (Retaining Wall Structures), the height of the turned-back portion may exceed 8 m, but shall not exceed 12 m as illustrated on sketch SK-B7 (Turned Back Wingwall Details for Bridge Skew Angles > 45 degrees) in Appendix B. Notwithstanding Section 6.4.4.4 (Approach Slabs), portions of the barrier may be integral with the approach slab over the turned back retaining wall wingwall;
  - The joints and connections between the bridge barrier and the approach barrier on moment slab is critical and very careful detailing is required to ensure that cracking of the barriers/wingwalls is minimized and leakage of roadway drainage does not occur through the joints;
  - Integral abutments are not permitted in conjunction with these turned back retaining walls as it becomes very difficult to accommodate the longitudinal thermal movements at the ends of the approach slabs while still controlling roadway drainage and avoiding leakage through the barrier joints. As a result, only conventional abutments with deck joints shall be used;
  - Other than for walls adjacent to under-passing railways, at all locations other than at the acute angled corner, retaining wall wingwalls shall be placed parallel to the under-passing roadway except that the retaining wall wingwall on the approaching traffic side for the under-passing roadway shall be flared away from traffic at a flare rate of 20:1 and the end of the retaining wall wingwall shall be buried into the ground; and
  - For walls adjacent to under-passing railways, at all locations other than at the acute angled corner, retaining wall wingwalls shall be placed parallel to the under-passing railway except that the retaining wall wingwall shall be flared away from the track at a flare rate of 20:1 beyond the abutment.

For both of the above-mentioned layouts, where the length of retaining wall wingwall on the approaching traffic side for the under-passing roadway extends more than 20 m beyond the abutment, only the end 20 m of the retaining wall wingwall shall be flared.

6.4.4 Abutment Design

Bridge ends shall be supported on piled foundations.

Any bridge components located immediately behind retaining walls, such as abutment seats, integral cantilevering wing walls, abutment deck joints, abutment bearings and traffic barriers, shall be designed to accommodate any movements resulting from vertical or lateral wall displacements.

MSE walls shall not be used behind abutments and soil reinforcing straps shall not be attached to abutment foundations, seats, backwalls or wingwalls to reduce lateral earth pressures. There is concern with excessive strains on the connections caused by long term differential settlement, which would occur under most circumstances.

The Department's preferred abutment design details shall be used for all major highway bridges. These are discussed in the following sections and are reflected on sketches contained in Appendix B:

- Sketch SK-B1 Conventional Abutment
- Sketch SK-B2 Conventional Abutment with Roof Slab
- Sketch SK-B3 Semi-integral Abutment
- Sketch SK-B4 Fully-integral Abutment

Bridge plaques and bench mark tablets shall be provided at bridge abutments in accordance with standard drawings *S-1847-17 (Standard Identification Plaques and Benchmark Tablets)* and *S-1848-17 (Standard Identification Tags)*.

6.4.4.1 Abutment Seats

The tops of abutment seats shall have a wash slope of 3% towards the front of the seat.

Proportion conventional abutments with maximum abutment seat height of 1.8m above grade (as illustrated in Section A (Headslope Option) on sketches SK-B1 and SK-B2).

Proportion integral abutment seats such that the maximum height from top of grade to underside of girders is 1.5 m (as illustrated in Section A (Headslope Option) on sketches SK-B3 and SK-B4).

For bridges with open headslopes, provide a minimum abutment seat embedment of 0.5 m below top of bench (as illustrated on sketches SK-B1 to SK-B4).

For bridges with abutment seats behind independent retaining walls, provide a minimum abutment seat embedment of 0.5 m below top of walkway (as illustrated on sketches SK-B1 to SK-B4).

6.4.4.2 Wingwalls

Bridge ends shall have cast-in-place wingwalls oriented parallel to the over-passing roadway.

For abutments without roof slabs, wingwalls shall be cantilevered from the abutment seat or the superstructure end diaphragm (as illustrated in the elevation views on sketches SK-B1 and SK-B2).

For conventional abutments, when wingwalls exceed 10m in length, the abutment shall be designed with roof slabs and grade beams (see Section 6.4.4.3 (Roof Slabs)).

Wingwalls shall extend beyond top of fill line by a minimum of 0.6 m, and shall be embedded a minimum of 0.6 m below top of grade at all locations (as illustrated on sketches SK-B1 and SK-B2).

For integral abutments, wingwalls shall meet the additional requirement of Section 6.4.4.6 (Integral Abutments).

Contractors often have difficulty achieving formwork alignment and surface tolerances for large flat vertical concrete surfaces. For highly visible bridges (eg. roadway grade separation bridges), poor finishing on large visible surfaces (eg. wingwall faces) can impact the overall aesthetics of the bridge. In order to prevent this concern (and reduce formwork challenges), if the wingwall has a flat face area greater than 5 m^2, the Consultant shall incorporate an aesthetically pleasing reveal finish on the wingwall face and provide details on the detailed design drawings.

6.4.4.3 Roof Slabs and Grade Beams

Abutments with roof slabs and grade beams are connected by the wingwalls to form a rigid box, and are considered to be very stable and provide significant resistance to movements and rotations.

Standard details for conventional abutments with roof slabs are shown on sketch SK-B2. Access to the cavity below the roof slab shall be provided through the abutment backwall, with access positioned between girders and accessible through the abutment diaphragm, in compliance with the requirements provided on Detail S "Access Door" on sketch SK-B2.

Roof slabs shall receive a 90 mm thick ACP deck protection and wearing surface system as per Section 5.2 (Bridge Decks, Roof Slabs and Approach Slabs) and standard drawings *S-1838-17*, *S-1839-17* and *S-1840-17 (Standard Waterproofing System for Deck and Abutments – Sheets 1, 2 and 3)*.

6.4.4.4 Approach Slabs

Approach slabs shall be in accordance with the provisions of the *CHBDC Clause 1.7.2*, except as modified in this Section 6.4.4.4.

Approach slabs shall have sufficient length to limit their rotation due to settlement to 0.5%, and shall have a minimum length of 6000 mm (measured parallel to centreline of roadway and as shown on sketches SK-B1 to SK-B4). Approach slabs for integral abutments shall be extended at least 1125 mm past the end of wing walls (as shown on sketches SK-B3 and SK-B4).

Approach slabs shall not be constructed with integral barriers or curbs, except for the barrier over the turned-back portion of an independent high retaining wall wingwall at the abutment (as shown on sketch SK-B6).

Approach slab thickness shall be as required by the Consultant but shall have a minimum thickness of 300 mm.

Approach slab reinforcement shall be as required by the Consultant but the bottom steel shall not be less than 20M @ 150 mm placed parallel to centreline of roadway and 15M @ 150 mm placed parallel to abutment backwall, and top steel shall not be less than 15M @ 300 mm each way.

Approach slabs shall receive a 90 mm thick ACP deck protection and wearing surface system as per Section 5.2 (Bridge Decks, Roof Slabs and Approach Slabs) and standard drawings *S-1838-17*, *S-1839-17* and *S-1840-17 (Standard Waterproofing System for Deck and Abutments – Sheets 1, 2 and 3)*.

Approach slabs shall be connected to the bridge in a manner that provides for free hinging rotation without causing restraining moments and forces.

Bridge approach fill may sometimes settle more than the specified design value based on 0.5% rotation of the approach slab. In this case, pavement smoothness is expected to be restored by milling and re-paving of the Asphalt wearing surface. Mud jacking of approach slabs is not recommended for integral abutments for the concern of bonding to the underside of the slab and preventing thermal movement.

6.4.4.5 Abutment Drainage Details

Abutment drainage details shall be incorporated into the design of abutments and shall include the following:

- The joints around the approach slab shall be well sealed to prevent water infiltration as detailed on standard drawing *S-1840-17 (Standard Waterproofing System For Deck and Abutments - Sheet 3)*;
- A 10 mm thick asphalt impregnated fibre board shall be placed between concrete slope protection or inspection walkway and conventional or semi-integral concrete bridge abutments (refer to sketches SK-B1, SK-B2 and SK-B3 in Appendix B). For fully-integral abutments, a similar detail shall be used with the addition of closed cell foam of adequate thickness (minimum 50 mm) to accommodate thermal movements between concrete headslope/walkway and the fully-integral concrete abutments (refer to sketch SK-B4 in Appendix B);
- A secondary system consisting of granular backfill, sheet wall drains and of sub-soil weeping drains shall be provided to collect, channel and remove any seepage. Sheet wall drains are typically spot-glued to the earth face of the abutment seat and wingwalls to intercept and channel seepage into a perforated weeping drain pipe complete with filter fabric sock. The pipe shall have a minimum positive drain slope of 2% and shall be day-lighted onto the headslope or sideslope and a galvanized screen shall be installed on the pipe opening to prevent small animals from entering the pipe;
- Sheet wall drains shall be omitted for MSE wall abutments with steel soil reinforcement, to eliminate a direct path for leakage to the steel reinforcement. Any leakage will then be forced to filter through the granular backfill and the steel reinforcement shall also be protected by an impermeable membrane;
- Clean, well graded, crushed granular backfill with a maximum aggregate size of 25 mm (Des 2, Class 25) shall be provided behind abutment seats and wingwalls complete with perforated weeping drains under the abutment seat and wingwalls;
- For abutments with MSE walls, drainage from deck joints and deck wick drains shall be drained into down spouts, which shall be installed in a channel recessed into the exposed face of the wall panels such that they can readily be inspected and serviced. Drainage swales along the top of MSE wingwalls shall always drain laterally away from the over-passing roadway at bridge abutments. Refer to the requirements in Section 7.2 (Mechanically Stabilized Earth Walls).

Bridge deck drainage shall always be controlled and channeled away from bridge components. Refer to Section 13 (Bridge Drainage).

6.4.4.6 Integral Abutments

Integral abutments shall include both fully-integral and semi-integral abutments. Integral abutments shall be used whenever practical, in accordance with this Section 6.4.4.6. For a more in-depth discussion on integral abutments, see Appendix A.

This Section 6.4.4.6 shall be read together with sketches SK-B3 and SK-B4 in Appendix B.

Integral abutments shall meet the general requirements for abutments identified in Section 6.4 (Bridge Embankments / Abutments). Additionally, integral abutments shall meet the following requirements:

- Integral abutments shall not be used for bridge spans greater than specified in “Table - Maximum Thermal Spans” below. The table below provides maximum thermal spans for joint types C1 and C2. The difference in concrete and steel bridge lengths reflects the effect of the greater thermal mass of concrete and the greater sensitivity of steel in reacting to ambient temperature changes. The thermal span shall be taken as the distance measured from the superstructure location which experiences zero longitudinal movement under temperature changes to the centreline abutment bearings (or centreline abutment piles);

Table - Maximum Thermal Spans

Joint Type	MAXIMUM THERMAL SPAN FOR STEEL GIRDER BRIDGES	MAXIMUM THERMAL SPAN FOR CONCRETE GIRDER BRIDGES
C1	*22.5 m*	*30 m*
C2	*45 m*	*60 m*

- Integral abutments shall not be used for bridges with skews greater than 20 degrees without Approval. The effects of skew, including the effects of earth pressure, the potential for twisting of the superstructure (in plan), and bi-axial bending of the piles, shall be analyzed and accounted for in all designs;
- The height of the abutment seat above grade shall not exceed 1.5 m, as shown on sketches SB-B3 and SK-B4 in Appendix B;
- Wingwalls for integral bridges shall be turned back and orientated parallel to the over-passing roadway. Wingwalls shall be attached to and cantilevered off the back of the superstructure end diaphragm. Maximum wingwall length shall be 8 m as measured from back of end diaphragm to end of wingwall;
- Integral approach slabs shall not be designed to move longitudinally in and out between stationary and parallel non-integral wingwalls;
- Integral abutments shall not be used when MSE walls are incorporated into the bridge abutment as wingwalls (for example with turned-back walls shown on sketch SK-B6 in Appendix B). In these

situations, it is too difficult to control salt laden water runoff at the sides and ends of the approach slabs, creating the potential for erosion and deterioration issues;

- Integral abutments should be avoided in situations where excessive settlement of the approach roadway/fills is expected;
- For fully-integral abutments:
  - The abutment foundation shall be a single row of H-piles oriented for weak axis bending wherever possible. For skewed bridges, it is common practice to orient the H-piles in the direction of the expected superstructure thermal movement;
  - For large movements exceeding the movement range of Type C1 control joints or when surrounding soils will restrict pile movement, piles shall be installed inside of permanent steel casings (refer to sketch SK-B4 in Appendix B). The casings shall be filled with Styrofoam beads. Styrofoam beads shall be new “Storopack” virgin polystyrene 14.4 kg/m3 (0.9 pounds per cubic foot) filler bead nominal diameter of 5 mm, or Approved equivalent - regrind or recycled polystyrene material is not acceptable and shall not be used. Steel casings shall be designed to last the same life as the bridge, and an appropriate sacrificial corrosion thickness and/or galvanizing shall be provided. The H-piles shall be embedded a minimum distance of two times the pile depth into the abutment seat;
  - When fully-integral abutments are being used at streams, the Consultant must consider the elevation of the top of the pile casing in relation to the normal water elevation to ensure that water is not getting trapped in the casing on a regular basis.
- Cycle Control Joint Types C1 and C2 shall be designed and detailed as shown on standard drawing *S-1840-17 (Standard Waterproofing System For Deck and Abutments - Sheet 3)* and as follows:
  - Cycle Control Joint Type C1 - The pavement shall be saw-cut at the end of the approach slab for crack control.
  - Cycle Control Joint Type C2 - A sleeper slab shall be provided under the end of the approach slab. The trench excavated for the installation of the sleeper slab and the granular base shall be extended across the full width of the road embankment and day-lighted onto the sideslope for drainage. The transverse wick drain in the control joint shall be drained into integrated the washed rock trench while the perforated drain pipe at the bottom of the trench shall be integrated with the abutment drain troughs if they are present as shown on standard drawings *S-1842-17 (Standard Drain Trough For Grade Separation Bridges With Integral Abutments)* and *S-1843-17 (Standard Drain Trough For Water Crossing Bridges with Integral Abutments)*.
- Cycle Control Joint Types C1 and C2 shall be located at least 1.125 m beyond the ends of the wingwalls by extending the length of the approach slab (refer to sketches SK-B3 and SK-B4 in Appendix B). Wick drains from the deck wearing surface and cycle control joints shall be day-lighted onto the sideslopes or connected to positive drainage;
- For monolithic abutment diaphragms which are cast around girder ends, the girders shall be erected on a minimum 150 mm high plinth to provide sufficient clear space between the girder bottom and previously cast concrete, to ensure proper flow of concrete under the ends of the girders. This is illustrated in Detail R on sketch SK-B4 in Appendix B;

- Two layers of polyethylene sheet shall be provided under the approach slab to minimize frictional forces due to horizontal movement as shown on standard drawing *S-1840-17 (Standard Waterproofing System For Deck and Abutments - Sheet 3)*. The connection between the approach slab and the superstructure shall be designed to resist these forces;
- The thrie beam approach rail transition shall be rigidly attached to the ends of bridge barriers, regardless of whether the barrier ends are stationary or moving;
- Where barriers are permitted to be constructed integral with approach slabs, the design shall be such that loss of barrier height due to settlement and overlay does not exceed 30 mm over the life of the structure. The joint between the barrier on the approach slab and the barrier beyond the approach slab shall be kept sealed;
- Provision shall be made for thermal movement between fully-integral abutment seats and the concrete headslope protection or inspection walkways. Gaps shall be protected against moisture ingress (refer to sketch SK-B4 in Appendix B).
- Additional deck reinforcement shall be provided for resisting negative bending moments at and near the ends of the superstructure due to flexural continuity of the girder-to-pile connection and the related torsional restraint of the stiff abutment diaphragms, adjacent girders, and earth pressure;
- Effects of construction sequence can have a very big impact of the success of bridges with integral abutments. There is often not one single approach that applies to all bridges. The Consultant must fully appreciate the potential effects of construction sequencing, shall choose the approach that is most appropriate for the site specific bridge, and must be explicitly clear if and how the Contractor is required to follow any procedures or provide any temporary measures to accommodate these effects. These requirements shall be stated on the detailed design drawings. The effects of construction sequencing shall include but not be limited to the following:
  - Effects of deck pour causing girder end rotation;
  - Effects of thermal changes. Thermal cycles during construction result in girder end rotations and lateral and longitudinal movements of the girders. This can then give rise to differential movement between the girders and the abutment with the girders shifting or the abutment seat moving. There are several options for accommodating these effects, including rigidly connecting the girder ends to the abutment with anchor rods (for lateral and/or longitudinal movements), or by simply allowing the girder ends to float on temporary bearing pads with or without temporary construction restraints.
  - Effects of post-tensioning on the abutment. Consideration shall be given to completing the post-tensioning before making the permanent connection between girder end and the abutment. Furthermore, temporary sliding bearings and/or anchoring of the abutment seat may be required to accommodate the longitudinal post-tensioning effects;
  - Stability of structure during all stages of construction; and
  - Backfilling behind abutments. It is typically recommended that backfilling behind abutments follow a balanced sequence so that the superstructure is not pushed out-of-position by backfilling operations.

- The installation of expansion foam material behind integral abutments for the purpose of relieving earth pressures shall not be permitted. The soft material can get compressed or punctured by backfill compaction during construction. In the long run, cyclical movements may cause progressive plastic compression, and promote unequal movements at the two abutments, resulting in a net shift to one side for the whole structure.

6.4.5 Slope Protection

Bridge slopes shall be protected from deterioration and erosion. For grade separations, concrete headslope protection shall be provided in accordance with standard drawing *S-1409-17 (Standard Concrete Slope Protection)*. For river crossings, headslope protection is provided using rock riprap, in accordance with the Department's *BCDG*. Occasionally, concrete slope protection may be used at river crossings above the high water level. This is typically reserved for aesthetic reasons in urban situations. Additional requirements for preventing slope protection are included in Section 13 (Bridge Drainage).

7. RETAINING WALL STRUCTURES

7.1 General

This Section 7 shall apply to all retaining wall types including gravity walls, piled walls, cantilever walls, anchored walls and Mechanically Stabilized Earth walls (MSE walls). In addition, this Section shall apply to all retaining wall locations, whether or not they are located at or in conjunction with a bridge structure. This Section does not apply to wingwalls that are cantilevered from the abutment seat or the superstructure end diaphragm, which are addressed in other sections of this document. Additional requirements that are specific only to MSE Walls are presented in Section 7.2 (Mechanically Stabilized Earth Walls).

Non-mechanically stabilized earth retaining walls shall be designed in accordance with the provisions of the *CHBDC*.

The height of any retaining wall, or the combined height of multiple retaining walls, shall not exceed 8.0 m at any location adjacent to roadways, or 12 m adjacent to railway grade separations. The height of retaining wall for this purpose shall be taken as the vertical height from top of coping to top of finished grade in front of the wall.

All walls shall be designed so that in the final position, they will be battered back against the retained soil from a vertical plumb line by a ratio of 50 vertical units to 1 horizontal unit.

Any bridge components located behind retaining walls, such as abutment seats, integral cantilevering wingwalls, abutment deck joints, abutment bearings and traffic barriers, shall be designed to accommodate any movements resulting from lateral wall displacements.

Where retaining walls are placed within the footprint of a bridge deck and in close proximity to a bridge abutment, a concrete walkway shall be provided between the retaining wall and the front of the abutment for inspection purposes (see Section 6.4.2.2 (Inspection Access)). The walkway shall be as detailed on sketch SK-B8 in Appendix B. The walkway include a minimum 600 mm wide swale and 3 percent wash slopes directing water away from the abutment seat and away from the face of the retaining wall towards the swale. The walkway swale shall be detailed so that it drains longitudinally with a minimum 0.5% grade towards the grassed swales outside of the bridge abutments. Where retaining walls are placed within the footprint of a bridge deck but further away from the bridge abutment, a headslope shall be provided between the top of the retaining wall and the bridge abutment. The headslope portion shall be protected with concrete slope protection and a concrete swale shall be incorporated at the base of the headslope to direct drainage to the sideslopes. Beyond the footprint of the bridge deck, grassed swales shall be provided along the top of retaining walls (see details on sketch SK-B8 in Appendix B). All swales shall be designed for the 1:100 year storm event without over-topping, but shall have a minimum width of 600 mm, a minimum depth of 150 mm and a minimum longitudinal (parallel to the face of the wall) slope of 0.5%. The swales shall include a non-degradable erosion control mat and have a bottom liner of impermeable geomembrane that has positive drainage to the ends of the walls. Swales and top of walls shall slope away from bridge abutments. Mitigating measures shall be designed to direct flow away from the toes and ends of walls, and to prevent erosion at these locations and at drainage swale discharge points.

In locations where traffic runs adjacent to the top of, and nominally parallel to a retaining wall, a rigid barrier shall be provided. For the purpose of determining the appropriate Test Level requirements for the rigid barrier, the retaining wall shall be considered to be a structure and the requirements of the *CHBDC Section 12 (Barriers* *and Highway Accessory Supports)* shall apply. The retaining wall shall be designed to fully resist the collision loads applied to the barrier, and loads from any attachments such as signs and lamp posts. Approach rail transitions shall be provided at the ends of the rigid barrier.

In locations where a sidewalk or a combined pedestrian/cyclist use pathway runs adjacent to the top of, and nominally parallel to a retaining wall, a pedestrian and bicycle barrier shall be provided. The retaining wall shall be designed to fully resist the loads applied to the barrier, and loads from any attachments such as signs and lamp posts.

Unless a traffic barrier, pedestrian rail or bicycle barrier is mounted directly on top of a retaining wall, a safety railing shall be mounted on the top concrete surface of all retaining walls and shall be designed as a “guard” in accordance with Part 9 of the *Alberta Building Code (2014)*. The Consultant shall design and detail the safety railing and anchorages on the detailed design drawings. Safety railings shall have a minimum height of 1070 mm and shall consist of vertical posts with not less than two horizontal rails. Chain link fence is not permitted. Safety railing anchorage assemblies shall be cast into the top concrete surface and shall not be field drilled. All steel components for safety railings shall be galvanized in accordance with ASTM A123/A123M and F2329. All steel components for safety railings shall conform to CSA G40.21M Grade 300W, except that anchor rods conforming to ASTM A307 are also acceptable. Retaining walls shall be designed to resist the loads from all barriers fastened to the walls.

Toe slopes within the clearzone in front of retaining walls that run nominally parallel to the adjacent roadway shall be covered with concrete slope protection and shall have a slope flatter than 1:3 horizontal to allow for safe vehicle recovery.

In locations where traffic runs adjacent to the top of, and nominally parallel to a retaining wall, roadway drainage and the allowable encroachment of rain runoff into adjacent traffic lanes shall be accommodated in the same manner as on bridges and in accordance with Section 13 (Bridge Drainage).

Dry cast and stacked concrete and/or masonry block walls with or without soil reinforcement are not permitted and shall not be used.

7.2 Mechanically Stabilized Earth Walls

The requirements in this Section 7.2 shall apply specifically to mechanically stabilized earth retaining walls (MSE walls). Additionally, there are requirements in other sections that also apply to MSE walls (see Section 5 (Durability and Materials), Section 6.4 (Bridge Embankments / Abutments) and Section 7.1 (General)).

MSE walls are proprietary designs and shall meet design/build requirements in *SSBC Section 25: Mechanically Stabilized Earth Wall*. Additional guidance is also available from *FHWA “Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes - Volumes I and II”, November 2009 (FWHA-NHI-10-024 and FWHA-NHI-10-025)*.

7.2.1 Geometric Requirements

The geometry of MSE walls, including associated headslopes and embankments, shall be sloped so that all drainage is directed away from bridge abutments.

MSE wall backfill shall extend a minimum of 0.5 m beyond the end of the soil reinforcement.

Acute corners less than 70° (measured between backfill sides of precast concrete fascia panels) shall not be allowed.

7.2.2 Surface Drainage

Sketches SK-B5 and SK-B6 in Appendix B illustrate a number of the following requirements.

Highway and bridge surface drainage shall be controlled and channeled away from the back of the MSE walls and the mechanically stabilized earth mass.

Water carrying appurtenances, such as catch basins, drainage inlets/outlets, culverts etc., shall be placed away from, or beyond the ends of the soil reinforcement zone, and provisions shall be made to mitigate the detrimental effects of potential leakage. No drain trough or wick drain carrying roadway drainage shall be located over the steel soil reinforcement, and no drain pipe carrying roadway runoff shall be located within the MSE wall soil reinforcement.

All galvanized steel soil reinforcement shall be protected from exposure to roadway de-icing salt by an impermeable geomembrane placed above the top layer of soil reinforcement. This shall include soil reinforcement directly below a roadway, as well as immediately adjacent to the roadway for a minimum width of 5 m parallel to the outer edge of roadway shoulder. In addition, for MSE walls that run parallel to the roadway, impermeable membrane shall be provided to intercept any drainage from the roadway base layer and direct it away from all MSE walls. The membrane shall be sealed to prevent leakage, sloped at a minimum 5% to drain away from the bridge and wall and be connected to an outlet beyond the MSE soil mass. A non-woven geotextile filter fabric layer shall be placed below and above the membrane to prevent puncture. In all cases the geomembrane material shall be made continuous and water-tight, and shall extend a minimum of 500 mm beyond the extent of the steel soil reinforcement. Any necessary joints shall be shingled in the direction of drainage and welded or bonded to prevent leakage.

Where required, Consultants shall include downspouts to accommodate deck joint and deck wick drain drainage. Downspouts shall not be directed through the mechanically stabilized earth mass, rather they shall be recessed behind the front face of the wall and shall deposit drainage directly onto the slope protection in front of the wall. This shall be accomplished by using special recessed precast concrete fascia panels with a painted steel plate covering the recess. Downspouts shall have a vertical slip joint with a dished top drain inlet cast into the wall coping.

7.2.3 Sub-Surface Drainage

Sub-surface drainage is defined as drainage occurring within the mechanically stabilized earth mass. While it is the MSE wall supplier's responsibility to incorporate sub-surface drains into their backfill, it is the Consultant's responsibility to determine the expected water level in the mechanically stabilized earth mass and to tie the sub-surface drainage in with the overall site drainage. In most circumstances, it is reasonable to assume that with properly designed backfill and sub-surface drainage, the water level within the mechanically stabilized earth mass will be at the invert level of the weeping drains. If the Consultant determines that a higher water elevation is likely to occur, the expected design water level shall be shown on the detailed design drawings as a loading condition so that the MSE wall supplier can incorporate into their design.

As a minimum, continuous weeping drains consisting of perforated 150 mm diameter pipe complete with filter sock shall be provided near the front and the back bottom corners of the mechanically stabilized earth mass. The weeping drains shall be day lighted into drainage ditches or connected to drainage collection lines for positive drainage and disposal in accordance with jurisdictional requirements. The high end of the drains shall be capped and sealed to prevent the ingress of native or backfill materials.

7.2.4 Waterways, Water Carrying Appurtenances and Utilities

MSE walls shall not be used adjacent to waterways.

MSE walls shall not be placed over or in the vicinity of any utilities or water carrying appurtenances, unless such utilities or water carrying appurtenances can be removed and repaired without disturbing the mechanically stabilized earth mass, excavation of such utilities or water carrying appurtenances can be executed without impact on wall stability, and agreement is obtained from the utility owners. Water carrying appurtenances include catch basins, drainage inlets/outlets, and culverts. As illustrated in section A on sketch SK-B5 and SK-B6 in Appendix B, utilities or water carrying appurtenances carrying potentially eroding materials shall not be permitted within 10 m of any MSE wall backfill unless the utilities or water carrying appurtenances are sufficiently enclosed within a containment structure that is designed to prevent exposure of any leakage from the utilities or water carrying appurtenances into or onto the MSE wall system, and the extent and design of the containment structure shall be sufficient to protect the MSE wall system against any discharges from this containment structure. No change of direction of utility lines, and no valves, valve chambers or any other discontinuity shall be permitted within the mechanically stabilized earth mass.

7.2.5 Facing

All MSE walls shall be faced with precast concrete fascia panels. The non-exposed side of MSE wall precast concrete fascia panels shall be in full contact with compacted backfill.

In locations where traffic is running adjacent to the bottom of, and nominally parallel to a MSE wall, and where thrie beam approach rail transitions are anchored to one or both ends of the MSE wall, anchor blocks with sufficient strength shall be provided for anchoring of the thrie beam transitions where they connect to the ends of the MSE wall. The Consultant shall identify the location, barrier load and any relevant details so that the MSE wall designer can incorporate into the wall design.

7.2.6 Coping Cap

A cast-in-place concrete coping cap shall be placed on the top of all MSE walls, except situations where the section of MSE wall requires a cast-in-place concrete barrier slab. The top of the cast-in-place concrete coping cap shall be smooth, have no steps or abrupt changes in height and shall have a 3% wash slope towards the back of the wall (see details on sketch SK-B8 in Appendix B).

Copings shall have control joints and shall have drip grooves in the soffit. Control joints and drip grooves shall be detailed in accordance with typical detail drawing *T-1680-17 (Typical Curb and Wingwall Details)*. Control joints shall be located to line up with the precast concrete fascia panel joints, shall be perpendicular to the wall alignment, and shall in no case exceed 4m spacing. At control joints, all longitudinal reinforcing in the coping cap shall be discontinuous and shall have 50 mm cover measured from the centre of the control joint.

7.2.7 Barriers/Railings

Barriers and railing on top of MSE walls shall be provided as per Section 7.1 (General). Where rigid bridge barriers are to be located on top of the MSE wall, they shall be supported on moment slabs to resist sliding and overturning. TL-4 bridge barriers on top of MSE walls shall be detailed in accordance with standard drawing *S-1798-17 (TL-4 Single Slope Concrete and Double Tube Type Barriers along Top of MSE Wall)*.

The Consultant shall fully design and detail all barrier and safety railing components. The Consultant shall show all barrier and safety railing loads on the detailed design drawings so that the MSE wall supplier can incorporate into their wall design.

7.2.8 Vertical Slip Joints

In instances where a continuous length of MSE wall cannot be built all at the same time as a result of an existing obstruction (e.g. existing bridge, or existing roadway, etc.), the continuous length of MSE wall may be divided into multiple sections and some or all of the sections constructed at different times. In such instances, if large differential settlement is expected between any of the various sections of the MSE wall, appropriately designed full height vertical slip joints shall be provided between adjacent MSE wall sections. This manner of staging MSE wall construction shall not be confused with Two Stage MSE Wall construction.

7.2.9 Obstructions within the Backfill

Obstructions within the mechanically stabilized earth mass, such as foundation piles and associated casings, casings for future pile installations, or other obstructions, shall be accommodated with appropriate arrangement of soil reinforcing around such obstructions. Consultants shall assume that the splay angle of the soil reinforcement shall not exceed 15° from the perpendicular of the precast concrete fascia panel.

7.2.10 Consultant Responsibilities with MSE Walls

The Consultant's responsibilities for MSE walls are summarized as follows:

- Perform geotechnical investigation, short term and long term global stability analysis, settlement and movement analysis, and confirm feasibility of MSE wall.
- Provide conservative estimates for long term settlement, rotation and translation, to ensure compatibility with bridge structural components.
- Perform site survey to establish wall location and layout.
- Prepare 'Wall Layout' detailed design drawing and special provisions to include the following information:
    - MSE wall location, layout, and geometry control;
    - Overall site drainage and sub-surface drainage, including coordination of bridge deck drainage and surface drainage with the MSE wall components;

  - Allowable bearing capacity, allowable rate of fill placement and settlement predictions;
  - Barrier and safety railing designs;
  - All loads being exerted onto the MSE wall components that are not part of the MSE wall supplier’s scope (eg: loads from roadway barriers, safety railing, attachments, high water level, etc.).
- Review design notes and shop drawings submitted by the contractor to ensure all specification requirements are met. This shall include a review of the contractor’s global stability check using actual soil properties of the backfill used for the project. This review is for conformance of specifications requirements only and not a design check.
- Monitor construction as required to meet the *Engineering Consultant Guidelines Highway, Bridge and Water Projects*, with particular attention to the following:
  - Confirm site preparation, site drainage and wall geometry is in accordance with the design;
  - Confirm fabrication is in accordance with the contractor’s design of the MSE wall components;
  - Confirm construction of the MSE wall is in accordance with the *SSBC Section 25: Mechanically Stabilized Earth Walls*, the contractor’s MSE wall construction specifications and the contractor’s MSE wall design.
- At the end of the construction, the Consultant shall update and submit the ‘Wall Layout’ detailed design drawing to reflect as-built conditions, and forward all reviewed supplier design notes and shop drawings to the Department for record keeping purposes.

8. BRIDGE BEARINGS

The Consultant shall select, design, and specify bearing components in accordance with Appendix D.

Fixed steel rocker plate bearings and plain unreinforced bearing pads shall be fully designed and detailed by the Consultant on the detailed design drawings.

For other bearing types, the bearing components between the sole plate and the base plate shall be designed and detailed by the bearing supplier in conformance with *SSBC Section 8: Bridge Bearings*. Shop drawings shall be stamped by the supplier's engineer, who shall be a Professional Engineer licensed to practice in Alberta.

The Consultant's responsibilities for bearings designed by a bearing supplier are summarized as follows:

1. The Consultant shall provide the following information on the detailed design drawings:
    - Bearing layout;
    - Bearing types and required number for each type;
    - Bearing schedule showing design loads, translation and rotation requirements;
    - Temperature setting graphs for construction and for long term to account for elastic, shrinkage and creep shortening of prestressed girders;
    - Sole plate dimensions and connection to girder details;
    - Base plate, temporary pintle supports, anchor rods, and grout pad details;
    - Reinforced concrete shear block details for lateral restraint; and
    - Bearing Setting Elevation Table showing top of sole plate elevations, estimated bearing heights and estimated top of grout pad elevations.
2. The Consultant shall review the design notes and shop drawings submitted by the contractor to ensure they meet all requirements of the detailed design drawings, these *Bridge Structures Design Criteria* and *SSBC Section 8: Bridge Bearings*. This review is for conformance of specification requirements only and is not a design check.
3. At the end of the construction, the Consultant shall update the 'Bearing' detailed design drawing(s) to reflect as-built conditions, including the Bearing Setting Elevation Table, and forward all reviewed supplier design notes and shop drawings to the Department for record keeping purposes.

9. GIRDERS / SUPERSTRUCTURES

9.1 General

Vertical clearance signs (WA-27) shall be provided on all bridge structures with under-passing roadways. The signs shall be mounted on the lower half of the “upstream” fascia girders at the location of the under-passing roadway where it is clearly visible to oncoming traffic. The sign connections shall be shown on the ‘Girder Layout’ detailed design drawings, along with the necessary shop drilled holes for steel girders or cast-in inserts for concrete girders.

Except for integral abutment designs, abutment diaphragms shall be steel brace type to provide open access for inspection and maintenance of bearings and abutment deck joints.

Continuous bridges shall have the same number of girders on adjacent spans or adjacent segments to be spliced in the field, such that each individual girder line is fully continuous from end to end of the structure.

Precast prestressed concrete and steel girders that are designed as composite girders shall be designed such that the non-composite girders carry the deck slab dead load in an unshored condition.

9.2 Precast Prestressed Concrete Girder Bridges

9.2.1 General

The Department’s preferred precast prestressed concrete girder for medium to long spans (20 – 65 m spans) is the NU girder, although other girder shapes are also occasionally used (eg. boxes). For smaller span bridges, side-by-side box girders are often used.

The requirements presented in this Section 9.2.1 shall apply to all precast prestressed girder bridges, with the exception of the Department’s standard SL, SLW and SLC girder bridges. The additional requirements presented in Section 9.2.2 apply specifically to I-shaped precast prestressed concrete girder bridges. The additional requirements presented in Section 9.2.3 apply specifically to side-by-side precast prestressed concrete girder bridges.

Precast prestressed concrete girders shall be designed to meet the following requirements:

(a) The maximum jacking stress for pretensioning strands shall be limited to 0.78 fpu. Jacking above this level may introduce unacceptable strand failure risks to personnel in a fabrication plant. Consultants should check with the precast manufacturer and obtain the correct allowances for prestress losses due to anchor set and/or elastic shortening of self-stressing forms, prior to deciding on the appropriate prestress value at transfer.

(b) To facilitate a reasonable turn-around time for girder casting, the specified concrete strength at release shall not be more than 45 MPa.

(c) The minimum age for girders before field cast continuity connection shall be 60 days. Girder and deck design and detailing shall consider the effects of differential camber between girders.

(d) Appropriate allowance for girder shortening due to prestress losses (pretension and post-tension) shall be included in the fabricated length of the girders. This value shall be shown on the detailed design drawings.

(e) Horizontal interface shear design for composite action shall satisfy the requirements from the *CHBDC* or *AASHTO LRFD Bridge Design Specifications*, whichever is more stringent. The longitudinal distribution of shear forces shall be taken to be the same as the ULS shear envelope, unless a more demanding shear flow condition can exist based on analysis.

(f) Girder ends shall have cast-in galvanized shoe plates anchored into the girders by Nelson Studs or welded deformed anchor rods. The shoe plate design shall account for the different support conditions at the abutments and piers and shall transfer all vertical and horizontal forces from the girders into the bridge bearings.

(g) The girder end zones shall be designed to account for the different support conditions at the abutments and piers at all stages of construction, and whether the girder ends will be permanently cast into a concrete diaphragm.

(h) For girders containing pretensioning strands, *Clause 8.15.4* of the *CHBDC* states "The number of stands where the bonding does not extend to the ends of the member shall not exceed 25% of the total number of strands." This requirement shall apply to girders with only pretensioning strands, as well as to girders with pretensioning and post-tensioning strands. For girders with pretensioning and post-tensioning strands, the 25% limit shall be applied to the total number of pretensioning strands only. In addition, the number of strands debonded in any one horizontal row shall not exceed 40% of the strands in that row. Not more than 40% of the debonded strands, or four strands, whichever is greater, shall have the debonding terminated at any one section. Debonded strands shall be symmetrically distributed about the centerline of the girder. Debonded lengths of pairs of strands that are symmetrically positioned about the centerline of the girder shall be equal. Exterior strands in each horizontal row shall be fully bonded and shall not be debonded at any location. The effect of debonding shall be such that all limit states are satisfied with consideration of the total developed resistance at any section being investigated.

(i) All miscellaneous steel that is attached to or embedded into girders, and has exposed surfaces, shall be galvanized. All intermediate steel diaphragms, including all associated plates, washers and bolts, shall be galvanized.

(j) Theoretical calculated cambers shall be determined by the Consultant and shall be shown on the detailed design drawings. Camber data shall be provided following the sequence of occurrence for each component, such as at prestress transfer, erection, deck pour, post-tensioning, superimposed dead loads (SIDL), gradeline profile, shrinkage and creep, etc. (see typical detail drawing *T-1757-17 (NU Girder Bridges Typical Details - Sheet 1)*). Camber for precast prestressed concrete girders can vary substantially from estimated values due to variations in concrete properties, storage conditions, and shrinkage and creep. Proper detailing of stirrup projections, girder end/bearings, and selection of deck haunch thickness are required.

9.2.2 Precast Prestressed Concrete I-shaped Girder Bridges

Precast prestressed concrete I-shaped girders are typically used by the Department in slab-on-girder bridges. The Department's preferred precast prestressed concrete I-shape girder shape is the NU girder. In addition to the requirements presented in Section 9.2.1, precast prestressed concrete I-shaped girder bridges shall meet the following requirements:

(a) NU girders may be pretensioned only or pretensioned and post-tensioned. NU girders shall be designed and detailed in accordance with the typical girder details shown on typical detail drawings *T-1757-17 (NU Girder Bridges Typical Details - Sheet 1)* and *T-1758-17 (NU Girder Bridges Typical Details - Sheet 2)*.

(b) Girder SLS and ULS design strengths shall be based on a nominal girder section and assuming a minimum deck haunch height of 13 mm between the bottom of the deck slab and the top of the precast girder.

(c) Pier diaphragms shall be continuous cast-in-place concrete diaphragms and shall be either pinned, fully monolithic with the pier top, or permit free expansion. Positive moment connections of girder over the piers shall consist of lapped and bent-up prestressing strands or lapped and cast-in hooked reinforcing steel. The minimum separation between girder ends shall be 300 mm. Where pier diaphragms are not monolithic with the pier top (cap or shaft), the ends of both girders shall be supported on separate reinforced elastomeric pads. Where pier diaphragms are connected monolithically to the pier top (cap or shaft) and are cast around girder ends, the girders shall be erected on plain unreinforced elastomeric pads on a minimum 150 mm high plinth to provide sufficient clear space between the girder bottom and previously cast concrete, to ensure proper flow of concrete under the ends of the girders.

(d) Stirrup projections from the top of precast girder into the deck shall satisfy the following:

- Stirrup projections shall meet all *CHBDC* requirements for lap splicing with vertical stirrups, and anchorage requirements for developing full composite action.
- All stirrup projections shall have 135° or 180° hooks around longitudinal deck bars. Longitudinal deck bars shall be detailed with a bar centred directly over the girder webs and the remaining bars spaced evenly between girder centre lines.
- Depending on the bridge gradeline and girder camber, the haunch height will vary along the length of the girder. If the haunch height varies significantly, two issues can arise:
    - Stirrup projections can be too short. When this occurs, the deck slab and the girder haunch are not connected sufficiently together to ensure full composite action with ductility in the connection between the girder and the deck. When the projection of stirrups is less than 40 mm above the underside of the bottom mat of deck bars, additional hat shape extension bars shall be lapped with the stirrups. These are shown on typical detail drawing *T-1758-17 (NU Girder Bridges Typical Details - Sheet 2)* and should be shown on the detailed design drawings; and
    - Stirrup projections can be too long. When this occurs, the stirrups can interfere with the top mat of deck reinforcing, causing significant problems during installation of deck reinforcing.

This problem is particularly significant when the bridge has a skew. This can typically be avoided by changing the stirrup projection at a few specific locations along the length of the girder. This must be addressed on the detailed design drawings.

(e) In pretensioning anchorage zones, additional stirrups shall be provided for the purpose of controlling web cracking at transfer of pretensioning force. Notwithstanding the stirrup area and spacing requirements presented in Clause 8.16.3.2 of the *CHBDC,* this crack control reinforcement shall be provided by vertical stirrups and shall meet the following requirements:

$P_r = f_s A_s$ where: f_s = stress in stirrup steel not exceeding 140 MPa;
A_s = total area of vertical crack control end zone reinforcement;
P_r shall not be less than 4 percent of the pretensioning force at transfer.

Half of these end zone stirrups (ie. $0.5A_s$) shall be concentrated within the end h/8 of the girder and the remaining half (ie. $0.5A_s$) shall be distributed over a distance from h/8 to h/2 (where h is the overall depth of the precast girder).

Experience has shown that cracking is minimized when the first crack control stirrup is placed as close to the end of the girder as possible. Therefore, the end cover for the crack control stirrup shall be 30 mm for exposed girder ends and 25 mm for girders encased into field cast diaphragms. The crack control stirrups shall be anchored beyond the anticipated extreme top and bottom cracks with sufficient embedment to develop at least a stress = 210 MPa. Since the crack control reinforcement is required to minimize the crack width, and not for strength, there is no need to develop the full yield strength beyond the locations of the top and bottom cracks. For I-shaped girders, the anticipated top and bottom cracks may be assumed for design to be at the junction between the web and the flanges. Therefore, the crack control stirrup anchorage into the flanges should be designed for a maximum stress of 210 MPa. (These requirements are based on the recommendations reported in *NCHRP Report 654 Section 3.8: Proposed Revisions to the AASHTO LRFD Bridge Design* Specifications).

The area of stirrups in the girder end region may need to be increased for other loading conditions, such as post-tensioning anchorage, shear resistance, etc.

(f) In pretensioning anchorage zones, 10M (or greater) closed ties shall be provided in the bottom flange to confine the pretensioning strands. Within the distance ‘h’ from the end of the girder, closed ties shall be provided as required for confinement, however spacing of closed ties shall not exceed 150 mm. Beyond the distance ‘h’ from the end of the girder, closed ties shall be provided at a minimum spacing of 300 mm. Closed ties are normally fabricated in two pieces with full tension lap splices. The top of the ties can be left open in the mid-span region of the girder where ever there is conflict with post-tensioning cables.

(g) Notwithstanding Clause 8.4.4.5.2 of the *CHBDC*, for precast prestressed concrete I-shaped girders with 28 day concrete strength greater than or equal to 65 MPa, the inside duct diameter can be increased to a maximum of 50% of the web thickness provided the inside duct area shall be > 250% of the strand area.

(h) For conventional abutments with deck joints, the superstructure end diaphragm shall be an open steel diaphragm to provide access for deck joint inspection and repair. The girder web at abutment ends shall be thickened and designed as part of the abutment steel diaphragm system for transferring laterals loads from the superstructure to the substructure.

(i) For NU girders, a minimum of four bonded pretensioning strands shall be incorporated in the top flange to assist in controlling stresses at transfer during transportation and during construction.

(j) Connections between the steel diaphragms and the exterior girders shall be detailed so that no connection hardware is visible on the exterior surface of the girders.

9.2.3 Side-by-side Precast Concrete Girder Bridges

Side-by-side precast concrete girder bridges are defined as bridges having precast girder units with little or no slab spanning transversely between girder units. In addition to the requirements presented in Section 9.2.1, side-by-side precast girder bridges (not including the Department's standard SL, SLW and SLC girder bridges) shall meet the following requirements:

(a) Precast concrete girders shall be fully composite with a 225 thickness cast-in-place concrete deck with two mats of orthogonal deck reinforcement;

(b) Side-by-side precast concrete box girders that are designed to be fully monolithic with the abutment or pier, may be erected on unreinforced elastomeric erection stage bearings on abutment and piers so long as permanent connection with the substructure is made through cast-in-place concrete diaphragms - between and under girder ends at piers and behind and under girder ends at abutments;

(c) Side-by-side precast concrete box girders that are not designed to be fully monolithic with the abutment or pier, shall be supported on two steel reinforced elastomeric pads at each girder end (one bearing nominally under each web). The bearing support surfaces on the substructure shall be fully detailed with control elevations on a plan layout to ensure the support surfaces lie in a plane that is parallel to the underside of the girders. The difference in elevations between the underside of the girders and the top of the concrete surfaces shall be equal to the thickness of the neoprene pads;

(d) Where permanent lateral distribution diaphragms are not incorporated prior to deck placement, the Consultant shall design and provide temporary measures to ensure lateral stiffness and load distribution is acceptable during deck placement and during all phases of construction, prior to the full strength and stiffness of the combined girder and deck system are achieved. The temporary measures and provision for inclusion of these measures during construction shall be considered in the girder and superstructure design and details. The measures to control differential deflection between girder units and to distribute deck placement and other construction loads during construction shall be adequate to prevent adverse effects to the specified properties and locations of deck, or barrier, or deck and barrier, or other reinforcement that can be affected by superstructure construction activities. Furthermore, the measures to control differential deflection between girder units and to distribute deck placement and other construction loads during construction shall be adequate to prevent concrete cover and thickness variations from design requirements;

(e) Pier diaphragms shall be continuous cast-in-place concrete diaphragms and shall be either pinned, fully monolithic with the pier top, or permit free expansion. Positive moment connections of girders over the piers shall consist of either one or a combination of grouted unstressed continuous tendons, lapped and bent-up strands, or lapped cast-in hooked rebar. For side-by-side girders, the minimum separation between girder ends on common supports shall be 400 mm with bent strands or hooked rebar.

9.3 Steel Girder Bridges

Bridges designed with welded steel plate girders shall meet the following requirements:

(a) Welded steel plate girders shall be designed and detailed in accordance with the typical girder details shown on typical detail drawings *T-1759-17 (Steel Plate Girder Bridges Typical Details - Sheet 1)* and *T-1760-17 (Steel Plate Girder Bridges Typical Details - Sheet 2)*.

(b) Vertical stiffeners and girder ends shall normally be square to the girder flanges, which are parallel to the road grade. Since abutment backwall is normally plumb, the abutment detailing shall account for the effects of girder end tilt.

(c) All welded steel girders, regardless of span, shall be cambered for 100% of dead load deflection and roadway gradeline profile.

(d) All bearing stiffeners, including jacking stiffeners, shall be “fit to bear bottom” and “fit only top”, and then fillet welded to both the top and bottom flanges and to the web. As defined in AWS D1.5, “fit to bear” requires minimum 75% contact and “fit only” allows a maximum gap of 1 mm.

(e) For long bridges with large expansion movements, the need for multiple vertical bearing stiffeners shall be checked. The check should ensure that when the bearing shifts away from the bearing stiffener(s), the girder bottom flange does not distort so much as to affect the performance of the sliding bearing.

(f) Jacking stiffeners for bearing replacement shall be provided on all girders and at all supports. Location of jacking stiffeners shall be based on estimated jack sizes required for bearing replacement, plus sufficient clearance to the edge of the abutment seat or pier cap.

(g) Diaphragm connector plates and intermediate stiffeners at stress reversal locations shall be “fit only bottom” and “fit only top”, and shall be welded to both the top and bottom flanges and to the web. Intermediate stiffeners, other than at stress reversal locations, shall be fitted and welded to the compression flange only, and cut short of the tension flange with web gap meeting the requirement of the *CHBDC Clause 10.10.6.4*.

(h) Vertical stiffener plates shall be corner coped (80 mm vertical x 35 mm horizontal) adjacent to the web (applicable for web thicknesses from 14 mm to 20 mm). Where stiffener plates project past the outside edge of the flange plates, the projecting stiffener corners shall be coped 45º.

(i) No intersecting welds are allowed. Longitudinal stiffeners are normally placed on the opposite side of the web as vertical stiffeners. Where horizontal stiffeners and vertical stiffeners intersect on the same side of the web, the horizontal stiffener shall run continuously through a slot in the vertical stiffener. The cut edges of the vertical stiffener at the intersection shall be corner coped (25 mm x 25 mm) adjacent to the web, and be welded to the horizontal stiffener.

(j) All weld ends for stiffeners, gussets, and other attachments to girders shall terminate 10 mm from the edge or end of plates.

(k) Gusset plates for attachment of horizontal bracing shall be bolted and not welded to girders.

(l) The following features shall be used to prevent staining of substructure concrete:

- At pier locations, the exterior edge of the bottom flange of exterior steel girders shall have a 19 mm x 19 mm x 8000 mm long rubber strip centred over the pier, in accordance with typical detail drawing *T-1760-17 (Steel Plate Girder Bridges Typical Details - Sheet 2)*.
- At abutments, exterior steel girders shall have the same rubber strip attached around the bottom flange at 2000 mm from the face of the abutment walls. Where steel girders are cast into integral abutment diaphragms, a second rubber strip shall be applied all around the bottom flange of all girders immediately in front of the concrete abutment diaphragm face.

(m) Changes in girder flange widths shall be tapered at a taper of 2.5 (longitudinal):1 (transverse) or with a 600 mm radius as shown on typical detail drawing *T-1760-17 (Steel Plate Girder Bridges Typical Details - Sheet 2)*.

(n) Shear Stud projections from the top of girder flanges into the deck shall meet all *CHBDC* requirements for stud development and anchorage requirements and ensure full composite action in accordance with design requirements. The design shear stud projection, measured from the underside of the head of the stud to the top of the bottom transverse deck reinforcement, should be designed to be a minimum of 50 mm (25 mm plus a construction tolerance of 25 mm).

(o) At all deck joint locations, the girder shall be protected with an approved bridge coating system in accordance with the *SSBC Section 6: Structural Steel*. Notwithstanding the exceptions listed below, the coating system shall be applied to the bottom flange (underside, top and edges), full height of the web (including any applicable bearing/jacking stiffeners), and to the underside of the top flange, and shall extend longitudinally from the girder end to a distance 100 mm beyond the bearing sole plate or 100 mm beyond the jacking stiffener, whichever distance is greater. The top coat colour shall conform to US Federal Standard 595C colour FS30045. The following exceptions apply:

  - Where bearing sole plates are galvanized, the faying surface of the underside of bottom flange in contact with the bearing sole plate shall only receive an organic zinc epoxy primer. The primer shall extend the full width of the flange and 15 mm beyond the projected contact surface of the bearing sole plate in the longitudinal direction;
  - Faying surfaces of bolted connections shall only receive the organic zinc epoxy primer; and
  - Any of the portions of the girder noted above that will be encased in cast-in-place concrete shall be left in the bare steel condition with no coating applied.

(p) At all pier bearing and semi-integral abutment bearings, Where bearing sole plates are galvanized, the, an organic zinc epoxy primer shall be applied to the faying surface of the underside of the bottom girder flange in contact with the bearing sole plate. The primer shall extend the full width of the flange and 15 mm beyond the projected contact surface of the bearing sole plate in the longitudinal direction.

9.3.1 Steel Girder Camber

Welded steel girders shall be cambered for 100% of dead load deflection and roadway gradeline profile. Camber data shall be shown on a camber diagram, at 10th span points, centreline of supports, and centreline of field splices, along with net camber values for individual girder segments between splices. For spans longer than 50 m, data shall be presented at 20th span points. Data shall be presented on the camber diagram detailed design drawing in accordance with the *Engineering Drafting Guidelines for Highway and Bridge Projects Appendix E – Bridge Drawing Checklists*, and shall include girder dead load, deck dead load, superimposed dead loads (eg. curb/barrier/median, wearing surface, underslung bridges, utilities, etc.), and vertical grade. Deck dead load deflections should include the effects of concrete deck creep and shrinkage.

For complex structures, such as those with long spans, curves or high skews, more rigorous analysis and camber diagrams for individual girder lines shall be considered.

Notwithstanding the *CHBDC Clause 10.7.4.1*, welded steel girders spanning less than 25 m shall be cambered to compensate for dead load deflection and highway grade profile.

Camber variations for steel girders are normally minor in nature and should be easily accommodated by adjusting deck formwork elevation and thickness of the deck haunch on top of the girders.

9.4 Intermediate Diaphragms

Intermediate diaphragms are required for all slab-on-girder bridge structures. Intermediate diaphragms for slab-on-girder bridges with precast concrete NU girders (or similar I-shaped precast concrete girders) shall have a maximum spacing of 13.0 m. Intermediate diaphragms for slab-on-girder bridges with steel girders shall have a maximum spacing of 8.0 m. Refer to typical detail drawing *T-1757-17 (NU Girder Bridges Typical Details - Sheet 1)* for NU girder diaphragm details and *T-1759-17 (Steel Plate Girder Bridges Typical Details - Sheet 1)* for steel girder diaphragm details.

For girder depths less than or equal to 1200 mm, intermediate diaphragms may be C-shape (channel) or W-shape sections of at least 1/3 (and preferably 1/2) the girder depth. For girders deeper than 1200 mm, intermediate diaphragms shall be X-bracing or K-bracing and top and bottom horizontals shall be provided.

Intermediate diaphragms and girders shall be designed for construction loads during deck concrete placement in accordance with requirements of the *CHBDC Clause 3.16* and *SSBC Section 4.10.6: Deck Formwork*. Typically, diaphragms provided shall become part of the permanent structure and be left in place for future maintenance, widening, and rehabilitation. The only exception to this is at the ends of NU girder bridges with integral abutments where the erection stage diaphragms may be removable as noted on typical detail drawing *T-1757-17 (NU Girder Bridges Typical Details - Sheet 1)*. Diaphragms, exterior steel girders, and exterior precast concrete girders carrying deck overhangs shall be checked to ensure sufficient strength and stability to handle concentrated loads from deck finishing machines, work bridges, and loads from temporary walkways outside the edge of the deck slab. Loads assumed for such design shall be based on realistic estimates for each bridge and shall be shown on the detailed design drawings, in accordance with *the Engineering Drafting Guidelines for Highway and Bridge Projects*.

Intermediate diaphragms are typically treated as secondary members, however for curved bridges and certain high skews situations, intermediate diaphragms should be treated as primary members and should be designed accordingly.

For bridges with precast concrete NU girders (or similar I-shaped precast concrete girders) with moderate skews, oversized or slotted holes may be used to accommodate moderate differential vertical camber or horizontal sweep between adjacent girders during erection. Oversized or slotted holes shall meet requirements of the *CHBDC Clause 10.18.*

For bridges with small radius curves or high skews, differential deflection between adjacent girders due to dead load application can be a concern. Steel bridges with skews greater than 30 degrees shall preferably be erected such that girder webs are plumb after all dead loads are applied (referred to as "Full Dead Load Fit"). Consultants shall assess the bridge to determine whether the skew effects will result in the webs being excessively out-of-plumb (refer to *NSBA G12.1-2016 Guidelines to Design for Constructability - Section 1.6 Differential Deflections*). Where applicable, the Consultant shall include the following note on the steel girder detailed design drawings, under the 'ERECTION' heading: "CROSS-BRACING SHALL BE DETAILED SUCH THAT GIRDER WEBS ARE PLUMB UNDER FULL DEAD LOADS".

10. DECKS, CURBS, CONCRETE BARRIERS, MEDIANS & SIDEWALKS

This Section 10 provides design information for decks, curbs, concrete barriers, medians and sidewalks.

10.1 Decks

Bridge decks shall normally be designed and detailed as cast-in-place deck slabs.

Partial depth precast concrete deck panel construction is not a preferred deck design due to the unknowns associated with long term performance. The Department will only consider the use of this system, on a trial basis, if submitted for Approval through the value engineering process by the contractor with demonstrated economic advantages. Submissions for partial depth precast concrete deck panels shall meet the design, fabrication and construction specifications in Appendix F.

Full depth precast concrete deck panels generally cost more than cast-in-place construction, but may be considered for special applications through a value engineering study. Shear connectors, transverse joint connections, negative moment over piers, durability, potential future rehabilitation, replacement and traffic accommodation issues need to be fully addressed.

All other types of stay in place deck soffit formwork, including corrugated steel or timber, are not allowed.

Decks shall meet the following requirements:

(a) Bridge decks shall consist of Class HPC concrete and Corrosion Resistant Reinforcing (CCR) bars.

(b) All bridge decks shall receive a 90 mm thick ACP deck protection and wearing surface system as per Section 5.2 (Bridge Decks, Roof Slabs and Approach Slabs) and *S-1838-17*, *S-1839-17* and *S-1840-17 (Standard Waterproofing System for Deck and Abutments – Sheets 1, 2 and 3)*. Bridge deck waterproofing membranes shall include wick drains along the gutter lines to allow for the controlled drainage of sub-surface water that penetrates the asphaltic wearing surface, and discharge of the sub-surface discharge at the bridge ends only. No intermediate discharge locations are permitted.

(c) Cast-in-place concrete deck slabs for slab-on-girder bridges shall be designed using the empirical method in accordance with *Clause 8.18.4* of the *CHBDC*, and in all cases shall have a minimum slab thickness equal to the greater of the girder spacing divided by 15.0 or 225 mm. Use of this method requires composite action between the slab and girder over the entire girder length.

(d) All cast-in-place concrete decks, even for the case where a deck is supported on side-by-side girder units with little or no slab span resulting, shall have a slab thickness not less than 225 mm. All cast-in-place concrete decks shall be reinforced with both upper and lower layers of reinforcing. Each layer of reinforcing shall consist of two reinforcing directions, one direction that can resist longitudinal or primarily longitudinal forces and one direction that can resist transverse or primarily transverse forces.

(e) Clause 5.7.1.6 of the *CHBDC* covers bending moments in concrete deck slabs. Any references to CL-625 in this Clause 5.7.1 shall be replaced with CL-800 and all stipulated forces and moments presented in Clause 5.7.1 shall be pro-rata increased by a factor equal to the ratio of 800/625.

(f) Deck reinforcement required to develop the capacity of bridge barriers is site specific and shall be included on the detailed design drawings.

10.2 Curbs and Concrete Barriers

Specific design information related to bridge barriers is provided in Section 12 (Bridge Barriers & Transitions). Curbs and concrete barriers shall meet the following requirements:

(a) Curbs and barriers shall be detailed using the details on typical detail drawing *T-1680-17 (Typical Curb and Wingwall Details)*.

(b) Concrete curbs and barriers shall have crack control joints as shown on the Department's standard drawings. These control joints shall have a maximum spacing of 3.0 m and shall be centred between bridgerail posts where bridgerail posts are used with the exception of the standard TL-5 barriers which have a maximum crack control joint spacing of 2.3 m. Longitudinal reinforcing in the curbs and barriers shall be discontinuous at the joints. Control joints shall extend down to the top of the concrete deck and the sealant shall be placed and cured prior to application of deck waterproofing membrane in accordance with standard drawing *S-1838-17 (Standard Waterproofing System For Deck and Abutments - Sheet 1)*.

(c) Curb reinforcement required to develop the capacity of bridgerail post anchors is site specific and shall be included on the detailed design drawings. Guidance for design of decks supporting bridgerail posts is available from *AASHTO LRFD Bridge Design Specifications Appendix A13*.

(d) The tops of curbs and barriers shall have a wash slope of 3% towards the roadway.

(e) Concrete paving lips along the edge of ACP are not permitted.

(f) Two electrical connections are required on bridge decks to accommodate the copper sulphate electrode ("CSE") or half-cell testing (Specialized Level 2 Inspections). The first electrical ground connection and associated hardware shall be located on top of the right hand side curb or barrier within 3 m of the centerline of bearing, of the first abutment encountered in the direction of travel. The second electrical ground connection shall be located at the opposite corner of the bridge.

10.3 Medians and Sidewalks

(a) Warrants for provision of sidewalks and pathways are provided in *RDG Section H9.2: Pedestrian/Cyclist Pathways on Bridges*.

(b) For raised concrete medians and sidewalks, the curb type (barrier or mountable) shall be compatible with roadside safety and barrier performance. For more detailed guidance, refer to *RDG*

Section H4.3: Curbs and Section *H11.3: Curbs*. The required median width (lip of gutter to lip of gutter) transition from roadway to bridge shall be maintained with lane markings on the bridge.

(c) The portion of the deck slab under the raised concrete medians and sidewalks shall be protected by a waterproofing membrane and protection board, in accordance with standard drawing *S-1838-17 (Standard Waterproofing System For Deck and Abutments - Sheet 1)*. The raised concrete median or sidewalk slab shall be poured after the membrane and protection board have been applied to the deck slab. The top slab surface of raised concrete medians and sidewalks shall have transverse tooled joints aligned perpendicular to and at a spacing matching the adjacent curb/barrier control joints. Slab reinforcing in the raised concrete medians and sidewalks shall be discontinuous at the transverse tooled joints.

(d) The sidewalk shall have a curb projecting 100 mm above the finished top of the sidewalk along the outside edge. If the roadway has a normal crown and the sidewalk is higher than the adjacent road surface, the sidewalk shall drain through slots in the traffic separation barrier onto the roadway gutter as shown on standard drawing *S-1838-17 (Standard Waterproofing System For Deck and Abutments - Sheet 1)*. If a sidewalk is located on the high side of a superelevated roadway, the sidewalk shall drain to the outside edge and the drainage shall be carried longitudinally down the edge of the sidewalk and acceptably discharged at the ends of the bridge.

(e) The tops of sidewalks and raised concrete medians shall slope 2% towards the roadway.

11. DECK JOINTS

(a) New bridge structures shall be fully continuous from end to end. Deck joints shall only be permitted at abutment ends of bridges.

(b) The Department has provided standard deck joint drawings as outlined in the table below. These standard deck joint types should work for most situations and shall therefore be used unless otherwise Approved. However, there may be situations where these standard deck joint designs are not appropriate. Examples of this might include bridges with very complicated geometry (high curves, high skews, etc.), bridges with very flexible superstructures, or existing bridges with older bearing types. The Consultant is expected to understand the behaviour of these deck joints (and their associated limits) and shall address any potential issues with the Department during the preliminary engineering phase (ie. when preparing the Structure Alternatives Report).

(c) In the following table, joint movement perpendicular to the deck joint has been designated “normal movement”, and joint movement parallel to the joint has been designated “shear movement”. Joint movement design values shall be based on unfactored values of thermal and other long term movements consistent with proper functioning of the joint type.

Standard Drawing	**Joint Type**	**Maximum Permissible Normal Movement**	**Maximum Permissible Shear Movement**[1]
S-1810-17 (Type I Strip Seal Deck Joint - Sheet 1) *S-1811-17 (Type I Strip Seal Deck Joint - Sheet 2)* *S-1812-17 (Type I Strip Seal Deck Joint - Sheet 3)*	Multi-cell strip seal	115 – 60 = 55 mm	13 mm [(2)]
S-1800-17 (Cover Plated V-Seal Deck Joint - Sheet 1) *S-1801-17 (Cover Plated V-Seal Deck Joint - Sheet 2)* *S-1802-17 (Cover Plated V-Seal Deck Joint - Sheet 3)*	Cover-plated 102 mm V-seal	90 – 60 = 30 mm	20 mm [(2)]
	Cover-plated 125 mm V-seal	115 - 60 = 55 mm	25 mm[(2)]
	Cover-plated 178 mm V-seal	150 - 60 = 90 mm	30 mm[(2)]

Standard Drawing	Joint Type	Maximum Permissible Normal Movement	Maximum Permissible Shear Movement[1]
S-1638-17 (Standard Finger Plate Deck Joint Assembly - General Layout) *S-1639-17 (Standard Finger Plate Deck Joint Assembly - Typical Cross-sections)* *S-1640-17 (Standard Finger Plate Deck Joint Assembly - Typical Drain Trough Details)*	Finger plate joint	300 mm	0 mm

Notes:

1. The maximum permissible shear movement shall be based on the maximum absolute temperature difference between the temperature at joint installation and the maximum or minimum design temperature.
2. For skew bridges, the movement along the longitudinal direction of bridge movement is resolved into the normal and shear components with respect to the joint axis. The governing movement limit is reached when either one of the component movement ranges exceed the respective permissible values listed above. It should be noted that the normal component gap can be set at the time of concreting the joint extrusion, in accordance with the temperature setting chart provided on the detailed design drawings, but the shear component is zero at the time of seal installation. The temperature of seal installation is assumed to be 15 ° C for the standard design, but can be fine-tuned if necessary for a specified installation temperature. A design example is provided in Appendix E.

(d) The standard deck joint drawings show most of the details required to fabricate and construct the joint. However, the standard drawings do not include all relevant bridge information and do not address all possible scenarios. Consultants shall ensure that sufficient information is provided on the detailed design drawings to address all site specifics. The following are some of the more common details that require attention by the Consultant:

- Coordination of the deck joint anchorage and the reinforcing in the abutment, deck, and barriers. This particularly important for bridges with challenging geometry, such as high skews;
- Coordination of longitudinal and lateral movements of the bridge superstructure at the deck joint location. This often is a concern for highly skewed and/or curved bridges, for long bridges, and for wide bridges. Coordination and attention to detailing is required at the barrier cover plates, finger orientation, bearings and concrete shear blocks; and
- The presence of different barrier types, sidewalks, or medians at the bridge site.

(e) The Type 1 strip seal deck joint is the simplest joint available and is therefore the Department's preferred deck joint system. The use of this deck joint system will be limited by the movement capacity of the seal, perpendicular and parallel to the joint. Since it is preferred to use this joint type as much as possible, Consultants are encouraged in their design to make full use of the 115 mm maximum gap limit. This may require gap adjustment in the field if the installation temperature varies by more than ± 5 °C from the assumed 15 °C.

(f) Deck joint assemblies are pre-set with a 15 °C joint gap in the fabrication shop and then clamped with erection angles for shipping. This temperature is representative of an average installation temperature. The standard cover plated V-seals and finger joints have very tight fabrication tolerances for sliding plates in contact and it is recommended the joints be installed as shipped without loosening the clamping bolts for adjustment. The design of these joints has built in tolerance for minor variations in effective bridge temperature.

(g) Setting of deck joints may be based on the effective bridge temperature at the time of installation which may be assumed to be the mean shade air temperature taken over the previous 48 hours for concrete structures and 24 hours for steel structures.

(h) Deck joints with seals (strip seal or V-seal) shall incorporate stop movement bars to maintain a minimum joint gap of 60 mm to facilitate extraction of seal for replacement without undue effort. Consultants should note that this is often larger than the minimum gap indicated on manufacturer's brochures, which provide gap widths suitable for first installation only.

(i) For multi-cell strip seal type deck joints with skew angles within the range of 20º to 45º, snow plow guard plates shall be installed in accordance with standard drawings *S-1810-17 (Type I Strip Seal Deck Joint - Sheet 1)*, *S-1811-17 (Type I Strip Seal Deck Joint - Sheet 2)* and *S-1812-17 (Type I Strip Seal Deck Joint - Sheet 3)* to prevent snow plow blades from dropping into the joint gap and catching the edge of the joint extrusion. Welded snow plow guard plates shall not be located directly under wheel paths.

(j) Finger plates and cover plates shall be fixed to the deck side to allow jacking and raising of the superstructure.

(k) The Department has historically not permitted the use modular seal deck joint systems due to fatigue concerns, premature failure of the seals and the difficulty in subsequently replacing the seals. However, in certain circumstances, modular deck joints may be the best solution. In these circumstances, success of the modular deck joint will require particular attention to detailing as well selection of appropriate seals. Modular deck joint systems shall be submitted for Approval.

(l) Deck joints shall run continuously across the full width of the deck, taking into account skew, crossfall and crown of roadway. Exterior bridge barriers and curbs shall have full cover plates on the inside face and across the top. Interior traffic separation barriers shall have full cover plates on both sides and across the top. For multi-web strip seal joints at raised medians, the deck joints shall follow the top surface of the median. For cover plated V-seal joints at raised medians, the deck joints shall continue through the median at the deck level complete with deck cover plates across the top of the median. Top of deck joints across the width of sidewalks or pathways shall have non-slip surfaces (by applying Devoe AS-2500 or Approved equivalent) and shall be detailed to avoid tripping hazards.

(m) Only neoprene seals shall be permitted for V-seal and multi-cell strip seal deck joints. No other seal materials will be permitted.

(n) The construction joint for deck joint block-outs shall be located outside the width of the concrete paving lip and shall be within the extents of the deck waterproofing system. This construction joint shall be shown on the detailed design drawings.

(o) Refer to Appendix E for a design example on allowable movement for strip seal deck joints.

12. BRIDGE BARRIERS & TRANSITIONS

12.1 General

Department standard bridge barriers and transitions shall be used, unless otherwise Approved. These barriers and their appropriate uses are described in Section 12.2. Other crash-tested bridge barrier and transition systems may be appropriate for special site conditions, but shall not be used without Approval. Except when sidewalks are present on a bridge, the exact same bridge barrier shall be used on both sides of the bridge.

For all projects, the bridge barrier exposure index shall be calculated in accordance with the *CHBDC*. Once the barrier exposure index has been calculated, the appropriate barrier test level can be determined from tables in the *CHBDC*.

Bridge barrier end transitions provide a gradual change in stiffness from the rigid bridge barrier to the flexible approach guardrail system. Beyond the bridge barrier end transition, the type of approach guardrail and terminal ends are determined by the roadway design requirements, in accordance with the *RDG*.

For low traffic volume roadways, the length of need of approach guardrails can be reduced in accordance with *RDG Table H3.12*.

12.2 Standard Bridge Barriers and Transitions

12.2.1 TL-2 Barriers and Transitions

The Department has three standard designs for TL-2 bridge barriers and approach rail transitions for different applications.

For bridges located on low volume gravel roads, the Department has two appropriate TL-2 bridge barrier systems. Standard drawing *S-1652-17 (TL-2 Thrie Beam Bridgerail)* presents the standard 850 mm high thrie beam bridge barrier and approach rail transition details. On bridges where the clear roadway width is less than 9 m and wide vehicles are expected (often farm equipment), consideration should be given to using the reduced height 680 mm barrier and approach guardrail transition details (standard drawing *S-1653-17 (TL-2 Low Height Thrie Beam Bridgerail)).* Both of these bridge barrier systems do not include a curb and should therefore only be used where there is no risk of exposure to de-icing salts on the bridge now or in the foreseeable future (without curbs on a bridge, de-icing salts will drain over the edge and will result in accelerated corrosion of the deck edge and the exterior girder).

All paved bridges are expected to receive de-icing salts to maintain safe traffic operation. Therefore, for bridges located on roadways that are paved or are expected to be paved in the foreseeable future, the standard bridge barrier incorporates a TL-2 thrie beam bridgerail and a 75 mm high curb for drainage control (standard drawing *S-1797-17 (TL-2 Thrie Beam Bridgerail on 75mm High Curb)*). The curb height for this design cannot be increased above 75 mm because any curb higher than 75 mm is not compatible with the semi-rigid bridge barrier transition design. The soft approach transition in this design can deflect too much on impact, and expose the vehicle wheel to snagging the curb end if it is a higher curb. This standard bridge barrier can be used with bridges that are located on gravel roads that will be paved in the future. In this case, the initial curb height will be

165 mm (75 mm curb + 90 mm allowance for future ACP overlay) and therefore, a 100 mm high rub rail is required to protect vehicles from snagging the curb end before the bridge is paved..

For long or high bridges, consideration should be given to using a TL-4 bridge barrier, even where a TL-2 bridge barrier would satisfy the minimum *CHBDC* requirements.

12.2.2 TL-4 Barriers and Transitions

The Department has five standard designs for TL-4 bridge barriers for different applications.

(a) TL-4 Double Tube Type Bridgerail Barrier

This is the Department's preferred standard TL-4 bridge barrier for use on rural roadways. The double tube type bridgerail on curb provides an open barrier that will not trap snow on the bridge. Standard drawing *S-1642-17 (TL-4 Double Tube Type Bridgerail - Bridgerail Details)* presents the bridge barrier details. This bridge barrier is typically not used as a separation barrier between roadway and sidewalk as the posts are a snagging hazard to pedestrians/cyclists. In addition, this barrier system is typically not used where the approach roadway uses a concrete roadside barrier.

There are two standard TL-4 approach barrier transition alternatives that can be used with this bridge barrier system:

- When the approach roadway requires a TL-3 roadside barrier design, the approach transition details shown on standard drawing *S-1643-17 (TL-4 Double Tube Type Bridgerail - Approach Rail Transition Details)* shall be used.
- When the approach roadway requires a TL-4 roadside barrier design, the approach transition details shown on standard drawing *S-1681-17 (Bridgerail to Modified Thrie Beam Transition Details)* shall be used in conjunction with the termination and curb end details on standard drawing *S-1643-17 (TL-4 Double Tube Type Bridgerail - Approach Rail Transition Details)*.

(b) TL-4 Single Slope Concrete Bridge Barrier

This is the preferred standard TL-4 bridge barrier for use on urban roadways for its aesthetic appeal. The bridge barrier has a top ledge originally developed as an architectural feature and for shielding the lower face of the barrier from wetting by rain. This feature has been adopted as a standard as it also provides extra strength in case of collision from a heavy vehicle. Standard drawing *S-1650-17 (TL-4 Single Slope Concrete Bridge Barrier Details)* presents the bridge barrier details.

There are three standard TL-4 approach barrier transition alternatives that can be used with this bridge barrier system:

- When the approach roadway requires a TL-3 roadside barrier design, the approach transition details shown on standard drawing *S-1651-17 (TL-4 Single Slope Concrete Bridge Barrier - Approach Rail Transition Details)* shall be used.
- When the approach roadway requires a TL-4 roadside barrier design, the approach transition details shown on standard drawing *S-1681-17 (Bridgerail to Modified Thrie Beam Transition*

Details) shall be used in conjunction with the termination and curb end details on standard drawing *S-1643-17 (TL-4 Double Tube Type Bridgerail - Approach Rail Transition Details)*.

- When the approach roadway barrier is a single slope concrete barrier, the end of the bridge barrier will need to be modified on the detailed design drawings to maintain the single slope geometry all the way to the end of the barrier so that it matches the geometry of the approach barrier. In addition, the interface between the bridge barrier and the concrete approach barrier will need to be modified on the detailed design drawings to include a concrete seat on the bridge barrier to support the approach barrier, following the details shown on standard drawings *S-1701-17 (TL-4 Combination Barrier - Barrier End Details)* and standard roadway drawing *RDG-B6.14 (TL-4 Single Slope Concrete Barrier Transition to PL-2 Standard Bridge Concrete Barrier)*. The Consultant needs to be very careful with the detailing of the interface and concrete seat to address the approach barrier loading and roadway drainage. This detail will not work well if excessive settlement of the approach roadway occurs.

For bridges with sidewalks, a traffic separation barrier is required between the traffic and the sidewalk. The separation barrier shall be as represented by standard drawing *S-1650-17 (TL-4 Single Slope Concrete Bridge Barrier Details)*, except that the top ledge shall be omitted to create a flat vertical face on the pedestrian side. The *CHBDC Clause 12.4.3.3* requires traffic separation barriers to have a smooth surface with no snag points and a minimum height of 0.60 m measured from the surface of the sidewalk. Note that this is a traffic barrier to prevent vehicle encroachment onto the sidewalk and is not provided as a pedestrian barrier. Therefore, no additional rail shall be mounted on the concrete barrier. The approach barrier transition may utilize any of the three alternatives discussed above. However, if one of the thrie beam approach transitions is utilized, project specific modifications to the transition details are required to protect pedestrians/cyclists from snagging the sharp corners at the top of the posts. This shall involve installing an HSS 50H x 100V rail, bolted to the sidewalk side of the thrie beam posts over the length of the thrie beam transition connecting to the end of the traffic separation barrier. The HSS 50x100 can be terminated when the sidewalk has moved away from the edge of the roadway to achieve a minimum separation of 600 mm from the back of the guardrail posts to the edge of the sidewalk.

(c) TL-4 Single Slope Concrete Combination Bridge Barrier

For bridges where the outside lane on the bridge is widened (typically widened to 4.2 m or 4.3 m) to accommodate cyclists, a 1400 mm high combination barrier shall be provided. This lane configuration is commonly provided on urban road bridges crossing over a mainline highway. This combination barrier was essentially developed by adding a rail to the Department’s previously developed (and crash tested) single slope concrete barrier. However, this combination barrier has not been crash tested and therefore the Department limits the use of this combination barrier to roadways with a design speed of less than or equal to 70 km/hr. Standard drawing *S-1700-17 (TL-4 Combination Barrier - Bridgerail Details)* presents the general barrier details.

There are three standard TL-4 approach barrier transition alternatives that can be used with this bridge barrier system:

- When the approach roadway barrier is a single slope concrete barrier, the concrete approach barrier transition details shall be as shown on standard drawing *S-1701-17 (TL-4 Combination Barrier - Barrier End Details)*. The approach barrier end and wingwall seat details shall be as shown on standard roadway drawing *RDG-B6.14*. Consultants will need to design and detail the approach barrier end and wingwall seat and on the detailed design drawings.

- When the approach barrier transition is a thrie beam system, the end of the bridge barrier and the transition shown on *S-1701-17 (TL-4 Combination Barrier - Barrier End Details)* will need to be modified and shown on the detailed design drawings. The end of the concrete bridge barrier shown on will need to be modified in accordance with *S-1650-17 (TL-4 Single Slope Concrete Bridge Barrier Details)* to show a transition from the sloped face barrier to a vertical face barrier and the appropriate approach rail transition anchorage details. In addition, the approach rail transition will need to be in accordance with standard drawings *S-1651-17 (TL-4 Single Slope Concrete Bridge Barrier - Approach Rail Transition Details)* or *S-1681-17 (Bridgerail to Modified Thrie Beam Transition Details)*, as appropriate for the approach roadway barrier.

(d) TL-4 Single Slope Concrete or Double Tube Bridge Barriers on Moment Slabs

This standard drawing was developed for traffic barriers located directly on top of MSE walls. Standard drawing *S-1798-17 (TL-4 Single Slope Concrete and Double Tube Type Barriers along Top of MSE Wall)* presents two barrier options: a single slope concrete barrier similar to *S-1650-17 (TL-4 Single Slope Concrete Bridge Barrier Details);* or a double tube rail on concrete curb barrier similar to *S-1642-17 (TL-4 Double Tube Type Bridgerail - Bridgerail Details)*. The moment slab barrier design is based guidelines presented in *NCHRP Report 663.*

The approach barrier transition details shall be provided as presented in Sections 12.2.2(a) or 12.2.2(b).

(e) TL-4 Thrie Beam Bridge Barrier on a 200 mm High Curb

This TL-4 bridge barrier is intended for use on smaller bridges that are less than 30 m in length (eg. 3 - 10 m span SLC bridge). For this barrier, the thrie beam from the approach transitions is continued across the bridge to provide a smooth continuous barrier without interruption. On the bridge, the thrie beam is mounted on top of a 200 mm high curb. Standard drawing *S-1648-17 (TL-4 Thrie Beam On Curb Bridgerail - Bridgerail Details)* presents the general barrier details and standard drawing *S-1649-17 (TL-4 Thrie Beam On Curb Bridgerail - Approach Rail Transition Details)* presents the standard thrie beam approach rail transition. This bridge barrier and the standard thrie beam approach rail transition both meet the TL-4 barrier requirements. Note that this approach transition requires a curb under the thrie beam, and it is slightly longer than the approach transitions shown on standard drawings *S-1643-17 (TL-4 Double Tube Type Bridgerail - Approach Rail Transition Details)* and *S-1651-17 (TL-4 Single Slope Concrete Bridge Barrier - Approach Rail Transition Details)*.

12.2.3 TL-5 Barrier with TL-4 Transitions

The Department has one standard TL-5 bridge barrier design, which is the 1270 mm high double tube rail on high concrete curb. This barrier is typically only necessary on bridges with very high AADT and truck traffic, as required by the *CHBDC*. The general TL-5 bridge barrier details are shown on standard drawings *S-1702-17 (TL-5 Double Tube Type Bridgerail - Bridgerail Details)*, *S-1703-17 (TL-5 Double Tube Type Bridgerail - Barrier End Details)* and *S-1704-17 (TL-5 Double Tube Type Bridgerail - Concrete Barrier Wall Details)*. The standard TL-4 thrie beam approach transition details are shown on standard drawing *S-1705-17 (TL-5 Double Tube Type Bridgerail - Approach Rail Transition Details)*. TL-4 is the highest test level that can be achieved with a thrie beam both in the approach transition and in the roadside barrier application. Where the roadway design requires the use of a TL-4 or TL-5 concrete roadside barrier, a site specific transition connection design is required with proper modifications to the bridge barrier end.

When a TL-5 barrier is required along the top of MSE walls, a site specific design is required. The design shall consist of a footing supported barrier (similar to that used for the TL-4 barrier on standard drawing *S-1798-17 (TL-4 Single Slope Concrete and Double Tube Type Barriers along Top of MSE Wall)*) and shall incorporate the 'above finished roadway' features of the TL-5 barrier shown on standard drawings *S-1702-17 (TL-5 Double Tube Type Bridgerail - Bridgerail Details)*, *S-1703-17 (TL-5 Double Tube Type Bridgerail - Barrier End Details)* and *S-1704-17 (TL-5 Double Tube Type Bridgerail - Concrete Barrier Wall Details)*, unless Approval for an alternate arrangement is granted.

When lamp poles or pier shafts are placed behind the bridge barrier, the height of the barrier shall be increased to 1370 mm by increasing the height of the lower concrete portion (see Section 12.3).

12.3 Modifications for Obstacles Behind Barriers

Obstacles mounted on top of or close behind bridge barriers (eg. signs, lamp posts, sign structure support columns, piers of adjacent bridges, etc.) have the potential to get snagged by vehicles that have collided with the barrier. This can adversely affect the post-collision behaviour of the vehicle. In addition, damage during the collision may result in portions of the obstacle falling onto a roadway below. As a result, the mounting of such obstacles on or closely behind the barrier shall be avoided whenever practical. However, when it is unavoidable, protective measures must be incorporated into the detailed design. The appropriate protective measures depend on the approach roadway barrier standard (as determined in accordance with the *RDG*). The following protective measures shall be incorporated into the detailed design:

1. When the approach roadway standard requires a TL-2 roadside barrier:
   - Provide a minimum 305 mm set-back from the traffic side face of the bridge barrier to the face of obstacle, measured at the top of the bridge barrier.
2. When the approach roadway standard requires a TL-3 roadside barrier:
   - Provide a minimum 610 mm set-back from the traffic side face of the bridge barrier to the face of obstacle, measured at the top of the bridge barrier.
3. When the approach roadway standard requires a TL-4 roadside barrier:
   - When TL-4 bridge barriers are required:
     - Provide a minimum 610 mm set-back from the traffic side face of the bridge barrier top steel rail to the face of obstacle; and
     - If design speed is less than or equal to 70 km/hr, provide the Department's *TL-4 Single Slope Combination Bridge Barrier* (*S-1701-17 (TL-4 Combination Barrier - Barrier End Details)*); or
     - If design speed is greater than 70 km/hr, provide a TL-5 bridge barrier with an overall height of 1370 mm. This requires the Consultant to provide a site specific design of the barrier. The Consultant shall use the Department's *TL-5 Double Tube Type Bridgerail* Barrier (*S-1702-17 (TL-5 Double Tube Type Bridgerail - Bridgerail Details)*) and modify it by increasing the height of the concrete base by 100 mm. This will result in a minor width reduction at the top of the

concrete base, but the horizontal location of the bridgerail mounted on the base should remain the same relative to the bottom of the barrier.

- When TL-5 bridge barriers are required:
  - Provide a minimum 610 mm set-back from the traffic side face of the bridge barrier top steel rail to the face of obstacle; and
  - Provide a TL-5 bridge barrier with an overall height of 1370 mm. This requires the Consultant to provide a site specific design of the barrier. The Consultant shall use the Department's *TL-5 Double Tube Type Bridgerail B*arrier (*S-1702-17 (TL-5 Double Tube Type Bridgerail - Bridgerail Details)*) and modify it by increasing the height of the concrete base by 100 mm. This will result in a minor width reduction at the top of the concrete base, but the horizontal location of the bridgerail mounted on the base should remain the same relative to the bottom of the barrier.

4. When piers of adjacent bridges are behind bridge barriers, provide a minimum 3000 mm set-back between the adjacent pier face and the traffic side face of barrier, as measured at the top of the barrier.

In addition to the above requirements, attachments on top of closely behind bridge barriers (eg. street light posts, etc.) shall be located close to the bridge supports (eg. abutments and piers) whenever practical, to avoid excessive vibration of the attachment from traffic loads.

12.4 Modifications to Approach Transition Barriers

(a) Beyond the bridge, the Department's standard TL-4 thrie beam approach transitions (*S-1643-17 (TL-4 Double Tube Type Bridgerail - Approach Rail Transition Details)* and *S-1651-17 (TL-4 Single Slope Concrete Bridge Barrier - Approach Rail Transition Details)*) are typically connected to a TL-3 approach roadway barrier such as a strong post guardrail system, a weak post guardrail system or a high tension cable barrier system. When connection to a TL-4 "Modified Thrie Beam" approach roadway barrier is required, the thrie beam approach transition shown on standard drawing *S-1681-17 (Bridgerail to Modified Thrie Beam Transition Details)* should be used. When connection to a TL-3 high tension cable approach roadway barrier is required, a FHWA approved connection is required. Currently, approved TL-3 proprietary systems are available from the FHWA Roadside Safety web links below:

- http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/barriers/pdf/b147.pdf
- http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/barriers/pdf/b147a.pdf

(b) Occasionally, intersections will be located too close to the end of a bridge to allow the full length approach transition to be installed. In such situations, consideration should be given to using standard drawing *S-1815-17 (TL-2 W-Beam Guardrail Approach Rail at Intersection Details)*. This is a crash tested TL-2 W-beam design suitable for use at intersecting ramps and street corners (there are currently not any similar designs that meet TL-3 design requirements, although some testing had been done but the results failed to meet criteria). This design is applicable for roadway design speed ≤ 80 km/hr. For higher design speeds, guardrails are required to stay aligned with the edge of the roadway for the full length of need and be terminated with an FHWA approved crash cushion.

No intersecting roadways are allowed within the length of need. For further guidance, refer to the *TAC Guide to Bridge Traffic Combination Barriers* Figures 6.21 to 6.24.

(c) Special attention and detailing is required at approach transition barrier posts in order to accommodate concrete drain troughs (see standard drawings *S-1841-17 (Standard Drain Trough For Conventional Abutments)*, *S-1842-17 (Standard Drain Trough For Grade Separation Bridges With Integral Abutments)* and *S-1843-17 (Standard Drain Trough For Water Crossing Bridges with Integral Abutments)*).

(d) The interaction between curb and guardrail is reported in *NCHRP Report 537*. The recommendations in the report are incorporated in the *RDG Section H4.3: Curbs* and *RDG Table H4.1*. Placement of guardrail posts and curbs is detailed in standard roadway drawings *RDG-B1.10* and *RDG-B1.11*. When there is a barrier curb that runs up to the end of a bridge barrier, it is important to build the end of the bridge barrier straight out to be continuous with the curb face, and not incorporate the large chamfer under the thrie beam connector as shown on the “CURB END DETAIL” on standard drawing *S-1642-17 (TL-4 Double Tube Type Bridgerail - Bridgerail Details)*, and on the “BRIDGE BARRIER ISOMETRIC VIEW” on standard drawings *S-1650-17 (TL-4 Single Slope Concrete Bridge Barrier Details)* and *S-1703-17 (TL-5 Double Tube Type Bridgerail - Barrier End Details)*.

12.5 Pedestrian and Bicycle Barrier

The Department’s standard pedestrian and bicycle barriers are shown on standard drawings *S-1845-17 (Standard Pedestrian and Bicycle Barrier - Sheet 1)* and *S-1846-17 (Standard Pedestrian and Bicycle Barrier - Sheet 2)* and are designed for use along the exterior edge of the sidewalk, with a standard single slope concrete barrier provided between the roadway and the sidewalk.

Pedestrian barriers are intended to be used with sidewalks where the volume of bicycle traffic is expected to be very low and infrequent, while the bicycle barrier should be used with sidewalks where the volume of bicycle traffic is expected to be significant enough to cause frequent potential conflict with other users. In some cases it may be necessary to conduct an assessment of the site and the expected volume of bicycle traffic. In general however, in lieu of an assessment, pedestrian barriers should be considered for rural applications where the sidewalk width is less than 2.5 m. For rural sidewalks that are 2.5 m or wider, and for all urban/urban fringe sidewalks, the bicycle barrier should be used. Typically, cyclists are only allowed to ride on a sidewalk if it is a multi-use path, which for bridges would consist of a sidewalk with a clear width of 4.2 m between barriers. Under current provincial legislation, cyclists (considered non-motorized vehicles) are not permitted to operate on sidewalks unless otherwise permitted by the Minister. Any sidewalks narrower than 4.2 m would typically require signage stating that cyclists should dismount. In areas with high bicycle traffic (existing or expected) past Department experience has shown that cyclists may not always dismount, and it is therefore recommended to use the bicycle barrier to protect users in such cases.

The standard pedestrian barrier rail shall be mounted on a concrete curb projecting 100 mm above the sidewalk for a total barrier height of 1250 mm. The standard bicycle barrier rail shall be mounted on a concrete curb projecting 100 mm above the sidewalk for a total barrier height of 1400 mm.

The standard bicycle barrier includes a steel rub rail at a height of 1070 mm from the top of the pathway to reduce the snagging hazard of bicycle handlebars on the vertical handrail bars and posts.

In accordance with *Clause 12.4.4.2 (Geometry)* of the *CHBDC*, the standard pedestrian/bicycle barrier is detailed so that the maximum allowable opening between the vertical handrail bars/posts is 150 mm. This requirement also applies at the expansion joint locations, where the 150 mm opening must be met at the coldest design temperatures. The expansion joint detailed on standard drawing *S-1846-17 (Standard Pedestrian and Bicycle Barrier - Sheet 2)* provides a total movement range of 120 mm (minimum rail gap is 20 mm and maximum rail gap is 130 mm). The expansion joint is detailed so that it can be installed at any temperature for thermal spans up to around 60 m long. For thermal spans up to around 110 m long, the same expansion joint detail may be used but the rail gap would need to be adjusted in the field to account for the installation temperature. This information will need to be shown on the detailed design drawings. For bridges with longer thermal spans, the expansion joint detail will need to be modified and shown on the detailed design drawings. In particular, careful detailing will be required to ensure that the maximum allowable opening between the vertical handrail bars/posts does not exceed 150 mm at the coldest temperatures.

Notwithstanding *Clause 12.4.4.2 (Geometry)* of the *CHBDC*, for bridges that are expected to have higher volumes of pedestrian traffic or bridges on which pedestrians might reasonably be expected to stop and congregate, consideration should be given to limiting the maximum opening between the vertical handrail bars/posts to 100 mm. Most river crossings and any bridges with rest areas or benches along the pathway would qualify. In addition, bridges with aesthetically pleasing or interesting viewpoints should be considered. This will require the Consultant to include modifications of the pedestrian and bicycle barrier rail on the detailed design drawings.

For situations where an intersection is located close to the end of the bridge, it is important that roadway vehicle drivers can detect pedestrians/cyclists on the pathway that may be crossing the intersecting road. Depending on the geometry of the intersection, the vertical bars on the pedestrian/bicycle barrier can actually obstruct the vision of the drivers, not allowing them to detect the pedestrians/cyclists early enough. In these situations, it is possible to improve the driver's ability to detect pedestrians/cyclists by utilizing a detail with staggered vertical bars. A detail for this is included on *S-1846-17 (Standard Pedestrian and Bicycle Barrier - Sheet 2)*. The direction of the stagger depends on the configuration of the intersection and the traffic movement. The Consultant must specific the use of the staggered vertical bars and must specify the direction of the stagger on the detailed design drawings.

At the bridge ends, the barrier end shall be cantilevered past the end of the wing walls and deflected outwards 400 mm away from the pathway, as shown on *S-1846-17 (Standard Pedestrian and Bicycle Barrier - Sheet 2)*.

12.6 Standard Bridge Barrier Detailing

Even though most of the design information is provided on the standard barrier drawings, certain design information is site specific and must be provided on the detailed design drawings. Consultants should be familiar with these requirements and ensure that all necessary information is provided.

(a) Bridgerail shall be detailed as follows:

- All dimensions for bridgerail layouts are to be given on centreline of bridgerail anchor bolts.

- Bridgerail expansion joints shall be provided at all deck joint locations. Additional bridgerail expansion joints shall be provided at a maximum spacing of 45 m.
- Standard barrier drawings show a standard bridgerail expansion joint with a gap of 100 mm, and a large expansion joint with a gap of 200 mm. Considering that most bridge abutments tend to move inwards over the life of the bridge, a large expansion joint should be selected when there is potential for the bridgerail joint to jam up before the deck joint closes.
- Steel railing for bridges with curve radius > 600 m can be chorded between field splices. Steel railing for bridges with curve radius ≤ 600 m shall be manufactured in curve. In the latter case, the Consultant shall clearly indicate such requirement on the detailed design drawings. Tube sleeves for splices and expansion joints shall be detailed accordingly.

(b) For attachments mounted on or closely behind bridge barriers (eg. street light posts, etc.), base plates and anchors shall be grouted and sealed with a penetrating sealer. A minimum 40 mm nominal thickness grout pad shall be provided under base plates. The grout shall sit in a grout pocket recessed 20 mm into the surface of the structure. The grout pocket shall be 40 mm larger than the base plate around the perimeter.

13. BRIDGE DRAINAGE

(a) Salt contaminated surface drainage shall be contained and controlled so that it does not come into contact with bridge components (eg. girders, bearings, diaphragms, substructure and retaining walls) with the exception of those components that are specifically allowed by the Department to be exposed to salt contaminated water (traffic face and tops of curbs/barriers/medians, bridge deck and roadway surface, deck joints, deck drains, drain troughs and concrete slope protection).

(b) The use of deck drains should be minimized where possible, but shall be provided in accordance with the Department's *Bridge Conceptual Design Guidelines* (further discussion is also provided in *BPG 12 - Bridge Deck Drainage)*. The goal is to restrict surface water runoff to within the width of the shoulders and avoid lane encroachment of runoff in order to minimize the risk of hydroplaning. Drainage issues should receive early attention at the planning stage, when there is opportunity to optimize bridge geometry.

(c) Any drains required to accommodate deck drainage, sub-surface deck drainage or drainage through deck joints shall be hidden from view where practical. Drains, including mounting brackets that cannot be hidden from view shall receive a finish that is acceptable to the Department and that causes them to blend into the surrounding structure.

(d) Any deck drainage that is discharged onto the headslopes or sideslopes shall be done in a controlled manner such that erosion of the headslope or sideslope does not occur. This applies at surface water deck drains along the length of the bridge, and at the ends of the bridge with deck joints and sub-surface deck drains. It is typically sufficient for the deck drainage to be discharged directly onto concrete or riprap headslope protection. Where it is not possible to drain onto the slope protection, additional measures shall be taken to prevent erosion.

(e) At the ends of bridges, concrete drain trough collectors shall be used to channel water off of the bridge and into concrete drain troughs. Drain troughs are required at the low corners of bridges and may also be required at the high corners of the bridges if the approach roadway has significant grade and the roadway runoff could result in erosion around the bridge wingwalls. Drain troughs may also be required at the ends of retaining walls where a roadway runs adjacent to the top of a retaining wall. Drain troughs shall drain directly down the slope (not across the slope), and shall extend to the bottom of the roadway approach fills. The drain troughs shall be designed to function as intended for the drainage volume and velocity while accommodating differential settlements and other movements between the bridge and the roadway approach fills. Drain troughs may be eliminated if the roadway drainage at the bridge barrier transitions is controlled by curbs/concrete barriers and catch basins. Standard drain trough details are shown on standard drawings *S-1841-17 (Standard Drain Trough For Conventional Abutments), S-1842-17 (Standard Drain Trough For Grade Separation Bridges With Integral Abutments)* and *S-1843-17 (Standard Drain Trough For Water Crossing Bridges with Integral Abutments)*. The Consultant shall include additional information on the detailed design drawings to address the various site specific details, including: approach slab dimensions and reinforcing; sleeper beam dimensions and reinforcing; side slope geometry; approach transition barrier type and geometry; connection details between the approach slab and sleeper beam; and dimensions and reinforcing of the drain trough collector. The standard drain trough details may also need to be modified for additional capacity in situations where excessive deck runoff may be expected (eg. very long, wide or steep bridges).

(f) Additional requirements around bridge abutments are provided in Section 6.4 (Bridge Embankments / Abutments) and Section 7 (Retaining Wall Structures).

14. DUCTS AND CONDUIT SYSTEMS

Ducts and conduit systems shall be fully detailed on the appropriate bridge detailed design drawings (e.g. abutment drawings, pier drawings, deck and barrier drawings, etc.). Ducts and conduit systems shall be installed in accordance with the following Sections. They shall not be installed within the thickness of the bridge deck, attached to the bridge girders, or attached to the underside of the bridge deck except as noted below.

14.1 Utility Ducts in Curbs and Barriers

The Contractor shall provide one 75 mm nominal outside diameter utility duct on each side of the bridge deck for the future accommodation of telecommunication or power utilities (ie. telephone, fibre-optic or street-lighting). The purpose of these ducts is to allow for unforeseen utility needs that are not currently anticipated at the design stage of the project. For bridges that are to be widened at Ultimate Stage, a duct will not be required on the side to which widening occurs. However, if widening takes place on both sides, a duct will be required on one side, and the side will be identified by the Department upon request. These utility ducts shall be placed within the bridge curbs and/or barriers and shall be terminated into a weather proof junction box on the outside of the wingwalls, near the wingwall ends and close to the ground level, where they are not too visible but can be easily accessed without damaging the existing road or bridge. The utility duct termination shall be detailed on the 'Abutment' detailed design drawings.

If additional utility ducts are required for the utility needs of the project, they may be placed within the bridge curbs/barriers which will not be removed at the Ultimate Stage. Where multiple ducts are placed in the same curb/barrier, the Consultant must ensure that strength of the curb/barrier has not been compromised.

Waterproof O-ring expansion fittings shall be provided at all bridge expansion joints and at locations where sidewalk curbs or barriers could undergo rotational settlement (e.g. sidewalk/roadway barrier over corbel supported approach slabs). Loose fit or tape connections are not permitted. At expansion joint, the O-ring expansion fitting shall be located within the deck joint block-out to facilitate future maintenance.

All utility ducts cast into curbs/barriers shall be rigid PVC DB2, meeting the requirements of *CSA C22.2 No. 211.1* and in accordance with the rules of the *Canadian Electrical Code, Part 1*. Coupling shall be solvent bell ends (SBE). Pull strings shall be installed and secured at each end of all ducts/conduits for future installations.

14.2 Conduit Systems for Under-bridge Lighting

Any conduits required for wiring to under-bridge lighting systems shall be cast within the bridge piers and pier caps and shall not be routed through abutment ends. If, at a specific bridge structure, no piers exist or other conditions exist that prevent routing of conduits for electrical supply through the piers, an alternative routing may be proposed for review by the Department.

The conduit system shall be concealed and shall comprise rigid PVC conduit having a minimum trade size of 41 mm, together with industry-standard junction boxes and fittings. The system shall provide a continuous concrete-proof and weatherproof conduit arrangement from below ground to the top surface of each pier cap.

Conduits shall be placed as follows:

(a) Conduits shall enter the bridge structure a minimum 1000 mm below finished ground elevation at the exterior of the pier as necessary and shall bend up to connect with a PVC junction box to be recessed on the exterior surface of the pier shaft 1000 mm above finished ground elevation. Minimum clear inside dimensions of this PVC junction box shall be 150 mm x 150 mm x 150 mm. The junction box may be larger if necessary for the proper connection and bonding of bridge wiring to incoming supply cables according to *Canadian Electrical Code* requirements. The PVC junction box is to be set flush with the surface of the pier shaft and shall be fit with a gasketed weatherproof securable cover.

(b) A riser conduit shall then extend up to a weather proof PVC access junction box secreted in the top surface of the pier cap. This box shall be sized for the number of luminaire conduits and wires to be accommodated at that point. For bridge structures where a concrete pier diaphragm precludes placement of an access junction box in the top of the pier cap, it may, subject to Approval, be placed unobtrusively in the face of the pier cap near its top edge. For bridge structures with integral pier cap/diaphragms, the riser conduit shall extend into the pier cap/diaphragm and up to the weather proof PVC access junction box secreted in the side surface of the pier cap/diaphragm.

(c) Additional weather proof access junction boxes may be installed in the pier cap as required by the width of the bridge and the number of luminaires to be serviced. These additional access junction boxes shall be supplied by a rigid PVC conduit not less than 27 mm trade size cast horizontally within the pier cap/diaphragm.

(d) Rigid conduits exiting the access junction boxes to service under-bridge luminaires shall be the minimum diameter consistent with *Canadian Electrical Code* requirements for the number and sizes of wires employed and the availability of attachment support points, but not less than 16 mm trade size.

(e) Luminaire conduits shall be run in neat vertical and horizontal alignments, supported as necessary to comply with *Canadian Electrical Code* requirements and to mitigate the effect of vibrations induced in the bridge by passing traffic.

(f) Luminaires shall be mounted on bridge pier caps or steel diaphragms as required. Where it is necessary to install a horizontal conduit run to access a luminaire, the conduit or any necessary conduit support tray or truss supports shall be fixed to the vertical face of the bridge girder haunches, with anchors cast into the haunch concrete at appropriate locations. No attachments shall be fixed to the girders or to the underside of the bridge deck.

(g) Luminaires conduits and/or conduit support equipment that are supported on the superstructure shall be located within interior girder bays.

(h) For solid slab superstructures or similar superstructure systems that do not have girders, an alternative fastening method shall be submitted for Approval.

(i) In the event that partial depth precast concrete deck panels are utilized, anchors for the purpose of supporting lighting conduits shall be cast into the underside of the precast deck panels. These anchors shall be positioned at the edges of the precast deck panel so that the conduits are located within 100 mm of the edge of the girder top flange. Spacing between anchors in the precast deck panels and between anchors on adjacent precast deck panels shall not exceed the maximum conduit support distance allowed in the *Canadian Electrical Code*.

(j) All wiring to under-bridge luminaires shall be RW90 of appropriate number and gauge to comply with voltage drop limitations. A continuous ground wire is required in all under-bridge lighting conduits to ensure the whole system is properly bonded. Conduits shall be sized to accommodate the noted wiring requirements.

(k) Prior to the wiring being installed, all conduits shall be proven to be free and clear of obstructions.

15. OVERHEAD SIGN STRUCTURES

Overhead sign structures, including bridge support or cantilever types, are procured by a design/build process in accordance with *SSBC Section 24: Overhead Sign Structures*.

Overhead sign structures shall be designed in accordance with the requirements of *AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals*. Overhead cantilever sign structures shall be designed for Fatigue Category 1. In addition, due to the difficulty in predicting the dynamic behaviour of overhead cantilever sign structures and determining which overhead sign structure types are susceptible to galloping, the Department requires that all overhead cantilever sign structures shall be designed for the galloping loads, without exceptions.

The Consultant's responsibilities for overhead sign structures are summarized as follows:

1. The Consultant shall determine placement, clearance requirements, need for barrier protection, and type of structure (bridge support or cantilever type) in accordance with guidance provided in *RDG Section H8.4: Overhead Signs*. The Consultant shall prepare a 'General Layout' detailed design drawing for each individual overhead sign structure in accordance with typical detail drawing *T-1721-17 (Typical Overhead Sign Structure General Layout)*.

2. Each overhead sign structure is treated as a small bridge, and tracked by a bridge file number for design, construction and BIM inspections. This structure classification is used for all sign support structures where the sign panel area is larger than 4 m^2 and the sign is fully or partially hanging over the traffic lanes or road shoulder.

3. The Consultant shall review the design notes and shop drawings submitted by the contractor to ensure all requirements of *SSBC Section 24: Overhead Sign Structures* are met. This review is for conformance of specification requirements only and is not a design check.

4. At the end of the construction, the Consultant shall update the 'General Layout' detailed design drawing(s) to reflect as-built conditions and forward all supplier design notes and shop drawings to the Department for record keeping purposes.

16. PREPARATION OF DETAILED DESIGN DRAWINGS

Bridge detailed design drawings shall be prepared in accordance with the Department's *Engineering Drafting Guidelines for Highway and Bridge Projects (Drafting Guidelines).* It is recommended that all bridge engineers working on Department bridge projects be intimately familiar with the *Drafting Guidelines* as it contains a lot of useful information, including:

- Requirements for drawing layout;
- Specific detailing preferences;
- Unit bid item quantities;
- Reinforcing bar marks and bar lists;
- Checklists for Bridge Drawings (in the Department's *Drafting Guidelines Appendix E);* and
- Standard Bridge Drawings Notes (in the Department's *Drafting Guidelines Appendix F*).

Consultants who are not familiar with AT bridge detailed design drawings are encouraged to obtain recently completed design drawing sets for their guidance.

The use of standard drawings and typical details is required wherever possible.

17. LIST OF DEFINITIONS AND ACRONYMS

17.1 Definitions

Definitions used in the Bridge Structures Design Criteria shall be in accordance with those provided in the *CHBDC*, unless otherwise noted below:

Approval (or Approved): approval, or approved, in writing by the Department's Director of Bridge Engineering.

Approved: when referencing a product or material, this shall mean products or materials listed as "Approved products" on the Department's Product List.

CHBDC: shall mean the CSA-S6-14 Canadian Highway Bridge Design Code.

Detailed Design Drawings: shall mean any drawings prepared by the Department's Consultant to be included in the construction tender package. These drawings shall be prepared in accordance with the Department's *Engineering Drafting Guidelines for Highway and Bridge Projects*.

Standard Bridges: shall mean any bridge built using the Department's Standard Drawings.

17.2 Acronyms

The following acronyms apply to the Bridge Structures Design Criteria document:

AADT: Average Annual Daily Traffic

AASHTO: American Association of State Highway and Transportation Officials

ACP: Asphalt Concrete Pavement

AISI: American Iron and Steel Institute

ASTM: ASTM International, formerly known as the American Society for Testing and Materials

AT: Alberta Transportation

AWS: American Welding Society

BIM: Alberta Transportation Bridge Inspection and Maintenance

BPG: Alberta Transportation Bridge Best Practice Guideline

BSDC: Alberta Transportation Bridge Structures Design Criteria (current version)

CSA: Canadian Standards Association

CRR: Corrosion Resistant Reinforcing

CSE: Copper Sulphate Electrode

FHWA: Federal Highway Administration

HPC: High Performance Concrete

LRFD: Load and Resistance Factor Design

MSE: Mechanically Stabilized Earth

NCHRP: National Cooperative Highway Research Program

NSBA: National Steel Bridge Alliance

PTFE: Polytetrafluoroethylene

PTI: Post Tensioning Institute

PVC: Polyvinyl Chloride

RDG: Alberta Transportation Roadside Design Guide

RWIS: Road Weather Information System

SSBC: Alberta Transportation Standard Specifications for Bridge Construction

SSPC: Society for Protective Coating Standards

TAC: Transportation Association of Canada

UNS: Unified Numbering System

WEAP: Wave Equation Analysis of Piles

18. REFERENCES

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APPENDIX A – INTEGRAL ABUTMENT GUIDELINES

A1. INTRODUCTION

This Appendix presents a general discussion of integral bridges as well as some of the more important issues to consider when selecting and designing integral abutments (includes semi-integral and fully-integral abutments).

Shorter bridges and older longer bridges in Alberta have typically been constructed using simple span girders with deck joints at every abutment and pier. However, decks joints have proven to be expensive and prone to maintenance problems:

- Neoprene seals may last 10-15 years and then will need to be replaced. If the joints are jammed (due to substructure movements), it can become very difficult to extract and replace the neoprene seals.
- The joint nosing concrete will normally last longer than the neoprene seal, but often suffers deterioration from shrinkage cracks and direct exposure to road salt. To remove and replace the complete joint assembly including the adjacent concrete generally requires extensive work and major traffic disruption for weeks.
- Joint leakage can also cause damage to structural components below the joint, such as bearings, girder ends, and abutment and pier components.

Most transportation jurisdictions in Canada have moved to minimize the use of expansion joints where practical. Since the 1970’s, the Department has designed bridges to be continuous over the piers (eliminating the joints at these locations) but still with deck joints at the abutment ends.

The development of integral abutment bridges has allowed the elimination of the deck joints at abutments for shorter to medium length bridges. For these types of bridges, thermal superstructure movements are accommodated at the end of the approach slabs, rather than at the end of the superstructure. Leakage through the control joints at the ends of the approach slab can be controlled and directed away from the bridge, avoiding leakage onto critical structural components. Joint maintenance and repair is generally limited to repairing the asphaltic wearing surface, and can be achieved with minimal disruption to traffic.

Today many jurisdictions routinely use integral abutment designs and recent reports of performance have generally been good. However, good understanding of the behaviour of integral abutments, and care in detailing and construction are required to ensure success.

In Alberta, pin connected semi-integral abutments have been used with short span precast girder bridges for a very long time and have been performing quite well (typically up to 60 m in total bridge length). For medium length bridges (<100m in total bridge length), AT encourages the use of integral abutments where appropriate.

The following is a list of the advantages of integral abutment bridges:

1. Reduced initial costs and long term maintenance costs;
2. Locates the control joints away from the end of superstructure and other structural components;
3. Elimination of bearings and deck joints, resulting in less tolerance restrictions and faster construction;
4. Continuous waterproofing membrane and ACP across the bridge results in better deck protection from de-icing salts;

5. Smoother ride across bridges due to continuous paving;
6. Reduced number of foundation piles;
7. Abutment ends are buttressed between headslope fills at each end, resulting in less potential for superstructure shifting and pier tilting;
8. Increased reserve load capacity and load distribution, resulting in more resistance to damaging effects of illegal overloads;
9. Provides resistance to uplift at abutment end;
10. Reduced end span to interior span ratio will allow longer interior spans for under-passing roadways and streams.

A2. Types of Integral Abutments

1. Fully-integral abutment on piles (Figure A1 and A2): Fully monolithic connection between end of superstructure and abutment. Single line of steel H-piles flex to accommodate thermally induced bridge deck movements. This is the most efficient design in most situations and every effort should be made to achieve full integral construction.

2. Semi-integral abutment with pinned connection (Figure A3): Pinned connection between superstructure end and abutment. Top of single line of H-piles move with thermal cyclic changes in superstructure length, without transfer of moments and rotations between girder ends and abutment piles.

3. Semi-integral abutment with sliding bearings (Figure A4): Superstructure slides over fixed abutment seat (with double row or stiff piles). Applicable where single line of flexible piles is overstressed, piles cannot flex due to embedment in stiff soils, or bridge has high skews.

4. Semi-integral abutment with partial backwall (Figure A5): Similar to Figure A4 above, except a partial height backwall will resist the earth pressure and the approach slab slides over the top of the back wall. This removes the earth pressure behind the end of the superstructure for cyclical movements. This tends to reduce the planar twisting of the bridge deck with high skews. This design is also suitable for conversion of existing conventional abutments to semi-integral in rehabilitation projects.

A3. DESIGN CONSIDERATIONS

There are many comprehensive and useful resources for designing integral abutments (see references). There is no point re-iterating everything from that is stated well in these documents. This Section A3 simply highlights some of the more important design considerations.

A.3.1 Thermal Movements

Integral abutments work well when the superstructure is free to expand and contract, due to thermal loading, with very little restraint. Restraint can be controlled by considering the following:

- Length of bridge: See Section A.3.2 (Cycle Control Joints) for further discussion.
- The size of integral abutments should be minimized in order to reduce the associated pressures and resistance to movement. As girders and abutment seats get deeper, the earth pressure

resisting thermal expansion can become considerable and harder to precisely quantify. The depth of the girders is generally controlled by the allowable bridge lengths. However, for deeper girders, semi-integral abutments with partial depth backwalls can be used to minimize the earth pressure (see Figure A5). For fully-integral abutments, the abutment seat height above grade should be minimized in order to reduce the earth pressure. The length of the wingwalls should also be minimized.

- Fully-integral abutment piles need to be sufficiently flexible to allow the abutment to move without developing large forces in the superstructure. Typically, this is accomplished by using a single row of H-piles oriented to bend about their weak axis (with casings around the H-piles for longer bridges).
- Care must be taken for bridges with skews. The bridge will want to expand/contract in a direction matching the thermal movement of the superstructure. Out-of-plane forces can result from earth pressure that develops as the skewed bridge abutment expands against the backfill. Additionally, if fully-integral piles are oriented in a direction that does not match the thermal movement of the superstructure, an out-of-plane force can develop. For small skews, it is typically acceptable to orient the H-piles parallel with the superstructure. For larger skews, more comprehensive analysis is recommended to determine the additional forces that may develop. In general, it is recommended to avoid integral abutments for bridges with skews larger than 20 degrees.
- Fully-integral abutment piles must be designed for vertical dead and live loads as well as moments that result from the superstructure movement and rotation. For bridges with longer movements, the moment capacity can often limit the pile design. Some design references present the option of carrying out a more refined pile design analysis to design the pile with the development of a plastic hinge. While this approach is acceptable, care must be taken with this method as it requires a more comprehensive analysis and requires careful design of the pile to ensure that the pile can develop a fully plastic stress distribution without flange local buckling.

A.3.2 Cycle Control Joints

The elimination of abutment deck joints does not eliminate the need for joints to accommodate cyclic thermal movement. Integral bridges will expand and contract in response to changes in temperature. These thermal movement control joints are located at the ends of the approach slabs.

The ambient temperature in Alberta generally ranges from +35 ºC to -45 ºC. Assuming an average construction temperature of +20 ºC (when approach slabs are cast), roughly 80% of the temperature range is on the contraction side. This causes the gap to open up in cold temperatures. It has been observed that under winter freezing temperatures, the gap becomes iced over, effectively providing a natural seal over the joint. Over the summer months, the joints are normally closed.

When the asphaltic pavement is placed, the pavement shall be saw cut directly over the control joints to produce a good square edge, and the sawcut gap shall be filled with an approved asphaltic hot pour rubberized crack sealing compound. These cracks may need to be re-sealed from time to time.

The control joints (and the ends of approach slabs) shall be located at least 1.125 m beyond the ends of the wingwalls, as shown on sketches SK-B3 and SK-B4 in Appendix B. The detailing of these control joints shall be coordinated with the approach slab, abutment drain trough and approach transition barrier or guardrail transition posts, as shown on standard drawings *S-1842-17 (Standard Drain Trough For Grade Separation Bridges With Integral Abutments)* and *S-1843-17 (Standard Drain Trough For Water Crossing Bridges with Integral Abutments)*.

The following Table A1 provides guidance for joint types and corresponding maximum thermal span limits. For symmetrical span arrangements, the total allowable bridge length is twice the maximum thermal span. The difference in concrete and steel bridge lengths reflects the greater thermal mass of concrete and the greater sensitivity of steel in reacting to temperature changes.

The thermal span is the length measured from point of thermal fixity of the superstructure to the centerline of the integral abutment.

Table A1 - Maximum Thermal Spans

Cycle Control Joint Type	Maximum Joint Gap	MAXIMUM THERMAL SPAN FOR STEEL GIRDER BRIDGES	MAXIMUM THERMAL SPAN FOR CONCRETE GIRDER BRIDGES
C1	*20 mm*	*22.5 m*	*30 m*
C2	*40 mm*	*45 m*	*60 m*

Cycle Control Joint Type C1 - Many short integral abutment bridges have been built with the only evidence of longitudinal thermal movement being the formation of a tight crack at the end of the approach slab. These cracks are of minor nature and do not present any problem. A simple saw-cut joint in the pavement is provided at the end of the approach slab for crack control and is subsequently filled with an approved hot pour rubberized crack sealant. This joint is shown on standard drawing *S-1840-17 (Standard Waterproofing System For Deck and Abutments - Sheet 3)*.

Cycle Control Joint Type C2 - For longer length bridges, a wider joint gap can allow more roadway drainage to reach the granular base course and subgrade. Therefore a sleeper slab is required under the end of the approach slab to protect the granular base course and subgrade, which can be damaged by the passage of heavy truck axles causing very high pore water pressure. The trench excavated for the installation of the sleeper slab and the granular base should be extended across the full width of the road and be daylighted at the sideslope to avoid forming a bathtub that traps water. The ends of the trench should be integrated with the abutment drain troughs if they are present. A transverse wick drain shall be installed on the sleeper slab under the AIFB to capture surface water that infiltrates into the joint gap. The transverse wick drain extends the full width of the sleeper slab and shall be integrated with the washed rock trench. This joint is shown on standard drawing *S-1840-17 (Standard Waterproofing System For Deck and Abutments - Sheet 3)* and integration of the wick drains is shown on standard drawings *S-1842-17 (Standard Drain Trough For Grade Separation Bridges With Integral Abutments)* and *S-1843-17 (Standard Drain Trough For Water Crossing Bridges with Integral Abutments)*.

A.3.3 Drainage

To achieve a successful and durable design, good drainage details must be incorporated. The goal is to prevent surface drainage from going below the surface at the abutments.

The joints between the approach slab and the wing walls should be sealed to prevent water infiltration. Even the best attempt to seal joints cannot prevent all leakage. It is therefore important to design a secondary system of sub-soil weeping drains to collect, channel and remove the seepage.

Drainage at the control joints includes surface drainage and sub-surface drainage of the deck wickdrains. This water must be collected and directed to the sideslopes,

Surface drainage at the ends of wingwalls is very difficult to control because the wingwalls are moving with the superstructure. This is further complicated on bridges with concrete approach barriers. Details for maintaining water tightness between the thermally cycling wingwall barrier and the stationary approach details shall be carefully thought through and detailed. Standard drawings *S-1842-17 (Standard Drain Trough For Grade Separation Bridges With Integral Abutments)* and *S-1843-17 (Standard Drain Trough For Water Crossing Bridges with Integral Abutments)* show details for accommodating the movement between the end of the wingwall barriers and drain troughs.

When MSE walls with galvanized steel soil reinforcement is incorporated as part of the abutment, an impermeable membrane is required over the top of the steel straps to keep salt contaminated water away from the straps.

A.3.4 Settlement

Integral abutments should generally be avoided in situations where excessive settlement of the approach roadway/fills is expected. With excessive settlement of the approach roadway/fill, the free end of the approach slab will also settle excessively. Not only will this result in an undesirable bump at the end of the bridge, but it can also not allow the integral abutment to function properly as the approach slab will not be free to move longitudinally. This can result in significant forces being built up at the connection between the superstructure and the approach slab. Additionally, with fully-integral abutments, excessive settlement of the approach roadway/fill can result in the pile casings being pulled down, which could allow backfill and water to enter into the casing void.

Once the approach slabs settle, it is very difficult and expensive to repair the situation, especially for integral abutments with Cycle Control Joint Type C2 (see Section A.4.3 (Cyclic Joint Repair))

A4. CONSTRUCTION AND MAINTENANCE CONSIDERATIONS

A.4.1 Construction Sequence

For fully-integral abutments, the girders are erected on abutments with a single line of flexible piles. Depending on the construction sequence, the abutment stability could be a concern and should be checked for all stages of construction - temporary support may be required.

Thermal cycles during construction result in girder end rotations and lateral and longitudinal movements of the girders. This can then give rise to differential movement between the girders and the abutment with the girders shifting or the abutment seat moving. There are several options for accommodating these effects, including rigidly connecting the girder ends to the abutment with anchor rods (for lateral and/or longitudinal movements), or by simply allowing the girder ends to float on temporary bearing pads with or without temporary construction restraints. Depending on the temporary bearing type used during girder erection (e.g. plain unreinforced elastomeric pad) and the size and rigidity of the piles, there may be sufficient friction between the girders and the abutment seat (before the concrete diaphragm is poured) to produce bending in the piles from thermal cycling of the girders.

For long precast girders with post-tensioning, sliding bearings should be provided and the abutment seat should be braced to prevent lateral movement during post-tensioning.

It is typically recommended that backfilling behind integral abutments follow a balanced sequence so that the superstructure is not pushed out-of-position by backfilling operations.

A.4.2 Backfill and Erosion Control

For all bridges, the development of a big bump at the end of the bridge is not desirable and this is even more so important for the proper functioning of integral abutments (see Section A.3.4 (Settlement)). While bumps can develop due to fill settlement, they can also develop due to poor abutment backfilling.

Poor backfilling if often related to poor compaction techniques. However, it can also happen when voids develop under the abutment seats and wingwalls during construction and are not properly filled. These voids often develop due to poor drainage control during construction as rainwater erodes around and under the abutment seats and wingwalls.

A.4.3 Cyclic Joint Repair

For longer bridges utilizing prestressed / post-tensioned concrete girders and Cycle Control Joint Type C2, the joint gap may open up more than desirable during the first two years after construction, due to creep and shrinkage shortening of the prestressed girders. However, this is a onetime permanent irreversible opening. The opened gap can be repaired as follows:

- Remove a minimum 1.2 m wide strip of ACP over the length of the joint;
- Clean out the existing joint gap down to the top of the existing transverse wick drain; Replace the transverse wick drain if damaged;
- Sand blast clean the exposed vertical face of the sleeper beam upstand;
- Install and spot glue new 20 mm asphalt impregnated fibre board (AIFB) to end face of approach slab;
- Fill remaining gap with an approved chemical grout such as Set 45, re-pave and saw cut the pavement over the top of the AIFB and re-seal the joint with an approved asphaltic hot pour rubberized crack sealant.

Settlement in bridge approaches often occurs within the first two years and can be repaired at the same time. Mudjacking the approach slabs is an option where Cycle Control Joint Type C1 is used. However, where Cycle Control Joint Type C2 is used, mudjacking may not be practical as it may result in a separation between the approach slab and the sleeper slab, thereby compromising the ability of the bridge to thermally expand/contract.

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A6. List of Figures

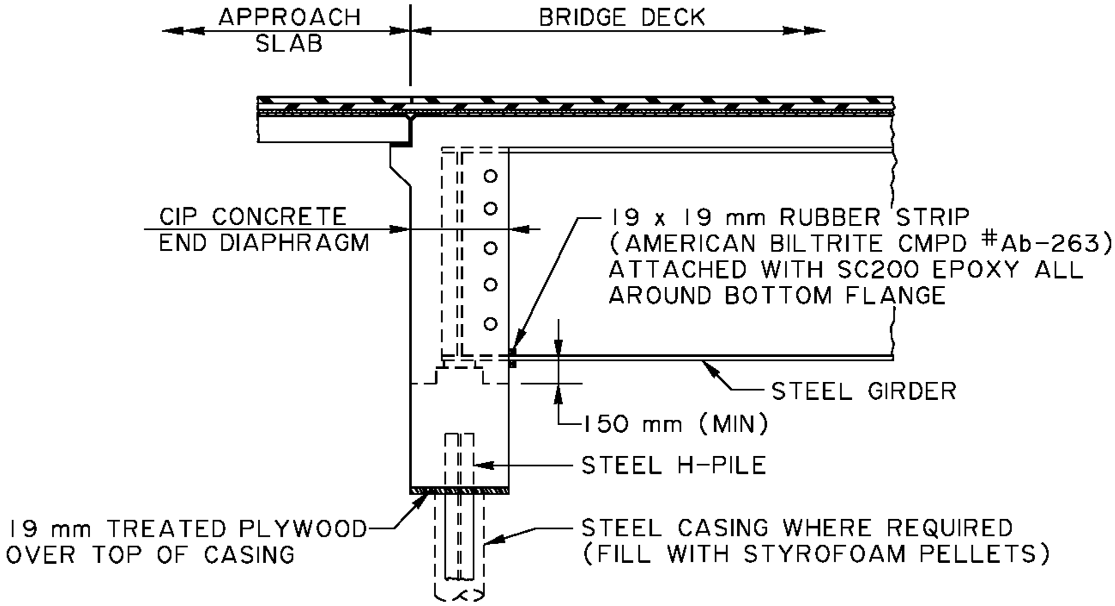


Figure A1: Fully-integral Abutment on Piles with Steel Girders

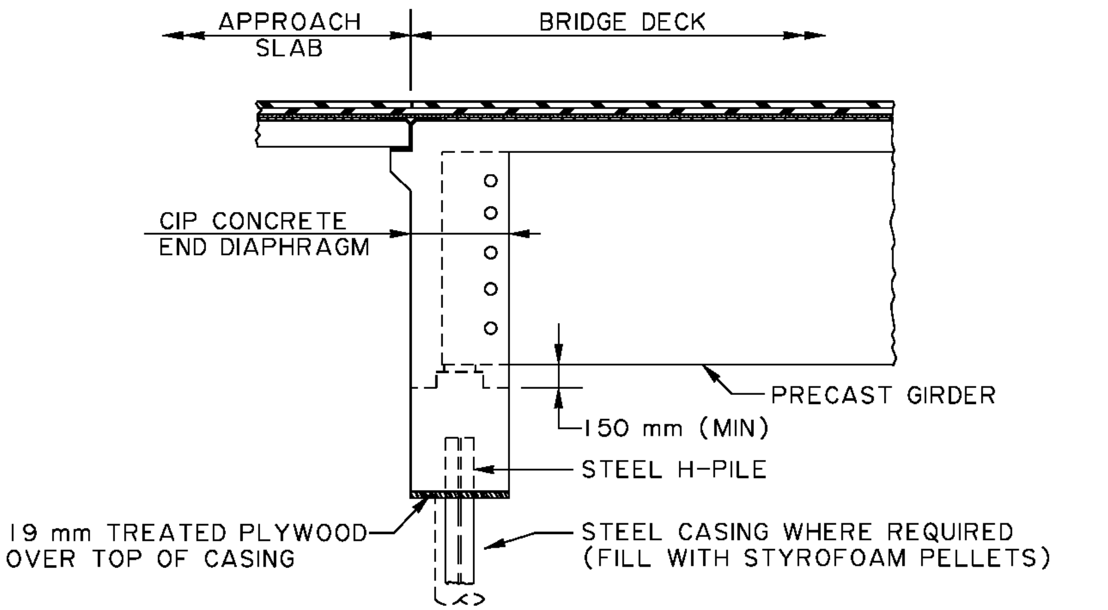


Figure A2: Fully-integral Abutment on Piles with Precast Concrete Girders

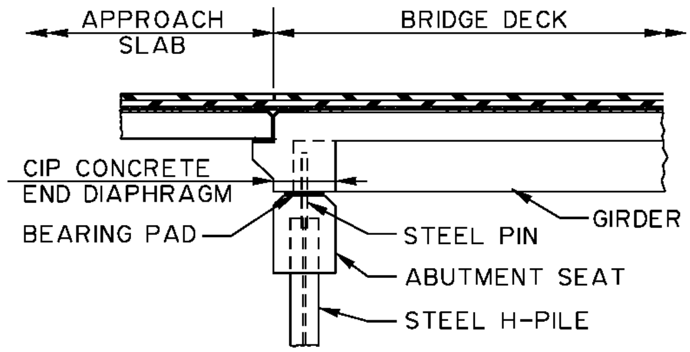


Figure A3: Semi-Integral Abutment with Pinned Connection

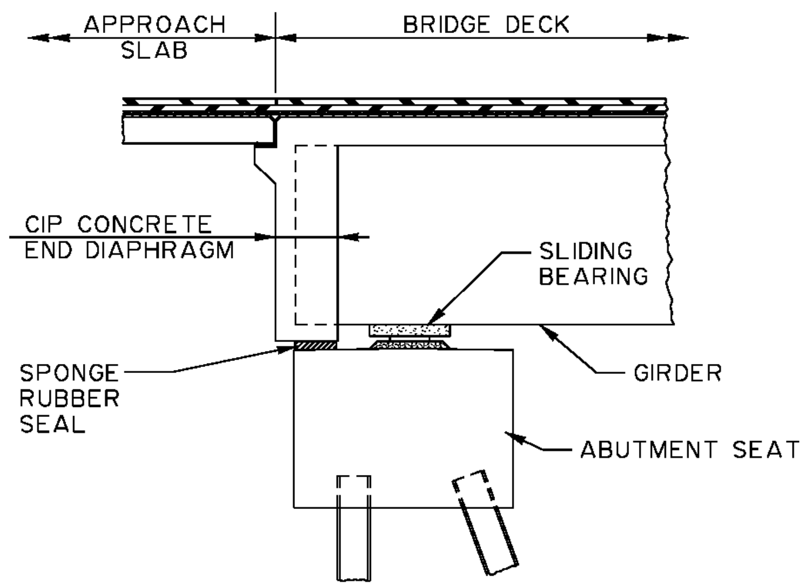


Figure A4: Semi-Integral Abutment with Sliding Bearings

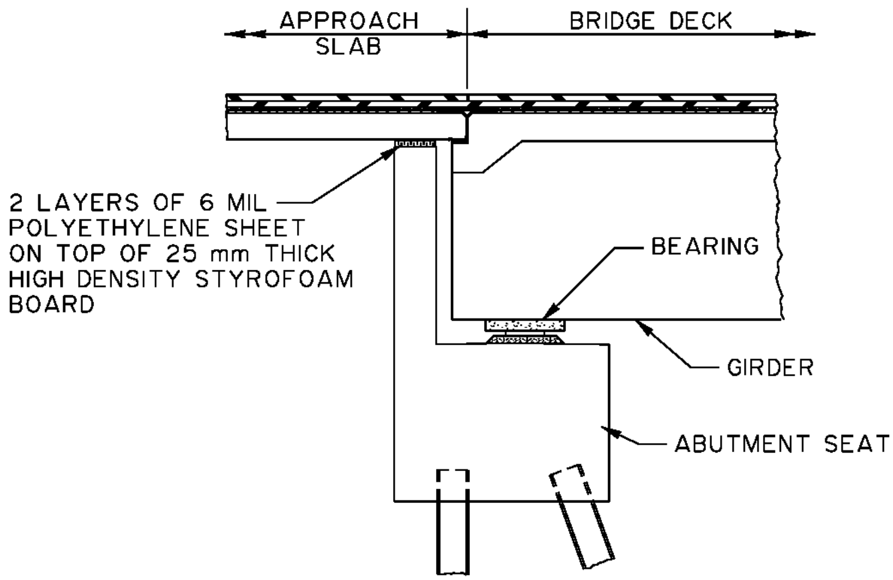


Figure A5: Semi-Integral Abutment with Partial Backwall

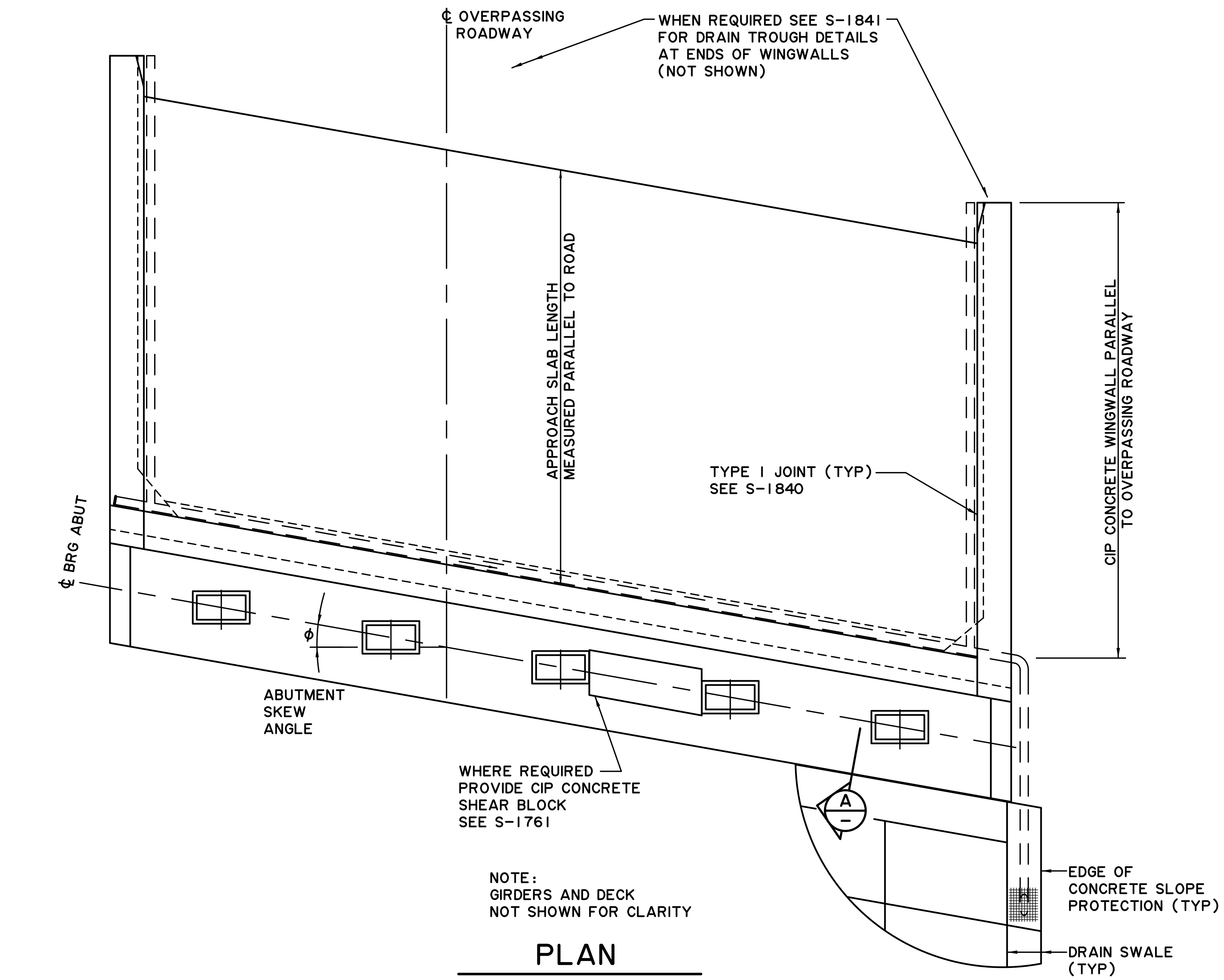
APPENDIX B – TYPICAL ABUTMENT AND EMBANKMENT SKETCHES

B1. Introduction

The sketches included in this Appendix were originally developed as part of the Department's Technical Requirements for P3 projects, but have been modified for conventionally delivered projects. The sketches show typical abutment and embankment details. They are provided for the sole purpose of illustrating selected requirements from the BSDC. They intentionally omit details that might detract from this purpose.

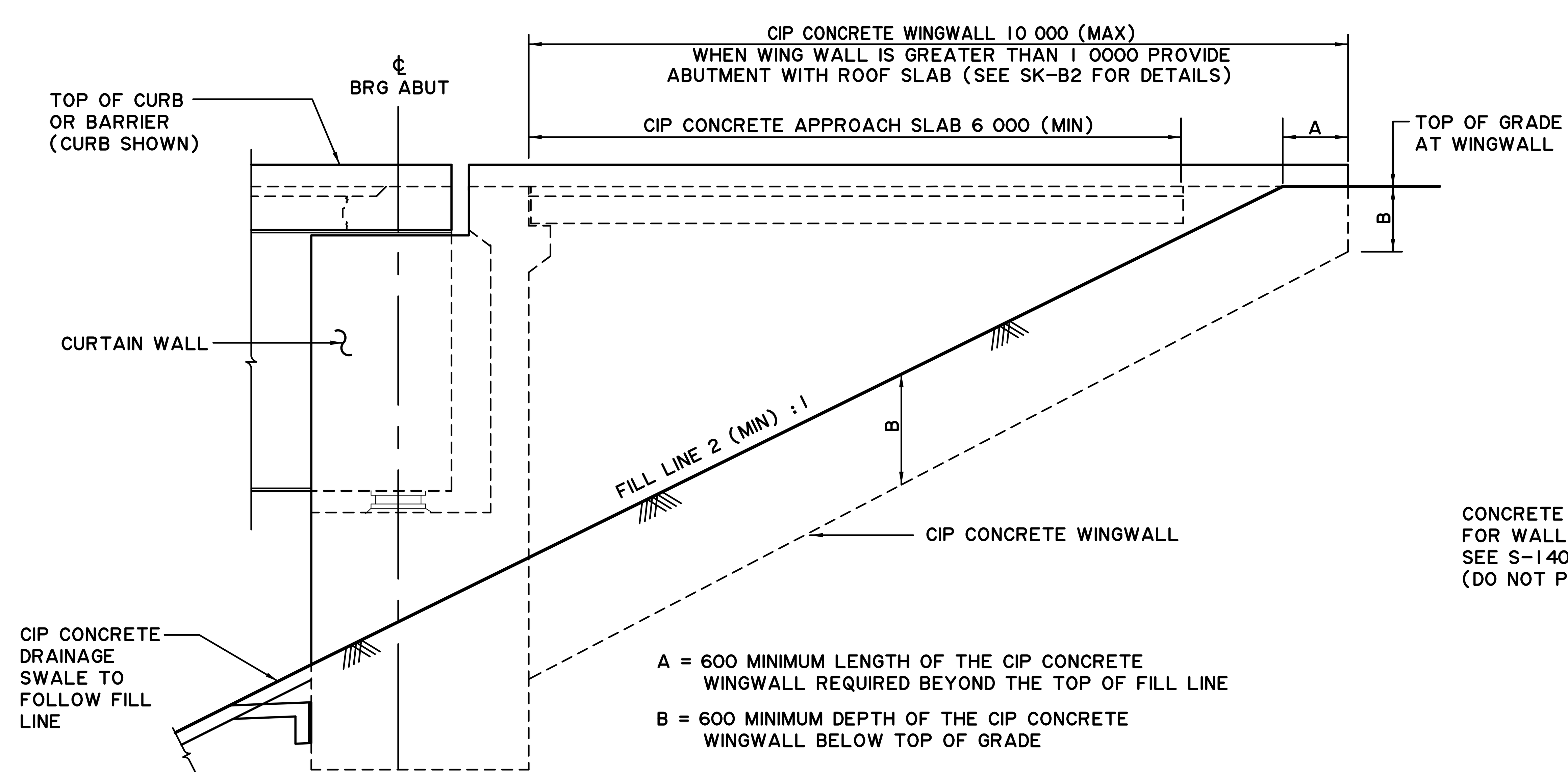
B2. List of Sketches

SK-B1	Standard Details for Conventional Abutment
SK-B2	Standard Details for Conventional Abutment with Roof Slab
SK-B3	Standard Details for Fully-integral Abutment
SK-B4	Standard Details for Semi-integral Abutment
SK-B5	Wall Layout and Site Drainage For Bridge Skew Angles ≤ 45 Degrees
SK-B6	Wall Layout and Site Drainage For Bridge Skew Angles > 45 Degrees
SK-B7	Turned Back Wingwall Details For Bridge Skew Angles > 45 Degrees
SK-B8	Standard Details Associated with MSE Walls
SK-B9	Bridge Approach Geometry



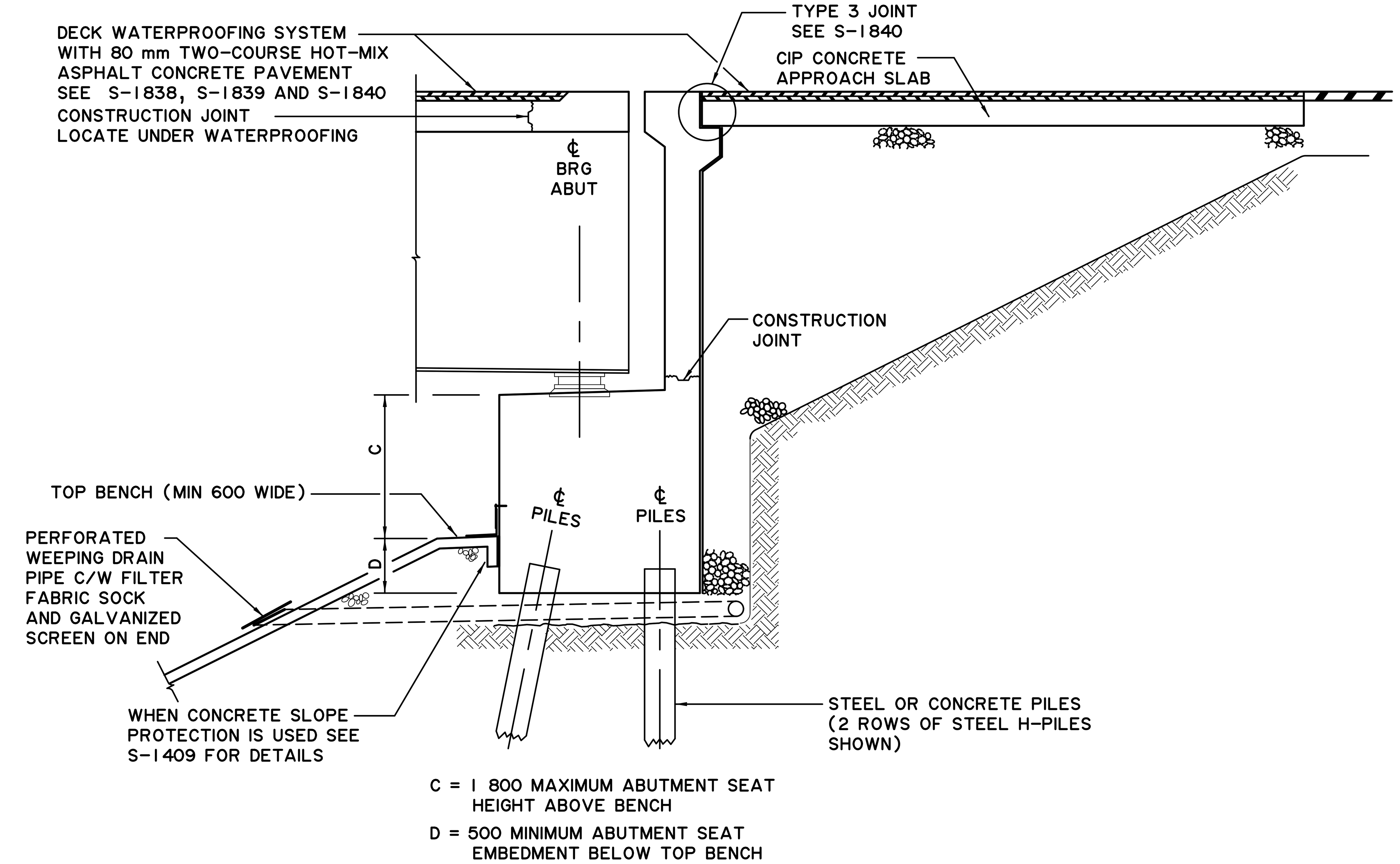
PLAN

(HEADSLOPE OPTION SHOWN)



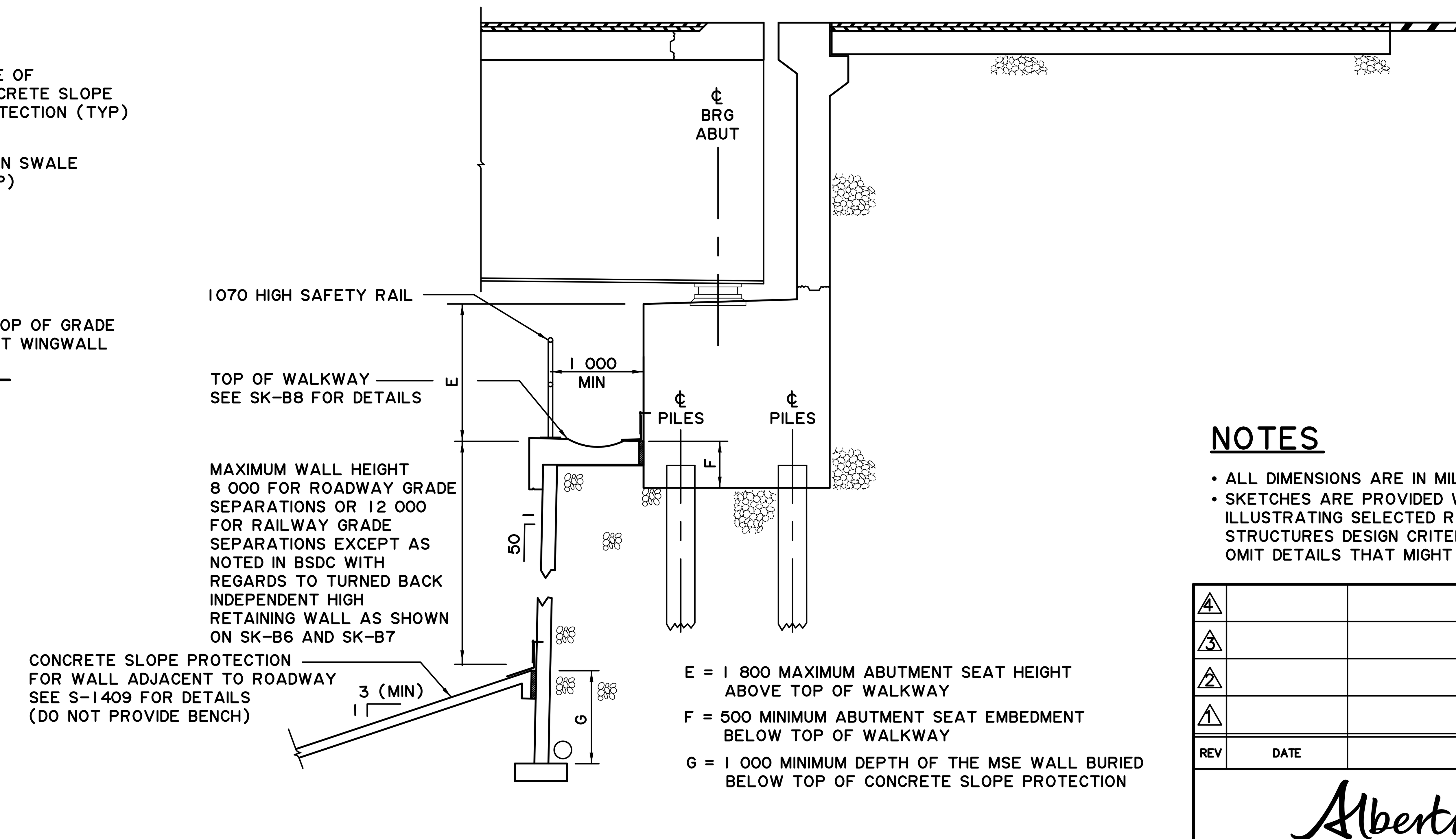
ELEVATION

(HEADSLOPE OPTION SHOWN)



A SECTION

HEADSLOPE OPTION



NOTE: MSE WALL SIMILAR TO HEADSLOPE OPTION EXCEPT AS NOTED

A SECTION

MSE WALL OPTION

NOTES

- ALL DIMENSIONS ARE IN MILLIMETRES
- SKETCHES ARE PROVIDED WITH THE SOLE PURPOSE OF ILLUSTRATING SELECTED REQUIREMENTS FROM THE BRIDGE STRUCTURES DESIGN CRITERIA (BSDC). THEY INTENTIONALLY OMIT DETAILS THAT MIGHT DETRACT FROM THIS PURPOSE

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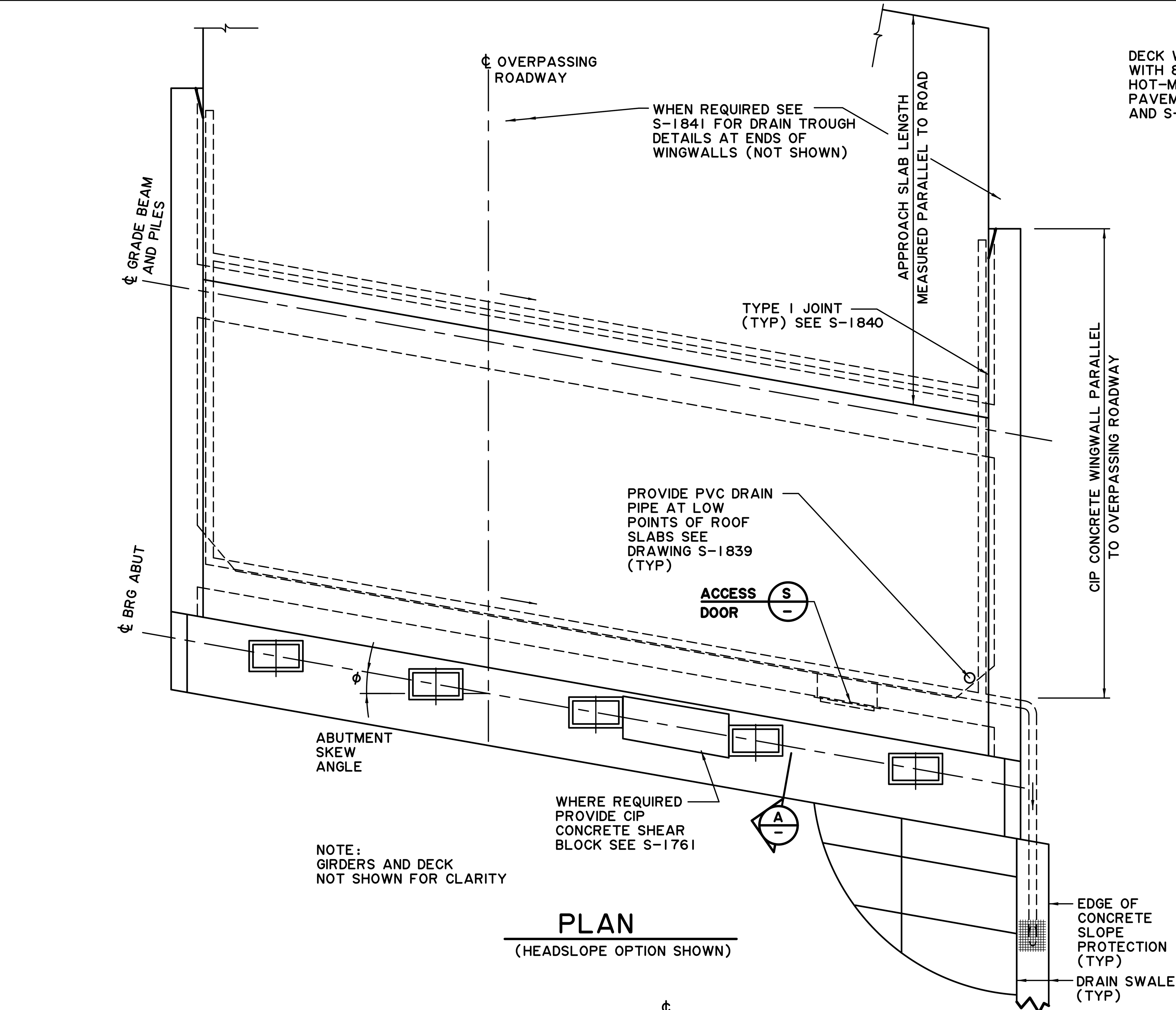
Alberta Transportation

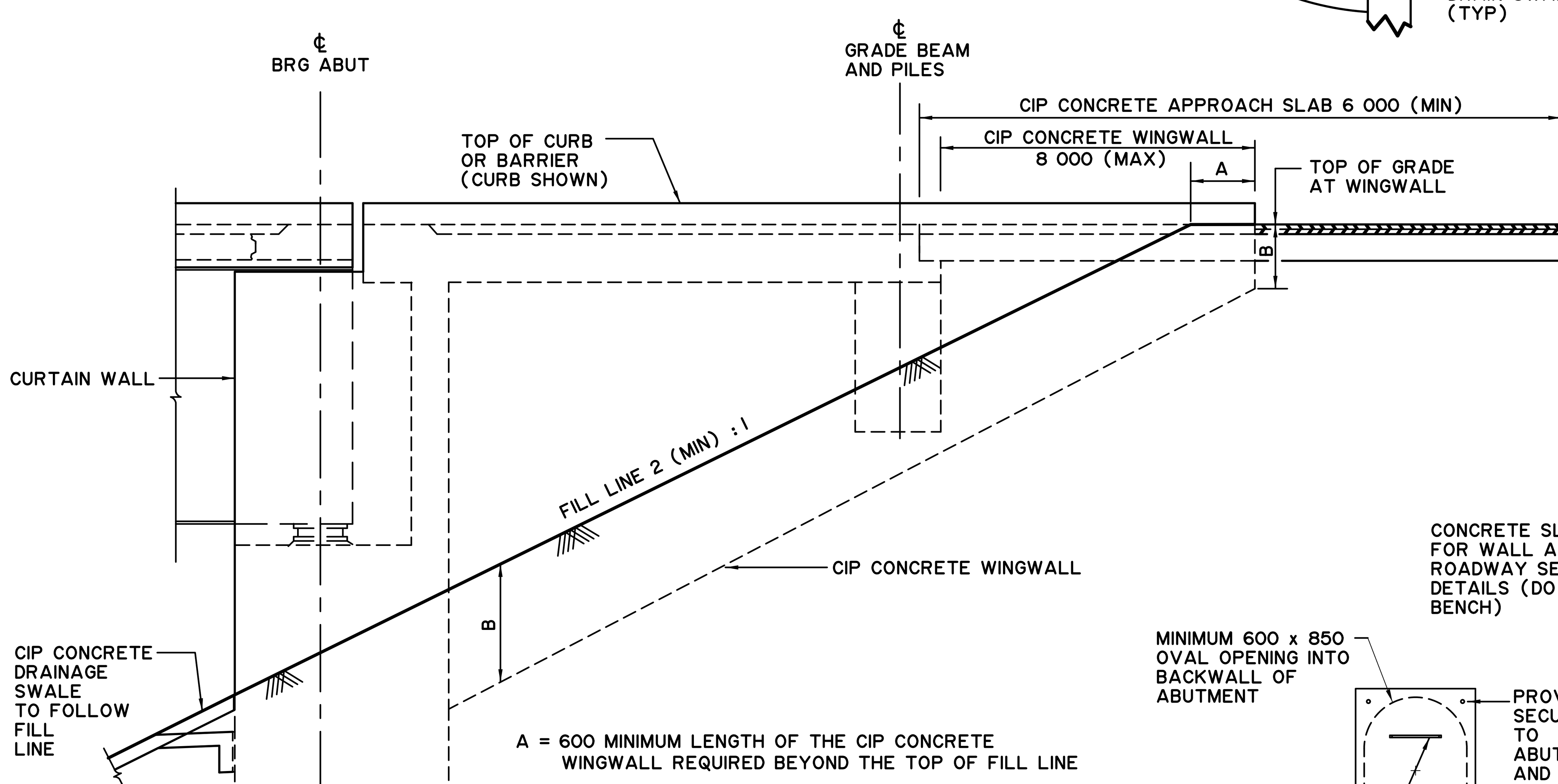
STANDARD DETAILS FOR CONVENTIONAL ABUTMENT

FILE NUMBER	DATE DRAWN	SHEET	DRAWING
–	2017-04-04	1 OF 9	SK-B1



PLAN

(HEADSLOPE OPTION SHOWN)

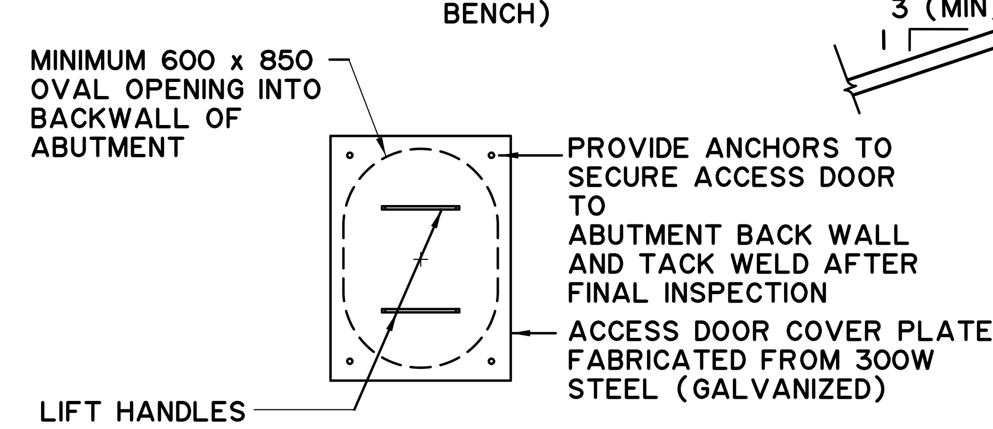


A = 600 MINIMUM LENGTH OF THE CIP CONCRETE WINGWALL REQUIRED BEYOND THE TOP OF FILL LINE

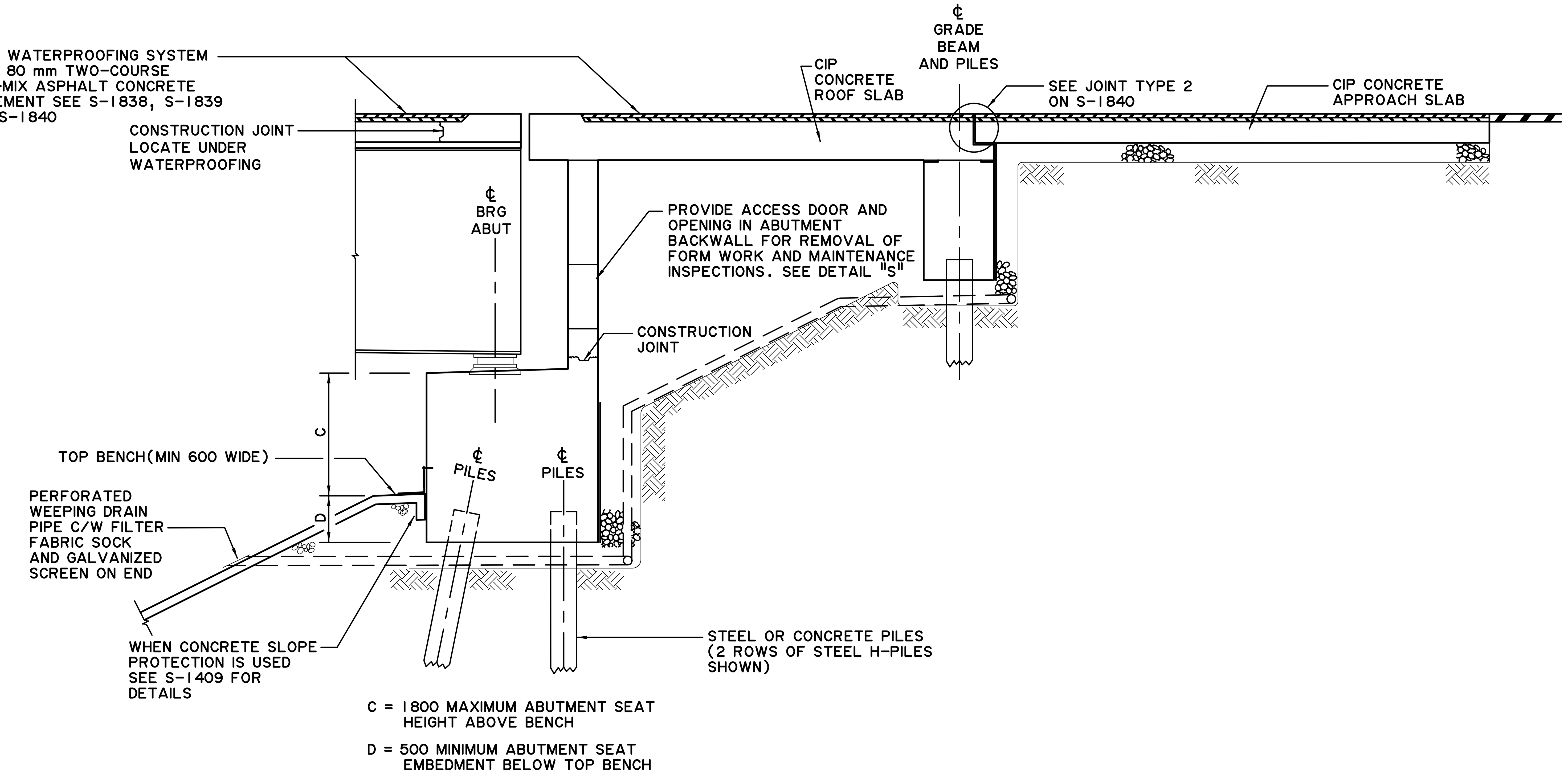
B = 600 MINIMUM DEPTH OF THE CIP CONCRETE WINGWALL BELOW TOP OF GRADE

ELEVATION

(HEADSLOPE OPTION SHOWN)



S ACCESS DOOR

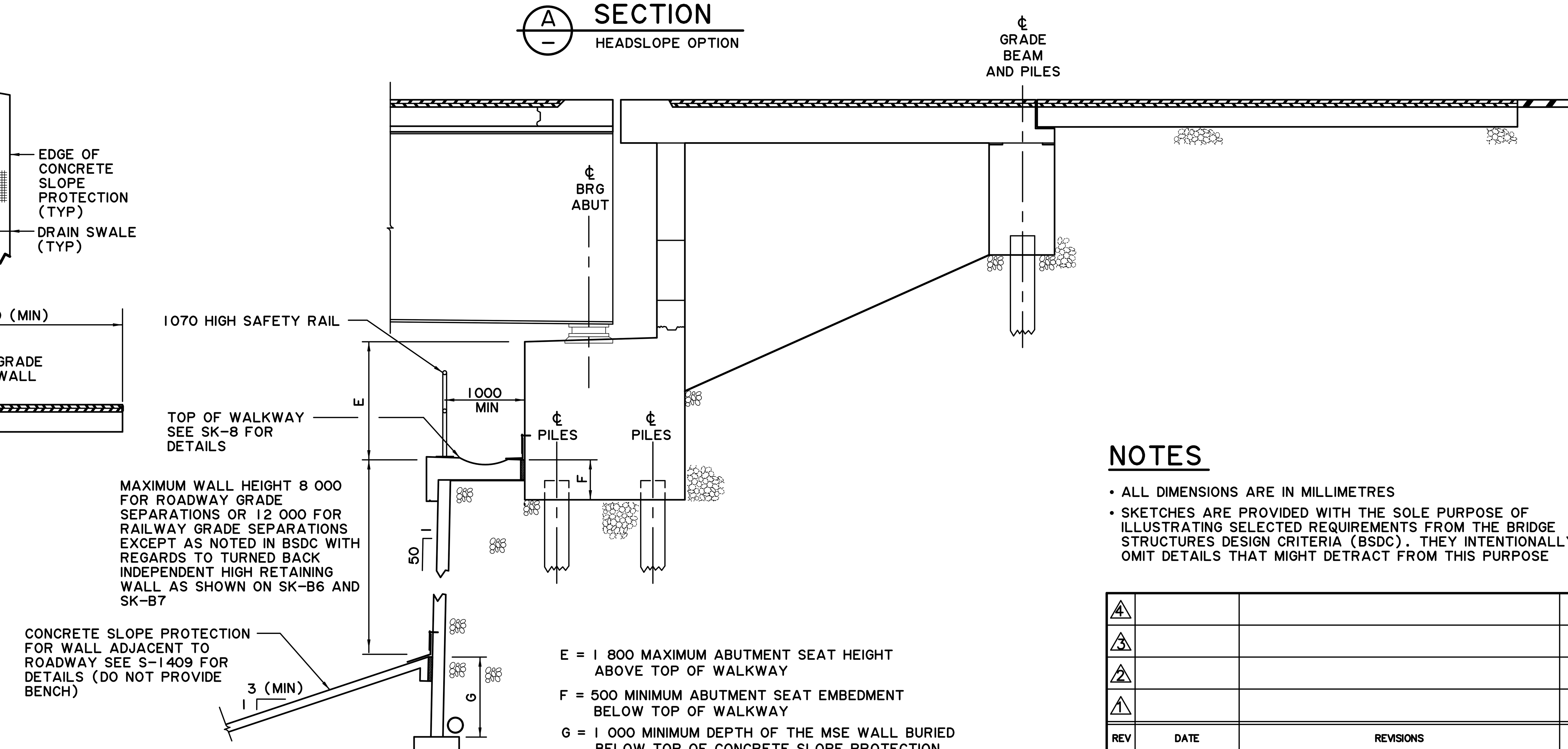


C = 1800 MAXIMUM ABUTMENT SEAT HEIGHT ABOVE BENCH

D = 500 MINIMUM ABUTMENT SEAT EMBEDMENT BELOW TOP BENCH

A SECTION

HEADSLOPE OPTION



E = 1 800 MAXIMUM ABUTMENT SEAT HEIGHT ABOVE TOP OF WALKWAY

F = 500 MINIMUM ABUTMENT SEAT EMBEDMENT BELOW TOP OF WALKWAY

G = 1 000 MINIMUM DEPTH OF THE MSE WALL BURIED BELOW TOP OF CONCRETE SLOPE PROTECTION

NOTE: MSE WALL SIMILAR TO HEADSLOPE OPTION EXCEPT AS NOTED

A SECTION

MSE WALL OPTION

NOTES

- ALL DIMENSIONS ARE IN MILLIMETRES
- SKETCHES ARE PROVIDED WITH THE SOLE PURPOSE OF ILLUSTRATING SELECTED REQUIREMENTS FROM THE BRIDGE STRUCTURES DESIGN CRITERIA (BSDC). THEY INTENTIONALLY OMIT DETAILS THAT MIGHT DETRACT FROM THIS PURPOSE

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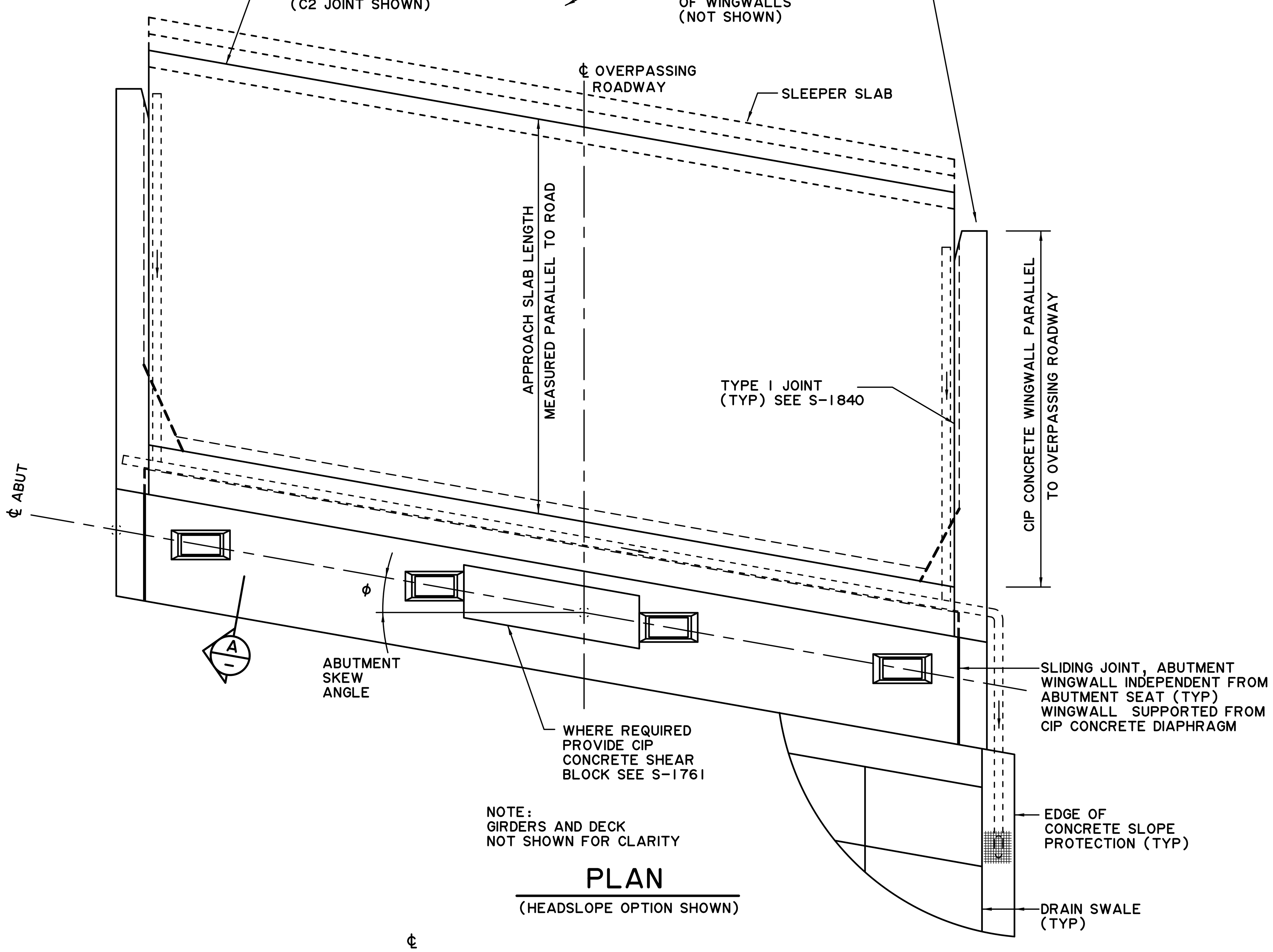
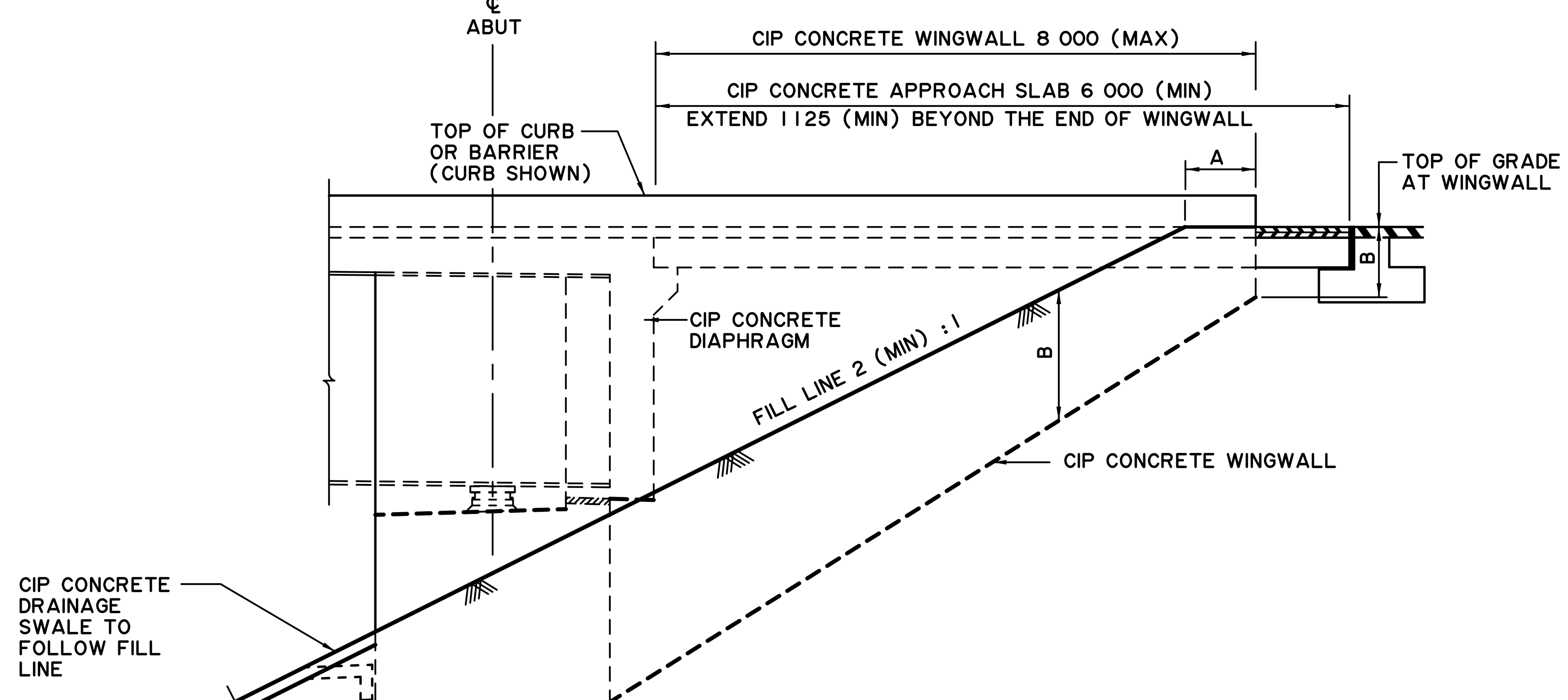
STANDARD DETAILS FOR CONVENTIONAL ABUTMENT WITH ROOF SLAB

FILE NUMBER	DATE DRAWN	SHEET	DRAWING
–	2017-04-04	2 OF 9	SK-B2



PLAN

(HEADSLOPE OPTION SHOWN)

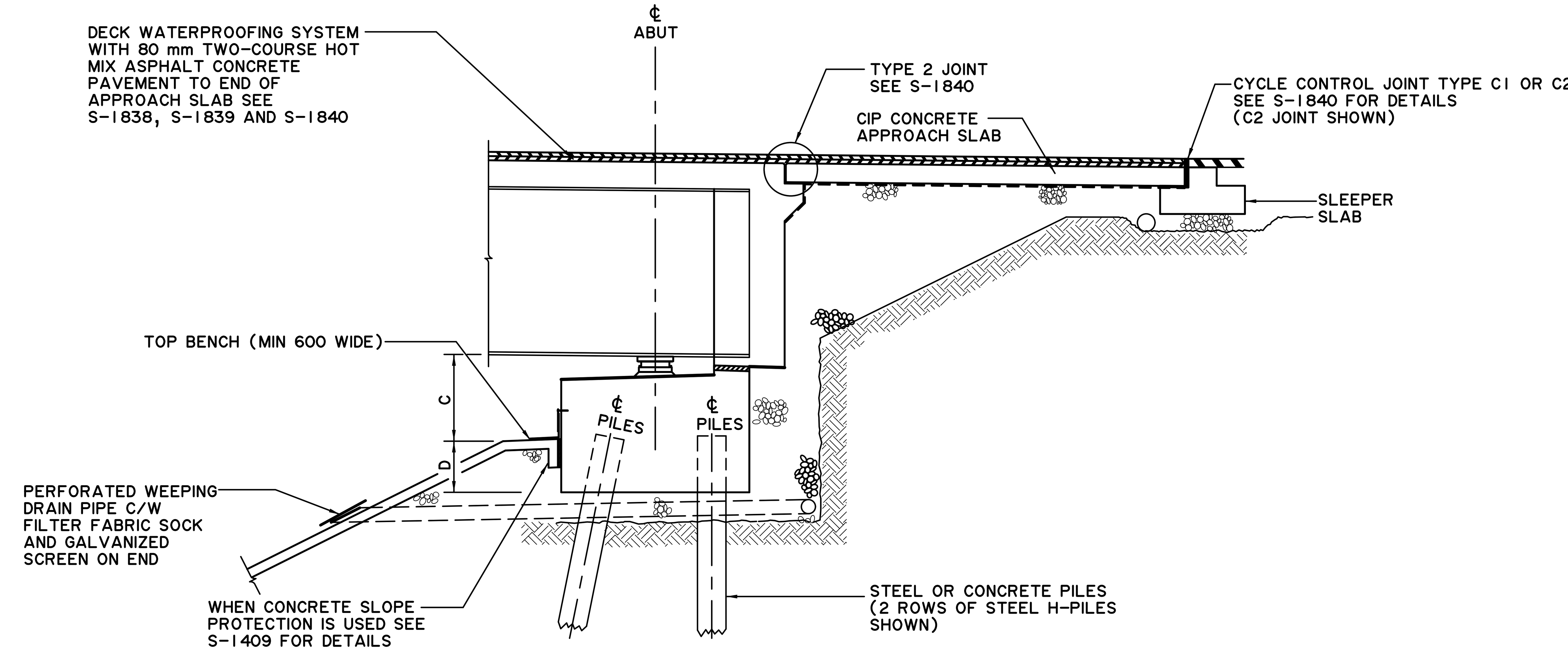


A = 600 MINIMUM LENGTH OF THE CIP CONCRETE WINGWALL REQUIRED BEYOND THE TOP OF FILL LINE

B = 600 MINIMUM DEPTH OF THE CIP CONCRETE WINGWALL BELOW TOP OF GRADE

ELEVATION

(HEADSLOPE OPTION SHOWN)

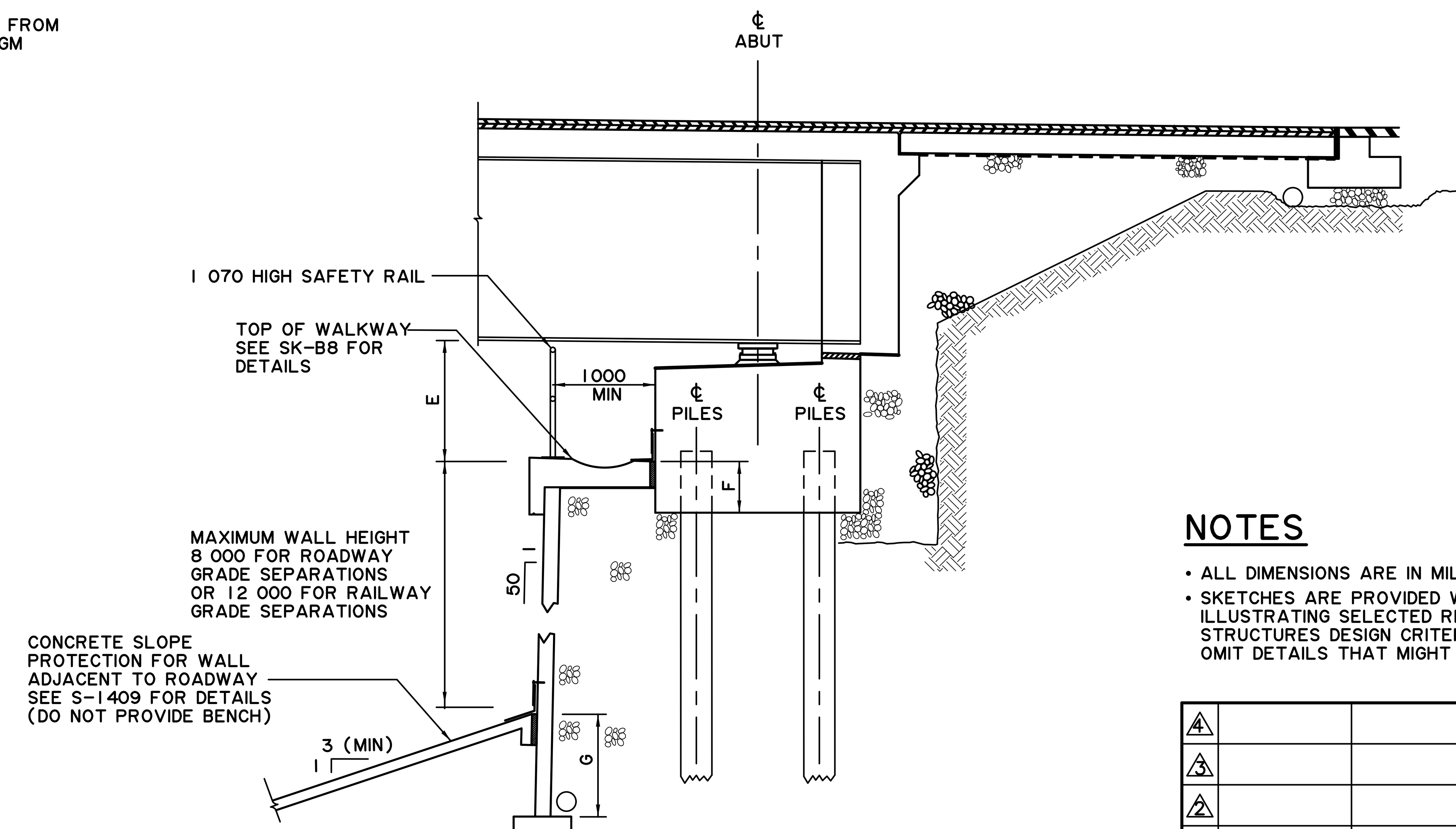


C = 1500 MAXIMUM VERTICAL DISTANCE BETWEEN UNDERSIDE OF GIRDER AND TOP OF BENCH

D = 500 MINIMUM ABUTMENT SEAT EMBEDMENT BELOW TOP OF BENCH

A — SECTION

HEAD SLOPE OPTION



E = 1 500 MAXIMUM VERTICAL DISTANCE BETWEEN UNDERSIDE OF GIRDER AND TOP OF WALKWAY

F = 500 MINIMUM ABUTMENT SEAT EMBEDMENT BELOW TOP OF WALKWAY

G = 1 000 MINIMUM DEPTH OF THE MSE WALL BURIED BELOW TOP OF CONCRETE SLOPE PROTECTION

NOTE: MSE WALL SIMILAR TO HEADSLOPE OPTION EXCEPT AS NOTED

A — SECTION

MSE WALL OPTION

NOTES

- ALL DIMENSIONS ARE IN MILLIMETRES
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4			
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2			
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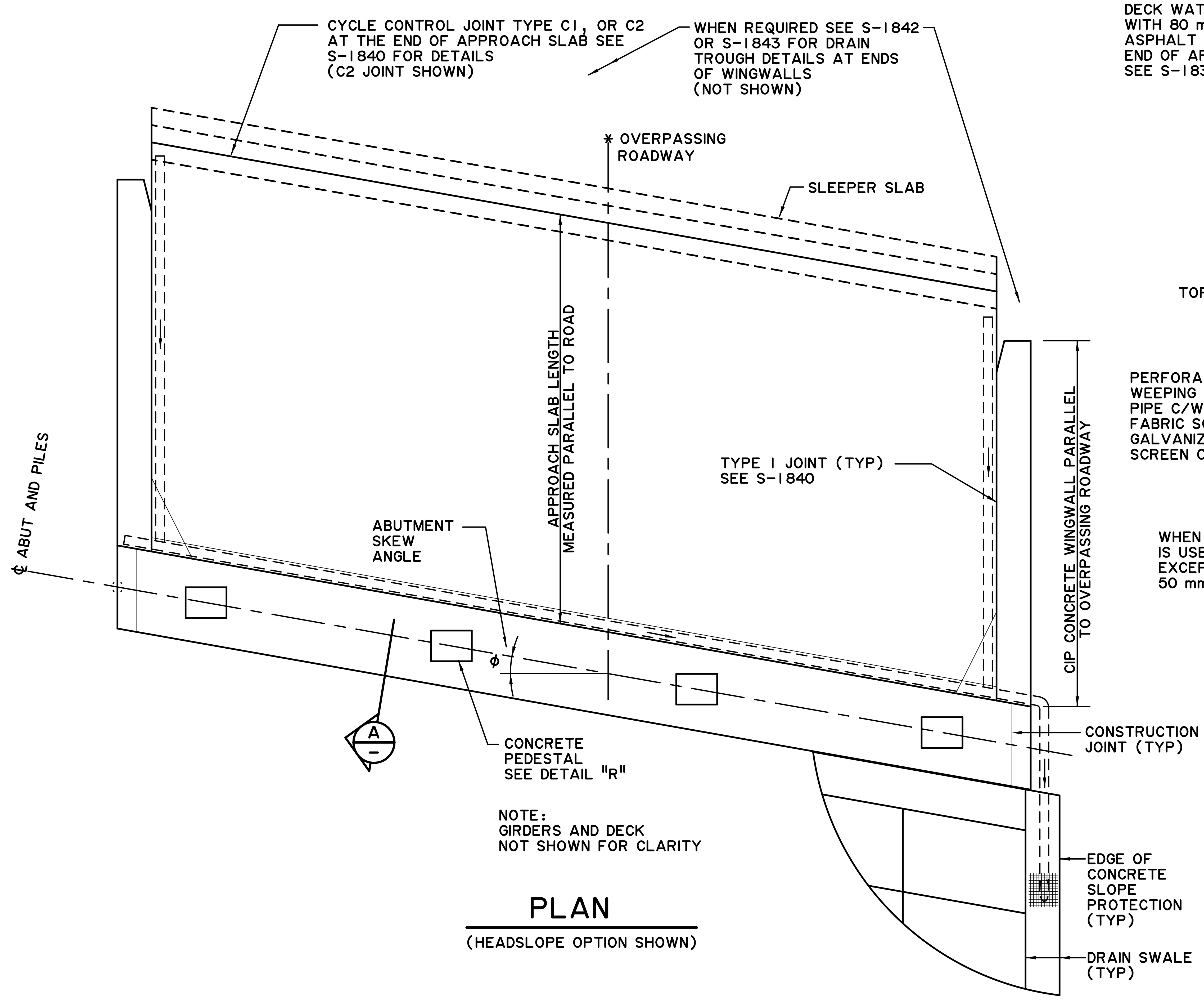
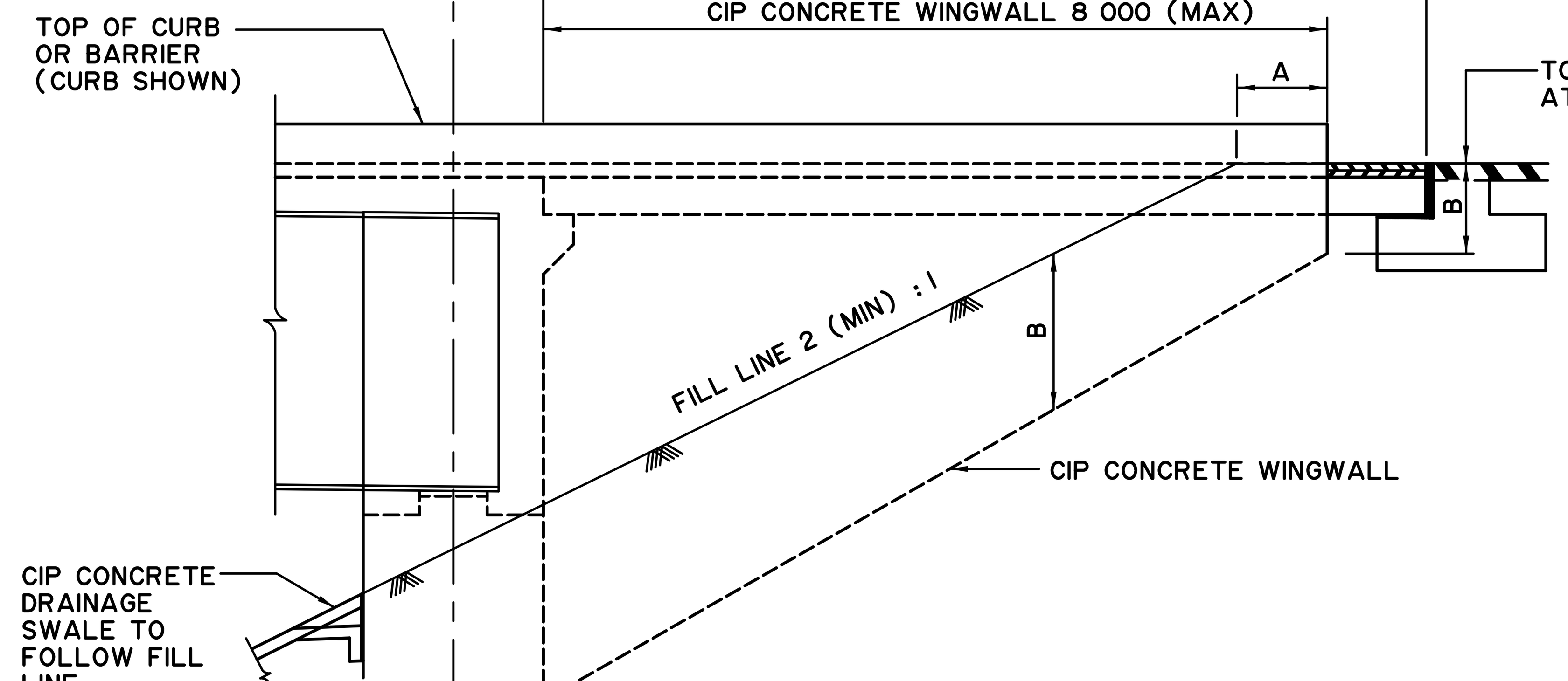
STANDARD DETAILS FOR SEMI-INTEGRAL ABUTMENT

FILE NUMBER	DATE DRAWN	SHEET	DRAWING
–	2017-04-04	3 of 9	SK-B3



PLAN

(HEADSLOPE OPTION SHOWN)

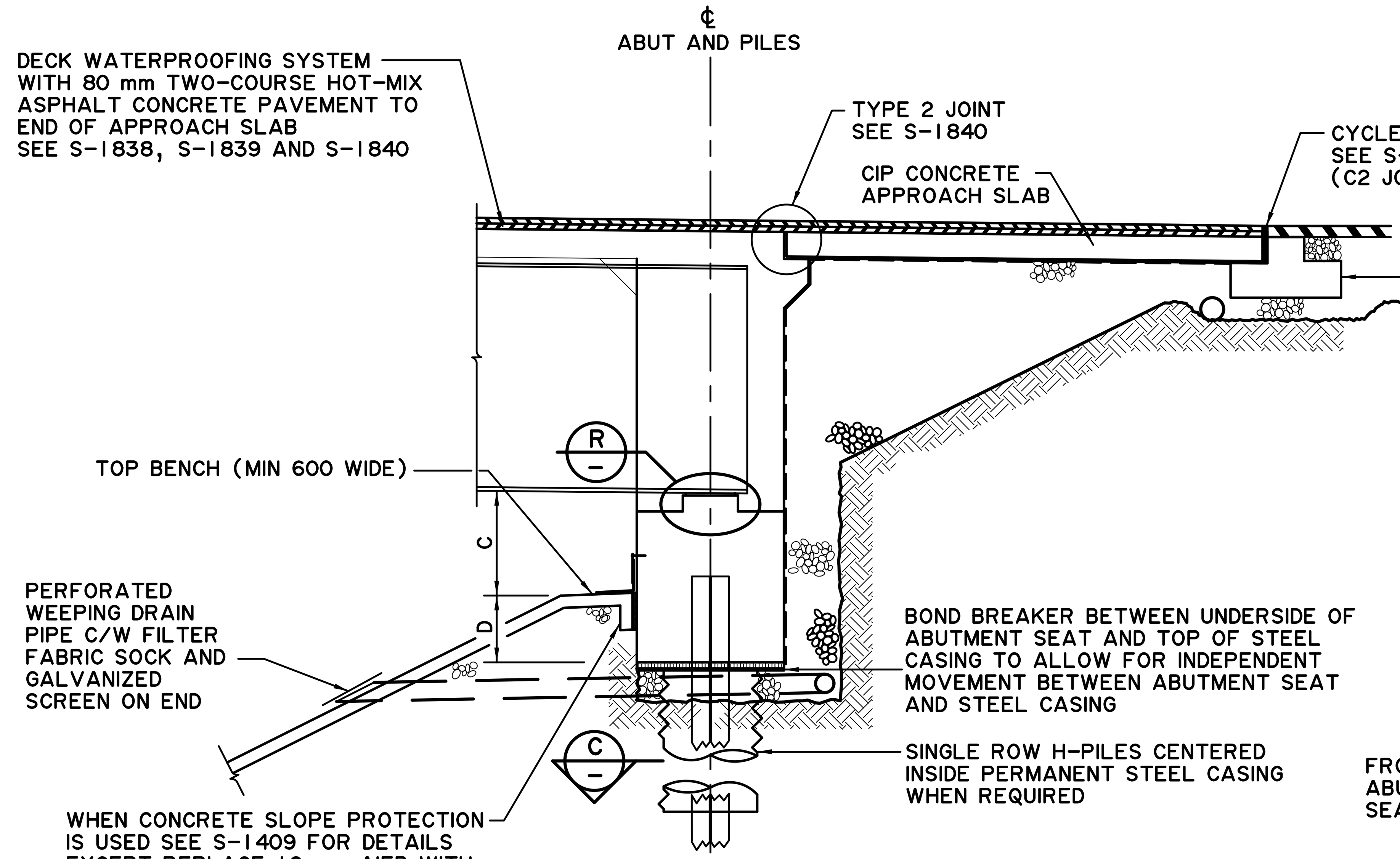


A = 600 MINIMUM LENGTH OF THE CIP CONCRETE WINGWALL REQUIRED BEYOND THE TOP OF FILL LINE

B = 600 MINIMUM DEPTH OF THE CIP CONCRETE WINGWALL BELOW TOP OF GRADE

ELEVATION

(HEADSLOPE OPTION SHOWN)

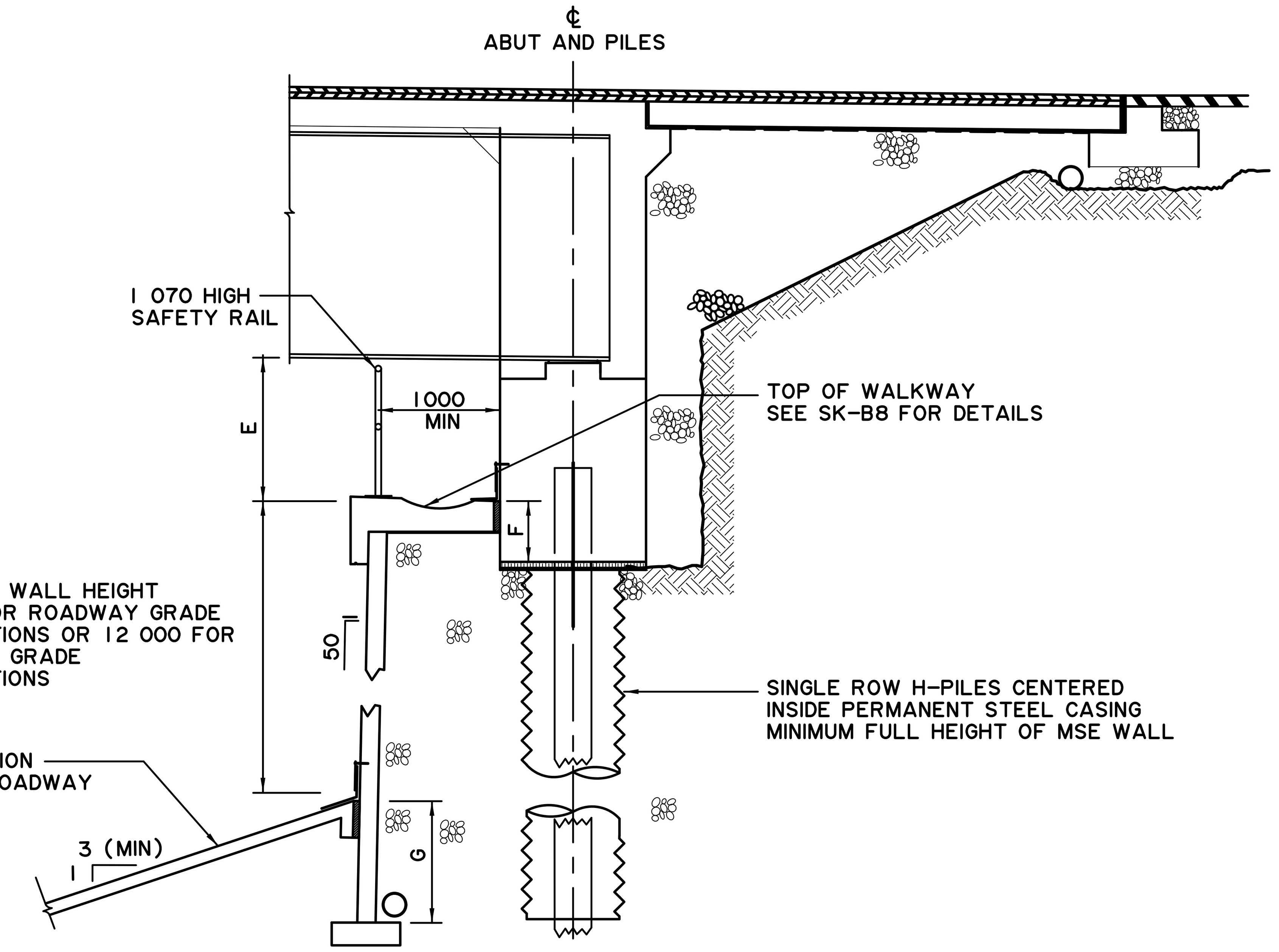


C = 1 500 MAXIMUM VERTICAL DISTANCE BETWEEN UNDERSIDE OF GIRDER AND TOP OF BENCH

D = 500 MINIMUM ABUTMENT SEAT EMBEDMENT BELOW TOP OF BENCH

(A) SECTION

HEADSLOPE OPTION



E = 1 500 MAXIMUM VERTICAL DISTANCE BETWEEN UNDERSIDE OF GIRDER AND TOP OF WALKWAY

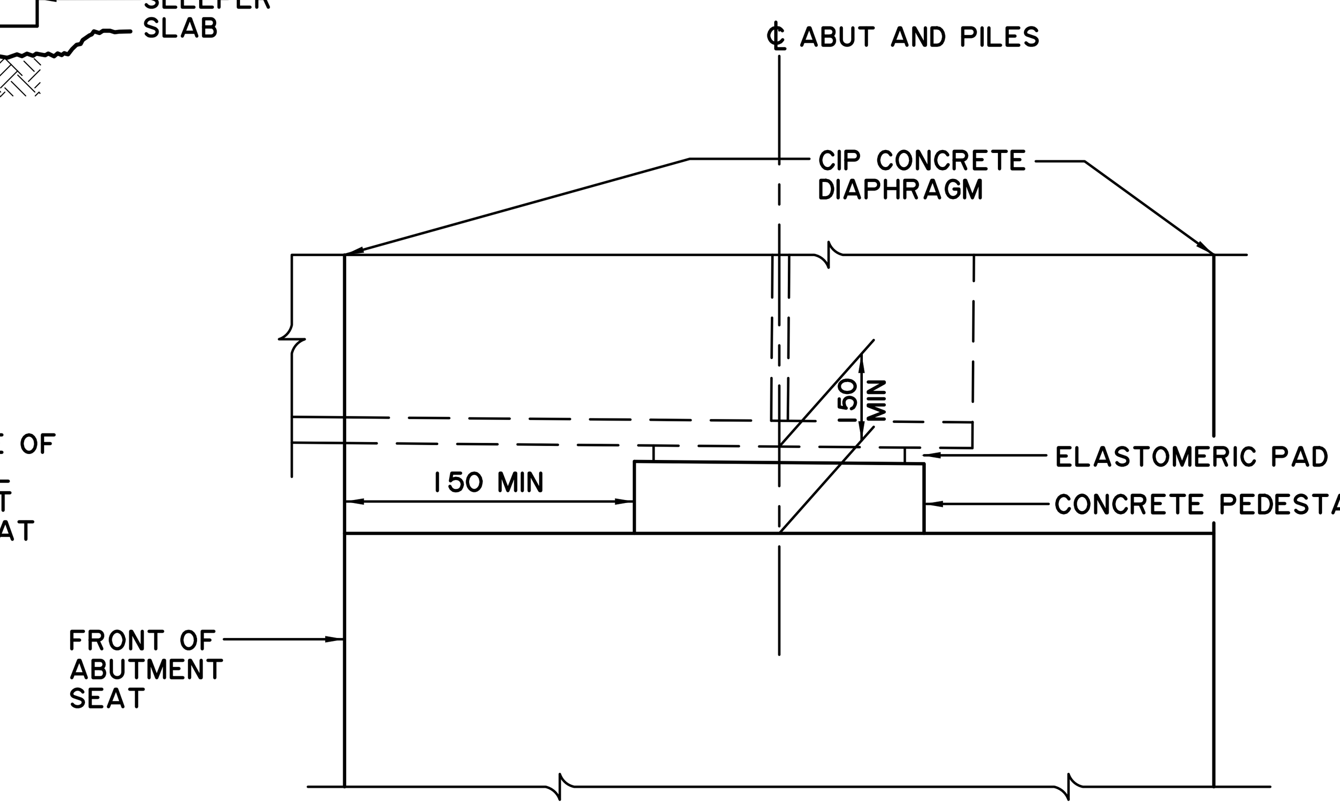
F = 500 MINIMUM ABUTMENT SEAT EMBEDMENT BELOW TOP OF WALKWAY

G = 1 000 MINIMUM DEPTH OF THE MSE WALL BURIED BELOW TOP OF CONCRETE SLOPE PROTECTION

NOTE: MSE WALL SIMILAR TO HEADSLOPE OPTION EXCEPT AS NOTED

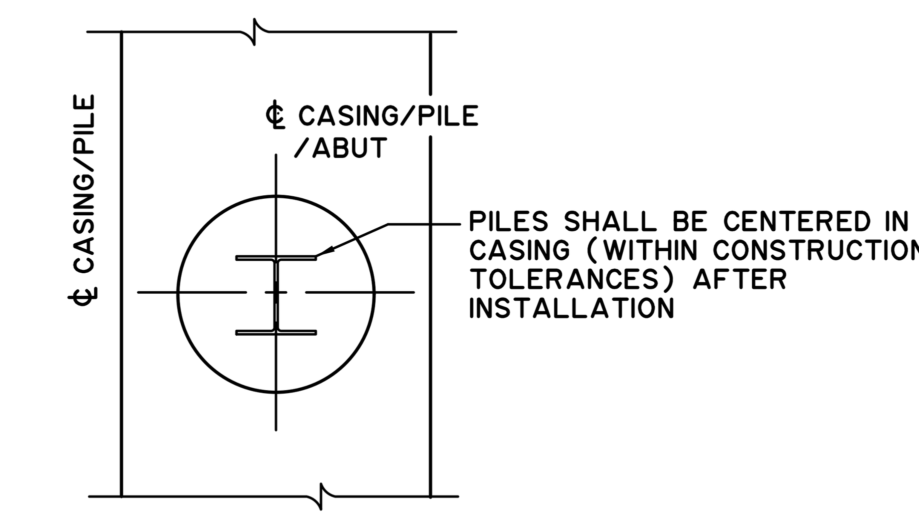
(A) SECTION

MSE WALL OPTION



(R) PEDESTAL DETAIL

NTS



(C) PILE POSITION INSIDE CASING

NTS

NOTES

- ALL DIMENSIONS ARE IN MILLIMETRES
- SKETCHES ARE PROVIDED WITH THE SOLE PURPOSE OF ILLUSTRATING SELECTED REQUIREMENTS FROM THE BRIDGE STRUCTURES DESIGN CRITERIA (BSDC). THEY INTENTIONALLY OMIT DETAILS THAT MIGHT DETRACT FROM THIS PURPOSE

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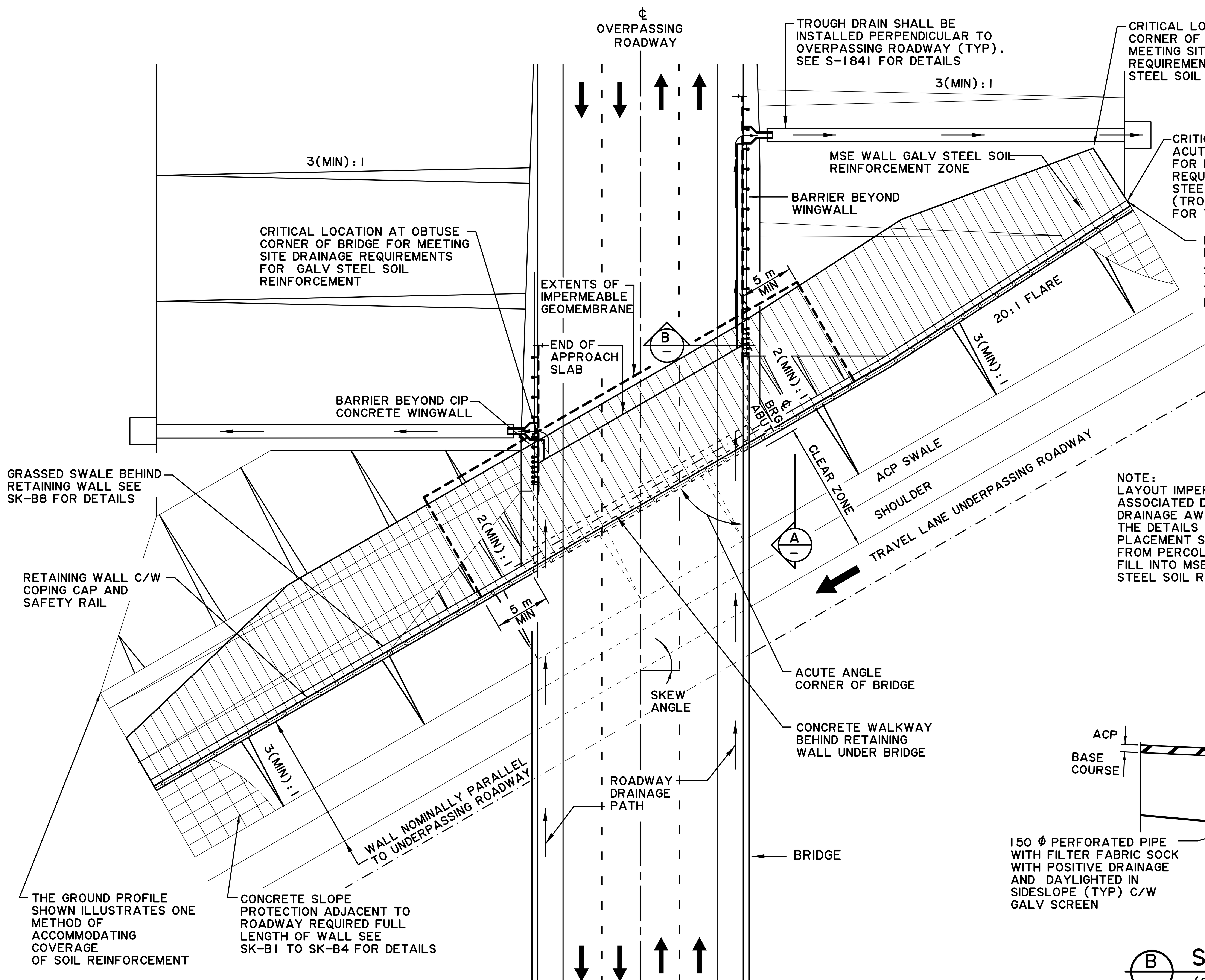
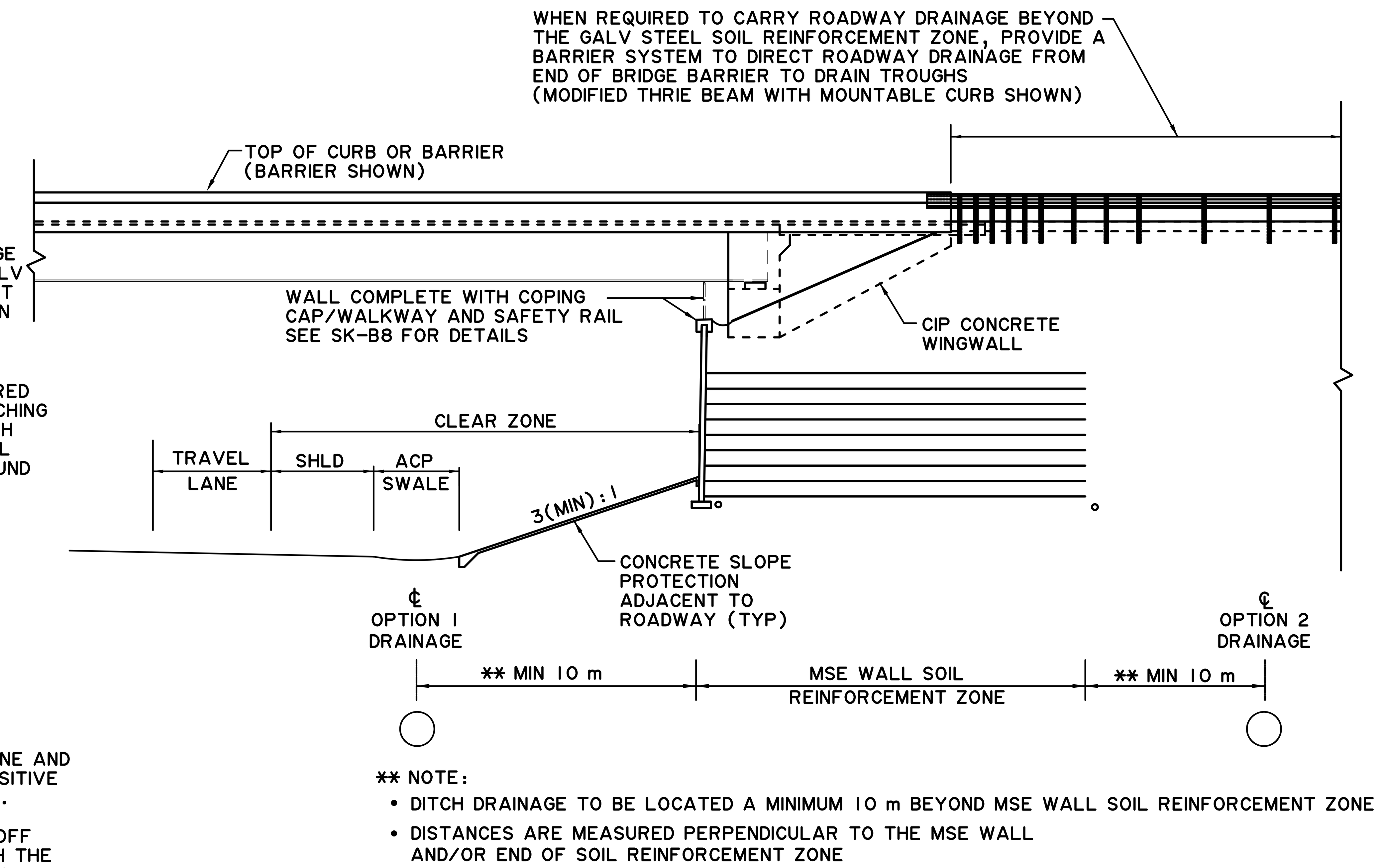
STANDARD DETAILS FOR FULLY-INTEGRAL ABUTMENT

FILE NUMBER	DATE DRAWN	SHEET	DRAWING
-	2017-04-04	4 OF 9	SK-B4



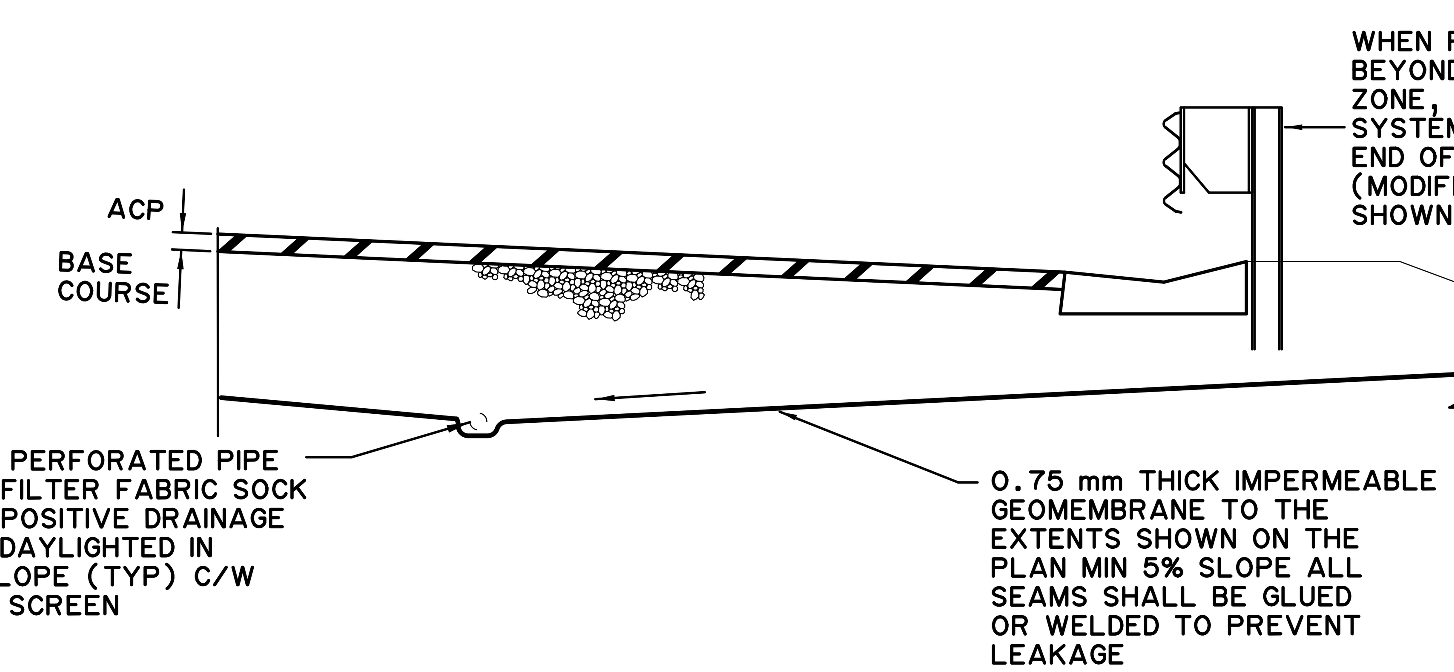
PLAN

NOTE: DITCH DRAINAGE AROUND INDEPENDENT WALLS NOT SHOWN FOR CLARITY



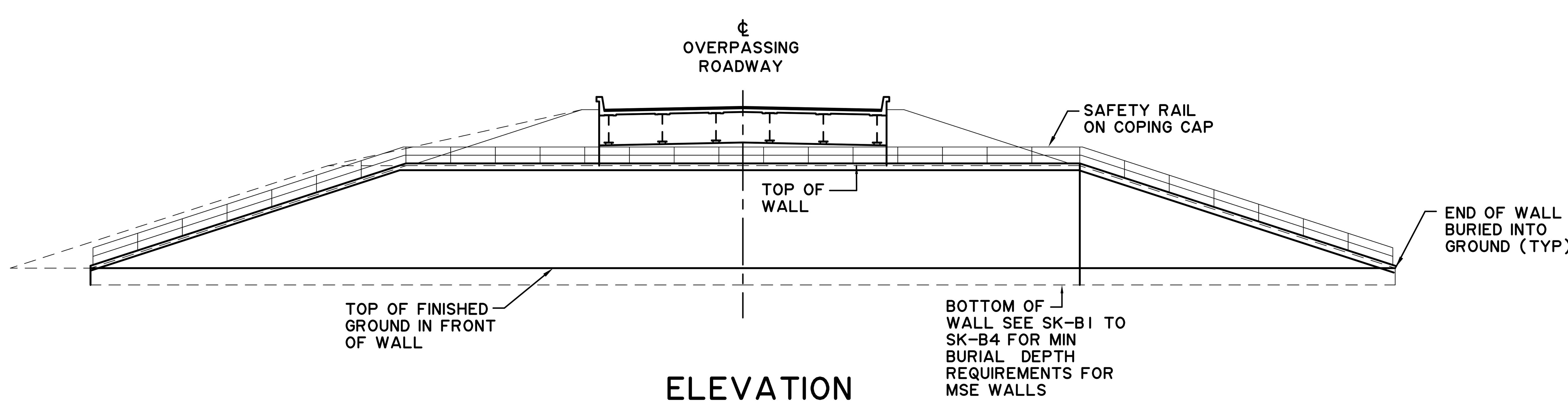
** NOTE:
- DITCH DRAINAGE TO BE LOCATED A MINIMUM 10 m BEYOND MSE WALL SOIL REINFORCEMENT ZONE
- DISTANCES ARE MEASURED PERPENDICULAR TO THE MSE WALL AND/OR END OF SOIL REINFORCEMENT ZONE

A SECTION



B SECTION

(SHOWING GEOMEMBRANE REQUIREMENT OVER GALVANIZED STEEL SOIL REINFORCEMENT)



ELEVATION

NOTES

- SKETCHES ARE PROVIDED WITH THE SOLE PURPOSE OF ILLUSTRATING SELECTED REQUIREMENTS FROM THE BRIDGE STRUCTURES DESIGN CRITERIA (BSDC). THEY INTENTIONALLY OMIT DETAILS THAT MIGHT DETRACT FROM THIS PURPOSE
- SKETCH ILLUSTRATES WALL LAYOUT TO BE USED FOR SKEW ANGLES LESS THAN OR EQUAL TO 45°
- FOR WALL LAYOUT TO BE USED FOR SKEW ANGLES GREATER THAN 45° SEE DRAWING SK-B6 AND SK-B7
- ROADWAY DRAINAGE TO BE DIRECTED PAST END OF WALL AND END OF STEEL SOIL REINFORCEMENT
- DRAIN TROUGHS SHALL NOT DRAIN ACROSS SLOPE

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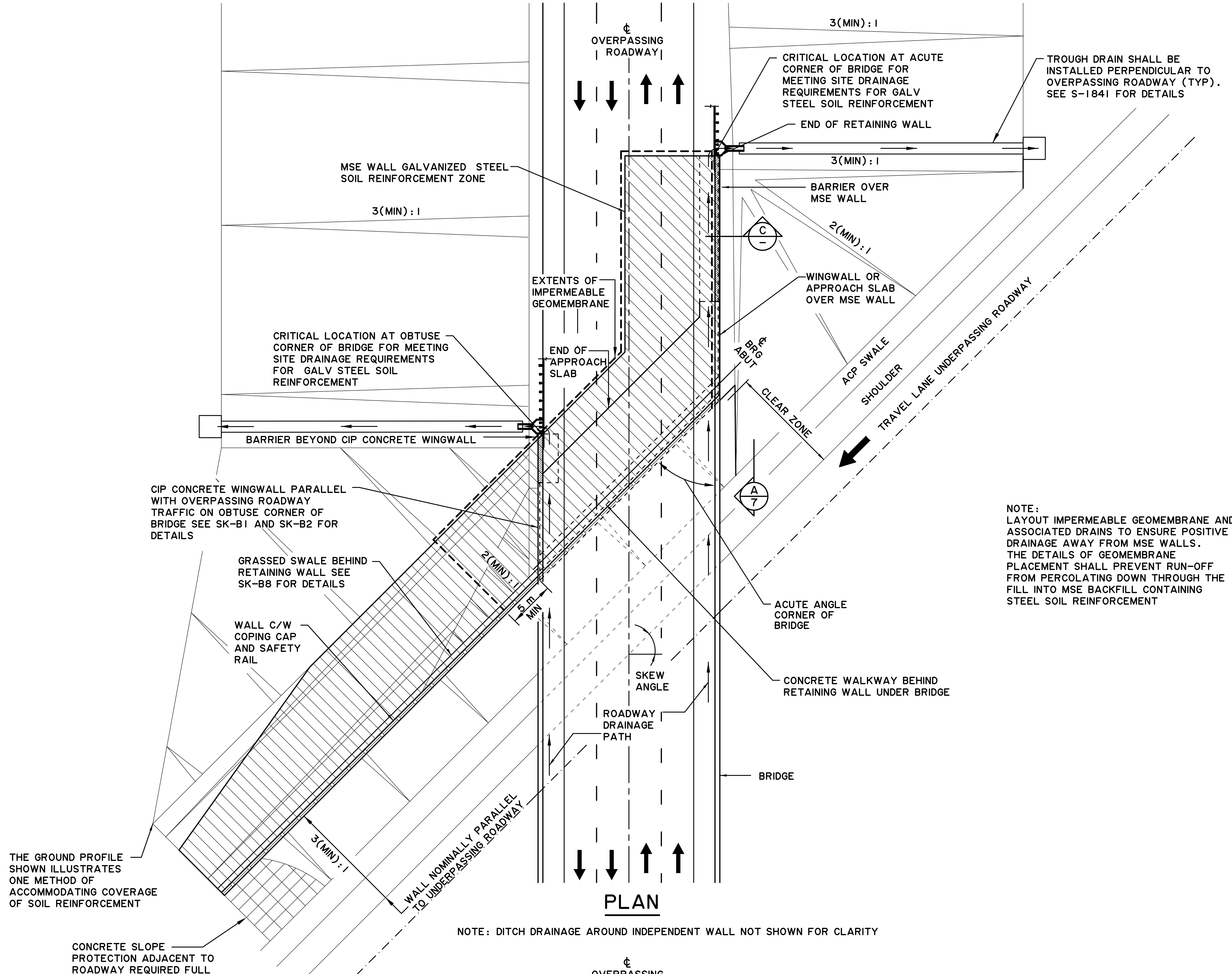
WALL LAYOUT AND SITE DRAINAGE FOR SKEW ANGLES ≤ 45°

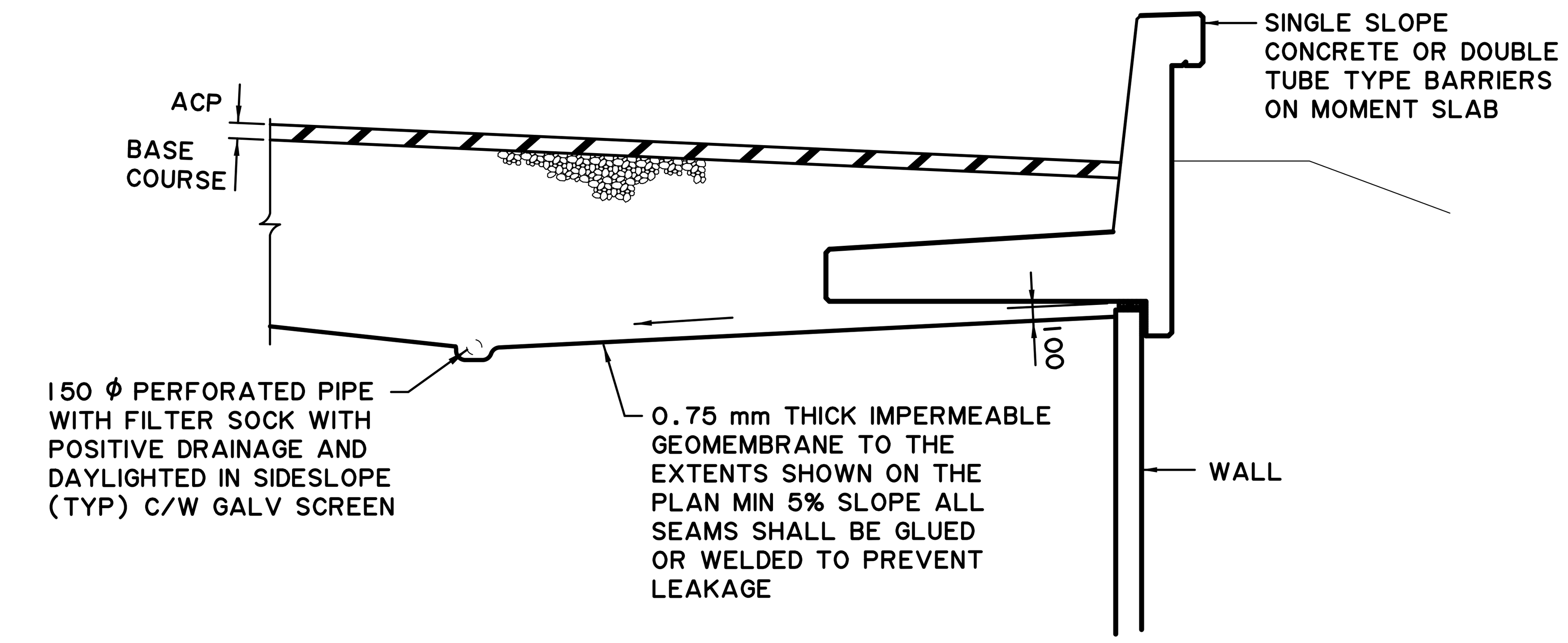
FILE NUMBER	DATE DRAWN	SHEET	DRAWING
–	2017-04-04	5 OF 9	SK-B5



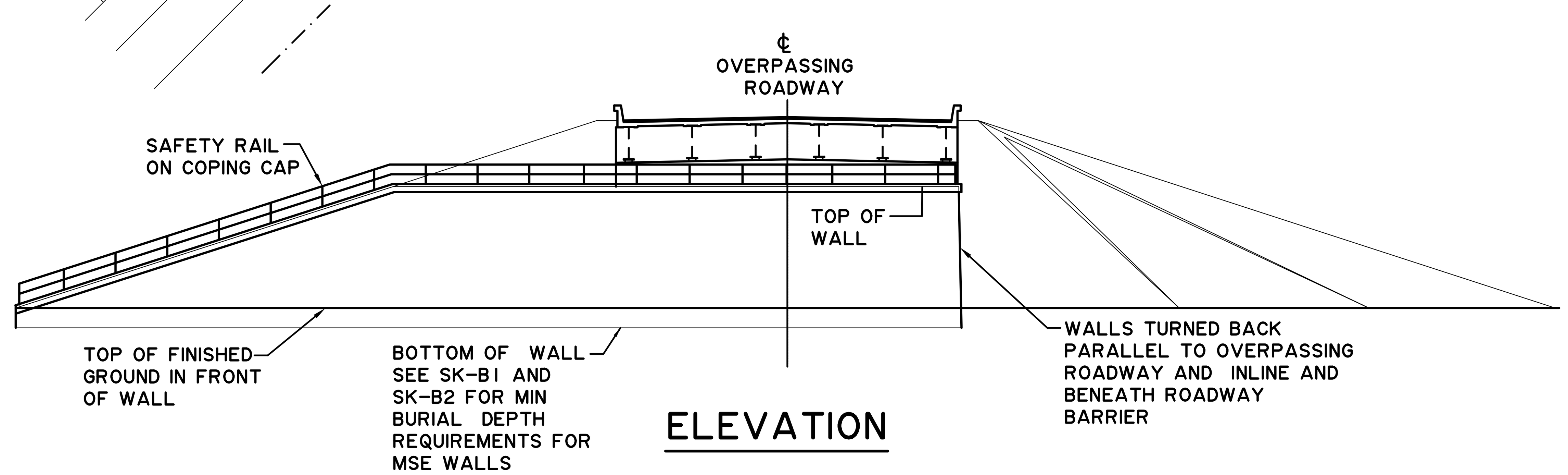
NOTE:
LAYOUT IMPERMEABLE GEOMEMBRANE AND ASSOCIATED DRAINS TO ENSURE POSITIVE DRAINAGE AWAY FROM MSE WALLS.
THE DETAILS OF GEOMEMBRANE PLACEMENT SHALL PREVENT RUN-OFF FROM PERCOLATING DOWN THROUGH THE FILL INTO MSE BACKFILL CONTAINING STEEL SOIL REINFORCEMENT

PLAN

NOTE: DITCH DRAINAGE AROUND INDEPENDENT WALL NOT SHOWN FOR CLARITY



C – SECTION
(SHOWING GEOMEMBRANE REQUIREMENT OVER GALVANIZED STEEL SOIL REINFORCEMENT)



ELEVATION

NOTES

- SKETCHES ARE PROVIDED WITH THE SOLE PURPOSE OF ILLUSTRATING SELECTED REQUIREMENTS FROM THE BRIDGE STRUCTURES DESIGN CRITERIA (BSDC). THEY INTENTIONALLY OMIT DETAILS THAT MIGHT DETRACT FROM THIS PURPOSE
- SKETCH ILLUSTRATES WALL LAYOUT TO BE USED FOR SKEW ANGLES GREATER THAN 45°
- FOR WALL LAYOUT TO BE USED FOR SKEW ANGLES LESS THAN OR EQUAL TO 45° SEE DRAWING SK-B5
- ROADWAY DRAINAGE TO BE DIRECTED PAST END OF WALL AND END OF STEEL SOIL REINFORCEMENT
- DRAIN TROUGHS SHALL NOT DRAIN ACROSS SLOPE
- INTEGRAL ABUTMENTS NOT PERMITTED WITH TURNED BACK WALLS

REV	DATE	REVISIONS	BY
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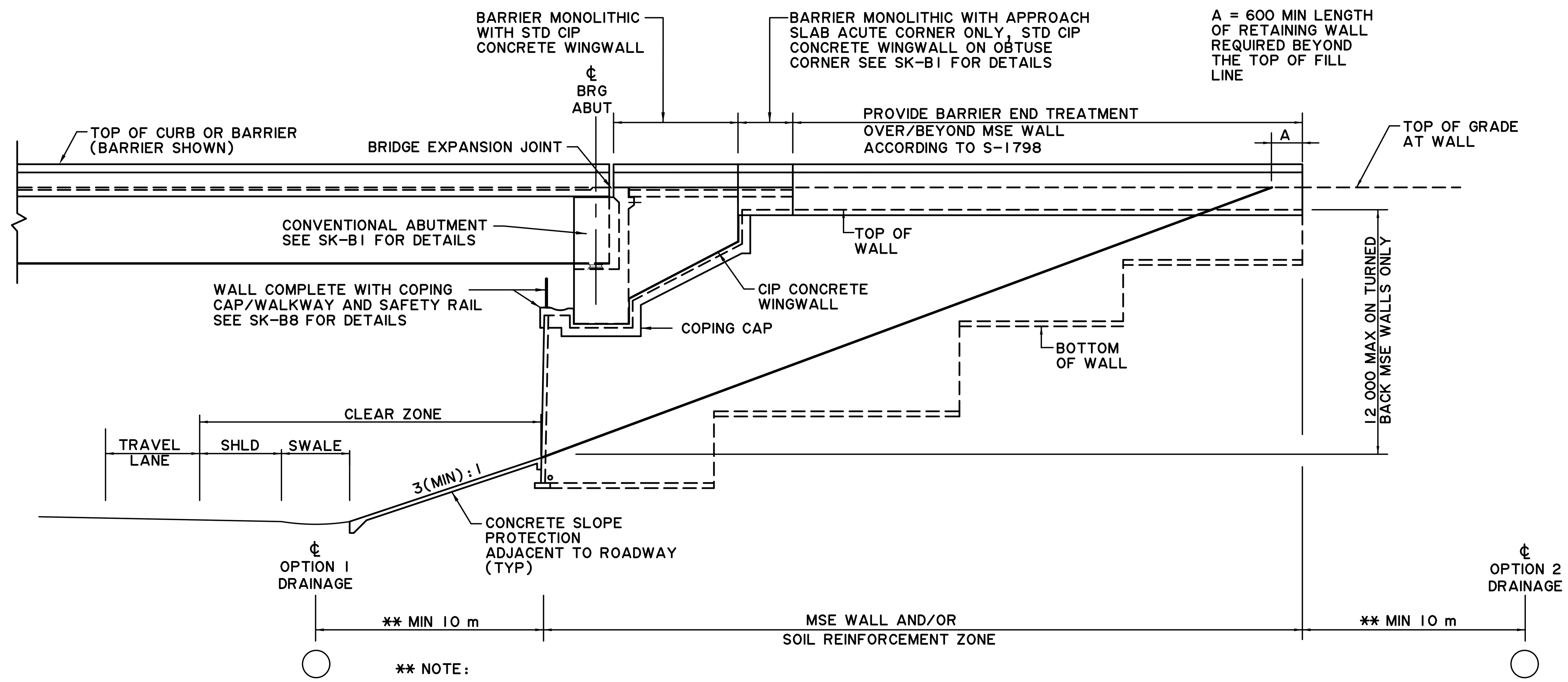
WALL LAYOUT AND SITE DRAINAGE FOR SKEW ANGLES > 45°

FILE NUMBER	DATE DRAWN	SHEET	DRAWING
-	2017-04-04	6 OF 9	SK-B6

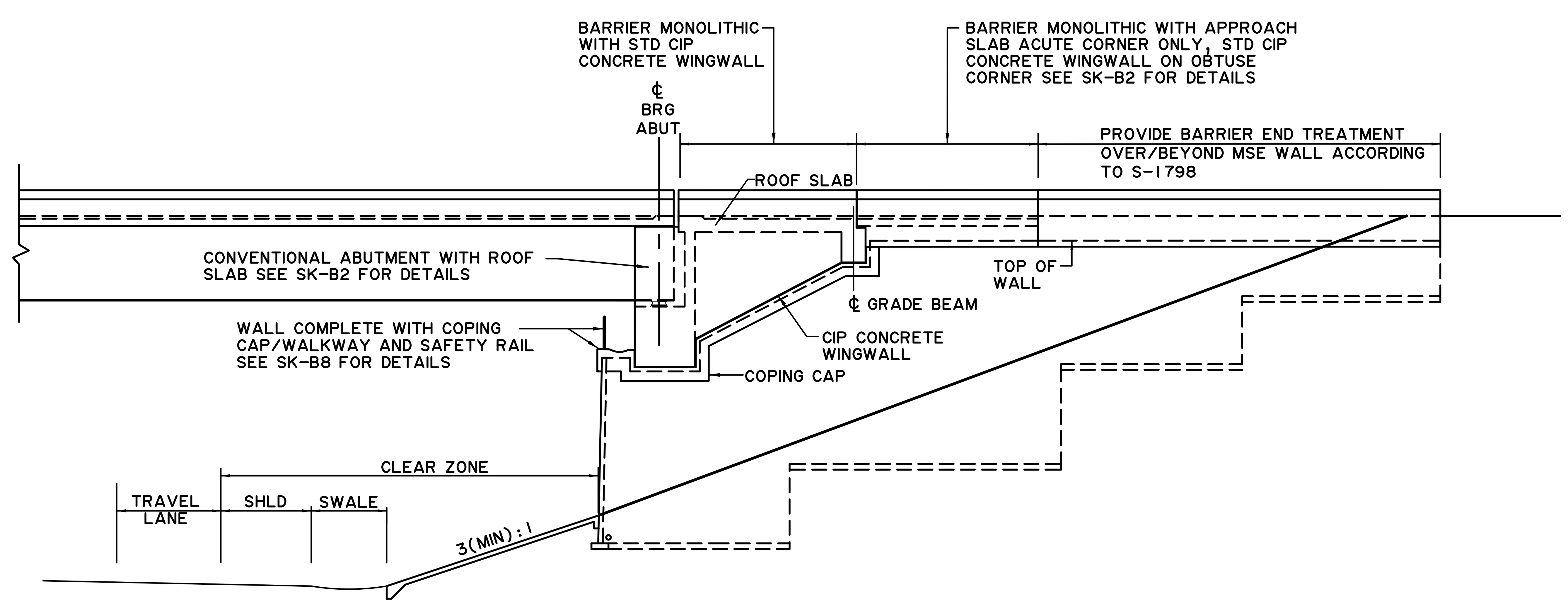


** NOTE:

- DITCH DRAINAGE TO BE LOCATED A MINIMUM 10 m BEYOND MSE WALL AND/OR SOIL REINFORCEMENT ZONE
- DISTANCES ARE MEASURED PERPENDICULAR TO THE MSE WALL AND/OR END OF SOIL REINFORCEMENT ZONE

A/6 SECTION

MSE WALL OPTION FOR CONVENTIONAL ABUTMENT



NOTE: ROOF SLAB OPTION SIMILAR TO CONVENTIONAL OPTION EXCEPT AS NOTED

A/6 SECTION

MSE WALL OPTION FOR CONVENTIONAL ABUTMENT WITH ROOF SLAB

NOTES

- DRAWING TO BE READ IN CONJUNCTION WITH SK-B6
- SEE GENERAL NOTES ON SK-B6

REV	DATE	REVISIONS	BY
4			
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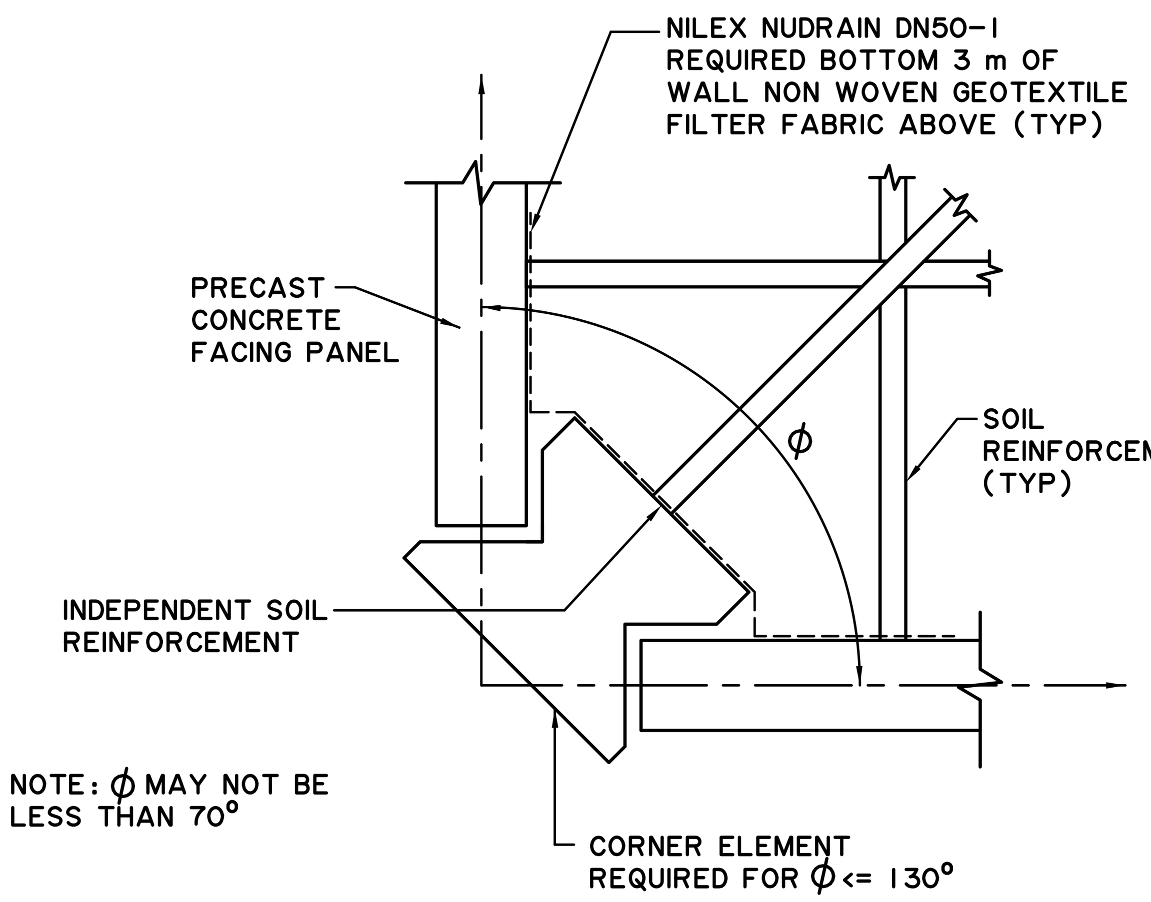
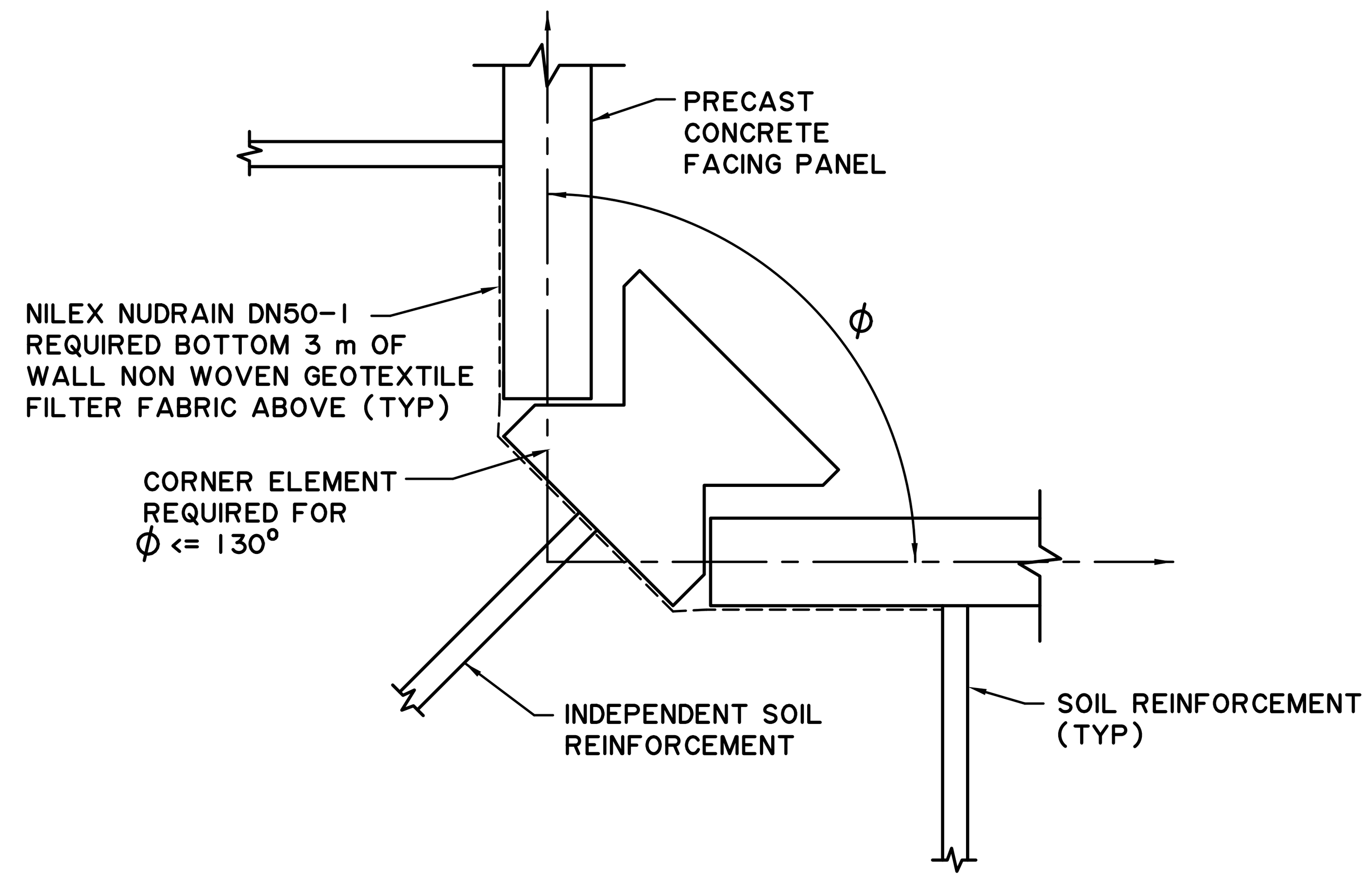
TURNED BACK WINGWALL DETAILS FOR SKEW ANGLES > 45°

FILE NUMBER	DATE DRAWN	SHEET	DRAWING
–	2017-04-04	7 OF 9	SK-B7

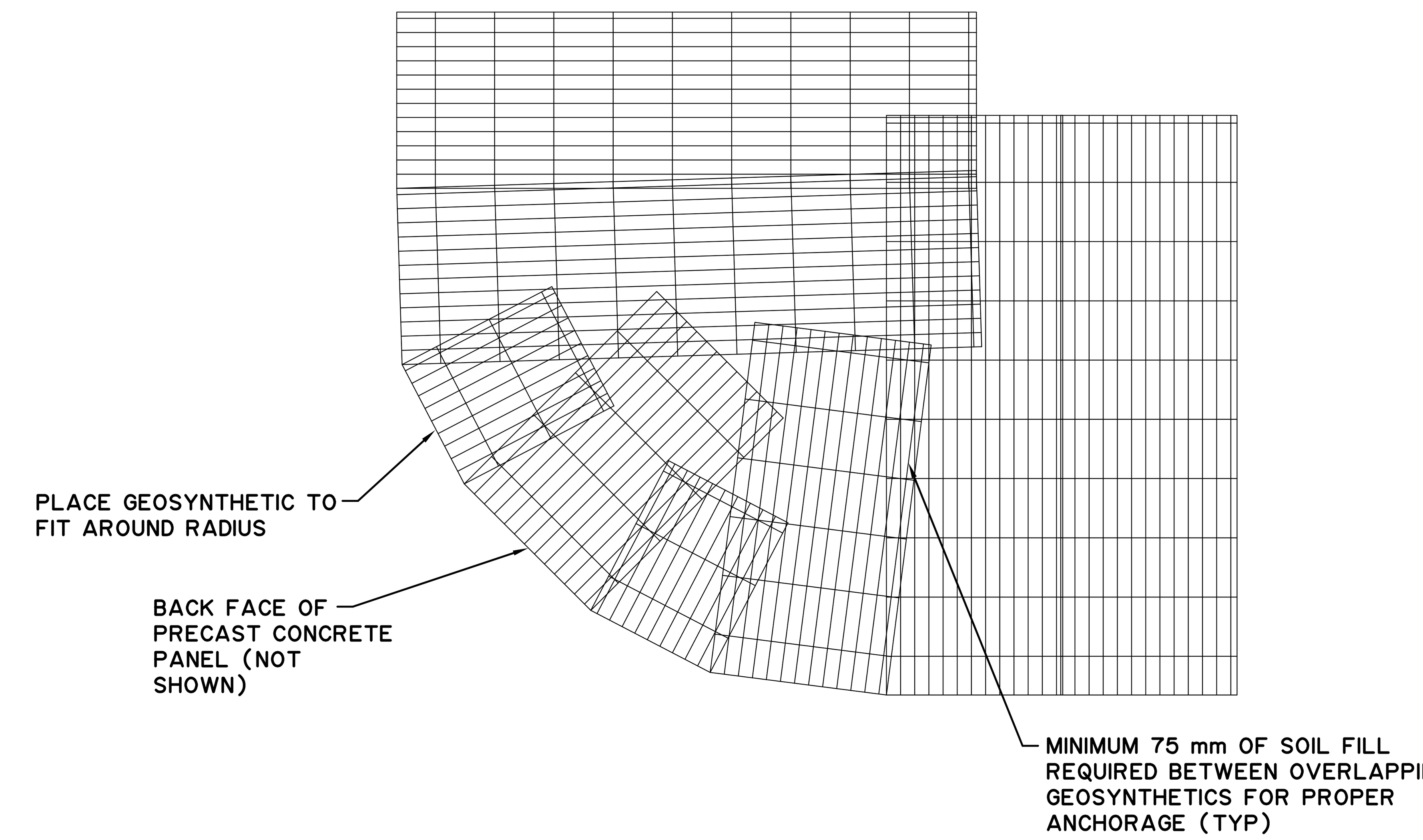


EXTERNAL CORNER

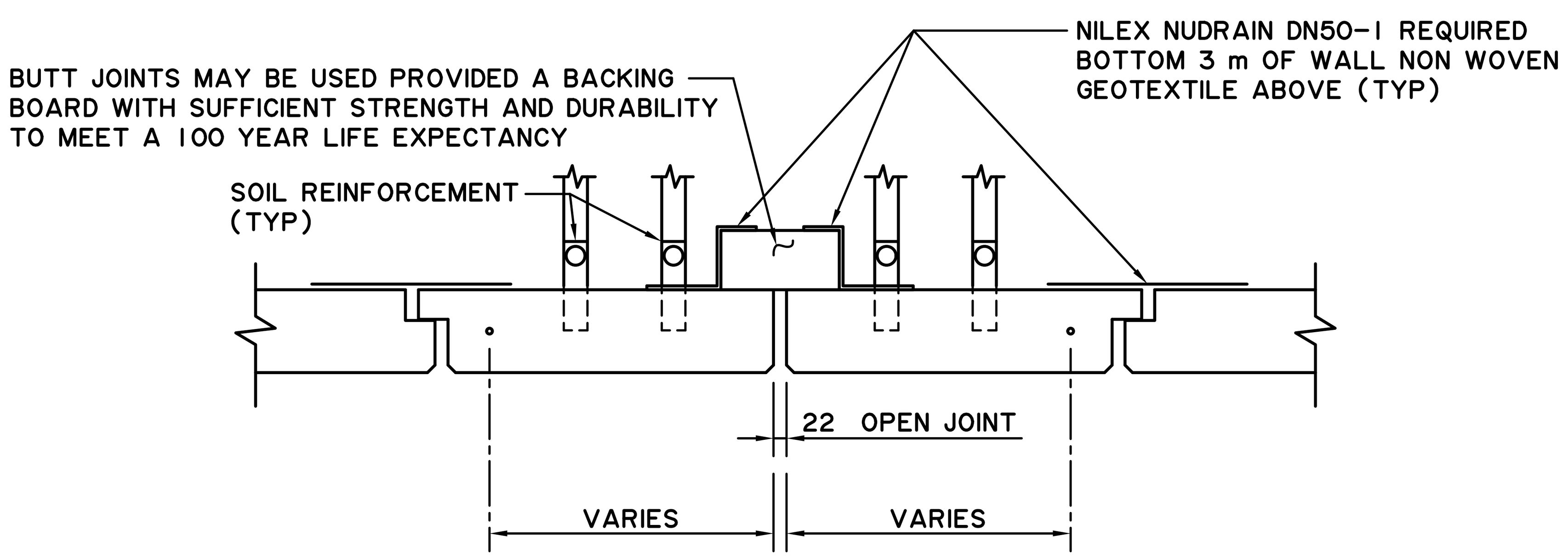


INTERNAL CORNER

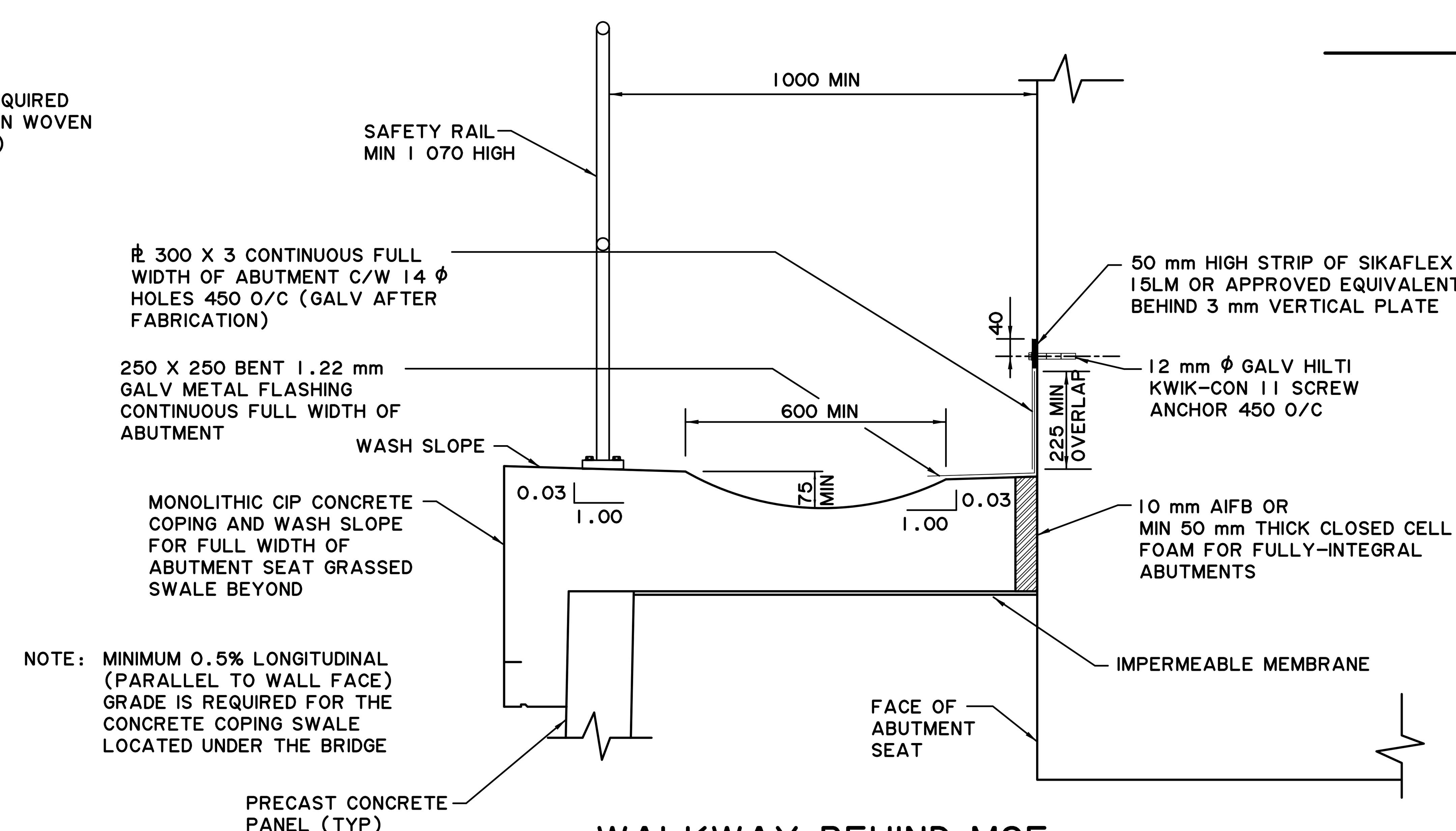
MSE WALL CORNER DETAILS



LAYOUT OF GEOSYNTHETIC REINFORCEMENT FOR CURVED WALLS

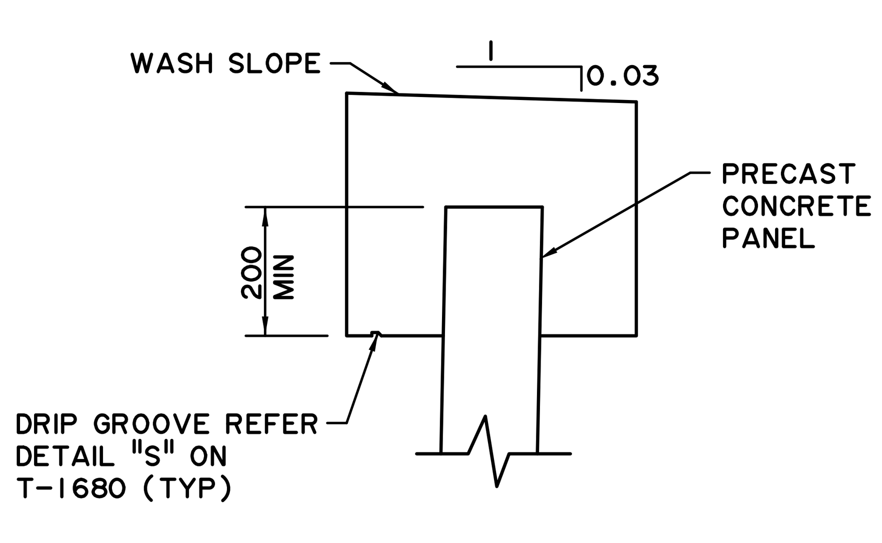


TYPICAL BUTT JOINT AND LIP AND RECESS(SHIP LAP) DETAILS FOR PRECAST CONCRETE FASCIA PANELS

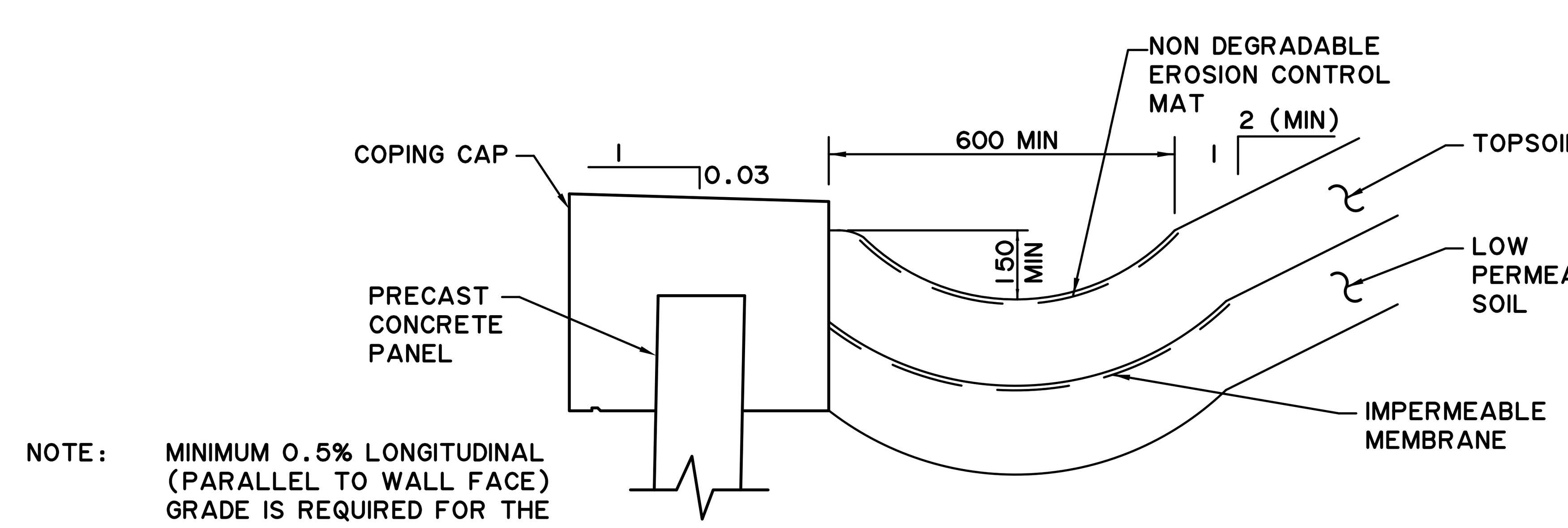


NOTE: MINIMUM 0.5% LONGITUDINAL (PARALLEL TO WALL FACE) GRADE IS REQUIRED FOR THE CONCRETE COPING SWALE LOCATED UNDER THE BRIDGE

WALKWAY BEHIND MSE WALL AT ABUTMENT



P/– COPING CAP DETAIL



NOTE: MINIMUM 0.5% LONGITUDINAL (PARALLEL TO WALL FACE) GRADE IS REQUIRED FOR THE GRASSED SWALE

GRASSED SWALE TOP OF MSE WALL BEYOND BRIDGE ABUTMENTS

NOTES

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4			
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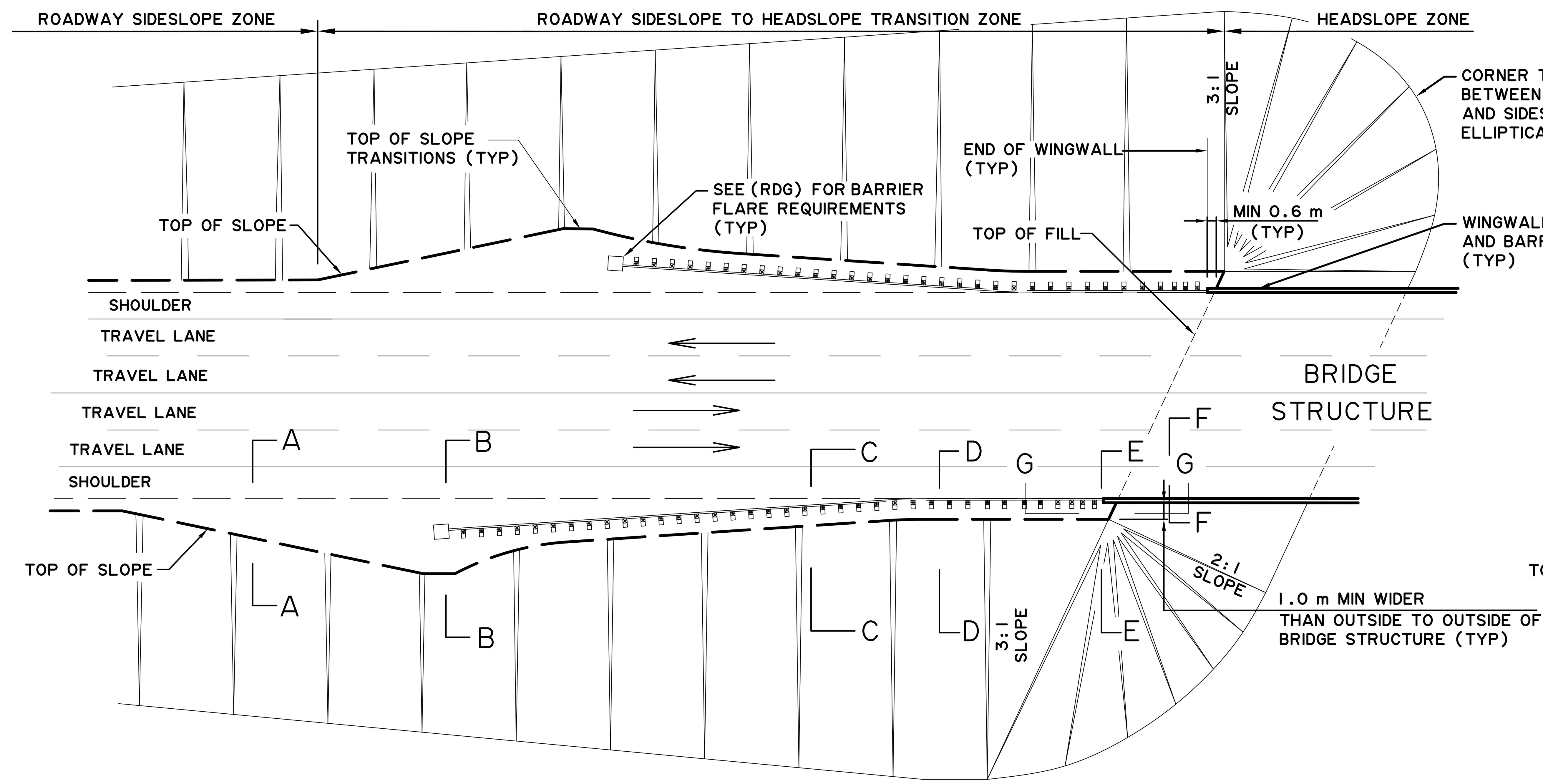
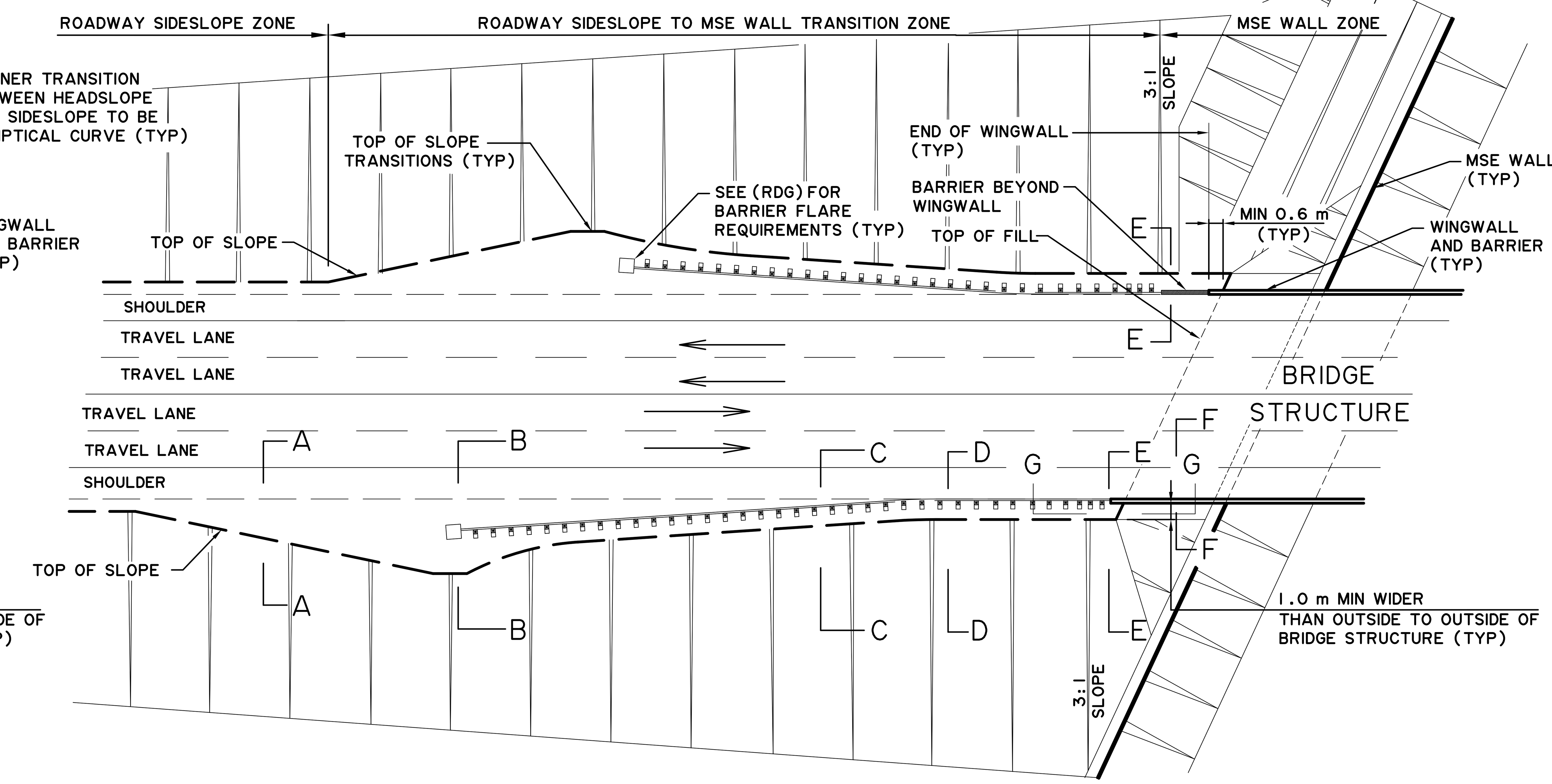
STANDARD DETAILS ASSOCIATED WITH MSE WALLS

FILE NUMBER	DATE DRAWN	SHEET	DRAWING
–	2017-04-04	8 OF 9	SK-B8



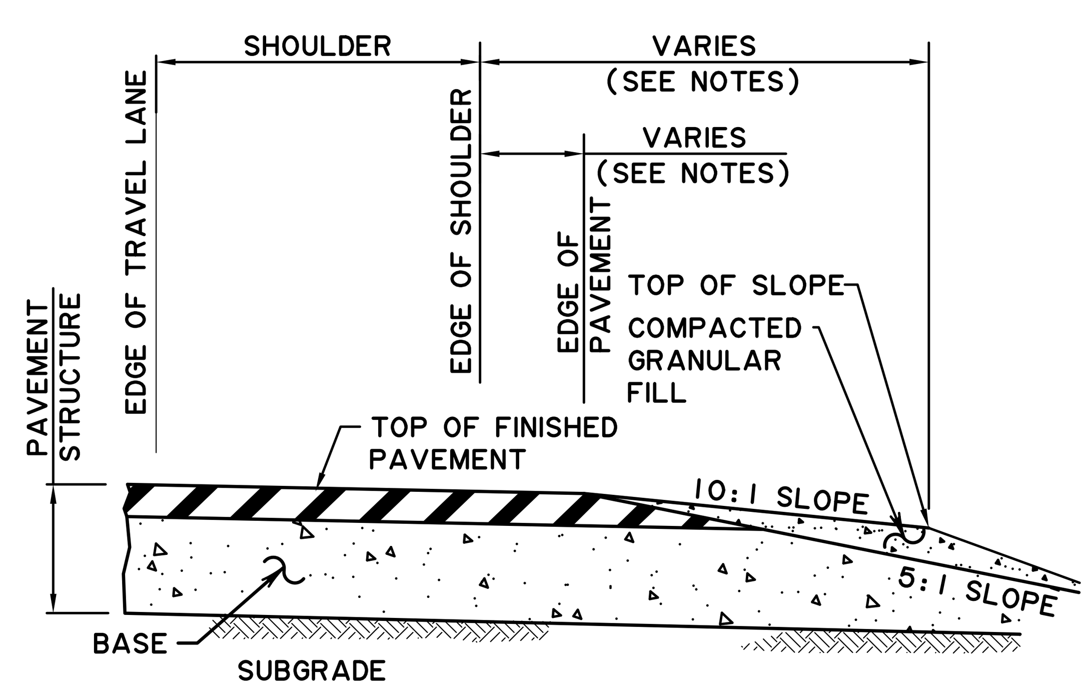
PLAN – BRIDGE WITH HEADSLOPE

(SKEW LESS THAN 45° SHOWN) NTS



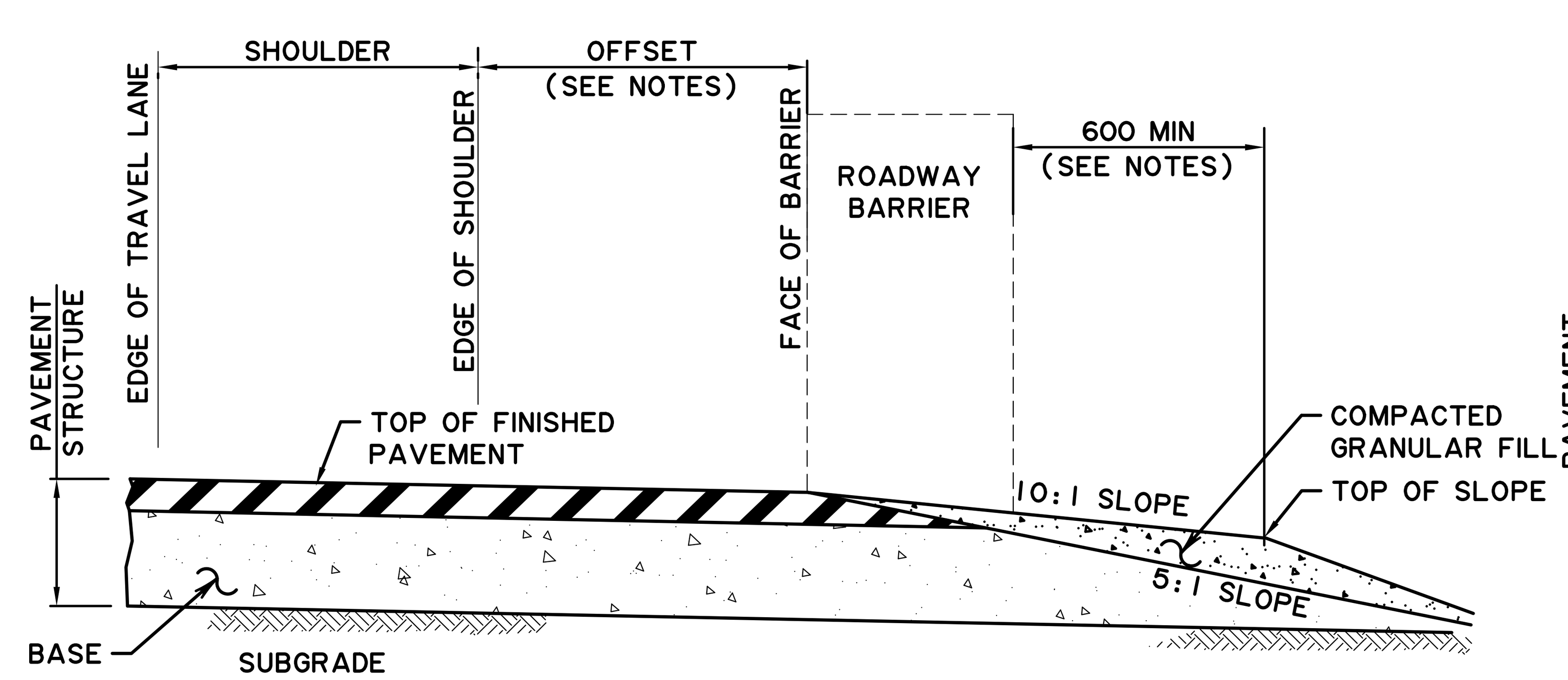
PLAN – BRIDGE WITH MSE WALL

(SKEW LESS THAN 45° SHOWN) NTS



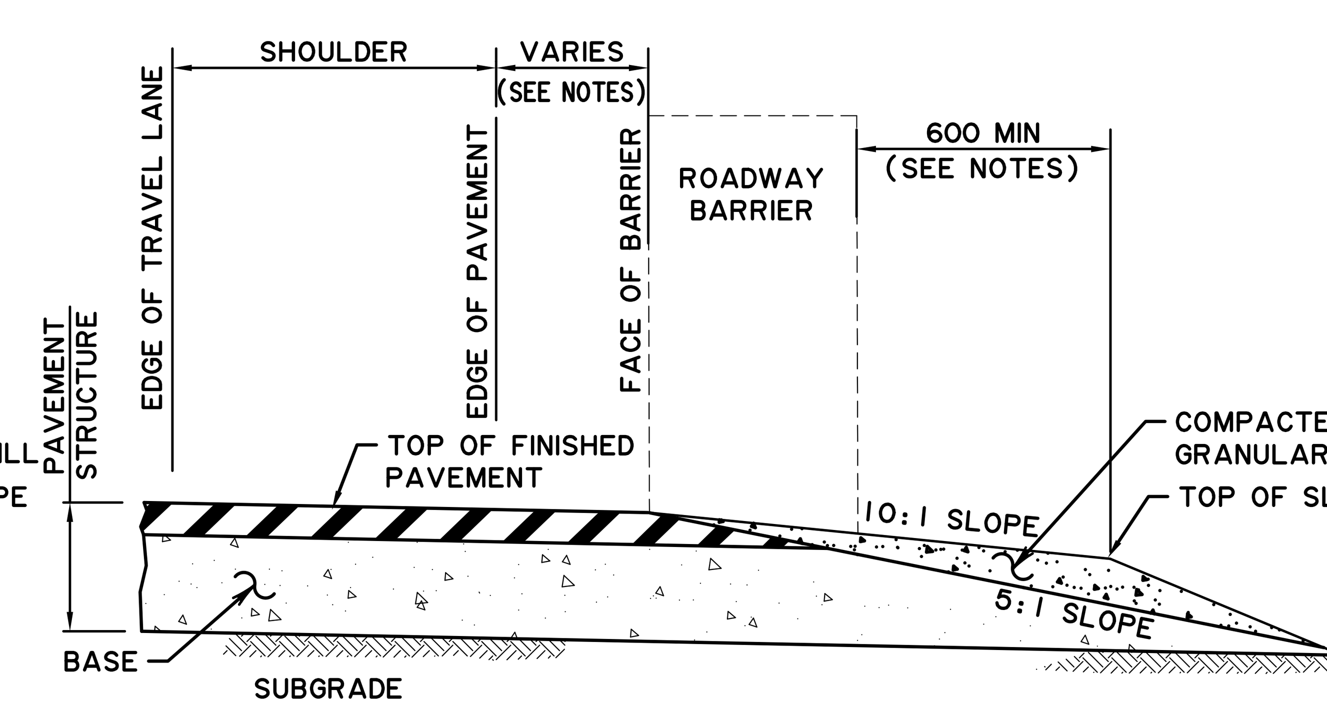
SECTION A–A

NTS



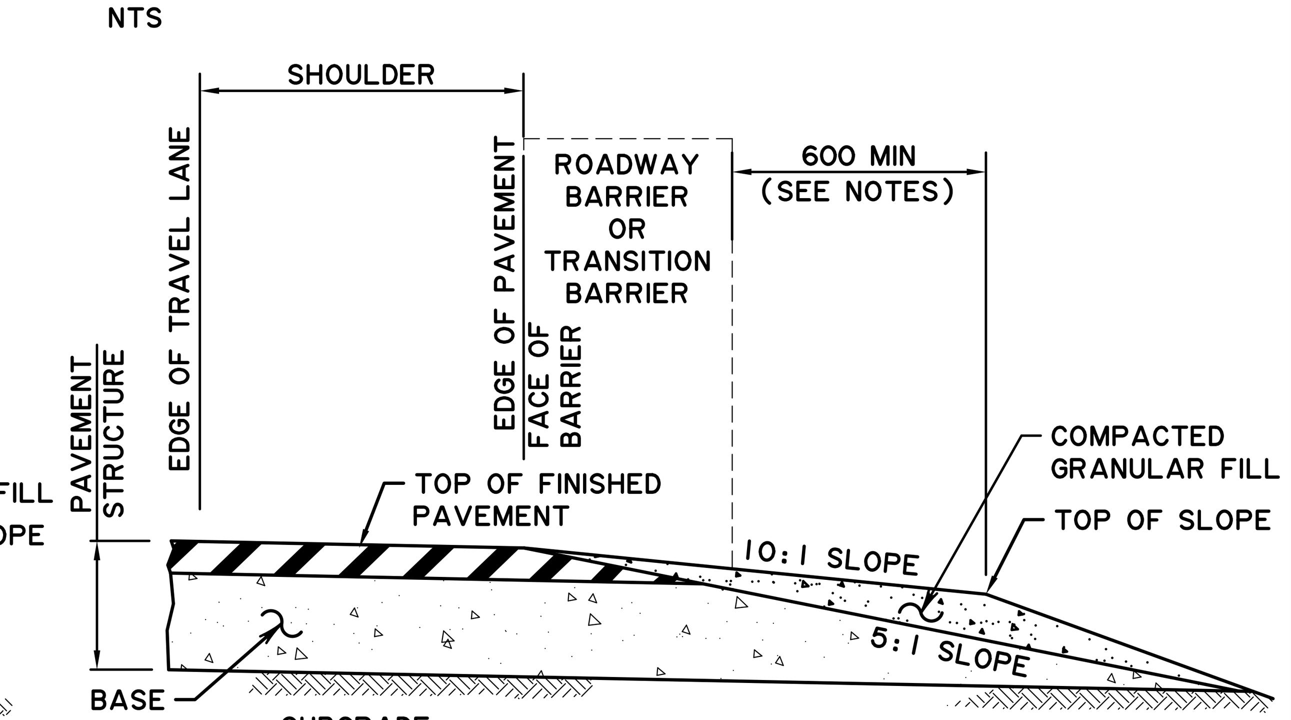
SECTION B–B

NTS



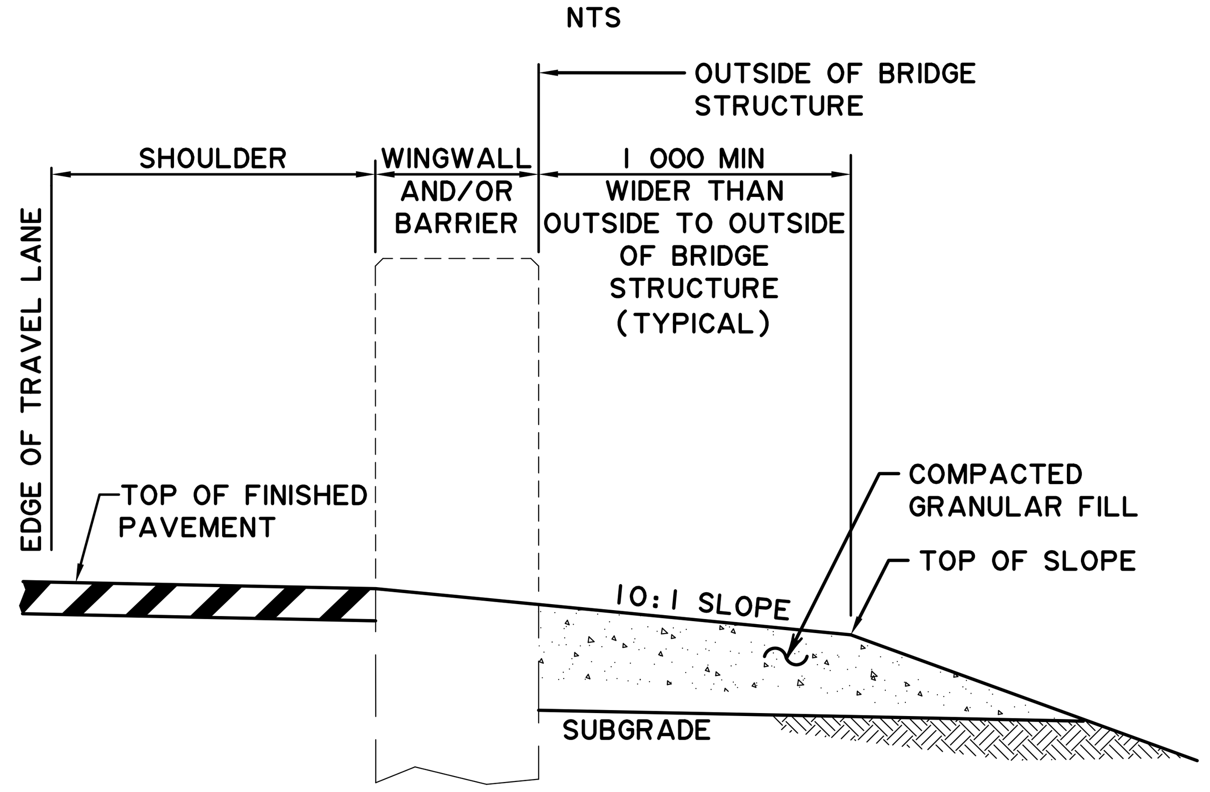
SECTION C–C

NTS



SECTION D–D

NTS



SECTION E–E

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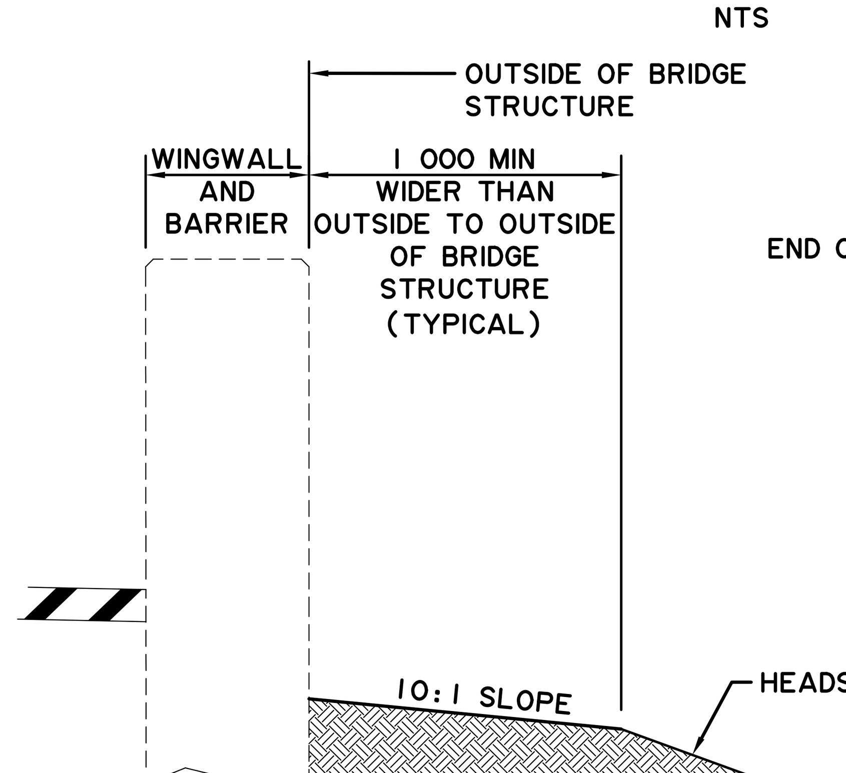
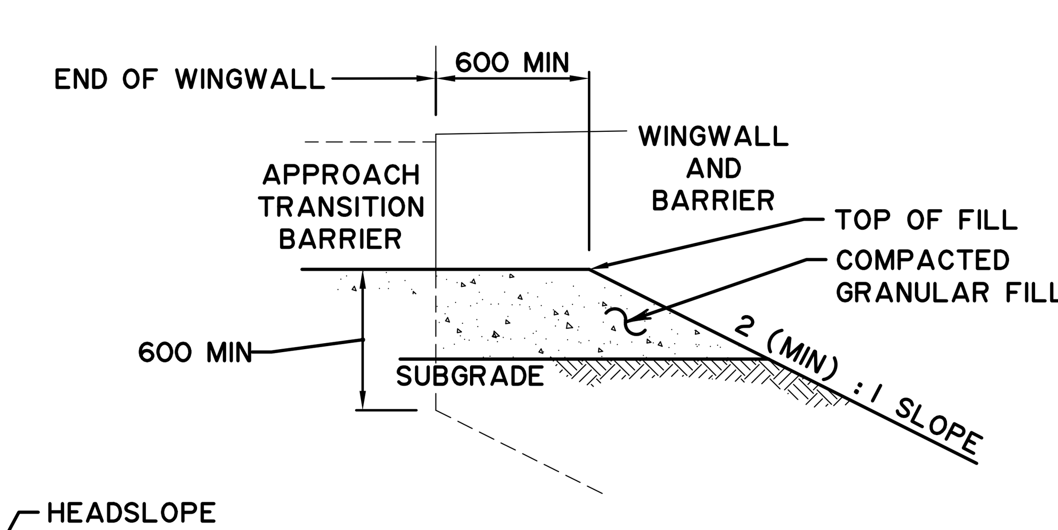


SECTION F–F

NTS



SECTION G–G

NTS

NOTES

- SKETCHES ARE PROVIDED FOR THE SOLE PURPOSE OF ILLUSTRATING SELECTED REQUIREMENTS FROM THE BRIDGE STRUCTURES DESIGN CRITERIA (BSDC). THEY INTENTIONALLY OMIT DETAILS THAT MIGHT DETRACT FROM THIS PURPOSE
- ROADSIDE BARRIERS AND TOP OF SIDESLOPE WIDTH TRANSITIONS SHALL BE AS PER THE ROADSIDE DESIGN GUIDE (RDG) AND THE SITE SPECIFIC ROADWAY DESIGN
- ALL DIMENSIONS ARE IN MILLIMETRES UNLESS NOTED OTHERWISE ROADWAY WITH TWO-WAY TRAFFIC AND TWO TRAVEL LANES EACH DIRECTION SHOWN. ADJUST FOR OTHER CONFIGURATIONS
- STEEL ROADWAY AND TRANSITION BARRIER TYPE SHOWN. ADJUST FOR OTHER BARRIER TYPES AND CONFIGURATIONS
- INFORMATION SHOWN IS FOR BRIDGES WITH SKEWS RANGING FROM 0 – 45 DEGREES. BARRIER AND WINGWALL CLEARANCES TO TOP OF SLOPE FOR BRIDGES WITH SKEW ANGLES EXCEEDING 45 DEGREES SHALL MATCH REQUIREMENTS INDICATED ON THIS DRAWING.

REV	DATE	REVISIONS	BY
4			
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BRIDGE APPROACH GEOMETRY

FILE NUMBER	DATE DRAWN	SHEET	DRAWING
–	2017-04-04	9 OF 9	SK-B9

APPENDIX C – CORROSION RESISTANT REINFORCING

C1. Introduction

Highway maintenance in Alberta involves the use of anti-icing and de-icing materials to maintain a safe and functional highway network in winter months. Anti-icing and de-icing materials are a significant cause of premature deterioration of bridge components and often result in the need for bridge rehabilitation. Over the past decades, the Department has refined its reinforcing standards to minimize the impact that winter roadway maintenance materials have on its bridge inventory. Alternative reinforcing steel materials with improved resistance to corrosion are now readily available and can provide a substantial reduction in life cycle and user costs with minimal impact on overall construction costs. These materials are referred to as Corrosion Resistant Reinforcing (CRR).

This Appendix presents the Department's approach for selecting the appropriate CRR as well as some important considerations when designing with CRR. Refer to Section 5.6.2 (Reinforcing Steel) for determining which bridge components should contain CRR.

C2. CRR MATERIALS

CRR materials shall be one of the following types:

1. Solid stainless reinforcing steel of Unified Numbering System (UNS) designations S31653, S31803 or S32304 meeting the requirements of ASTM A276 and A955/A955M. The minimum yield strength shall be 420 MPa.

2. Low carbon/chromium reinforcing steel meeting the requirements of ASTM A1035. The alloy type shall be 1035 CS, which has a high chromium content of 8.0 - 10.9 %. The minimum yield strength based on the 0.2% offset method shall be equal to 690 MPa. The common trademark name for ASTM A1035 compliant material is MMFX.

The National Research Council of Canada (NRC), US Federal Highway Administration (FHWA), Virginia Center for Transportation Innovation & Research, and several other research entities have reported that ASTM A1035 and stainless steel reinforcing can improve long term durability over traditional materials. It is also reported that the long term corrosion performance of stainless steel exceeds that of ASTM A1035 steel. Given the difference in material cost, the appropriate selection and use of CRR materials is important in order to maximize benefit to cost ratios.

C3. SELECTION OF CRR MATERIALS

Selection of CRR materials shall be based on a structure's exposure class. Exposure class charts have been developed from life cycle cost analyses including road user costs. It is important to note that societal factors, such as length of detour or local/industry economic impact are not incorporated in the following exposure class figures. Consideration of potential societal factors shall be reviewed at the conceptual design stage. Any deviation from the exposure class selection charts shall be reviewed and approved by the Department.

The appropriate exposure class shall be determined using Figure C1 for undivided highways and Figure C2 for divided highways. Selection charts account for an estimated 2% growth rate of AADT over the life of the structure. Therefore the AADT used for selection shall be the traffic in year 1 of operation.

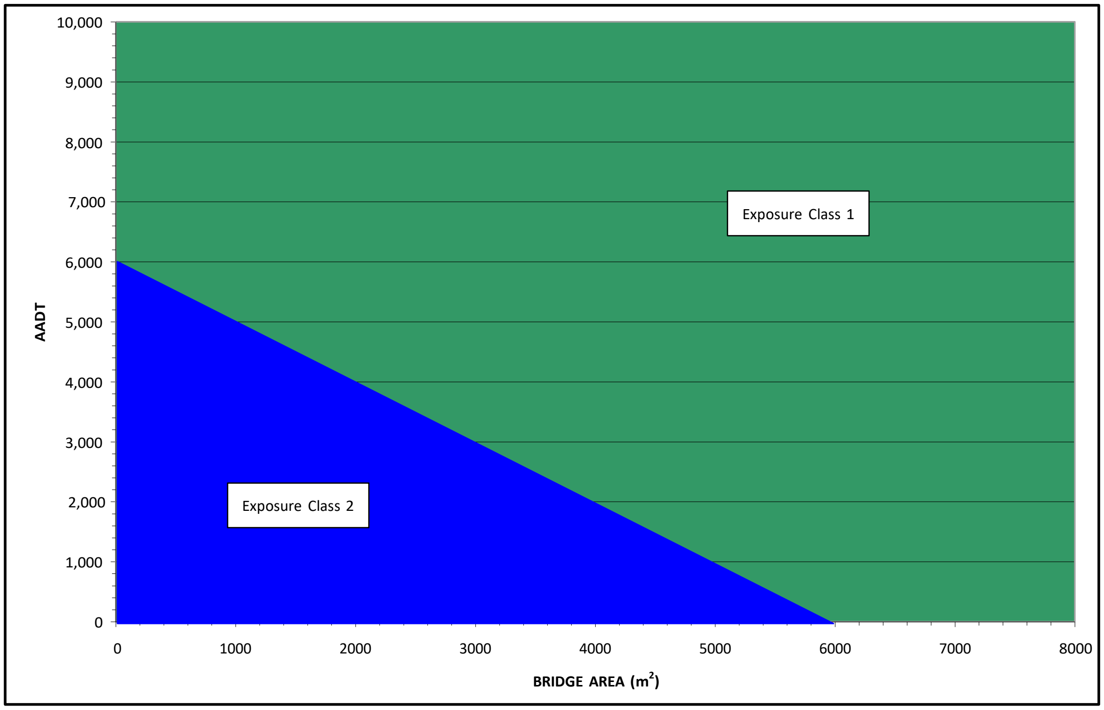

Figure C1: Reinforcing Steel Exposure Class Selection Chart for Undivided Highways

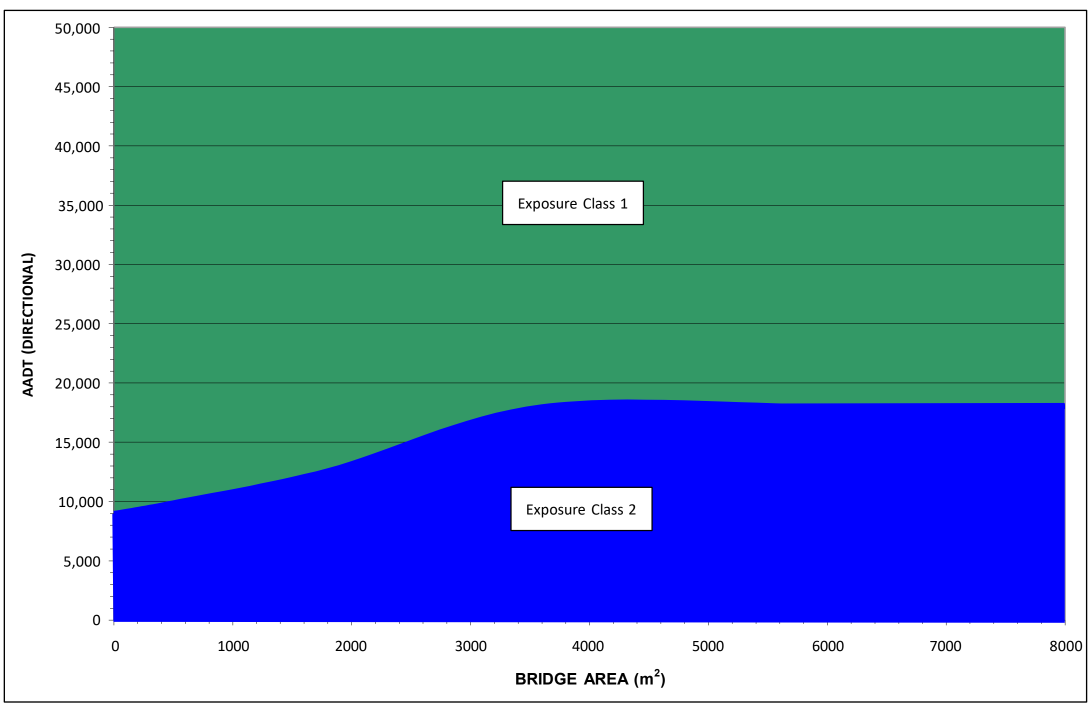

Figure C2: Reinforcing Steel Exposure Class Selection Chart for Divided Highways

Allowable CRR material types and grades for the applicable exposure class are listed in Table C1.

Table C1 – List of CRR Materials

Exposure Class	Allowable CRR Materials	Grades
1	*Solid Stainless Reinforcing Steel*	*UNS S31653* *UNS S32304* *UNS S31803*
2	*Corrosion Resistant Reinforcing Steel*	*Solid stainless reinforcing steel* *(as noted above in exposure class 1)* *or* *ASTM A1035* *(Low carbon/ chromium reinforcing steel)*

C4. DESIGN REQUIREMENTS

The design of bridge components containing CRR shall be based on the reinforcing steel having a yield strength of 420 MPa. This includes all hooks, development lengths and bar splices.

The design of stirrups or projecting girder stirrups with low carbon/chromium ASTM A1035 reinforcing shall be based on the reinforcing steel having a yield strength of 500 MPa. This includes all hooks, development lengths and bar splices.

For bridges in Exposure Class 2, ASTM A1035 or solid stainless reinforcing is permitted and final selection will be determined by the Contractor. However, given current market pricing it is most probable that ASTM A1035 reinforcing would be selected. Currently ASTM A1035 reinforcing is only produced in imperial bar sizes. Stainless steel reinforcing is available in both metric and imperial sizes. The design of bridge components within Exposure Class 2 shall take into consideration the high probability that a Contractor will request a substitution of imperial bars for metric bars. All detailed design drawings shall specify metric bars and only at the request of the Contractor and acceptance by the Consultant may imperial bars be substituted.

The majority of CRR applications will utilize 15M and 20M bars. Metric 15M bars and imperial #5 bars have cross-sectional areas within 0.5% and direct substitution will have negligible impact on a components design. When 20M bars are required the Consultant should design using the dimensions of a #6 bar (19 mm diameter, 284 mm^2 cross-section area). The imperial #8 bar has a cross-sectional area 2% greater than its metric 25M counterpart and will have negligible impact on a components design. When 30M bars are required the Consultant should design using the dimensions of a #9 bar (29 mm diameter, 645 mm^2 cross-section area). The proposed substitution of imperial for metric bars should be as outlined in Table C2. Any proposed substitution of metric for imperial bars must be accepted by the Consultant.

Table C2 – Substitutions of Metric for Imperial Bars

Metric Bar Designation	10M	15M	20M	25M	30M	35M	40M	45M
Imperial Bar Designation	*4*	*5*	*6*	*7*	*8*	*9*	*10*	*11*
Mass (kg/m)	*0.785*	*1.570*	*2.355*	*3.925*	*5.495*	*7.850*	*11.775*	*19.625*

C5. DETAILING CONSIDERATIONS

The available sizes and lengths of CRR materials are listed in Table C3.

Table C3 – CRR Available Bar Sizes and Lengths

	CRR: Solid Stainless	**CRR: Low Carbon/Chromium ASTM A1035**
Bar Sizes	*All Sizes – Metric and Imperial**	*All sizes – Imperial Only*
Bar Lengths	*Up to 18288 mm (60') for sizes #4, #5 and #6, and up to 12192 mm (40') for all other sizes*	*Up to 18288 mm (60') for all sizes***

* Not all grades of stainless steel are available in metric sizes.

** ASTM A1035 #3 or #4 bars are available in coil.

All hooks and bends shall be detailed using the pin diameters and dimensions as recommended for "black reinforcing" in the *Reinforcing Steel Institute of Canada (RSIC)*, *Reinforcing Steel - Manual of Standard Practice*.

C6. REFERENCES

1. *ASTM A276-10* (2010). *Standard Specification for Stainless Steel Bars and Shapes*. ASTM International, West Conshohocken, PA.

2. *ASTM A955/A955M-15* (2015). *Standard Specification for Deformed and Plain Stainless-Steel Bars for Concrete Reinforcement*. ASTM International, West Conshohocken, PA.

3. *ASTM A1035/A1035M-15* (2015). *Standard Specification for Deformed and Plain, Low-Carbon, Chromium, Steel Bars for Concrete Reinforcement*. ASTM International, West Conshohocken, PA.

4. AT. *Standard Specifications for Bridge Construction, Edition 16*. Alberta Transportation, Edmonton, AB (2017).

5. RSIC. *Reinforcing Steel- Manual of Standard Practice, Fourth Edition.* The Reinforcing Steel Institute of Canada, Richmond Hill, ON (2004).

APPENDIX D – BRIDGE BEARING DESIGN GUIDELINES

D1. Introduction

With this document, the Department aims to specify the appropriate or preferred bearing types, and to standardize the design approach of these bearings. It is anticipated that a standardized approach to bearing selection and design will lead to a more robust and predictable bearing inventory with a proven track record of requiring little to no maintenance.

In the past, steel bearings such as roller nests, pinned rockers, and tall expansion rockers have been used to accommodate unidirectional displacements and rotations. These required costly precision fabrication and were susceptible to corrosion causing the bearings to freeze. Steel roller nests require regular maintenance and lubrication and are susceptible to contamination from dirt and binding. Tall expansion rocker bearings require frequent re-setting and can be susceptible to catastrophic failure if their movement capacity is exceeded. None of these bearings are suitable for wide or highly skewed bridges as they can only accommodate displacements and rotations along or about a single axis.

Bearings types specified for new bridges in Alberta are plain or reinforced elastomeric bearings, pot bearings, or fixed steel plate rocker bearings. Continuous plain elastomeric sheets are used for the Department's standard SL, SLW and SLC girder bridges.

D2. Preferred Bearing Types

D2.1 Steel Reinforced Elastomeric Bearings

Steel reinforced elastomeric bearings have no moving parts, require no maintenance, and have a long history of successful performance in Alberta. Elastomeric bearings also have significant overload capacity beyond the first signs of distress and generally allow ample time for identification and repair of problems.

Elastomers are very flexible in shear but very stiff in bulk compression. When compressed, unconfined elastomeric pads will expand laterally. Layers of steel reinforcing limit the lateral expansion that can occur in reinforced bearings, increasing the compressive stiffness and strength. Thinner elastomer layers lead to less bulging and increased compressive strength and stiffness, but also higher rotational stiffness. Larger rotations can be accommodated by increasing the elastomer layer thicknesses or increasing the number of elastomer layers. Selection of the number and thickness of elastomer layers is a compromise between the need for compressive strength and rotational flexibility.

Although elastomeric bearings can accommodate horizontal displacements through shear deformation of up to 50% of the total elastomer thickness, Alberta practice is to accommodate long term and thermal movements by attaching a PTFE sheet and stainless steel slider to the top of expansion bearings allowing the girder ends to slide across the PTFE surface. Friction between the PTFE sheet and slider will still cause some shear deformation in the bearing, which must be considered in the design. Rapidly applied movements of the bottom flange, such as those resulting from girder rotation under live load, cause a higher coefficient of friction between the PTFE and stainless steel slider than movements that occur over a longer period of time. The higher coefficient of friction prevents any sliding from occurring and the small rapidly applied movements are accommodated through shear deformation of the elastomer. This two-pronged approach to accommodation of movement is preferable as small rapidly applied sliding movements have been shown to cause rapid deterioration of the PTFE sheet. Accommodating the rapidly applied movements by deformation of the elastomer improves the durability of the PTFE sheet. The low magnitude of the rapidly applied movements does not generally affect the elastomer thickness. Accommodating large movements with a PTFE and stainless steel slider reduces the overall thickness of the elastomer and increases the strength of the bearing.

Moderate (up to 25 mm) horizontal movements can be accommodated by shear deformations within the elastomeric pad. Movements larger than 25 mm shall be accommodated by adding a polytetrafluoroethylene (PTFE) and stainless steel sliding interface to the top surface of the elastomeric bearing.

For NU girders, wide elastomeric pads provide more stability during girder erection than compact pot bearings.

D2.2 Pot Bearings

Pot bearings shall be used only where steel reinforced elastomeric bearings are too large and become difficult to fabricate. Pot bearings can accommodate very high vertical loads and moderate rotations about any axis. A pot bearing does not permit any horizontal displacements, but a PTFE and stainless steel slider may be added to accommodate them. Lateral guides can also be added to restrict the direction of movement, but concrete shear blocks are preferred for this purpose.

A pot bearing relies on the total confinement of an elastomeric disc. The disc operates in a nearly hydrostatic state of stress and is very flexible in rotation, but extremely stiff against changes in volume. The total confinement of the elastomeric disc is essential to the performance of the bearing.

Moderate bearing rotations are accommodated by deformation of the elastomeric disc, but large deformations may cause the disc to slip inside the pot, causing abrasion of the disc. The abrasion is mitigated by ensuring that the inside surfaces of the pot wall and piston are as smooth as possible. The rotational limit of a pot bearing is reached when metal components come into contact or when the piston binds with the sidewalls of the pot.

Other common problems with pot bearings include leaking of the elastomer and binding of the steel components. Leaking of the elastomer is prevented by a system of sealing rings, of either rectangular or circular section, that is crucial to the satisfactory performance of the bearing. Binding of the steel components is prevented by specifying proper clearances and by proper leveling during installation.

Pot bearings require precision fabrication and very tight quality control and quality assurance during installation to avoid performance problems.

D2.3 Fixed Steel Plate Rocker Bearings

Fixed steel plate rocker bearings can accommodate high loads and very large rotations about a single axis. They are most commonly used for fixed pier bearings on moderate to long span steel girder bridges. They can provide solid longitudinal and lateral force transfer through direct shear of high strength steel anchor rods. Steel plate rocker bearings only allow for rotation about a single axis and are not suitable for bridges with movements in multiple directions due to curve, skew, or very large bridge width.

D2.4 Plain unreinforced bearing pads

Plain unreinforced bearing pads without sliding surfaces shall only be used for support of girders where the total movement range of the bearing in any direction over the life of the bridge does not exceed 25 mm or for temporary erection stage bearings for integral bridge girder supports.

D3. Selection of Bearing Type

Bearing types shall be selected according to the following order of preference:

1. Plain unreinforced bearing pads shall only be used in the following situations:
    a. As continuous unreinforced elastomer sheets for short span concrete box girders such as the Department's standard SL, SLW and SLC girder bridges; or
    b. As temporary girder bearings for fully-integral piers or abutments.
2. For fixed connections to piers on medium to long span bridges with minimal curvature or skew, the Department's preferred bearing is fixed plate rocker bearings.
3. For all other situation, the Department's preferred bearing is steel reinforced elastomeric bearings.
4. Pot bearings shall only be used when fixed plate rocker bearings or steel reinforced elastomeric bearings are not practical.

Table D1 summarizes approximate design limits on the design loads, displacements, and rotations for the preferred bearing types.

Table D1 – Approximate Maximum Bearing Capacities at SLS

Bearing Type	Min Reaction (kN)	Max Reaction (kN)	Max Displacement (mm)	Max Rotation (rad)
Unreinforced Continuous Elastomeric Sheet	*0*	*450*	*±15*	*± .02*
Steel Reinforced Elastomeric Pad	*200*	*3000*	*±25*†	*± .04*
Pot Bearing	*1500*	*12000*	*No limit with slider plates*	*± .025*
Fixed Steel Plate Rocker	*1000*	*4000*	*0*	*No practical limit*

† Displacement capacity may be increased by adding a PTFE / stainless sheet slider to the top of the bearing.

For loads exceeding 12000 kN, alternative bearing types may need to be considered. Bearings for high skew and curved bridges may require special considerations or special design features.

If the Consultant wishes to propose alternative bearing types or bearings with special features, they submit specific bearing details and specifications to the Department's Bridge Engineering Section for review and approval.

D4. Design of Specific Bearing Types

D4.1 Steel Reinforced Elastomeric Bearings

The design of steel reinforced elastomeric bearings shall meet the requirements of this document, the *CHBDC* and *SSBC Section 8*: *Bridge Bearings,* and shall incorporate the details shown on typical detail drawing *T-1761-17 (Typical Expansion Bearing Details)* for steel reinforced elastomeric bearings.

Steel reinforced elastomeric bearings shall be designed for rotations that take place after the bearings are grouted plus an allowance for uncertainties of 0.005 radians at SLS. Rotations need not be considered at ULS.

Steel reinforced elastomeric bearings shall include steel sole plates and base plates.

A self-rocking pintle welded to the underside of the base plate shall be used to ensure uniform contact between the elastomeric pad and the girder bottom flange at erection. A single pintle centred beneath the bearing is preferable as it allows rocking in all directions ensuring uniform contact between the pad and the girder. For wide bearing pads such as those for NU girders, at least one additional pintle is usually required to distribute the load. In this case, the pintles shall be centred beneath the bearing along a line perpendicular to the longitudinal axis of the girder. The pintle or pintles shall be supported on galvanized steel shim stacks of the appropriate thickness to achieve the correct bearing elevation and the appropriate size to support the loads applied prior to grouting the baseplate. Typical pintle details are provided on typical detail drawing *T-1761-17 (Typical Expansion Bearing Details)*.

At the time of erection, the self-rocking pintle will bring the elastomeric pad into uniform contact with the girder underside, or the sole plate if one is provided. The bearing base plate shall be grouted prior to the deck pour, locking the base plate into position. Subsequent girder end rotations due to deck pour and other permanent loads will bring the elastomeric pad into a wedge shape, and the base plate will not be parallel to the sliding plane. The design rotation shall therefore include all permanent rotation components after girder erection.

When selecting the elastomer layer thicknesses and the number of layers, the acceptable configuration that produces the smallest overall bearing height should be used. *AASHTO LRFD Section 14.7.6.1* provides guidance on selecting the number and thickness of interior elastomer layers. The minimum shim plate thickness shown on typical detail drawing *T-1761-17 (Typical Expansion Bearing Details)* is generally adequate. *AASHTO LRFD Section 14.7.5.3.5* provides guidance on checking shim plate thickness.

Notwithstanding *Clause 11.6.6.2.2 of* the *CHBDC*, material requirements for elastomers shall conform to *AASHTO M251-06 Standard Specification for Plain and Laminated Elastomeric Bridge Bearings*. Cured elastomeric compounds shall be low temperature Grade 5 and meet the minimum requirements listed in *Table X1* of *AASHTO M251 Plain and Laminated Elastomeric Bridge Bearings (AASHTO M251)* and Shore A durometer hardness of 60.

The entire bearing assembly, between the sole plate and the base plate shall be replaceable without damage to the structure and without removal of any concrete, welds or anchorages permanently attached to the structure and without lifting the superstructure more than 5 mm. Bearings shall not be recessed into plates that are permanently attached to the structure.

Elastomeric pads shall be restrained from walking out from the design position by means of 10 mm high corner keeper bars bolted to the top of the base plate, as shown on typical detail drawing *T-1761-17 (Typical Expansion Bearing Details)*.

Sliding bearings shall allow translation by sliding of a stainless steel surface against a mating polytetrafluoroethylene (PTFE) element. The PTFE element shall be a 4.8 mm thick unfilled, unlubricated PTFE sheet. The flat PTFE sheet shall be recessed and bonded into a 2.5 mm deep recess in the top of a 10 mm thick galvanized steel plate. The galvanized plate shall be vulcanized to the top of the elastomeric pad. The galvanized steel plate shall have the same plan dimensions as the elastomeric pad and act as the top laminate in the elastomeric bearing. The stainless steel sliding surface shall conform to AISI Type 304, No. 8 finish and shall be welded to the bottom of the sole plate as shown on typical detail drawing *T-1761-17 (Typical Expansion Bearing Details)*.

Elastomeric bearing pads on skewed bridges shall typically be oriented perpendicular to the longitudinal girder axis. When the direction of rotation is uncertain, such as for severe skews or for bridges with stiff concrete diaphragms, round elastomeric bearing pads shall be considered.

Field welding adjacent to elastomeric pads shall be performed with care to avoid damage to the elastomer. The temperature of the steel adjacent to the elastomer should be kept below 120°C. The distance between the weld and the elastomer should be at least 40 mm.

For side-by-side precast concrete box beams, two separate reinforced elastomeric bearing pads shall be provided at each end of each girder, under the girder webs. The bearing pads shall be installed directly on top of the substructure and steel shear pins shall be provided to keep the bearings from walking. The pins shall not project higher than the top of the bottom internal reinforcing shim plate in the bearing. For additional information for box beams, see Section 9.2.3 (Side-by-side Precast Concrete Girder Bridges).

D4.2 Pot Bearings

The design of pot bearings shall meet the requirements of this document, the *CHBDC* and *SSBC Section 8*: *Bridge Bearings*.

D4.2.1 General Pot Bearing Design

The following design considerations are applicable to the Consultant's bearing design responsibilities as well as to the specific pot bearing components that will be designed by the bearing supplier.

Pot bearings can be installed level or inclined, depending on the orientation requirement for the final sliding surface (see Section D5.5 (Tapered Sole/base Plates and Bearing Setting Plane)). Where a level sliding plane is required, the bearings shall be supported on a level base plate sitting on four galvanized shim stacks. Where an inclined sliding surface is required, the bearings shall be supported on a tapered base plate with the bottom surface of the base plate level and sitting on four shims stacks. The bearing shall be grouted after erection of the girders but before any additional dead load (deck, etc.) is placed on the girders. In both cases, an initial rotation will be forced into the bearing elastomer due to initial girder camber at the time of erection. This initial rotation will be negative, but will be cancelled out with the use tapered sole plates or tapered base plates, as the girder end rotates into its final camber position.

Provision for translation shall be through sliding of a stainless steel surface against a mating PTFE element. Sliding surfaces shall allow translation by sliding of a metal surface against a mating PTFE element. For plane surfaces, the metal surface shall be stainless steel. The metal surface shall overlap the PTFE by at least 25 mm at extremes of movement on each side and, except for guides for lateral restraint, shall be positioned above the PTFE element. The translational capacity in an unrestrained direction shall be specified on the detailed design drawings.

The coefficient of friction between stainless steel sliding surfaces and lubricated virgin PTFE shall be as per *Section 14.7.2.5* and *Table 14.7.2.5-1* of the *2012 AASHTO LRFD Bridge Design Specifications*.

Notwithstanding *clause 11.6.1.1* of the *CHBDC*, pot bearings shall be designed for all rotations that take place after grouting, plus a fabrication and construction tolerance of 0.005 radians plus an allowance for uncertainties of at least 0.005 radians. These additional rotations are unfactored but shall be considered at SLS and ULS. The rotational capacity about the vertical axis through the centre of the bearing shall be minimum ±1°.

The rotational capacity about any horizontal axis shall be specified on the detailed design drawings.

Rotational bearings shall be capable of resisting the specified lateral loads in any direction in combination with the applicable vertical loads.

The entire bearing assembly, between the sole plate and the base plate shall be replaceable without damage to the structure and without removal of any concrete, welds or anchorages permanently attached to the structure and without lifting the superstructure more than 5 mm. Bearings shall not be recessed into plates that are permanently attached to the structure.

Allowable contact pressures for PTFE sliding surfaces are less than those allowed by the *CHBDC*. Maximum average contact pressures for confined and unfilled PTFE are provided in *SSBC Section 8 Table 8-1*. The average contact pressure at SLS for PTFE sliding surfaces filled with up to 15% mass of glass fibers used to face mating surfaces of guides for lateral restraints shall not exceed 45 MPa.

Pot bearing components shall be metalized or galvanized and shall be attached to galvanized plates by bolting. Surfaces in contact with elastomer shall not be metallized or galvanized.

D4.2.2 Pot Bearing Component Design

The following design considerations are applicable to the specific pot bearing components that will be designed by the bearing supplier. They match the requirements outlined in *SSBC Section 8*: *Bridge Bearings* and are provided here for information purposes only.

Provision for rotation about any horizontal axis shall be by means of a single disc of confined elastomer. Brass rings shall not be considered in determining the effective thickness of the elastomeric disc. The effective thickness of the elastomeric disc to evaluate the rotational capacity shall be limited to the thickness of the disc excluding the brass rings.

Brass sealing rings shall be flat and smooth on all surfaces and conform to the requirements of the *CHBDC*.

The depth of the pot wall shall be such that a minimum vertical distance of 2.5 mm remains between top of the pot wall and the closest point of contact of the brass sealing rings with the pot wall upon rotating the piston an amount equal to the specified rotation at ULS.

The pot and piston surfaces in contact with the confined elastomer shall be lubricated with silicone grease. The bearing shall be sealed by a one-piece continuous preformed closed-cell compressible ring against entry of dirt, dust, and moisture between the elastomer and the pot and piston contact surfaces. Any joint in the ring shall be bonded and the strength shall be at least equal to the strength of the ring.

Except when used as mating surfaces for guides for lateral restraint, the PTFE resin shall be virgin material and shall be used as unfilled sheets and shall contain spherical reservoirs for lubricant pressed into its surface. The diameter of the reservoirs shall not exceed 8 mm measured at the surface of the PTFE, and the depth shall not be less than 2 mm nor more than half the thickness of the PTFE. The reservoirs shall be evenly distributed across the surface of the PTFE and shall occupy 20% to 30% of the surface. PTFE used as mating surface for guides for lateral restraint shall not be dimpled or lubricated. All PTFE elements shall be fully bonded and recessed in a rigid backing material,

All PTFE surfaces except those that act as mating surfaces for guides for lateral restraint or that are subjected to a contact pressure of less than 5 MPa shall be permanently lubricated with silicone grease.

Notwithstanding *clause 11.6.3.6* of the *CHBDC*, for PTFE elements filled with up to 15% by mass of glass fibres and used to face mating surfaces of guides for lateral restraint, the maximum average contact pressure for 'all loads' shall not exceed 45 MPa at SLS and 55 MPa at ULS.

Notwithstanding *clause 11.6.5.4* of the *CHBDC*, the average stress in the elastomer at serviceability limit states loads shall not exceed 30 MPa.

Notwithstanding *clauses 11.6.5.2* and *11.6.6.2.2* of the *CHBDC*, cured elastomeric compounds shall be low temperature Grade 5 and meet the minimum requirements listed in *Table X1* of *AASHTO M251 Plain and Laminated Elastomeric Bridge Bearings (AASHTO M251)* and Shore A durometer hardness of 50.

Fasteners, anchorages and translational elements with lateral restraints shall at least be capable of resisting either of the following lateral loads:

For bearings with a capacity of 5,000 kN or less at serviceability limit state, 10% of the vertical load capacity;

For bearings with a capacity over 5,000 kN at serviceability limit state, 500 kN plus 5% of the vertical load in excess of 5,000 kN.

Guides for lateral restraint shall be arranged to permit the required rotations about both the horizontal and vertical axis. The translational elements of guides for lateral restraint shall be faced with stainless steel and shall provide lateral restraint by sliding against mating surfaces faced with PTFE.

The beneficial effect of friction shall be neglected in proportioning fasteners and anchors, except for slip resistant connections which shall be designed to the requirements of the *CHBDC clause 10.18.2*.

Bearings shall be designed to prevent moisture and dirt from entering the internal surfaces. The bearings shall be fabricated from materials that are durable and are protected against corrosion so as to perform acceptably over the service life of the bridge.

Notwithstanding *clause 11.6.3.6* of the *CHBDC*, the average contact pressure for unfilled PTFE elements, based on the gross area of the PTFE, shall not exceed the values given in the following Table D2:

Table D2 - Average Contact Pressure for unfilled PTFE Elements

Limit State	Permanent Load (MPa)	Total Load (MPa)
SLS	25	35
ULS	40	55

The maximum contact pressures at the extreme edges of flat and curved PTFE elements shall not exceed 1.2 times the values indicated in Table D2 above.

D4.3 Fixed Steel Plate Rocker Bearings

The design of steel plate rocker bearings shall meet the requirements of this document, the *CHBDC* and *SSBC Section 8: Bridge Bearings*. Steel plate rocker bearings shall be fully designed and detailed by the Consultant and all loads, translations and rotations shall be shown on the detailed design drawings.

Fixed steel plate rocker bearings consist of a curved steel rocker plate and a base plate, connected with anchor rods or pintle pins. The curved surface of steel rocker plates shall have a maximum radius of 750 mm. The curved surface of the rocker plates and the top central 250 mm width of the base plates shall be machined to a surface finish of 6.4 μm and a flatness tolerance of 0.001 × the bearing length.

Lateral loads are transferred through steel anchor rods that project through the base plate and the rocker plate. In order to allow the rocker plates to rotate longitudinally (about the transverse axis) while still providing lateral restraint, the anchor rod holes in the rocker plate must be detailed carefully. The bottom half of the holes in the rocker plates shall be the required diameter of the anchor rod plus standard tolerance, while the top half of the holes should be oversized to allow rotation of the rocker plate without bending the anchor rods.

The entire bearing assembly, between the rocker plate and the base plate shall be replaceable without damage to the structure and without removal of any concrete, welds or anchorages permanently attached to the structure and without lifting the superstructure more than 5 mm. A coupler shall be provided under the base plate allowing the top portion of the anchor rod to be removable. The top portion of the anchor rod shall have two nuts torqued against each other. The lower nut shall be finger tight on a 10 mm thick neoprene washer and the upper nut shall be tightened against the first nut. The neoprene washer shall be big enough to cover the top of the hole and shall also ensure that the nuts are not tightened onto the rocker plate so as to prevent free rotation.

Base plates shall be installed level on galvanized steel shim stacks, and shall be grouted prior to casting deck concrete. Due to the large rotational capacity of rocker bearings, there is normally no need for tapered sole plates.

Notwithstanding *clause 11.6.1.1* of the *CHBDC*, fixed steel plate rocker bearings shall be designed for all rotations that take place after grouting, plus a fabrication and construction tolerance of 0.005 radians plus an allowance for uncertainties of at least 0.005 radians at ULS.

D4.4 Plain unreinforced bearing pads

The design of plain unreinforced bearing pads shall meet the requirements of this document and the *CHBDC*. Plain unreinforced bearing pads shall be fully designed and detailed by the Consultant and all loads, translations and rotations shall be shown on the detailed design drawings.

Plain unreinforced bearing pads shall only be used for support of girders where the total movement range of the bearing in any direction over the life of the bridge does not exceed 25 mm or for temporary erection stage bearings for integral bridge girder supports.

For permanent bearings, the bearing support surfaces on the substructure shall be fully detailed with control elevations given on a plan layout. The bearing pads shall be designed and detailed so that they have full contact on both the underside of the girders and on the substructure supports with consideration given to girder camber and other geometric conditions.

Temporary erection stage bearings shall not be included in the design of girder-to-substructure connections for dead and live load support during the in-service life of the bridge.

These shall be as detailed on the standard bridge drawings and shall be laid directly on top of the substructure.

Plain unreinforced elastomeric bearing pads shall be moulded individually, cut from moulded strips or slabs of the required thickness, or extruded and cut to length.

Cured elastomeric compounds for plain unreinforced elastomeric bearing pads used as permanent bearings shall be low temperature Grade 5 and meet the physical and low temperature brittleness requirements listed in *Table X1* and *Section 8.8.4* of *AASHTO M251 Plain and Laminated Elastomeric Bridge Bearings (AASHTO M251)* and shall have a Shore A durometer hardness of 60;

Cured elastomeric compounds for plain unreinforced elastomeric bearing pads used as temporary supports during erection stage shall be low temperature Grade 3, 4, or 5 and meet the physical and low temperature brittleness requirements listed in *Table 1* and *Section 8.8.4* of *AASHTO M251* and shall have Shore A durometer hardness of 50.

D5. GENERAL DESIGN REQUIREMENTS

D5.1 General

Bearing components between the sole plate and the base plate shall be designed and detailed by the bearing supplier in conformance with the most current edition of the *SSBC Section 8*: *Bridge Bearings*. Shop drawings shall be stamped by the supplier’s engineer, who shall be a Professional Engineer licensed to practice in Alberta.

The Consultant shall review the bearing shop drawings for conformance with the requirements of *SSBC Section 8*: *Bridge Bearings*. Any adjustment required for sole plates and base plates shall be approved by the Consultant. The Consultant shall familiarize himself with the requirements of *SSBC Section 8*: *Bridge Bearings*.

Consultant designed components shall be in accordance with this document and typical detail drawing *T-1761-17 (Typical Expansion Bearing Details)*. Materials for bearings shall meet the requirements of the Section 5 (Durability and Materials).

The maximum vertical and horizontal bearing reactions caused by all SLS and ULS load combinations shall be considered. When establishing the horizontal design reactions, the designer shall include anticipated bearing loads at all stages of construction, as well as seismic forces specified in the *CHBDC* Section 4. Wherever practical, reinforced concrete shear blocks shall be used to transfer lateral loads between the superstructure and the substructure, in accordance with Section D5.8 (Concrete Shear Blocks).

All steel bearing components, except stainless steel, shall be hot-dip galvanized or metalized.

No uplift is permitted at bearings as determined for the FLS limit state without Approval. Where uplift for the FLS limit states cannot be avoided, a hold-down device shall be provided that is independent of the bearings, accessible for inspection and easily serviceable for repairs.

D5.2 Information to be Included on Detailed Design Drawings

The Consultant shall provide the following on the detailed design drawings:

1. Bearing layout;
2. Bearing types and the required number of each type;
3. Bearing schedule showing design loads, and translation and rotation requirements. A sample bearing schedule is provided in Figure D1 below;
4. Temperature setting graphs for positioning expansion bearing components according to the girder temperature after girder erection and prior to grouting. For prestressed girders, the setting graph shall include one graph for initial construction, and a second long term graph with allowances for post-tensioning and long term movements due to shrinkage and creep;
5. Sole plate details including tapered dimensions and attachment details;
6. Base plate details including anchor rods, bearing attachment details, and grout pad details;
7. Concrete shear block details including reinforcement details and/or details to provide lateral restraint;
8. Bearing Setting Elevation Table showing top of sole plate elevations plus two empty rows for bearing height and top of grout pad elevations, to be filled in after bearing heights are obtained from the contractor.

Figure D1 – Sample Bearing Schedule

BEARING SCHEDULE											
Bearing				1		2		3		4	
				Value	LC[4]	Value	LC	Value	LC	Value	LC
Design Bearing Reaction (kN)	SLS	Vert.	max.								
			perm.								
			min.								
		Long.									
		Trans.									
	ULS	Vert.	max.								
			perm.								
			min.								
		Long.									
		Trans.									
	FLS[1]	Vert.	live								
Design Bearing Movement[2] (mm)	SLS	Long.									
		Trans.									
Design Bearing Rotation[3] (rad)	SLS	Long.									
		Trans.									
	ULS	Long.									
		Trans.									

Notes:

1. The component of the vertical reaction at FLS due to live load only.
2. Design bearing movement shall include the maximum unfactored movements, including post tensioning shortening and long term creep, obtained from analysis, plus the excess travel capacity required in the design guidelines.
3. Design bearing rotation shall include the maximum factored rotation obtained from analysis, plus allowances for uncertainties, at SLS for elastomeric bearings, and at ULS for pot bearings.
4. For ULS load effects, indicate governing load case from *Table 3.1* of the *CHBDC*.

D5.3 Tolerance and Uncertainties for Bearing Rotation

D5.3.1 Current Code and Specification Requirements

There is considerable disagreement between the *CHBDC*, the *AASHTO LRFD Bridge Design Specifications (AASHTO LRFD)*, and the *Ontario Provincial Standard Specification 1203 Material Specification for Bearings – Rotational and Sliding Surface – November 2008 (OPSS 1203)*, on what allowances should be made for fabrication and installation tolerances and for unknown or unaccounted for rotations.

The *CHBDC Clause 11.6.1.1* specifies that “for bearings other than elastomeric bearings, the design rotation θu, shall be taken as the sum of the rotations due to ULS loads and tolerances in fabrication and installation, plus 1° (0.0175 rad)”. No rotational tolerance requirement has been specified for elastomeric bearings.

AASHTO LRFD Section 14.4.2.1 specifies that plain and reinforced elastomeric bearings shall be designed for the applicable SLS rotations plus an allowance for uncertainties of 0.005 radian. *AASHTO LRFD Section 14.4.2.2* specifies that pot bearings shall be designed for the applicable ULS rotations plus a fabrication and installation tolerance of 0.005 radian, plus an allowance for uncertainties of another 0.005 radian.

OPSS 1202 Section 07.05 specifies that the fabrication of elastomeric pads allows a deviation from the plane parallel to theoretical surface of 0.005 radian.

OPSS 1203 Section 04.01.03 specifies that an additional 1.2° (0.02 rad) be added to the ULS rotation to account for fabrication and installation tolerances and uncertainties.

D5.3.2 Evaluation of Fabrication and Installation Tolerances

There are several different aspects of bridge component fabrication and installation that will affect the design rotation of bridge bearings. The three factors that most significantly affect the design tolerance of bearings are the initial set of the bearing with respect to rotation, the girder camber at erection, and the initial set of the bearing with respect to elevation.

The *CHBDC Clause 11.6.1.1* specifies that bearings shall be set to the specified plane within a tolerance of ±0.2° (±0.0035 rad). Within the Department’s practices, this would apply to pot bearings which are set on top of shim stacks to a level or specified plane, but would not apply to elastomeric pads with rocker pintels, which will rock the bearing pad into uniform contact with the underside of the girder or tapered sole plates attached to the girder bottom flange.

For steel girders, *SSBC Section 6: Structural Steel* specifies that girders should be fabricated to the design girder camber within a tolerance of ±(0.2L + 3) mm, where L is the length of the girder section in meters. The end rotation tolerance is directly proportional to the girder camber tolerance, and therefore increases as the girder length decreases. For a worst case scenario of a 10 m section length, the end rotation tolerance related to camber is 0.0016 rad based on elastic deflection of a simply supported member. For steel girders, this is likely a conservative value as steel fabricators are generally able to very tightly control camber. Furthermore, it is very unlikely that a tolerance sensitive bearing would be used on a girder as short as 10 m.

For prestressed concrete girders, *SSBC Section 7: Precast Concrete Units* specifies a maximum camber tolerance of ±(20L ÷ 50) mm which results in an end rotation tolerance of 0.0013 rad. However, there are many factors such as variations in design and construction sequence, concrete modulus, creep and shrinkage properties, and age, which will affect girder camber. The fabrication tolerance specified in the *SSBC* cannot account for these variables, so additional allowances are required.

SSBC Section 6: Structural Steel specifies that bearings are to be set to the exact elevation specified on the detailed design drawings, but poor quality control could still result in some discrepancy.

Long term movements caused by substructure translation or rotation are difficult to accurately predict at the time of design.

For elastomeric bearings, the Department's practice of using self-rocking pintles ensures uniform contact with the underside of the girder or sole plate, and eliminates any need for including any construction tolerances and uncertainties at the time of girder erection. Therefore, a tolerance allowance of 0.005 radians at SLS is considered adequate.

It should be recognized that rotation capacity demand due to construction misalignment and other uncertainties in many cases is much larger than the sum of other calculated rotations. As failure of pot bearings can lead to serious damage to bridge structural components and expensive repairs, a generous tolerance should be allowed for. For pot bearings, which are set to the specified plane on shim stacks, a fabrication and construction tolerance of 0.005 radians plus an allowance for uncertainties of at least 0.005 radians shall be provided. These additional rotations are unfactored but shall be considered at SLS and ULS.

Fixed steel rocker bearings have a very large rotational capacity in the longitudinal girder direction and do not require consideration for tolerances.

D5.4 Sole Plates, Base Plates and Grout Pads

The thickness of tapered sole plates shall be sufficient to transfer all loads without significant distortion of the sole plate or the girder bottom flange. The thickness shall also be sufficient to provide threaded holes long enough to develop full torquing capacity of A325 connecting bolts used to connect the sole plates to the bottom flanges of steel girders.

For precast girders, attachment of sole plates shall be by welding in the longitudinal direction along the edge of the shoe plate. All galvanizing damaged by field welding shall be metallized after welding. The weld design size shall account for weld contamination effects (which occurs if the galvanizing is not removed in advance of welding). This typically results in an increase in the weld size. Transverse welding requiring underhand welding shall not be permitted. Transverse ends shall be sealed with Sikaflex 1a or an Approved equivalent caulking material in accordance with typical detail drawing *T-1761-17 (Typical Expansion Bearing Details)*.

For weathering steel girders, sole plates shall be connected to the bottom flanges with galvanized A325 bolts. Bolted connections shall be designed as slip-critical connections and bolt spacing shall meet sealing requirements. The bolts shall be installed through holes in the girder bottom flange into threaded holes in the sole plate. Girder bottom flanges at bearing connections shall be prime coated all around (bottom, top and edges) with an Approved organic zinc epoxy primer meeting the requirements of a Class B coating. The galvanized top surface of the sole plate shall be hand wire brushed to the requirements of a Class C surface. The slip coefficient k_s from the *CHBDC Table 10.9* shall be taken as 0.4.

The thickness and size of the base plate shall be sufficient to distribute loads at all stages of construction, and sufficient to distribute all SLS and ULS loads through the grout pad to the concrete substructure at completion.

Base plates and anchor rods shall be grouted after girders are erected, elevations are checked and confirmed, and before any additional deck deadload is applied. After girder erection and immediately before grouting, the longitudinal location of bearing base plates shall be adjusted in accordance with the bearing setting charts.

The grout pad shall have a nominal thickness of 80 mm and shall be 75 mm larger than the base plate all around the perimeter. The grout pad shall be keyed into the substructure 40 mm and project above the substructure 40 mm. This will raise the bearings above the top of the substructure and will also allow for some adjustment of girder elevations if necessary.

The underside of galvanized base plates in contact with grout shall have the contact surfaces protected by a barrier coating in accordance with similar requirements set forth in *SSBC Section 12: Bridgerail*.

Shim plates used for shim stacks shall be CSA G40.21 Grade 300W or 350W steel and shall be hot-dip galvanized.

D5.5 Tapered Sole/base Plates and Bearing Setting Plane

Tapered sole plates are required for steel reinforced elastomeric bearings and pot bearings to bring the sliding surface as close to level as possible, bearing in mind there can be uncertainties in girder cambers and deflections, especially in the case of precast concrete girders. When the design taper is less than 0.003 radian, consideration may be given to eliminating the tapered sole plate and increasing the rotational capacity of the bearing by a corresponding amount instead.

When finger plate abutment expansion joints or cover plated abutment expansion joints are used, the sole plates and/or base plates shall be tapered such that the sliding plane of the abutment expansion bearings is parallel to the roadway grade for proper functioning of the joint. Effects of longitudinal forces generated by the inclined sliding bearings on the structure shall be addressed.

Tapered sole plates are not typically required for fixed steel plate rocker bearings due to their large rotational capacity.

The taper of the plate shall be calculated based on the following:

- Other bearing components are of uniform thickness;
- Unfactored theoretical girder cambers and rotations for all dead loads and permanent deformations applied after girder erection, without an allowance for tolerances and uncertainties (the theoretical rotation due to the long term component of camber growth from shrinkage and creep, after the bearings are grouted in place, shall be included);
- A correction for the roadway grade at the centreline of bearing is applied;
- No allowance for rotation due to cyclical thermal changes is applied.

D5.6 Horizontal Movement and Sliding Surfaces

Elastomeric bearings designed to accommodate thermal movements through shear deformations shall assume an installation temperature of +20°C.

Expansion bearings with a PTFE and stainless steel sliding surface shall be centred at -5°C.

Notwithstanding the *CHBDC Clause 11.6.3.7*, the coefficient of friction between stainless steel and PTFE sliding surfaces shall be as per *AASHTO LRFD Section 14.7.2.5* and *Table 14.7.2.5-1*. For reinforced elastomeric bearings, use values for unfilled PTFE. For pot bearings, use values for dimpled lubricated PTFE. For lateral guides on pot bearings, use values for filled PTFE.

Sliding bearings shall be designed for all relative horizontal displacements between the superstructure and substructure at the bearing location. In addition, sliding bearings shall provide an excess travel capacity in each direction equal to 25% of the theoretical thermal movement, but not less than 25 mm in the longitudinal direction and 10 mm in the transverse direction. When establishing design movements, all movements described in the *CHBDC Clause 3.9* shall be considered, including elastic shortening due to post-tensioning, and long term movements resulting from creep, shrinkage, relaxation, substructure movement and any other internal or external causes.

The stainless steel sliding plate shall be AISI Type 304 with No. 8 (0.2 μm) mirror finish, with a minimum thickness of 3.2 mm, and shall be shop welded to the bottom of the sole plate with matching stainless steel electrodes.

D5.7 Load Bearing Plates - Flatness and Machining Requirements

Steel load bearing plates in contact shall be machined to a surface finish of 6.4 μm and a flatness tolerance of 0.001 × longer bearing plan dimension. Surfaces in contact with an elastomeric pad (except sliding surfaces), grout, or cast-in-place concrete do not require machining. Where required, machining shall be performed prior to hot-dip galvanizing. Where the galvanizing process may cause distortion, metalizing shall be used instead.

D5.8 Concrete Shear Blocks

Concrete shear blocks are the Department's preferred approach for transferring lateral loads from the superstructure to the substructure and shall be used wherever practical. In the past, anchor rods cantilevered out from the concrete base have been used to resist lateral loads was provided by anchor rods. However, Alberta experience is that these anchor rods often get bent or broken and due to limited accessibility are often very difficult to repair or replace. Independent concrete shear blocks between girders have proven to be more effective than anchor rods for transferring horizontal forces into the substructure. These concrete shear blocks provide higher resistance to lateral forces are usually easy to access for any required repairs. The use of shear blocks also limits the function of bearings to the transfer of gravity loads and rotations only. This not only reduces initial bearing cost by removing lateral guide components, but will also increase bearing service life by eliminating components that are more susceptible to damage. Concrete shear blocks shall be designed in accordance with the details shown on typical detail drawing *T-1761-17 (Typical Expansion Bearing Details)*.

D5.9 Design For Jacking and Bearing Replacement

Bridges and bearings shall be designed and detailed to allow for bearing replacement. Typical bearing replacement includes simultaneously jacking all girder lines to avoid damage to the deck, diaphragms, and deck joints. Jacking locations shall be clearly shown on the detailed design drawings, along with assumed jack and distribution plate sizes. The following assumptions shall be made for a typical bearing replacement procedure.

- All girder lines are simultaneously jacked to avoid damage to deck, diaphragms, and deck joint components.
- After raising the structure, jacks are shimmed around the piston or locked to prevent catastrophic hydraulic failure.
- Bearings are pulled and replaced one at a time with overhead traffic being directed away from the bearing being removed and replaced.
- At abutments, jacking shall typically take place in front of the bearing, and the bearings shall be pulled out from the side.
- For girders with a single pier bearing, jacks shall be placed in pairs on either side of the bearing and the bearings shall be pulled out from the side.
- For precast concrete girders with double bearings, the pier diaphragm shall be designed for girder jacking.

D6. REFERENCES

1. AT. *Specifications for Bridge Construction*. Alberta Transportation, Edmonton, AB (2016).

2. AASHTO. *AASHTO LRFD Bridge Design Specifications, Customary U.S. Units*, 6th Edition. American Association of State Highway and Transportation Officials, Washington, DC (2012).

3. AASHTO. *Standard Specifications for Highway Bridges*, 17th Edition. American Association of State Highway and Transportation Officials, Washington, DC (2002).

4. CSA. (2014). *CSA-S6-14 Canadian Highway Bridge Design Code*. Canadian Standards Association, Toronto, ON.

5. MTO. *OPSS 1202 Material Specification for Bearings – Elastomeric Plain and Steel Laminated*. Ontario Ministry of Transportation, Toronto, ON (2008).

6. MTO. *OPSS 1203 Material Specification for Bearings – Rotational and Sliding Surface*. Ontario Ministry of Transportation, Toronto, ON (2008).

7. Stanton, J.F. et. al., *NCHRP Report 596: Rotation Limits for Elastomeric Bearings*. Transportation Research Board, National Research Council, Washington, DC (2008).

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APPENDIX E – SKEWED STRIP SEAL DECK JOINT DESIGN EXAMPLE

E1. Introduction

Deck joints shall be selected and designed in accordance with Section 11 (Deck Joints) and the appropriate standard drawings. The standard designs may need to be adjusted for some special site conditions. The following design example illustrates the process of calculating the allowable movement, as measured parallel to the roadway, for a Type 1 Strip Seal deck joint given assumed skew, maximum and minimum effective temperatures, and the effective joint installation temperature. The same procedure is applicable for the cover plated V-seal deck joint but with the limiting movement values applicable to that deck joint type.

E2. Design Example

For this example, assume the following:

i. Maximum effective Temperature = +40 ºC;
ii. Minimum effective Temperature = -40 ºC;
iii. Effective temperature at time of joint seal installation = +10 ºC;
iv. Skew angle $\theta = 25°$ and roadway has a tangent horizontal alignment;
v. Multiweb Strip seal deck joint movement limitations:
   - Maximum allowable movement range perpendicular to joint = 55 mm based on maximum and minimum allowable joint gaps of 115 mm and 60 mm respectively;
   - Maximum allowable movement parallel to joint (shearing of seal) is 13 mm.
vi. Movement parallel to roadway $= \Delta$;
vii. Movement perpendicular to deck joint $= \Delta_p = \Delta \cos\theta$;
viii. Movement parallel to deck joint $= \Delta_t = \Delta \sin\theta$

See Figure E1 for illustration of movement directions.

Step 1: Calculate joint movement limitation measured parallel to roadway based on movement constraint measured perpendicular to joint:

- Movement range limit perpendicular to joint is 55 mm.
- Calculate $\Delta = {}^{\Delta_p}/_{\cos\theta}$ $\quad \Delta = {}^{55}/_{\cos 25} = 61\ mm$
- Therefore the limit to joint movement measured parallel to roadway based on the constraint for movement measured perpendicular to joint is 61 mm.

Step 2: Calculate joint movement limitation measured parallel to roadway based on movement constraint measured parallel to joint:

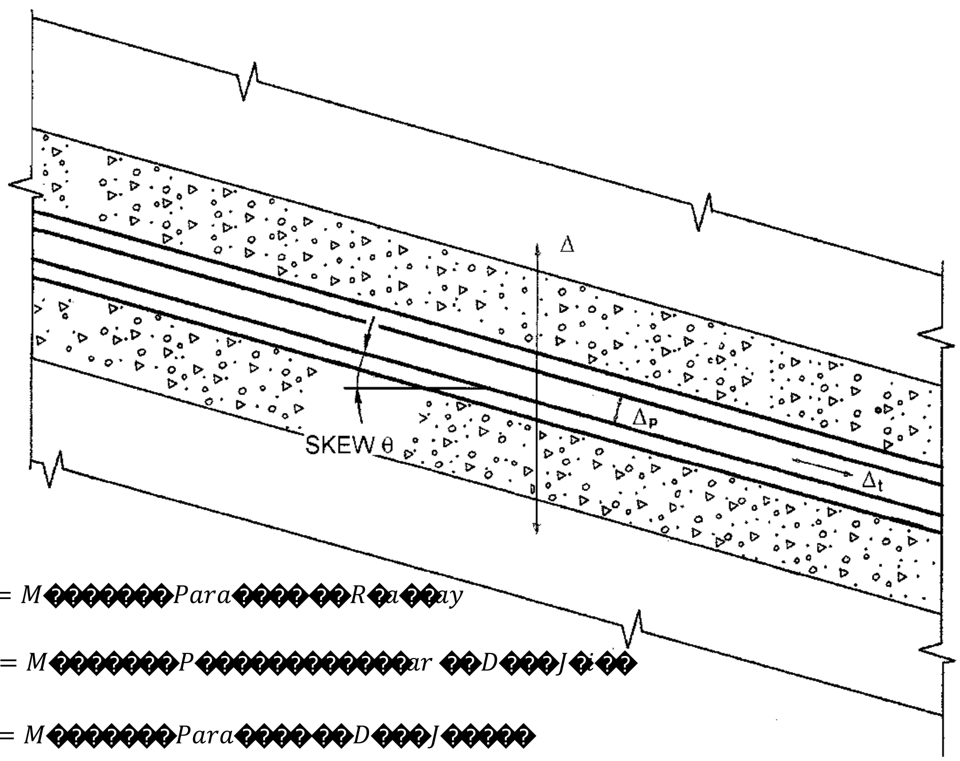
- Maximum allowable movement parallel to joint is 13 mm.
- Calculate $\Delta = {}^{\Delta_t}/_{\sin}$ θ

$\Delta = {}^{13}/_{\sin 25} = 31\ mm$

- However, this value does not account for the seal setting temperature. At the time of seal installation the strain in the seal in a direction parallel to the joint is zero. The temperature range that creates strain in this direction is based on the maximum temperature differential that takes place after seal installation. In this example, the maximum temperature differential is 10 ºC - (-40 ºC) = 50 ºC. The total temperature differential for the joint is 40 ºC - (-40 ºC) = 80 ºC. Therefore, the ratio of maximum temperature differential after seal installation to total temperature differential for the joint is 50 ºC / 80 ºC = 0.625. The resulting joint movement parallel to roadway required to create a 13 mm joint movement measured parallel to the joint is 31 mm / 0.625 = 49 mm. (Similarly, by going through the same calculations with an installation temperature of 15 ºC, the allowable movement parallel to roadway would be only 45 mm).

Step 3: Determine limiting joint movement measured parallel to roadway from steps 1 and 2:

- Based on calculations from steps 1 and 2, the minimum value is 49 mm. Therefore, the maximum allowable joint movement as measured parallel to the roadway is 49 mm for the 25º skew condition, based on the assumed maximum, minimum, and seal installation temperatures.



$\Delta =$ M[illegible]Para[illegible]R[illegible]a[illegible]ay

$\Delta_p =$ M[illegible]P[illegible]ar [illegible]D[illegible]J[illegible]

$\Delta_t =$ M[illegible]Para[illegible]D[illegible]J[illegible]

Figure E1: Deck Joint Movement Directions

APPENDIX F – PARTIAL DEPTH PRECAST CONCRETE DECK PANELS

F1. INTRODUCTION

Precast concrete partial depth deck panel construction is not the Department's preferred deck system due to a number of reasons including the potential for cracking that may affect long term performance and the additional level of detailing required. The Department will only consider their use, on a trial basis, if submitted for Approval through the value engineering process by the contractor with demonstrated economic advantages. Submissions for precast concrete partial depth deck panels shall meet the following design, fabrication and construction specifications.

F2. DESIGN

Deck slabs using precast concrete partial depth deck panels shall be permitted with the following design requirements:

1. Deck slabs using precast concrete partial depth deck panels shall consist of a cast-in-place concrete deck slab on precast concrete partial depth deck panels;

2. The cast-in-place concrete deck slab shall be designed to be fully composite with the precast concrete partial depth deck panels;

3. The minimum composite deck slab system (precast concrete partial depth deck panels and cast-in-place concrete deck together) thickness shall be the greater of the girder spacing divided by 15.0 or 225 mm (ie. the minimum allowable combined thickness of the precast concrete partial depth deck panels and the cast-in-place deck is 225 mm). In addition, the following shall be satisfied:

    a. The precast concrete partial depth deck panels shall have a minimum thickness of 90 mm;

    b. The cast-in-place concrete portion of the composite deck slab system shall have a minimum thickness of 115 mm; and

    c. The cast-in-place concrete portion of the composite slab system shall have sufficient thickness to satisfy all reinforcement cover requirements and maintain adequate spacing between reinforcement bars.

4. The precast concrete partial depth deck panels shall be fully pretensioned and the stresses in the precast concrete partial depth deck panels shall not exceed the following:

    a. From transfer until the 28 day strength is attained:

        - Compression: 0.6 f'_{ci};

        - Tension: 0.5 f_{cri};

    b. After the 28 day strength is attained and at serviceability limit states:

        - Tension: f_{cr};

c. The average compressive stress in the precast concrete partial depth deck panels at pretension strand release shall be ≤ 7.0 MPa;

5. The empirical design method in accordance with the *CHBDC Clause 8.18.4* shall not be permitted for design of the composite deck slab system using precast concrete partial depth deck panels;

6. The composite deck slab system shall be designed using flexural design methods based on elastic moments:

    a. For ‘square’ (ie. deck span is perpendicular to girder axes, not skewed) deck slabs continuous over three or more girder lines, the maximum positive and negative transverse moments shall be determined using the simplified elastic method in accordance with the *CHBDC Clause 5.7.1.2*, with P adjusted to 112 kN to correspond with the CL-800 Design Truck. These moments shall be used to design the maximum transverse positive moment reinforcing requirements in the precast concrete partial depth deck panels and the composite cast-in-place concrete slab as well as the transverse maximum negative moment reinforcing requirements in the cast-in-place concrete portion of the deck slab. In addition, reinforcement development and cut-off locations shall be determined using moment envelopes based on elastic analysis;

    b. For curved or skewed bridges, all moments shall be determined by elastic analysis;

    c. For all bridges the following minimum transverse positive moment reinforcing shall be provided over supporting girder lines:

        - In addition to the required pretensioning strands, transverse stainless steel reinforcing bars, with a minimum reinforcement ratio “*ρ*” of 0.003, shall be provided throughout the precast concrete partial depth deck panels and shall project over the girder lines and into the cast-in-place concrete portion of the composite deck slab system. The reinforcement ratio “*ρ*” shall be calculated for “*d*” equal to the effective depth of the composite deck slab system. The spacing of the transverse stainless steel reinforcing bars shall not exceed 300 mm;

        - At interior girder lines, the transverse stainless steel reinforcing bars shall project out of the precast concrete partial depth deck panel edges and over the girder flanges as required to provide a full lap splice (ie. adequate to develop the bar yield capacity) with the bars projecting from opposing precast concrete partial depth deck panels supported on the same girder. At exterior girder lines, the transverse stainless steel reinforcing bars shall be extended at least one full development length beyond the exterior girder centreline;

7. Precast concrete partial depth deck panels shall consist of Class HPC Concrete.

8. The composite deck slab system shall conform to the following:

    a. The precast concrete partial depth deck panels shall have a minimum age of 45 days and a maximum age of 120 days when the cast-in-place portion of the deck is cast;

b. The cast-in-place concrete portion shall have 15M continuous bottom longitudinal reinforcing bars (parallel to girders lines) spaced at a maximum of 300 mm on centre placed directly on top of the precast concrete partial depth deck panels;

9. Pretensioning strands shall be 9.5 mm diameter;

10. Pretensioning strands shall not project beyond the edges of the precast concrete partial depth deck panels;

11. Pretensioning strands cast into the precast concrete partial depth deck panels shall be uncoated steel;

12. With a steel girder superstructure, the following additional provisions shall apply:

    a. The precast concrete partial depth deck panel length shall be set to provide a minimum 75 mm long bearing zone (as measured perpendicular to the girder line) on the haunch concrete. A minimum 50 mm thick haunch shall be provided beneath the underside of the precast concrete partial depth deck panels;

    b. The girder top flange shall have a minimum width of 450 mm;

    c. Shear studs attached to the girder top flange shall project above the top surface of the flange to provide at least 25 mm clearance between the underside of the shear stud head and the top of the precast concrete partial depth deck panels. If this clearance is not met, additional ;

13. With precast concrete girder superstructures, the following additional provisions shall apply:

    a. For NU girders or any other girder shape where the top flange is less than 150 mm thick at the flange edges, the precast concrete partial depth deck panel length shall be set to provide a minimum 200 mm long bearing zone (as measured perpendicular to the girder line) on the haunch concrete. For all other girders, the precast concrete partial depth deck panel length shall be set to provide a minimum 75 mm bearing zone (as measured perpendicular to the girder line) on the haunch concrete. A minimum 50 mm thick haunch shall be provided beneath the underside of the precast concrete partial depth deck panels;

    b. Stirrups projecting from the top girder flange shall project above the top surface of the flange to provide at least 25 mm clearance between the underside of the stirrup tops and the top of the precast concrete partial depth deck panels;

14. The maximum allowable haunch height with precast concrete partial depth deck panels shall be 150 mm, as measured at the outer edge of the girder flange. If the haunch exceeds this height, additional analysis is required to determine whether the unreinforced section of the haunch (ie. that part of the haunch supporting the precast concrete partial depth deck panel) requires additional reinforcing;

15. Vertical bleed holes shall be provided through the precast concrete partial depth deck panels and evenly distributed along the two supported panel edges. The holes shall be not less than 25 mm diameter, and shall be located adjacent to the formed edge of the haunch to facilitate the escape of entrapped air;

16. When a bridge includes a traffic separation barrier between a sidewalk and the traffic, any reinforcement required to anchor the separation barrier to the deck shall be cast into the precast concrete partial depth deck panels and project into the barrier; and

17. No portion of any hardware associated with deck formwork, including deck overhang formwork, shall be visible after removal of all formwork.

F3. FABRICATION AND CONSTRUCTION

Unless otherwise noted in this Appendix, all the requirements of the *BSDC* shall apply to the design of deck systems using partial depth precast concrete deck panels.

Unless otherwise noted in this Section, all the requirements of *SSBC Section 7: Precast Concrete Units* shall apply to the supply, manufacture, delivery and erection of partial depth precast concrete deck panels.

Unless otherwise noted in this Section, all the requirements of *SSBC Section 4: Cast-in-place Concrete* shall apply to the construction of deck systems using partial depth precast concrete deck panels.

F3.1 MANUFACTURE

The panels shall be cast flat.

All edges of the panel shall have a minimum 20x20 mm chamfer, except the transverse joint which shall have a 55x55 mm chamfer along the top edges.

Panel identification tags cast into the surface of the precast concrete partial depth deck panels are not permitted. Panel identification methods shall be acceptable to the Department.

F3.1.1 Stressing Strand

All pretensioning strands shall be cut flush with the precast concrete partial depth deck panel edges, and the ends of the pretensioning strands shall be sealed with Sikadur-31 or an Approved equivalent.

F3.1.2 Surface Finish

The top surface of precast concrete partial depth deck panels shall be clean, free of laitance, and roughened to 3 mm amplitude with spacing not greater than 15 mm with grooves parallel to strands. Formed chamfer surfaces that will be in contact with cast-in-place concrete shall be sandblasted to remove all laitance and uniformly expose aggregate particles.

F3.1.3 Tolerances for Panels

Precast concrete partial depth deck panels shall meet the following tolerances:

a) Panel lengths: ± 5 mm (as measured perpendicular to the girder lines);

b) Panel widths: ± 10 mm (as measured parallel to the girder lines);

c) The maximum difference in plan view diagonal dimensions (squareness) of rectangular panels shall not be greater than 3.5 mm per meter of diagonal length;

d) Thickness of panel: + 5 mm, - 3 mm;

e) Pretensioning strands shall be located at the centroid of the panel with a vertical tolerance of +0 mm, - 3 mm, measured from the soffit and a horizontal tolerance of ± 10 mm;

f) Deviation from straightness of panel edges along the transverse joint between adjacent panels shall not exceed 1.5 mm per metre length;

g) Vertical bowing of panels out of plane, after casting and immediately prior to erection, in the direction of measurement, shall not be greater than the panel length/360 or the panel width/360, whichever is less, and in no case shall it exceed 10 mm maximum; and

h) Warping of panels shall not be greater than 5 mm per metre of distance from the nearest adjacent corner.

Tolerance measurement results shall be provided to the Department forthwith, upon request. Panels not meeting any of the tolerances listed above shall be rejected.

F3.1.4 Defects and Deficiencies Causing Rejection

A precast concrete partial depth deck panel having any one of the following defects or deficiencies shall be rejected:

a) Panels with honeycombing or spalls when the depth exceeds 15 mm or when the area of defect exceeds 25 mm x 25 mm;

b) Panels with any voids or spalls in the bottom of the panel;

c) Panels with any crack located parallel to or over the pretensioning strands or reinforcing steel;

d) Panels with any crack at the edges and / or with cracks at the bottom; and

e) Panels with cracks that are deeper than 25 mm and/or wider than 0.1 mm.

F3.2 ERECTION AND CONSTRUCTION

The precast concrete partial depth deck panels shall be erected on the girders with temporary supports. The precast concrete partial depth deck panels shall be erected so that the transverse joints between adjacent panels are never greater than 5 mm. All transverse joints shall be sealed with Sikaflex 15LM or an Approved equivalent to prevent mortar leakage.

The Contractor shall survey all girders at locations corresponding with those detailed on the camber diagram and determine girder haunch dimensions required to achieve design grades. The Consultant shall perform an additional and independent haunch dimensional check to confirm deck thickness and grades prior to deck reinforcing steel placement. In the event that actual girder camber values vary significantly from the estimated values indicated on the detailed design drawings, the Contractor may raise or lower the grades when accepted by the Consultant.

All precast concrete partial depth deck panel system formwork drawings shall be prepared and sealed by a Professional Engineer registered in the Province of Alberta, and inspected prior to placing concrete to confirm conformance with the detailed design drawings. The Contractor shall design and install support brackets such that no damage to girder flanges and webs will result. Where brackets bear against girder webs, the Contractor shall protect the contact surface with timber or neoprene softeners. No drilling of additional holes, or any other modifications including field welding, shall be made to the superstructure elements. Effects of concentrated loads on thin webs shall be checked, and where necessary, sufficient means shall be provided to distribute or carry such concentrated loads to the supporting flanges or stiffeners. Formwork for deck overhangs, curbs, sidewalks and parapets shall be fabricated so that the lines and grades shown on the detailed design drawings are achieved, with adjustments made where necessary to compensate for variances in girder dimensions, positioning, alignment and sweep. All lifting hooks and deck panel levelling bolts shall be cut flush with the top of the deck panel after the profile, deck concrete thickness and girder haunch dimensions have been completed, checked and accepted by the Consultant, and before reinforcing steel placement.

The haunches shall be formed to be flush with the edge of the girder flanges. Formwork shall be sealed against girder flanges such that concrete paste leakage does not occur. All haunch forming material, including sealants, shall be completely removed after casting the deck to fully expose the haunch concrete.

No portion of any hardware associated with deck formwork, including deck overhang formwork, shall be visible after removal of all formwork. For precast concrete girder superstructures, anchors for the exterior deck overhang formwork may be cast into the girder top flanges above the web. For steel girder superstructures, anchors for the exterior deck overhang formwork may be shop attached to the girder top flanges. Field welding or drilling of the girders or precast concrete partial depth deck panels shall not be permitted.

Prior to the placement of deck reinforcing steel and prior to the placement of deck concrete, the surfaces of precast concrete partial depth deck panels, girder flanges, and all formwork shall be thoroughly cleaned with high pressure water. Cleaning shall be completed in a controlled and progressive manner from the high to low ends of the deck pour area in both transverse and longitudinal directions. Appropriate wash water drains shall be incorporated into haunch and bulkhead formwork. All surfaces shall be free of dirt, debris or foreign materials. All hardened concrete surfaces to receive deck concrete shall be brought to and kept in a saturated surface dry condition, free of standing water, a minimum of 2 hours prior to concrete placement.

The deck and haunch concrete shall be cast monolithically in a two stage process to ensure full consolidation of concrete in the haunch area.

The first stage shall include placement of concrete in the haunch area and over the girder top flange in continuous strips. The depth of the first stage pour shall be above the bottom surface of the precast concrete partial depth deck panel, but shall not exceed the top surface of the precast concrete partial depth deck panel, and shall not extend in front of the second stage pour by more than 6 m. Placement and consolidation of concrete in the first stage shall be completed in such a manner that entrapped air on the vertical and horizontal formed surfaces of the haunch is minimized.

The second stage shall include placement of the remaining deck concrete. Concrete placement shall occur in a timely manner as to not result in any cold joint between the first and second stages. If cold joints are produced, the entire deck section shall be removed and replaced including but not limited to the cast-in-place HPC concrete, steel reinforcing bars and precast concrete partial depth deck panels.

Voids, cavities, or areas of honeycombing found in the haunch concrete meeting the following parameters shall be repaired by the Contractor:

a) Any defects with depth greater than or equal to 20 mm;

b) Defects greater than or equal to 25 mm high or 25 mm wide x 10 mm deep;

c) 10 or more defects between 20 mm wide or 20 mm high x 15 mm deep per lineal metre; or

d) 30 or more defects between 10 mm wide or 10 mm high x 15 mm deep per lineal meter.

Proposed repair procedures shall be submitted for review and acceptance by the Department.

All lifting hooks for the precast concrete partial depth deck panels shall project through the top surface of the precast panel and shall be removed by cutting flush with the top surface of the precast panel after erection.

F4. REFERENCES

1. AT. *Standard Specifications for Bridge Construction, Edition 16*. Alberta Transportation, Edmonton, AB (2017).

2. CSA. (2014). *CSA-S6-14 Canadian Highway Bridge Design Code*. Canadian Standards Association, Toronto, ON.