Superceded by Version 8.0
April 31, 2017

Bridge Structures Design Criteria Version 7.0

Original: September 09, 1986
Date Published: May 31, 2012

Government

Superceded by Version 8.0
April 31, 2017

P R E F A C E

The Alberta Transportation Bridge Structures Design Criteria (BSDC) is a summary of items that the Bridge Engineering Section consider important in the design of Alberta bridges. Some items are extractions from the *CAN/CSA-S6 CODE*, highlighted because of their fundamental importance; others are defined to the Department's particular preference. There are other items including geometry, detailing, and materials, that are defined to produce a reasonably uniform product that will simplify construction, reduce maintenance, and extend service life as much as possible. In the Department's experience, these items have been found to reduce design, construction, and maintenance problems; and assure the best quality at reasonable cost.

Maintenance, repair and upgrading of the Province's bridges is a major ongoing activity, and uniformity of design, detailing and drawing systems is an important factor in achieving efficiency. It is intended that this document will help facilitate the achievement of this goal. However, it is not the intent of the document to limit progress or discourage innovation, and Consultants are encouraged to explore all engineering options they deem appropriate for a specific site.

The following BSDC refers to many Department documents, which can be found on the Department's website. The latest version of these documents shall be used. The following websites are provided for reference:

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

Specifications for Bridge Construction (SBC): http://www.transportation.alberta.ca/2653.htm

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

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

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

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

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

Superceded by Version 8.0
April 31, 2017

Table of Contents

Superceded by Version 8.0
April 31, 2017

APPENDICES:

APPENDIX A	INTEGRAL ABUTMENT DESIGN GUIDELINES
~~APPENDIX B~~	~~ABC STRUCTURES DESIGN GUIDE~~ (REMOVED FROM CIRCULATION, ALL DESIGNS TO CONFORM WITH THE REQUIREMENTS OF THE LATEST VERSION OF *CAN/CSA-S6 CODE SECTION 7 – BURIED STRUCTURES*)
APPENDIX C	PROTECTION SYSTEM STANDARDS FOR BRIDGE COMPONENTS
APPENDIX D	TYPICAL ABUTMENT DETAILS – SKETCHES SK-9 TO SK-19
APPENDIX E	DESIGN-BUILD SPECIFICATIONS FOR PARTIAL DEPTH PRECAST CONCRETE DECK PANELS
APPENDIX F	DESIGN EXAMPLE FOR STRIP SEAL DECK JOINT MOVEMENT WITH SKEW
APPENDIX G	BRIDGE BEARING DESIGN GUIDELINES

Superceded by Version 8.0
April 31, 2017

1. DESIGN

Highway and Pedestrian Structures shall be designed in accordance with the latest version of “CAN/CSA-S6 Canadian Highway Bridge Design Code” (*CAN/CSA-S6 CODE*), including the latest supplements, and in accordance with other codes and standards, referred to in this “Bridge Structures Design Criteria” (BSDC), or with the prior written approval of the Department. Exceptions to *CAN/CSA-S6 CODE* requirements are noted as follows and in the rest of this document:

1. Design exceptions to the *CAN/CSA-S6 CODE* and the BSDC requirements may be justified under special circumstances. Such design exceptions shall be fully documented by the Consultant, reviewed and approved in writing by the Bridge Engineering Section.

2. Live load distribution factors used for girder design shall not be less than the empirical factors specified by *CAN/CSA-S6 CODE*. 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 design drawings.

3. In *Clause 5.7.1.3* of *CAN/CSA-S6 CODE*, the width (B) of the bridge may be assumed to be reduced to a width that provides a value of B < 10. The number of design lanes (n) shall be reduced as required to be consistent with the assumed bridge width (B).

4. Notwithstanding *Clause 1.4.2.5* of *CAN/CSA-S6 CODE*, approval shall not be given for the use of single load path structures. Slab and girder bridge structures shall have a minimum of four girder lines.

5. Piers with one column shall have a minimum cross-section of 2.8 m^2. Piers with two and three columns shall have a minimum cross-section area of 2.8 m^2 for each column, or alternatively smaller columns shall be linked together with a strut extending from 1 m below to 1.4 m above the adjacent ground. In no case shall any pier column have a diameter less than 1.5 m nor an equivalent cross sectional area less than that of a 1.5 m diameter column. These requirements do not apply to Standard SL, SLW and SLC bridges.

6. Bridge decks shall normally be designed and detailed as cast-in-place slab. Stay-in-place partial depth precast concrete deck panel construction is not a preferred design due to the potential for cracking around the perimeter of the precast panels that may affect long term performance. 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 partial depth precast concrete deck panels shall meet the design, fabrication and construction specifications in Appendix E.

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

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

2. DESIGN LOADS

2.1 Highway Bridges

For highway bridges, the design vehicle shall be a CL-800 Truck as defined in *CAN/CSA-S6 CODE*. No adjustments are required for the 9 kN/m uniformly distributed load for lane load.

2.2 Pedestrian Bridges

The minimum pedestrian bridge live load shall be in accordance with *CAN/CSA-S6 CODE Clause 3.8.9*. For flexible structures, dynamic response and side sway, which could cause discomfort to pedestrians due to crowd loading, shall be considered.

As it relates to Clause 3.4.4 (Serviceability limit states) of the *CAN/CSA-S6 CODE*, the anticipated degree of pedestrian use for all bridges with sidewalks will normally be "With sidewalks - occasional pedestrian use". When a higher anticipated degree of pedestrian/cyclist use is identified, a request for the application of "frequent pedestrian use" shall be submitted for approval by the Department.

2.3 Fatigue

All new bridges shall be designed to comply with Class A Highway requirements (*CAN/CSA-S6 CODE Clause 1.4.2.2*). This requirement shall apply to all bridge components for considerations of structural fatigue.

2.4 Vehicle / Pier Collision Load

Bridge structural supports located ≤ 10 m from the edge of the ultimate stage pavement shall be designed for a vehicle collision force. For roadways with a design speed < 80 km/hr, a 1400 kN collision load shall be applied in accordance with *CAN/CSA-S6 CODE Clause 3.15*. For roadways with a design speed ≥ 80 km/hr, the collision force shall be increased to 1800 kN, and applied in any direction in a horizontal plane located 1.2 m above ground.

2.5 Rating and Strengthening of Existing bridges

Bridge rating shall be based on *CAN/CSA-S6 CODE Section 14 – Evaluation*. For normal traffic the vehicles for Evaluation Levels 1, 2 and 3 in *Clause 14.9.1* shall be replaced by AT rating truck configurations CS1, CS2 and CS3, in combination with the uniformly distributed portion for lane loads as specified in *Clause 14.9*. The live load factors for normal traffic shall be as specified in Table 14.8, except these factors shall be increased by 10% for short span bridges. Short span bridges are defined as spans with lengths less than 10 m for shear forces and spans with lengths less than 15 m for bending moments.

Design for bridge strengthening is normally based on meeting *CAN/CSA-S6 CODE Section 14 – Evaluation* requirements for the current legal loads of CS1 28t, CS2 49t and CS3 63.5t, or other site specific heavy vehicle. It should be recognized that once a decision is made to proceed with strengthening, it may be an opportunity to add extra capacity for the longer term at minimal additional costs. Considerations should include type and age of structure, expected remaining life, and practicality limits.

3. SPAN LENGTHS, SUB-STRUCTURE STATIONING & BEARING SETTING

3.1 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' 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 sub-structure 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 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.

The following standard notes shall be incorporated on the 'General Layout' design drawing:

"GIRDER LENGTHS SHOWN ARE MEASURED ALONG BOTTOM (TOP) FLANGE AND ARE CORRECT AT +20˚ C. ABUTMENT AND PIER STATIONINGS ARE LOCATED SUCH THAT BEARINGS ARE CENTRED AT -5˚ C".

"DIMENSIONS ARE SHOWN TO GROUND STATIONS AND CONTROL POINTS ARE SHOWN TO 3TM GRID COORDINATE VALUES" (this note shall only be included if the initial survey and roadway design is completed with grid coordinates, or modified to other surveying system used).

3.2 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' 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 sub-structure 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.

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.

The following standard note shall be incorporated on the ‘General Layout’ design drawing:

“GIRDER LENGTHS SHOWN ARE MEASURED ALONG BOTTOM FLANGE AND ARE CORRECT AT +20˚ C. ABUTMENT AND PIER STATIONINGS ARE LOCATED SUCH THAT BEARINGS ARE CENTRED AT -5˚ C. PRECAST SUPPLIERS SHALL MAKE APPROPRIATE ALLOWANCE FOR PRESTRESS SHORTENING, SHRINKAGE & CREEP UP TO THE TIME OF GIRDER ERECTION”.

“DIMENSIONS ARE SHOWN TO GROUND STATIONS AND CONTROL POINTS ARE SHOWN TO 3TM GRID COORDINATE VALUES” (this note shall only be included if initial survey and roadway design is completed with grid coordinates, or modified to other surveying system used).

3.3 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’ design drawing, with a cross-reference to the detailed ‘Girder Layout’ design drawing showing complete geometry of all girders.

4. BRIDGE GEOMETRY

Control line designations shall be selected from the following list and be used consistently throughout the same set of design drawings: Centreline NBL Hwy XX, Centreline N-W RAMP, Centreline RDWY, Centreline CROWN, Centreline BRG ABUT #X, Centreline ABUT #X (for integral abutments), Centreline PIER #X.

Bridges shall be on tangent horizontal alignments where practicable. 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.

In general, bridge deck clear roadway width shall match the width of the approaching roadway (existing or future planned). Minimum bridge clear roadway width shall be 9 m (*RDG Section H5.4.1: Shy Line Offset* and *BPG 10 - Minimum Bridge Width for SLC Girder Structures*). For long bridges, shoulders on bridges may be reduced for economic reasons. This shall be reviewed at the preliminary engineering stage in conjunction with the Department.

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 or a scupper system for peak storm flows may be required to meet drainage requirements (*BPG 12 - Bridge Deck Drainage*). Minimum sight lines may also require widening.

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 TL2 curved corner guardrail is provided on standard drawing *S-1815*. 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 drawing *RDG-B1.11*.

The bridge deck shall have a 2% normal crown unless the bridge structure is superelevated. The tops of sidewalks and raised concrete medians shall slope 2% towards the roadway. The tops of abutment seats, pier caps, curbs and barriers shall have a wash slope of 3%.

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

The longitudinal grade for proper bridge deck drainage (*BPG 12 - Bridge Deck Drainage*) shall be a minimum of 1%. However, the minimum longitudinal grade may be reduced to 0.5% due to site specific restraints or excessive costs. 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 result in accelerated structure deterioration.

Grade separation structures may require crest curves that result in portions of the bridge deck having a longitudinal grade of less than 1%. Wherever feasible, the tops of crest curves shall be located off of the bridge.

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

Clear zone requirements, calculated critical vertical clearances with their critical locations for current construction as well as the ultimate stage construction shall be shown on the 'General Layout' design drawing for all grade separation structures.

For bridges over an underpassing roadway, the design vertical clearance (including any planned future ultimate stage), shall be 5.4 m at the most critical location, as illustrated in *RDG Figures H7.1 to H7.3*. Note that the final posted vertical clearance for any bridge shall not be less than 5.2 m as determined by the standard procedure below.

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

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

ii. Round down to the nearest decimetre (i.e. 5.3 m); then

iii. Subtract one decimetre for tolerance (i.e. Post vertical clearance as 5.2 m).

Vertical clearance signs for bridges shall be centered over the traffic lanes (one per direction) and mounted close to the girder bottom flange.

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 underpassing 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.

The design vertical clearance for pedestrian bridges shall be a minimum of 5.7 m.

The design vertical clearance measured to the bottom edge of the sign panel of overhead sign structures shall be 6 m. The bottom edge of the sign structural framing shall be at least 0.6 m higher than the bottom edge of the sign panels.

The headslope width at the top of fill shall be out-to-out bridge structure end width plus 2 m. Beyond the bridge end, the width of fill shall be sufficient to meet guardrail standard requirements. Where guardrails are no longer required, the headslope fill width shall be transitioned at 30:1 or flatter to the approach roadway width.

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

Skew angles are to be given to the nearest minute; i.e., 12˚ 41' L.H.F. not 12˚ 40' 35" L.H.F.

“Top of Centreline Finished Crown” or “Top of Centreline Finished Roadway”, if there is no crown, stations and elevations are to be shown for each end of the structure. Top of Centreline Finished Crown is defined as the point where the headslope line intersects the finished centreline roadway profile. Station is given to the nearest decimetre and elevation to the nearest centimetre.

Design high water elevation, high ice elevation, low water elevation (with date of survey), design general and local scour elevations shall be shown on the General Layout for all river structures.

Substructure elements are to be numbered in the direction of increasing chainage; i.e., ABUTMENT #1 or PIER #1 occurs at the lower chainage location and the numbering increases from there.

All dimensions shall be ground dimensions. For more information on ground and grid coordinates refer to *Design Bulletin #34 – Grid-to-Ground Survey Application*.

5. GIRDER DEFLECTION & CAMBER

5.1 Steel Girders

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 include girder DL, deck DL, SIDL (including curb/barrier/median + wearing surface), and vertical grade.

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 *CAN/CSA-S6 CODE 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.

5.2 Precast Concrete Girders

Forms for DBC type girders are adjustable to allow a sag to be built into the girder to account for camber resulting from prestressing/post tensioning. The required form sag is to be presented on the design drawings.

Forms for NU type girders do not allow for form sag. However, theoretical calculated cambers based on best estimates shall be shown on the 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 standard drawing *S-1757*).

Camber for precast 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.

5.3 Cast-in-Place Concrete Superstructures

Data must be presented on design drawings to allow setting of form elevations. The deflection data used in the determination of the form elevations should be presented.

5.4 Standard Note for Camber Correction

The following standard note shall be shown on the 'Deck' design drawing and shall apply to the nominal deck haunch thickness and the outside of curb/fascia dimensions:

"THESE DIMENSIONS WILL VARY DUE TO VARIATIONS IN GIRDER CAMBER. THE CONTRACTOR SHALL DETERMINE THE ADJUSTMENTS REQUIRED AND SUBMIT THEM TO THE CONSULTANT FOR APPROVAL."

Superceded by Version 8.0
April 31, 2017

6. MATERIALS

6.1 Material Specifications on Engineering Drawings

Materials selected for incorporation in a design shall be specified in the general notes of the appropriate design drawing complete with material properties used for design such as the 28 day strength for concrete and yield or ultimate strength for steel.

6.2 Concrete:

DESCRIPTION	CLASS	f'c @ 28 days
Precast girders (refer to *SBC Section 7: Precast Concrete Units*).		50 to 70 MPa
All splash zone concrete[(1)]; Cast-in-place decks, concrete deck overlays, abutment & pier diaphragms, deck joint blockouts; Tops of abutment backwalls (300 mm minimum), abutment roof slabs, approach slabs, sleeper slabs, curbs, bridge barriers, sidewalk, raised concrete median; Precast MSE wall panels, MSE wall copings; Girder shear keys (14 mm max. aggregates), except SLW standard design.	HPC	45 MPa
Pile caps; Substructure elements and monolithic concrete other than splash zone; Sign structure foundations (with the exception that cement shall be type HS or HSb); Drilled caissons above the frost line; MSE wall levelling pads. Concrete slope protection; Concrete drain trough.	C	35 MPa
Pipe pile infill concrete; Drilled caisson concrete below frost line.	Pile	25 MPa

(1) Splash zone is defined as (a) top 300 mm of all pier and abutment concrete, and entire horizontal members of trellis structures and straddle bents, that projects beyond the bridge deck/bridge abutment footprint, to a distance of 6 m from inside edge of barrier/curb, and (b) any substructure elements, monolithic concrete protection barriers, that fall within 6 m of edge of lane of under-passing roadway, to a height of not less than

3 m above the edge of lane elevation. Bridge headslope protection concrete is not included.

6.3 Reinforcing and Prestressing Steel

The following reinforcing steel bar types shall be used in the specified bridge locations. For selection and detailing of Corrosion Resistant Reinforcing (CRR) types see Appendix C – Protection System Standards for Bridge Components:

- Corrosion Resistant Reinforcing CRR (Low Carbon/Chromium or Stainless Steel)
    - full depth cast-in-place decks and partial depth cast-in-place decks over precast panels;
    - reinforcing bars projecting from partial depth precast concrete 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, 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;
    - concrete within 300 mm of the tops of abutment backwalls, diaphragms and wingwalls; and
    - concrete within 300 mm of Splash Zone Surfaces, unless otherwise specified.

- Low Carbon/Chromium CRR (ASTM A1035 – MMFX)
    - stirrups projecting from precast girders into deck slabs.

- Stainless Steel CRR
    - deck joint blockouts;
    - corbels and dowels connecting approach slabs to corbels.

- Epoxy Coated Reinforcing
    - MSE wall panels.

- Carbon Steel Reinforcing
    - precast girders – grade 400W or deformed welded wire, except for CRR stirrups projecting from precast girders into deck slabs;
    - all locations not otherwise specified.

Superceded by Version 8.0
April 31, 2017

The following yield strengths shall be used for design with the applicable reinforcing steel grades:

REINFORCING TYPE	SPECIFICATION	DESIGN YIELD
Stainless steel bars	ASTM A276, ASTM A955/A955M, UNS S31653, S31603, S31803, S30400, S32101, S32304	420 MPa
Low Carbon/Chromium:		
• for stirrups projecting from precast girders into deck slabs	ASTM A1035	500 MPa
• at all other locations	ASTM A1035	420 MPa
Epoxy Coated bars	CSA G30.18M, ASTM A775, OPSS 1442, OPSS 1443	400 MPa
Carbon steel bars	CSA G30.18M	400 MPa
Deformed welded wire mesh	ASTM A615	480 MPa

Welding of structural reinforcing shall be prohibited. Welding of additional non-structural reinforcing shall be reviewed and approved by Bridge Engineering Section. If approved, Grade 400W steel shall be used.

Prestressing strands shall be ASTM A416 low relaxation strands, fpu = 1860 MPa,

High Strength Rods for prestressing shall be ASTM A722, fpu = 1030 or 1100 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.

Superceded by Version 8.0
April 31, 2017

6.4 Structural Steel and Structural Bolts

The following material properties shall be used for design of structural steel and structural bolts:

DESCRIPTION	GRADES
Girders and any material welded to girders; Any bracing member considered a Primary component and bolted to the girders (ie. for heavily skewed girders, curved girders or kinked girders).	CSA G40.21M-Grade 350AT CAT 3 or ASTM A709 Grade 345WT Type B with Charpy value of 27 J @ -30˚ C
Any bracing member considered a Secondary component and bolted to the girders.	CSA G40.21M-Grade 350A or ASTM A709 Grade 345 Type B
Bearing sole plates and rocker plates bolted to steel girder bottom flange; Shoe plates cast into precast girders and sole plates welded to shoe plates; Bearing base plates.	Galvanized CSA G40.21M-Grade 300W or 350W
Deck joint components for strip seal, cover plated strip seal, and cover plates for curb, barrier and raised median.	Galvanized CSA G40.21M-Grade 300W
Finger joint components except cover plates for curb, barrier and raised median.	CSA G40.21M-Grade 350A or ASTM A709 Grade 345 Type B
Bolts for weathering steel structural connections.	22 mm diameter ASTM A325M Type 3
Bolts connecting galvanized bearing components.	Galvanized 22 mm diameter ASTM A325M

6.5 Anchor Rods Projecting from Concrete Base

The following material properties shall be used for design of anchor rods projecting from concrete base:

DESCRIPTION	GRADES
Galvanized mild steel anchor rods in contact with galvanized bearing plates.	Galvanized CSA G40.21M Grade 300W or ASTM A307
Galvanized high strength anchor rods in contact with galvanized bearing plates and galvanized bridgerail posts base plates	Galvanized ASTM A193 GRADE B7 (Fy = 725 MPa, Fu = 860 MPa). Note galvanizing of high strength material requires special procedure, see standard drawing *S-1642*.

7. CLEAR COVER TO REINFORCING AND PRESTRESSING STEEL

The following covers for reinforcing steel shall be specified as "minimum clear cover" on the appropriate design drawings. These are minimum requirements that shall be met for construction inspection acceptance, and shall not be reduced by placement tolerances. Where not specified below, clear concrete cover shall be as specified in the *CAN/CSA-S6 CODE*.

Minimum clear cover to reinforcing steel in cast-in-place components shall be as follows:

LOCATION	COVER
For all concrete except when listed below.	50 mm
Top layer of cast-in-place decks and slabs protected with waterproofing membrane and ACP wearing surface.	50 mm
Bottom layer of cast-in-place decks and slabs.	40 mm
Bottom layer of approach slabs on clean granular fill and polyethylene sheeting.	40 mm
Near vertical traffic faces of curbs, medians and barriers.	100 mm
Cast-in-place elements, which are not protected by a waterproofing membrane and ACP wearing surface, that will come into contact with de-icing salts, including splash zone surfaces, but excluding the near vertical traffic faces of curbs, medians and barriers.	70 mm
Concrete cast in contact with soil (no form).	75 mm

Minimum clear cover in precast prestressed concrete girders shall be as follows:

LOCATION	COVER
Minimum clear cover to reinforcing steel.	30 mm
Minimum clear cover to prestressing steel and post-tensioning ducts when girder concrete has 28 day compressive strength greater than or equal to 65 MPa	45 mm
Minimum clear cover to prestressing steel and post-tensioning ducts when girder concrete has 28 day compressive strength less than 65 MPa	50 mm

8. REINFORCING STEEL DETAILS

8.1 Bar Lists

Complete reinforcing bar details are to be shown on the ‘Bar List’ design drawing(s).

All reinforcing steel bar marks are to be as per the *Engineering Drafting Guidelines for Highway and Bridge Projects*.

Bar marks should not be duplicated on a project unless the bars are identical.

Incremented bars should each have their own bar mark.

On the ‘Bar List’ design drawing, separate mass totals for each different bar type, as well as a combined total, are to be given for each list for each bridge component, i.e. abutments, piers, deck, etc. Mass for individual bar types is to be calculated and shown to the nearest kilogram.

In the Quantity Summary Table to be included on the ‘Information Sheet’ design drawing, separate totals for each bar type shall be shown separately for substructure and superstructure.

For reinforcing bars other than carbon steel reinforcing bars, the following bar mark suffixes shall be used:

C	Epoxy coated bars
CR	Generic bar mark for alternative corrosion resistant rebar types
MX	Low carbon/chromium steel bars (ASTM A1035 - MMFX)
SS	Solid stainless steel bars

When preparing the project Record Drawings (C-drawings), the consultant shall update the drawings to reflect the bar type and grade that was actually used for construction. Refer to Appendix C for details.

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

9. SUB-STRUCTURE & FOUNDATIONS

9.1 General Requirements

Bridge sub-structure components provide support to the superstructure. Premature deterioration of substructure components can result in costly repairs or reduced bridge life.

Past problems with substructures have been related to the following. New designs shall address these issues fully to avoid their occurrence:

- Abutment or pier footings settlement or scour
- Long term headslope movements due to geotechnical issues
- Bridge or highway drainage around abutments causing undermining of approach slabs, abutment seats, concrete slope protection, and general slope erosion
- Abutment and retaining wall translation or rotation
- Pile foundation problems

Spread footing foundations shall not be used for foundations at the outside edge of river bends, on the banks of highly mobile streams or within stream beds. For more background information, see *BPG 7 - Spread Footings*.

For land based foundations, most bridges are also supported on piles unless competent rock is available close to the 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 rock protection for stream related sub-structure elements see *BPG 9 - Rock Protection for Stream Related Infrastructure*.

Headslopes and retaining walls 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 longterm 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.

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 design drawings.

9.2 Piling

Test holes shall extend a minimum of 3 m below the estimated pile tip elevation.

For substructure elements founded on driven steel H-piles, HP 310 or larger piles shall be used for better driveability.

The following design pile load information for abutment and/or pier piles are to be shown in the General Notes on the Information Sheet:

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

The SLS extreme loads shown on the design drawings shall be used with the Bearing Formula in *SBC Section 3: Bearing Piles*.

When there are a limited number of driven piles, the ultimate geotechnical resistance may be estimated by static analysis. The geotechnical resistance at ULS is then obtained by multiplying the ultimate geotechnical resistance with a geotechnical resistance factor of 0.4.

Where justified for sufficiently large projects, more accurate field testing using PDA or static load tests can be specified in the Special Provisions. In this case, the economics of pile design can be improved by increasing the geotechnical resistance factor to 0.5 or 0.6 respectively in accordance with *CAN/CSA-S6 CODE* Table 6.1.

Welded pile splices shall be avoided where tensile or flexural capacity is critical to the structural integrity of the bridge (for example within the flexing length of integral abutment piles). The design drawings shall identify the extent of such sensitive zones, and shall identify that any welded splices within those zone shall require testing using non-destructive testing techniques. 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"

Full length piles shall be provided wherever possible to avoid field splicing.

Outline of foundations and estimated pile tip elevations are to be shown relative to test holes on the geotechnical information sheet.

Steel piles designed to be exposed shall be hot-dip galvanized to a minimum of one metre below grade or stream bed. All damaged galvanizing shall be zinc metallized.

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

9.3 Bridge Piers

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

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 bottom and previously cast concrete, to ensure proper flow of concrete under the ends of the girders.

End of pier cap cantilevers shall have cast in stainless steel drip sheets across the full underside width of the pier cap.

The up-stream end of river piers shall be protected by a galvanized nose plate.

9.4 Bridge Abutments

Bridge ends shall be supported on piles.

The most preferred abutment type for highway structures is the pile supported stub abutment seat located near the top of an open headslope, to maintain the openness of the highway cross-section. This allows for easy access for inspection and maintenance of bearings, deck joints and girder ends, and potentially allows for future expansion of underpassing highway by trimming back the toe of the headslopes.

The use of high abutments and MSE 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. Where considered justifiable, the high abutments or walls shall be presented during the choose design process with full documentation of supporting rationale. It should be noted that high walls are often associated with long term movements and rotations that may increase future maintenance. These issues shall be addressed clearly at the choose design stage for review by the Department

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.

Sketches SK-9 to SK-19 (Appendix D) show typical abutment details. These sketches were originally developed as part of the Department’s Technical Requirements for P3 projects and are included for reference.

- Bridge ends shall have cast-in-place wingwalls oriented parallel to the overpassing roadway. Wing walls up to 8 m long can generally be designed as cantilevers from the end of the superstructure or the abutment seat. Longer wing walls shall be supported by roof slabs and grade beams to reduce the cantilever length.

- Abutments with roof slabs and grade beams are connected to form a rigid box, and are considered to be more stable and provide more resistance to movements and rotations.

- 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 lateral wall displacements.

- MSE reinforcing straps shall not be attached to pile supported abutments or retaining walls 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. Voided space between the MSE backfill and the back face of the abutment or wing walls is not allowed.

- Geofoam blocks may be considered for light weight abutment fill. Proposals for their use will be considered only if submitted with well thought out details to ensure long term performance. Protection of geofoam from damage due to exposure to hydrocarbons, erosion due to leakage of highway drainage around abutments and wing walls, effects of differential settlement and other movements, compatibility of material properties such as long term creep and short term elastic characteristics, etc., are all important considerations. The space between the geofoam blocks and abutments or wall panels shall be filled with an approved material.

- Soil reinforcement shall not be attached to pile supported abutments or other bridge components to reduce soil pressure, due to potential damaging effects of long term differential settlement and corrosion of steel connectors.

- For abutments with cavity under roof slabs, access to the cavity below the roof slab shall be provided through the abutment backwall, with an access hatch positioned between girders.

- All abutments shall be designed for inspection access without need of specialized equipment.

- For conventional headslopes, provide minimum 0.6 m wide bench in front of abutment seats. For high abutments and MSE walls, provide a minimum 0.5 m wide inspection walkway in front of abutment seat.

- The ends of wing walls shall extend a minimum of 0.6 m past the finished top of headslope and 0.6 m minimum depth below adjacent ground.

- Bridge plaques and bench mark tablets shall be provided at bridge abutments in accordance with standard drawings *S-1477*, *S-1617*, and *S-1478*.

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 (Reference standard drawings *S-1411* and *S-1443*);

- 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 shall be spot-glued to the earth face of the abutment seat and wingwalls to intercept and channel seepage into a perforated weeping drain with a minimum positive drain slope of 2% that will be day-lighted on the sideslope;

- 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 MSE wall abutments, 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, Refer to the requirements in *SBC Section 25: Mechanically Stabilized Earth Wall*.

- Drainage swales along the top of MSE wing walls shall always drain laterally away from the overpassing roadway at bridge abutments.

- Bridge deck drainage shall always be controlled and channelled away from bridge components. Refer to Section 21 - Bridge Drainage

9.5 Layout of Retaining Walls at Abutments

- Typical layouts of independent high retaining walls at abutments adjacent to roadways and railways, as shown in Sketches SK-16 and SK-17 in Appendix D, were developed for P3 projects and are provided here for guidance. Project specific layouts shall be provided in choose design report.

- Retaining wall wingwalls shall preferably be placed parallel to the underpassing roadway except that the retaining wall wingwall on the approaching traffic side for the underpassing 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 underpassing 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.

- Any toe slope in front of MSE walls that project into the clear zone shall be no steeper than 1 on 3.

The MSE abutment and wing walls shall also meet the requirements of Section 10 – Retaining wall structures.

9.6 Integral Abutments

Historically, deck joints are expensive to maintain and have been a drain on bridge maintenance budgets. Integral abutments and jointless bridges shall be used whenever possible for bridges meeting the thermal span limits of the Table below. The thermal span shall be taken as the span measured from the structure's fixed point to the centreline abutment bearings/centreline piles. For detailed requirements on integral abutment design, see Appendix A.

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*

10. RETAINING WALL STRUCTURES

In locations where traffic runs adjacent to the top of, and nominally parallel to a retaining wall, a rigid bridge barrier shall be provided that meets the appropriate Performance Level requirements of the *CAN/CSA-S6 CODE Section 12 – Barriers and Highway Accessory Supports*. The retaining wall shall be designed to fully resist the collision loads applied to the barrier, and loads from any attachments such as sign and lamp posts. Details for the Department's standard PL-2 Barrier System on top of MSE walls can be found on standard drawing *S-1798*.

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 the *Alberta Building Code, Part 9*. Safety railing 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. Retaining walls shall be designed to resist the loads from all barriers;

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.

Non-mechanically stabilized earth retaining walls shall be designed in accordance with the provisions of the *CAN/CSA-S6 CODE*.

Mechanically Stabilized Earth retaining walls are proprietary designs and shall meet design/build requirements in *SBC 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)*.

Consultant responsibilities for MSE walls are summarized as follows:

- Perform geotechnical investigation, 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' design drawing showing location, geometry control, site drainage, allowable bearing capacity, settlement prediction, and special provisions.

- Review design notes and shop drawings submitted by the contractor to ensure all specification requirements are met. This review is for conformance of specifications requirements only and not a design check.

- At the end of the construction, the Consultant shall update and submit the wall layout to reflect as-built conditions, and likewise ensure that the contractor update all design notes and shop drawings.

Dry cast concrete block walls are not permitted.

11. BRIDGE BEARINGS

Fixed steel rocker plate bearings shall be fully designed and detailed by the Consultant.

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 *SBC 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 *SBC Section 8: Bridge Bearings*.

The Consultant shall provide the following information on the 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, tapered dimensions and connection to girder details;
- Base plate, anchor rods, and grout pad details;
- Reinforced concrete shear block details for lateral restraint;
- Bearing Setting Elevation Table showing top of sole plate elevations and two empty rows, for bearing heights and top of grout pad elevations, to be filled in after bearing heights are obtained from the contractor.

Consultant designed bearing components shall be in accordance with Appendix G – Bridge Bearing Design Guidelines.

12. GENERAL BRIDGE GIRDER REQUIREMENTS

Attachment of utilities to bridge girders or other primary load carrying members shall not be permitted.

Vertical clearance signs, where required, shall be mounted on the lower half of the girder and shall be shown on the 'Girder Layout' design drawings. Shop drilled holes for steel girders or cast-in inserts for concrete girders shall be incorporated during girder fabrication.

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.

13. INTERMEDIATE DIAPHRAGMS

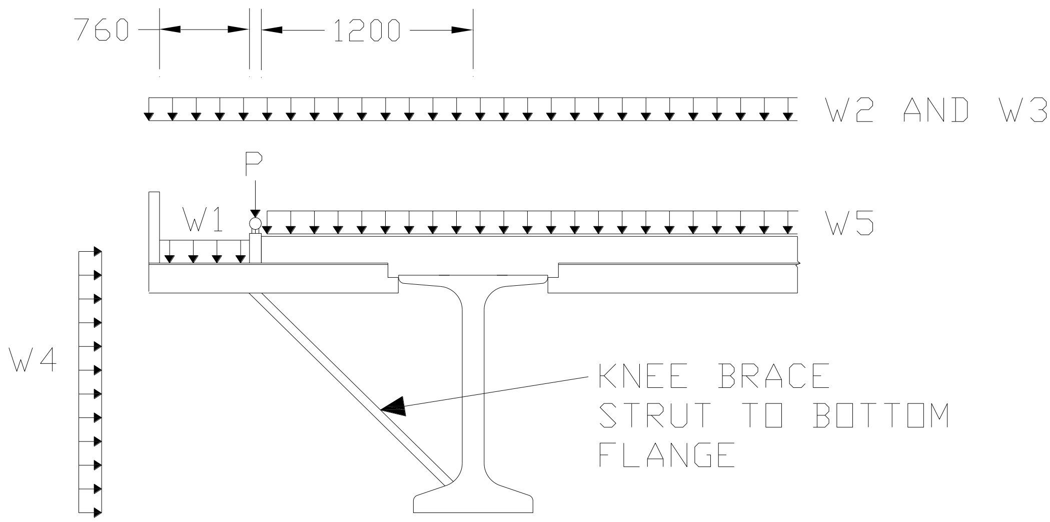
Intermediate diaphragms for steel girder and slab superstructures shall have a maximum spacing of 8.0 m. Intermediate diaphragms for precast NU girder (or similar I-shaped precast girder) and slab superstructures shall have a maximum spacing of 13.0 m. Refer to standard drawing *S-1757* for NU girder diaphragm details and standard drawing *S-1759* for steel girder diaphragm details.

Intermediate diaphragms for steel girders or precast NU girder (or similar I-shaped precast girder) with depths 1200 mm or shallower, can be channel or W shape sections of at least 1/3 and preferably 1/2 the girder depth. For girders deeper than 1200 mm, X-bracing or K-bracing with 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 *CAN/CSA-S6 CODE Clause 3.16* and *SBC Section 4.10.6: Deck Formwork*. Typically, diaphragms provided shall become part of the permanent structure and be left in place for possible future maintenance, i.e. widening, rehabilitation, etc. The only exception to this is at the ends of NU girder bridges with integral abutments where the diaphragms may be removable as noted on standard drawing *S-1757*. Diaphragms, exterior steel and precast 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 design drawings. The following is a sample of drawing notes:

- THE GIRDER CAPACITY HAS BEEN EVALUATED FOR THE FOLLOWING UNFACTORED CONSTRUCTION LOADS IN ADDITION TO THE DEAD LOADS OF THE NU GIRDERS:
  - LIVE LOAD ON WALKWAY: W1 = 2.4 kPa
  - FORMWORK DEAD LOAD: W2 = 1.0 kPa
  - WIND LOADS: W3 and W4 = 70 km/h MAXIMUM WIND SPEED CORRESPONDING TO 1:10 YEAR RETURN PERIOD WITH q = 0.35 kPa
  - CONCRETE DECK DEAD LOAD: W5 = 5.4 kPa
  - SCREED MACHINE LOAD ON ONE SCREED RAIL: P = 40 kN

Superceded by Version 8.0
April 31, 2017



IF KNEE BRACE STRUT DOES NOT EXTEND TO THE BOTTOM FLANGE, CONTRACTOR TO DETERMINE WHETHER TEMPORARY STRUTS TO INTERIOR GIRDERS ARE REQUIRED TO DISTRIBUTE LOAD.

- IF THE CONTRACTOR PROPOSES TO USE ANY CONSTRUCTION LOADS OR PROCEDURES THAT DEVIATE FROM THOSE SHOWN ON THE DRAWINGS, THE CONTRACTOR SHALL SUBMIT FOR REVIEW TO THE CONSULTANT, NEW PLANS, SIGNED AND SEALED BY A PROFESSIONAL ENGINEER REGISTERED IN THE PROVINCE OF ALBERTA, INDICATING ALL LOADS, PROPOSED METHODS AND SEQUENCES OR CONSTRUCTION, AND ANY TEMPORARY SUPPORT SYSTEMS REQUIRED. THE SUBMISSION SHALL VERIFY THAT THE GIRDERS ARE CAPABLE OF RESISTING THE ACTUAL LOADS SAFELY AND WITHOUT DAMAGE

For precast concrete NU or I-shaped girder bridges 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 *CAN/CSA-S6 CODE Clause 10.18.4.2*, except for slotted holes with the slot parallel to the direction of the force, the slip resistance shall be taken as $0.60V_s$ (reference *AASHTO LRFD Bridge Design Specifications Table 6.13.2.8.2*).

For bridges with small radius curves or high skews, differential deflection between adjacent girders due to dead load application can become 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 (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 Guidelines for Constructability* - Section 1.6 Differential Deflections). Where applicable, the Consultant shall include the following note on the steel girder design drawings, under the 'ERECTION' heading: "CROSS-BRACING SHALL BE DETAILED SUCH THAT GIRDER WEBS ARE PLUMB UNDER FULL DEAD LOADS".

14. PRECAST PRESTRESSED CONCRETE NU OR SIMILAR I-SHAPED GIRDER BRIDGES

Bridges designed with precast prestressed concrete I-girders shall meet the following requirements:

1. 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.

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

3. For NU or similar I-shaped precast girders, typical girder details shall be in accordance with the standard drawings *S-1757* and *S-1758*.

4. 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 at piers shall be developed by bent-up strands. Minimum separation between girders ends shall be 300 mm. Where pier diaphragms are not monolithic with the pier top, the ends of both girders shall be supported on separate reinforced elastomeric pads. For monolithic pier diaphragms which are cast around girder ends, the girders shall be erected on plain 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.

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

6. Girder SLS and ULS designs shall be based on a nominal girder section assuming a minimum deck haunch height of 13 mm between the bottom of the deck slab and the top of the precast girder.

7. Stirrup projections from the top of the precast girder into the deck shall meet all code requirements for developing full composite action. All stirrups shall be hooked around longitudinal anchor bars. When 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 provided to tie the slab and the deck haunch together.

8. Horizontal interface shear design for composite action shall satisfy the requirements from *CAN/CSA-S6 CODE* or *AASHTO LRFD Bridge Design Specifications*, whichever is more stringent. The longitudinal distribution of shear forces shall be taken conservatively to be the same as the ULS shear envelope.

9. For NU girders and other I shaped girders with pretensioned strands, the end zone reinforcement provided by vertical stirrups at SLS 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 reinforcement located within the distance from the end of the beam (mm^2);
   h = overall depth of precast member (mm).

   The resistance P_r shall not be less than 4 percent of the prestressing force at transfer. End cover for the end vertical reinforcement shall be 30 mm for exposed girder ends and 25 mm for girders encased into field cast diaphragms. One half of that reinforcement shall be concentrated in the end h/8 of the member while the balance of the 4% is distributed over a distance from h/8 to h/2. The reinforcement in the end h/2 shall be not less than that required for shear resistance. The end zone reinforcement shall be anchored beyond the anticipated extreme top and bottom cracks, an embedment adequate 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, which are assumed for design to be at the junction between the web and the flanges. The bar 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).

10. Additional end zone reinforcement shall be provided to confine the prestressing steel for the distance of 1.5d or at least 60 strand diameters from the end of the beams, other than box beams. The reinforcement shall not be less than 10M deformed bars, with spacing not exceeding 150 mm, and shaped to totally enclose the strands. The same amount of confinement steel must be provided at the bonded end of all debonded strand groups. For box beams, transverse reinforcement shall be provided and anchored by extending the leg of stirrup into the web of the girder. The top of the ties can be left open in the midspan region wherever there is conflict with post-tensioning cables.

11. For post-tensioning ducts in precast concrete 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.

12. NU and I- shaped girder ends shall have cast-in galvanized shoe plates anchored into the girders by Nelson Studs or welded deformed anchor rods. The shoe plate shall transfer all horizontal shear forces from the bridge bearings into the girder ends.

13. Girder end vertical shear stirrup design and strand development shall 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.

14. For conventional abutments with deck joints, the superstructure end diaphragm shall be an open steel diaphragm to provide access for joint inspection and repair. The girder web at abutment ends shall be thickened and designed as part of the abutment steel diaphragm for transfer of lateral forces into concrete shear blocks.

15. 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.

16. For connecting steel diaphragms to exterior girders, no connection hardware shall be visible on the exterior surface of the girders.

17. For girders containing pretensioning strands, *Clause 8.15.4* of the *CAN/CSA-S6 CODE* 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 pretension only, as well as girders with combined pretension and post-tension. For girders with combined pretension and post-tension, 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. 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.

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

19. Appropriate allowance for girder shortening due to prestress losses (pretension and post-tension) shall be included in the fabricated length of the girders.

15. PRECAST PRESTRESSED BOX GIRDER BRIDGES

Bridges designed with side-by-side precast prestressed box girder bridges (not including AT standard bridge girders) shall meet the following requirements:

1. Side-by-side box girder bridges shall be composite with a 225 thickness cast-in-place concrete deck with two mats of orthogonal deck reinforcement.

2. Side-by-side box girder bridges, fully monolithic with abutment or pier, may be erected on continuous plain neoprene sheets on abutment and piers. Permanent connection with substructure shall be made through the cast-in-place diaphragms between girder ends at piers and behind girder ends at abutments.

3. Side-by-side box girders, with permanent bearings in non-monolithic construction with the substructure, shall be supported on two steel reinforced elastomeric pads at each girder end, to be placed directly on top of the supporting concrete surface.

4. 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.

16. STEEL GIRDER BRIDGES

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

1. Typical welded steel plate girder details shall be in accordance with the standard drawings *S-1759* and *S-1760*.

2. Vertical stiffeners and girder ends shall normally be square to the girder flanges, which are parallel to the road grade, whereas the abutment backwall is plumb. The dimension of the corbel overhang at the top of the backwall shall account for the effects of girder end tilt.

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

4. All bearing stiffeners, including jacking stiffeners, shall be “fit to bear bottom” and “fit only top”, and then fillet welded to both 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.

5. 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

6. Jacking stiffeners shall be provided for future bearing replacement. 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.

7. 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. Corner cope of plates shall normally be 80 mm vertical x 35 mm horizontal for web thicknesses of 14 mm to 20 mm. 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 *CAN/CSA-S6 CODE Clause 10.10.6.4*.

8. Corners of stiffener plates projecting past the outside edge of flange plates shall be coped 45º.

9. No intersecting welds are allowed. Longitudinal stiffeners are normally placed on the opposite side of the web as vertical intermediate stiffeners. Where horizontal stiffeners and vertical diaphragm stiffeners intersect on the same side of the web, the horizontal stiffener shall run continuously through a slot in the diaphragm stiffener. The cut edges of the diaphragm stiffener at the intersection shall be corner coped (25 mm x 25 mm) adjacent to the web, and be welded to the horizontal stiffener. Refer to standard drawing *S-1760* for details.

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

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

12. The following features shall be used to prevent staining of sub-structure concrete:

    a. 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 standard drawing *S-1760*.

    b. 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 fully integral abutments, a second rubber strip shall be applied all around the bottom flange of all girders immediately in front of the concrete abutment face.

13. 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 standard drawing *S-1760*.

14. Shear Stud projections from the top of girder flanges into the deck shall meet all *CAN/CSA-S6 CODE* 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 a minimum of 25 mm plus a design tolerance of 25 mm.

17. DECKS, CURBS & CONCRETE BARRIERS

(a) Cast-in-place concrete deck slabs for beam and slab bridges shall be designed with the empirical method in accordance with *Clause 8.18.4* of *CAN/CSA-S6 CODE*, and 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.

(b) *Clause 5.7.1.6* of *CAN/CSA-S6 CODE* covers deck slab moments due to loads on the cantilever overhang in concrete decks supported on longitudinal girders. This clause of *CAN/CSA-S6 CODE* shall be amended as follows:

- the third paragraph of *Clause 5.7.1.6.1.1* shall be amended to read:

  "For the design moment intensity due to the vertical axle loads of the CL-800 Truck, the effects of individual loads shall be obtained and superimposed or, alternatively, the design moment intensity due to the CL-800 Truck may be obtained directly by multiplying the maximum cantilever moments in Table 5.10 by a factory of 1.28, for stiffened and unstiffened overhangs, as applicable (Table 5.10 includes the factor [1+DLA])."

- In *Clause 5.7.1.6.2*, wheel load P shall be changed from 87.5 kN to 112 kN..

(c) Deck and curb reinforcement required to develop the capacity of bridgerail post anchors are site specific designs. Guidance for design of decks supporting bridgerail posts is available from *AASHTO LRFD Bridge Design Specifications Appendix A13*.

(d) For standard curb details see standard drawing *S-1680*.

(e) Concrete curbs and barriers shall have crack control joints at a maximum spacing of 3 m (centred between bridgerail posts where bridgerail posts are used), with the exception of the standard PL-3 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 shall be caulked prior to application of deck waterproofing membrane in accordance with standard drawing *S-1443*.

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

(g) 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.

18. SIDEWALK & RAISED CONCRETE MEDIAN

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

(b) For sidewalks and raised concrete medians, 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 sidewalk shall have a curb projecting 100 mm minimum 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 (see standard drawing *S-1443*). 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.

(d) The portion of the structural deck slab under sidewalks and raised concrete medians shall be protected by a waterproofing membrane and protection board, in accordance with *S-1443*. Sidewalk slab or raised concrete median shall be cast-in-place concrete poured after the membrane and protection board have been applied to the structural deck slab. The top slab surface of sidewalks and medians shall have transverse tooled joints at a spacing matching adjacent curb/barrier control joints. Slab reinforcing shall be discontinuous at the joints.

19. DECK PROTECTION AND WEARING SURFACE

(a) For standard deck protection system requirements see *BPG 3 - Protection Systems for New Concrete Bridge Decks* and standard drawing *S-1443*. 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.

(b) Asphalt Wearing Surface Type H2 is used basically on Primary Highway bridges and/or high traffic volume highways or local roads. Asphalt Wearing Surface Type M1 is used on low traffic volume highways or local roads when an asphalt overlay is applied.

Superceded by Version 8.0
April 31, 2017

20. DECK JOINTS

(a) New structures shall be fully continuous from end to end. Deck joints shall only be permitted at abutments.

(b) The following standard deck joints described below shall be used unless prior written acceptance is obtained from the Department to use other deck joints. 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 un-factored 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 to *S-1812* (Type I Multi-web Strip Seal Deck Joint)	Multi-cell strip seal	115 – 60 = 55 mm	13 mm [(2)]
S-1800 to *S-1802* (Cover Plated V-Seal Deck Joint)	Cover-plated V-seal (102 mm V-seal)	90 – 60 = 30 mm	20 mm [(2)]
S-1800 to *S-1802* (Cover Plated V-Seal Deck Joint)	Cover-plated V-seal (125 mm V-seal)	115 - 60 = 55 mm	25 mm[(2)]
S-1800 to *S-1802* (Cover Plated V-Seal Deck Joint)	Cover-plated V-seal (178 mm V-seal)	150 - 60 = 90 mm	30 mm[(2)]
S-1638 to *S-1640* (Standard Finger Plate Deck Joint Assembly)	Finger plate joint	300 mm	0 mm

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 design drawings, but the shear component is zero at the time of seal installation. The temperature of seal installation is assumed to be 15 º C for the standard design, but can be fine tuned if necessary for a specified installation temperature. A design example is provided in Appendix F.

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

(d) The Type 1 multi-web strip seal has no moving plates, and is the simplest joint available. Therefore, it is advantageous to use this joint type as much as possible, and allow the design to use up 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.

(e) 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.

(f) Only approved strip seals listed on the Department's deck joint standard drawings shall be used. Multi-cell strip seal deck joints are the Department's preferred deck joint system where their use is not limited by the movement capacity of the seal perpendicular and parallel to the joint.

(g) Deck joints 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.

(h) 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* to *S-1812* 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.

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

(j) Modular seal deck joint systems are not permitted.

(k) Deck joints shall run continuously across the full width of the deck. 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.

(l) Only neoprene V-seal and multi-cell strip seals shall be permitted.

(m) Refer to Appendix F for design example on allowable movement for strip seal deck joints.

(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.

21. BRIDGE BARRIERS AND TRANSITIONS

The bridge barrier exposure index for the project shall be calculated in accordance with the latest version of *CAN/CSA-S6 CODE*. Once the barrier exposure index has been calculated, the appropriate barrier performance level can be determined from tables in *CAN/CSA-S6 CODE*.

AT standard bridge barrier and the transition for connecting to the approach guardrails shall be selected based on the required performance level. Other crash-tested bridge barrier and transition systems may be appropriate for special site conditions, but shall not be used without written approval from the Bridge Engineering Section.

For bridge barriers, the weak link in the barrier system shall be the bending over of bridgerail posts, with minimal damage to anchor bolts and bridge curbs. Refer to Section 17(c) for curb and reinforcement design at bridgerail posts.

The transitions provide a gradual change in stiffness from the rigid bridge barrier to the flexible approach guardrail system. Beyond the bridge end transition, the type of approach guardrail and terminal ends are determined by the roadway design requirements, in accordance with the *RDG*. The approximate equivalency between the Test Levels from *NCHRP Report 350* and the current Performance Levels from *CAN/CSA-S6 CODE* is provided in *RDG Section H2.2.5: Bridge Barrier Selection Requirements*. It should be noted that *CAN/CSA-S6 CODE* is in the process of converting from Performance Levels to Test Levels in a future edition of the code.

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

For applications of the high tension cable roadside barrier in general, refer to *Design Bulletin #75 – Alberta Roadside Design Guide High Tension Cable Barrier System - Median and Roadside Installation*. Where snow drifting is a problem, it may be advantageous to minimize the length of approach beam type guardrail by switching to high tension cable after the end of the bridge end transition. In such cases, the high tension cable is to be connected to the end of the approach transition with a FHWA approved connection. 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

21.1 Traffic Barriers and Transitions

21.1.1 PL-1 (TL-2) Barriers and Transitions

Where the barrier exposure index indicates a PL-1 requirement, the standard PL-1 thrie beam bridge barrier with no curb (standard drawing *S-1652*) can be used, provided there is no salt application on the bridge now or in the future, because de-icing salt can drain over the edge and accelerated corrosion of the edge girder can be expected.

Standard drawing *S-1652* presents the standard 850 mm thrie beam bridge barrier and approach rail transition details. For bridges with a clear roadway width less than 9 m, a reduced height 680 mm PL-1 barrier can be considered to allow passage of wide vehicles over the top of the lower bridge barrier (standard drawing *S-1653*). standard drawing *S-1653* presents the standard 680 mm barrier and approach guardrail transition details. These bridge barriers and the approach rail transitions meet the TL-2 barrier requirements.

All paved bridges are expected to receive de-icing salt to maintain safe traffic operation. Whenever a bridge is paved or expected to be paved in the future, a PL-1 thrie beam bridgerail modified with a 75mm high curb for drainage control shall be used, as shown on standard drawing *S-1797*. 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 (deflection 330 mm at top of thrie beam) and 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. For bridges on gravel roads that will be paved in the future, a 100 mm rub rail is required to protect the curb end until the bridge is paved.

For larger bridges, consideration should be given to upgrading to a PL-2, even where a PL-1 barrier would satisfy the minimum CAN/CSA-S6 CODE requirements.

21.1.2 PL-2 (TL-4) Barriers and Transitions

The Department has developed 5 standard designs for PL-2 bridge barriers for different applications.

1. **STANDARD 850 MM PL-2 DOUBLE TUBE BARRIER AND TL-4 THRIE BEAM TRANSITION (STANDARD DRAWINGS *S-1642* AND *S-1643*)**

This is the standard PL-2 barrier for rural bridges. The double tube rail on curb provides an open barrier that will not trap snow on the bridge. Standard drawing *S-1642* presents the general barrier details and standard drawing *S-1643* presents the standard thrie beam transition. This bridge barrier and the thrie beam transition both meet the TL-4 barrier requirements.

2. **STANDARD 850 MM PL-2 SINGLE SLOPE CONCRETE BARRIER AND TL-4 THRIE BEAM TRANSITION (STANDARD DRAWINGS *S-1650* AND *S-1651*)**

This single slope concrete barrier is preferred in the urban area for its aesthetic appeal. The 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* presents the bridge barrier details and standard drawing *S-1651* presents the standard thrie beam transition. Except for the connection detail to the bridge barrier, this thrie beam transition is identical with that shown on standard drawing *S-1643*. It should be noted that towards the end of the bridge barrier, the single sloped face is transitioned to a vertical face to facilitate the connection to the vertical thrie beam. However, if the approach roadside barrier is a matching single slope concrete barrier, the sloped bridge barrier face should be continued to the end. The interface between the bridge end and a concrete approach barrier is shown on the standard drawings *S-1701* and *RDG-B6.14*. If the approach roadside barrier is a safety shape, the cross-section transition can be done in the transition length shown on standard drawing *RDG-B6.14*.

For bridges with sidewalks, a traffic separation barrier is required between the traffic and the sidewalk. *CAN/CSA-S6 CODE 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. A height less than 0.6 m is considered less obvious and could become a tripping hazard. The separation barrier shall be as represented by standard drawings *S-1650* and *S-1651*, except that the top ledge shall be omitted to create a flat vertical face on the pedestrian side. Note that this is a traffic barrier to prevent vehicle encroachment onto the sidewalk and is not provided as a pedestrian barrier. Over the length of the thriebeam transition connecting to the end of the traffic separation barrier, provide an HSS 50H x 100V rail, bolted to the sidewalk side of the thriebeam posts, to protect pedestrians/cyclists from snagging the sharp corners at the top of the posts. The HSS 50x100 can be terminated when the sidewalk has moved away and has achieved a minimum separation of 600 mm from the back of the guardrail posts.

3. STANDARD 1400 MM PL-2 SINGLE SLOPE CONCRETE COMBINATION BARRIER (STANDARD DRAWINGS *S-1700* AND *S-1701*)

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 combination barrier shall be provided. This normally happens on urban road over mainline bridges where a design speed ≤ 70 km/hr would only demand a TL-2 barrier. Standard drawing *S-1700* presents the general barrier details and standard drawing *S-1701* presents the details at the bridge barrier end. A connection to a rigid single slope concrete approach roadway barrier is shown here to reflect the urban situation. However, in situations where the approach roadway barrier is a more flexible barrier system, the transition details on standard drawings *S-1651* or *S-1681* should be used and the end of the bridge barrier should be transitioned from a sloped face to a vertical face.

4. STANDARD 850 MM PL-2 THRIEBEAM BRIDGE BARRIER WITH 200 MM CURB (STANDARD DRAWINGS *S-1648* AND *S-1649*)

This PL-2 barrier is used for short bridges less than 30 m (such as a 3 - 10 m span SLC bridge), where the thriebeam from the end transitions is continued across the bridge to provide a smooth continuous barrier without interruption. Over the bridge portion the thriebeam is mounted on top of a 200 mm curb on W200 posts. Standard drawing *S-1648* presents the general barrier details and standard drawing *S-1649* 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 transition requires a curb under the thriebeam, and is slightly longer than the ones on standard drawing *S-1643* and *S-1651*.

5. STANDARD 850 MM PL-2 SINGLE SLOPE CONCRETE OR DOUBLE TUBE BARRIERS ON MOMENT SLABS (STANDARD DRAWING *S-1798*)

This standard drawing was developed for traffic barriers located directly on top of MSE retaining walls, and can also be used where there is a need to extend barriers on grade beyond the end of the bridge for drainage control or other reasons. Standard drawing *S-1798* presents the option to use the double tube rail on concrete curb barrier or the single slope concrete barrier. The barrier design is based guidelines presented in *NCHRP Report 663*. The approach barrier transition details shall be provided in accordance with standard drawings *S-1643*, *S-1651* or *S-1681*, as applicable.

6. CONNECTING TL-4 THRIE BEAM TRANSITION TO DIFFERENT APPROACH GUARDRAIL SYSTEMS

Several additional strong posts are shown at the end of the thriebeam transition on standard drawings *S-1643* and *S-1651*. Beyond the strong posts shown, the approach guardrail can either continue with a strong post guardrail system, or change to a weak post guardrail system, or a cable guardrail system. These approach guardrails meet TL-3 barrier requirements. For connection to TL-4 "Modified Thrie Beam" approach guardrail, the transition shown on standard drawing *S-1681* should be used.

In cases where a TL-3 high tension cable is to be connected to the end of the strong post transition, 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

21.1.3 Standard 1270 mm PL-3 (TL-5) Double Tube Barriers with TL-4 Transitions

The preferred PL-3 barrier is the double tube rail on high concrete curb, represented by standard drawings *S-1702*, *S-1703*, *S-1704* and *S-1705*. This barrier is typically only necessary on bridges with very high AADT and truck traffic, as required by the *CAN/CSA-S6 CODE*. Standard drawings *S-1702*, *S-1703* and *S-1704* present the general PL-3 barrier details and standard drawing *S-1705* presents the TL-4 standard thrie beam transition. This bridge barrier is an FHWA approved design meeting TL-5 barrier requirements, and the thrie beam transition meets the TL-4 requirements. TL-4 is the highest level achieved with a thriebeam both in the transition and roadside barrier application. TL-5 roadside barriers are concrete barriers and a site specific connection design with proper modifications to the bridge end is required. When used to protect vehicles from catching lamp poles or pier shafts placed behind the barrier on highways, the height of the barrier shall be increased to 1400 mm by increasing the height of the lower concrete portion (see Section 21.3).

21.1.4 Miscellaneous Details

1. TL-2 W-BEAM CORNER GUARDRAIL AT INTERSECTIONS (STANDARD DRAWING *S-1815*)

This is a crash tested TL-2 W-beam design suitable for use at intersecting ramps and street corners. There had been some crashing of TL-3 thrie beam designs 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 TAC Guide to Bridge Traffic Combination Barriers Figures 6.21 to 6.24.

2. MODIFIED THRIEBEAM TRANSITION AT ABUTMENT DRAIN TROUGH TERMINAL (SKETCHES SK-14 AND SK-15 IN APPENDIX D)

Sketches SK-14 and SK-15 (Appendix D) present bridge end details around abutment concrete trough drain terminals where the transition posts are tightly spaced.

3. CURBS AND CURB/GUARDRAIL INTERACTION (*RDG TABLE H4.1*, STANDARD DRAWING *RDG-B1.10*)

The interaction between guardrail and curb 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 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 thriebeam connector as shown on the "CURB END DETAIL" on standard drawing *S-1642*, and on the "BRIDGE BARRIER ISOMETRIC VIEW" on standard drawings *S-1650* and *S-1703*.

4. PLACEMENT OF GUARDRAIL END CUSHIONS IN CURBED ROADWAYS (STANDARD DRAWING *RDG-B1.11*)

Details for placing crash cushion ends for guardrails along curbed roadways is provided on standard drawing *RDG-B1.11*.

21.2 Pedestrian / Cyclist Railing

The standard 1150 mm Vertical Bar Type Handrail as shown on standard drawing *S-1401* is designed for use at the outside of sidewalks with a traffic separation barrier on the road side. The handrail shall be mounted on a concrete curb projecting 100 mm above the sidewalk for a total handrail height of 1250 mm. For urban ring road projects, the standard 1150 mm high handrail design shall be modified to 1300 mm for a total rail height of 1400 mm. This shall be achieved by extending the height of the vertical bar panels and the posts.

For situations where there is an intersection close to the end of the bridge and enhanced vehicle visibility is required, the standard handrail shall be modified as shown on standard drawing *S-1426*. Orientation of the staggered bars must be described on the site specific design drawings to suit the site situation.

For combined pedestrian/cyclist use pathways 3.0 m or wider, provide a 50 diameter 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.

At the bridge ends, cantilever the railing out for a length of 1.2 m past the end of the wing walls and deflect the end of the railing outwards 400 mm away from the pathway.

21.3 Lamp Poles, Sign Structure Columns, and Pier Columns Behind Bridge Barriers

The presence of obstacles, such as signs, lamp posts, sign structure support columns, piers of adjacent bridges, etc., on top of or close behind bridge barriers can potentially cause snagging of errant vehicles or cause debris to fall on the roadway below. The mounting of such hazards are to be avoided whenever practical. However, when it becomes unavoidable, the following set-back requirements or protective measures shall apply. It should be noted that roadside barrier standards on the approaching roadways are determined independently from bridge barriers in accordance with the *RDG*:

1. When applicable roadside barrier standard is TL-2:
   - Provide minimum 305 mm set-back from traffic face at top of barrier.
2. When applicable roadside barrier standard is TL-3:
   - Provide minimum 610 mm set-back from traffic face at top of barrier.
3. When applicable roadside barrier standard is TL-4:
   - When PL-2 bridge barriers are required:
     - Provide PL-2 combination barrier (*S-1700* & *S-1701*) with a height of 1400 mm;
     - Provide minimum 610 mm set-back from traffic face of top steel rail.
   - When PL-3 barriers are required:
     - Provide PL-3 barrier (*S-1702* to *S-1705*) with the overall height increased to 1370 mm by increasing the height of the concrete base;
     - Provide minimum 610 mm set-back from traffic face of top steel rail.
4. When piers of adjacent bridges are behind bridge barriers, a 3,000 mm minimum set-back is required between the adjacent pier and the traffic face at top of barrier.

All lamp poles and overhead sign structures 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.

Attachments behind barriers (such as street light posts etc.) shall be mounted on top of curb or concrete barrier at locations close to the centreline of piers, whenever practical, to avoid excessive vibration from traffic.

21.4 Standard Bridge Barrier Detailing

(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. For long bridges, additional bridgerail expansion joints shall be provided at a maximum spacing of 45 m.
- Standard bridgerail 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 later case, the Consultant shall clearly indicate such requirement on the site specific design drawing. Tube sleeves for splices and expansion joints shall be detailed accordingly.

(b) For attachments mounted behind bridge barriers (such as 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.

22. BRIDGE DRAINAGE

(a) In general, salt contaminated surface drainage shall be contained and not be allowed to come into contact with bridge components.

(b) For Guidance on bridge deck drainage design, see *BPG 12 - Bridge Deck Drainage*. The goal is to avoid lane encroachment for risks of hydroplaning, and restrict water flow spread within the width of shoulders. Drainage issues should receive early attention at the planning stage, when there is opportunity to optimize bridge geometry.

(c) Drains required to accommodate 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) Concrete drain trough collectors shall be located at low corners of bridges to channel water off of the bridge and into drain troughs lined with compacted granular filled “Geoweb” ditch or equivalent accepted by the Department. Typical drain trough details are shown on Sketches SK-14 and SK-15 (Appendix D). The drain troughs shall be designed to function as intended while accommodating differential settlements and other movements between the bridge and the roadway approach fills. The same drain trough may also be required at the high end when there is sufficient grade and approach pavement draining towards the high end of the bridge.

(e) Additional requirements around bridge abutments are provided in Section 9.4 Bridge Abutments.

23. APPROACH SLABS

Approach slabs shall be in accordance with the provisions of *CAN/CSA-S6 CODE Clause 1.7.2*, except as modified below:

1. 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). Approach slabs for integral abutments shall be extended at least 1125 mm past the end of wing walls (see Appendix A);

2. 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;

3. Consist of Class HPC concrete;

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

5. Approach slab reinforcement shall be as required by the designer but 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;

6. All approach slabs shall receive a 90 mm thick ACP deck protection and wearing surface system as per Section 19 and standard drawing S-1443; and

7. 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.

24. UTILITY ACCOMMODATION

24.1 Utility Ducts in Curbs and Barriers

The Contractor shall provide one 75 mm outside diameter utility duct on each side of the bridge deck for the future accommodation of utilities. 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. The utility ducts shall be placed within the bridge curbs and/or barriers and shall be terminated into a weather proof junction box near the wingwall ends on the outside, where they can be accessed without damage to any road or bridge construction. The utility duct termination shall be detailed on the 'Abutment' design drawings. Utility ducts shall not be placed within the bridge deck or attached to the bridge girders.

All utility ducts shall be continuous and free and clear of obstructions, and shown to be so by passing a spherical object of the appropriate size through the entire length.

O-ring expansion fittings shall be provided at all bridge expansion joints. The o-ring expansion fittings shall be located within the deck joint block-out to facilitate future maintenance. At any locations where the curb/barrier may undergo rotation and/or vertical displacement, other appropriate fittings shall be used to accommodate the movements.

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. Rigid conduit shall be bent only with a standard conduit bender.

24.2 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 so that routing of conduits for electrical supply through the abutment ends is desired, a proposed alternative routing may be proposed for review by the Department.

The concealed conduit system shall comprise rigid PVC conduit having a minimum trade size of 38 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:

1. 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 dimensions for 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 fitted with a gasketted weatherproof cover.

2. 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 Department approval of detailed design, be placed unobtrusively in the face of the pier cap near its top edge. For bridge structures with integral pier cap/diaphragm, 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.

3. 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 25.4 mm trade size cast horizontally within the pier cap/diaphragm.

4. 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 12.7 mm inch trade size.

5. 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.

6. 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 shall be fixed to the vertical face of the bridge girder haunch. No attachments shall be fixed to the girders or to the underside of the bridge deck.

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

8. Luminaire conduits and/or conduit support equipment shall be attached to the bridge structure with anchors cast into the haunch concrete at appropriate locations.

9. In the event that precast 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*.

10. 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.

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

25. OVERHEAD SIGN STRUCTURES

Overhead sign structures, including bridge (non-cantilever) or cantilevered types, shall be designed to the requirements of *AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals* Fatigue Category I. Due to the difficulty in predicting the dynamic behaviour of cantilevered sign structures and determining which sign structure type is susceptible to galloping, the Department requires that all cantilevered sign structures shall be designed for the galloping load in the *AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals* without exceptions. These structures are procured by a design/build process, inclusive of sign panels, structural framing and foundations, in accordance with *SBC Section 24: Sign Structures and Panels*. Consequently, the Consultant's responsibilities differ from those of the conventional procurement process.

Consultant responsibilities are summarized as follows:

1. 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 partially hanging over the traffic lanes or road shoulder.

2. The Consultant shall determine placement, clearance requirements, need for barrier protection, and type of structure (bridge or cantilevered) in accordance with guidance provided in *RDG Section H8.4: Overhead Signs*, and prepare a 'General Layout' design drawing for each individual sign structure in accordance with standard drawing *S-1721*.

3. The Consultant shall review the design notes and shop drawings submitted by the contractor to ensure all requirements of *SBC Section 24: Sign Structures and Panels* 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 layouts to reflect as-built conditions and forward all shop drawings for Department records.

26. QUANTITIES

Bridge Contracts are tendered on a unit price basis for most bid items. The following items, with their indicated units, are among the most commonly used:

Piling (Type and size) – supply	-	m
Piling (Type and size) – drive	-	m
Piling (Type and size) – set up	-	pile
Concrete – Type	-	m3
Reinforcing steel - plain	-	kg
Reinforcing steel – corrosion resistant	-	kg
Reinforcing steel – stainless steel	-	kg
Concrete Slope Protection	-	m2
Rock Rip-Rap – Class	-	m3
Bridgerail	-	m
Handrail	-	m
Bridge Deck Waterproofing	-	m2
Wearing Surface – Two course hot-mix ACP (Type)	-	lump sum or tonne
Wearing Surface – Hot-mix ACP	-	lump sum or tonne

Piling, concrete, and rip-rap require a separate quantity for each size or type used.

Quantities, with the exception of slope protection and rip-rap, are to be calculated and shown to the nearest whole unit. Slope protection and rip-rap quantities are to be calculated and shown to the nearest 10 units.

Individual quantity estimate tables are to be shown on the applicable design drawings for the abutments, piers, and deck and are to be summarized on the quantity estimate table shown on the 'Information Sheet' design drawing.

Quantities done by other than the site contractor are to be so identified on the quantity estimate tables.

Structural steel mass for steel girder superstructures shall be calculated and the mass, in tonnes, shall be shown in the 'Girder Notes' on the 'Girder' design drawings. The mass shall include girders, diaphragms, stiffeners, and splice plates but does not normally include deck joints, bearings, and bolts.

27. STANDARD DETAILS & ENGINEERING DRAFTING GUIDELINES

The use of standard drawings and details are encouraged wherever possible.

Drafting standards and standard details shall be in accordance with *Section 2: Guidelines for Bridge Projects* of the *Engineering Drafting Guidelines for Highway and Bridge Projects*.

The preferred design drawing order for bridge type structures is as follows:

General Layout
Information Sheet(s)
Abutment(s)
Pier(s)
Bearing(s)
Girders
Deck
Deck Joints
Other (such as Bridge Lighting, RWIS/FAST System, etc.)
Bar List(s)
Standard Drawings

Other types of structures (culverts, etc.) should follow the same basic order with drawings added and/or deleted as necessary.

Bridge Engineering Section drawing numbers are to be used in all cases. Numbers will be established when exact number of design drawings in set is known.

Index listing all drawings included in the drawing set is to be shown on the first sheet of the set. The index is normally orientated from the bottom up; i.e., sheet No. 1 shown at the bottom and successive sheets listed upward from there.

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

28. LIST OF 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: Bridge Inspection and Maintenance

BPG: Alberta Transportation Bridge Best Practice Guideline

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

CSA: Canadian Standards Association

CAN/CSA-S6 CODE: CAN/CSA-S6-06 Canadian Highway Bridge Design Code.

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

SBC: Alberta Transportation Specifications for Bridge Construction

SSPC: Society for Protective Coating Standards

TAC: Transportation Association of Canada

UNS: Unified Numbering System

29. REFERENCES

1. Alberta Municipal Affairs. *Alberta Building Code 2006*. Alberta Transportation, Edmonton, AB (2006).

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. AASHTO. *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*, 5th Edition*, with 2010 and 2011 Interim Revisions*. American Association of State Highway and Transportation Officials, Washington, DC (2009).

5. AASHTO/NSBA *Guidelines for Constructability, NSBA G12.1.* American Association of State Highway and Transportation Officials/National Steel Bridge Alliance Steel Bridge Collaboration, Chicago, IL (2003).

6. *ASTM A193/A193M-12* (2012). *Standard Specification for Alloy-Steel and Stainless Steel Bolting for High Temperature or High Pressure Service and Other Special Purpose Applications*. ASTM International, West Conshohocken, PA.

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

8. *ASTM A307-10* (2010). *Standard Specification for Carbon Steel Bolts and Studs, 60 000 PSI Tensile Strength.* ASTM International, West Conshohocken, PA.

9. *ASTM A325M-09* (2009). *Standard Specification for Structural Bolts, Steel, Heat Treated 830 MPa Minimum Tensile Strength [Metric]*. ASTM International, West Conshohocken, PA.

10. *ASTM A416/A416M-10* (2010). *Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete*. ASTM International, West Conshohocken, PA.

11. *ASTM A615/A615M-12* (2012). *Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement*. ASTM International, West Conshohocken, PA.

12. *ASTM A709/A709M-11* (2011). *Standard Specification for Structural Steel for Bridges*. ASTM International, West Conshohocken, PA.

13. *ASTM A722/A722M-12* (2012). *Standard Specification for Uncoated High-Strength Steel Bars for Prestressing Concrete*. ASTM International, West Conshohocken, PA.

14. *ASTM A775/A775M-07b* (2007b). *Standard Specification for Epoxy-Coated Steel Reinforcing Bars*. ASTM International, West Conshohocken, PA.

15. *ASTM A955/A955M-12* (2012). *Standard Specification for Deformed and Plain Stainless-Steel Bars for Concrete Reinforcement*. ASTM International, West Conshohocken, PA.

16. *ASTM A1035/A1035M-11* (2011). *Standard Specification for Deformed and Plain, Low-carbon, Chromium, Steel Bars for Concrete Reinforcement*. ASTM International, West Conshohocken, PA.

17. AT. *BPG 3 - Protection Systems for New Concrete Bridge Decks*. Alberta Transportation, Edmonton, AB (2003).

18. AT. *BPG 7 - Spread Footings*. 2003. Alberta Transportation, Edmonton, AB (2003).

19. AT. *BPG 9 - Rock Protection for Stream Related Infrastructure*. Alberta Transportation, Edmonton, AB (2006).

20. AT. *BPG 10 - Minimum Bridge Width for SLC Girder Structures*. Alberta Transportation, Edmonton, AB (2008).

21. AT. *BPG 12 - Bridge Deck Drainage*. Alberta Transportation, Edmonton, AB (2010).

22. AT. *Design Bulletin #34/2006 – Grid-to-Ground Survey Application*. Alberta Transportation, Edmonton, AB (2006).

23. AT. *Design Bulletin #63/2009 (Revised April 2012) – Revisions/Updates to the Roadside Design Guide*. Alberta Transportation, Edmonton, AB (2007).

24. AT. *Design Bulletin #75/2012 (revised April 2012) – Alberta Roadside Design Guide High Tension Cable Barrier System - Median and Roadside Installation*. Alberta Transportation, Edmonton, AB (2012).

25. AT. *Engineering Drafting Guidelines for Highway and Bridge Projects*. Alberta Transportation, Edmonton, AB (2005).

26. AT. *Roadside Design Guide, with Design Bulletin #63/2009 (Revised April 2012)*. Alberta Transportation, Edmonton, AB (2007).

27. AT. *Specifications for Bridge Construction*. Alberta Transportation, Edmonton, AB (2010).

28. *Bligh, R., et. al.*, *NCHRP Report 663 – Design of Roadside Barrier Systems Placed on MSE Retaining Walls*. Transportation Research Board, National Research Council, Washington, DC (2010).

29. CSA. (2006). *CAN/CSA-S6-06 Canadian Highway Bridge Design Code,* including *S6S1-10, Supplement #1* and *S6S2-11, Supplement #2*. 2006. Canadian Standards Association, Toronto, ON.

30. CSA. (2012). *C22.1-12 Canadian Electrical Code*, 22nd Edition. Canadian Standards Association, Toronto, ON.

31. CSA. (2009). *CSA G30.18M-09 Carbon steel bars for concrete reinforcement*. Canadian Standards Association, Toronto, ON.

32. CSA. (R1998). *CSA G279-M1982 (R1998) Steel for Prestressed Concrete Tendons*. Canadian Standards Association, Toronto, ON.

33. CSA. (R2009). Canadian Standards Association. R2009. *CSA G40.20-04/G40.21-04 (R2009) General Requirements for Rolled or Welded Structural Quality Steel/ Structural Quality Steel*. Canadian Standards Association, Toronto, ON.

34. Federal Highway Administration (FHWA). 2009. *Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes - Volumes I and II (FWHA-NHI-10-024 and FWHA-NHI-10-025)*.

35. MTO. *OPSS 1442 Material Specification for Epoxy Coated Reinforcing Steel Bars for Concrete*. Ontario Ministry of Transportation, Toronto, ON (2007).

36. MTO. *OPSS 1443 Material Specification for Organic Coatings for Reinforcing Steel Bars for Concrete*. Ontario Ministry of Transportation, Toronto, ON (2007).

37. Ross Jr, H., D. Sickling, R. Zimmer, and J. Michie. *NCHRP Report 350 – Recommended Procedures for the Safety Performance Evaluation of Highway Features*. Transportation Research Board, National Research Council, Washington, DC (1993).

38. Plaxico, C. et. al., *NCHRP Report 537 – Recommended Guidelines for Curb and Curb–Barrier Installations*. Transportation Research Board, National Research Council, Washington, DC (2005).

39. *Tadros, M., S. Badie, and C. Tuan. NCHRP Report 654 – Evaluation and Repair Procedures for Precast/Prestressed Concrete Girders with Longitudinal Cracking in the Web*. Transportation Research Board, National Research Council, Washington, DC (2010).

Appendix A - INTEGRAL ABUTMENT DESIGN GUIDELINES

~~APPENDIX BABC STRUCTURES DESIGN GUIDE~~ (REMOVED FROM CIRCULATION, ALL DESIGNS TO CONFORM WITH THE REQUIREMENTS OF THE LATEST VERSION OF *CAN/CSA-S6 CODE SECTION 7 – BURIED STRUCTURES*)

Appendix C – PROTECTION SYSTEM STANDARDS FOR BRIDGE COMPONENTS

Appendix D – TYPICAL ABUTMENT DETAILS – SKETCHES SK-9 TO SK-19

Appendix E – DESIGN-BUILD SPECIFICATIONS FOR PARTIAL DEPTH PRECAST CONCRETE DECK PANELS.

Appendix F – DESIGN EXAMPLE FOR STRIP SEAL DECK JOINT WITH SKEW

Appendix G – BRIDGE BEARINGS DESIGN GUIDELINES

Superceded by Version 8.0
April 31, 2017

Alberta Transportation Bridge Structures Design Criteria v. 7.0

Appendix A

INTEGRAL ABUTMENT DESIGN GUIDELINES

Date Published: May 31, 2012

Government

Table of Contents

Superceded by Version 8.0
April 31, 2017

A.1 INTRODUCTION

Older bridges in Alberta are simple span structures with deck joints at every abutment and pier. Since the 1970's, bridge decks have been typically designed to be continuous over the piers eliminating the joints at these locations. However, joints were still common at the abutment ends. In many instances, due to long term headslope movement and settlement, abutments moved forward until they became jammed against the end of the superstructure, and the deck joints remain closed. These bridges would then behave as if they were integral bridges.

Deck joints are expensive to fabricate and install. Neoprene seals may last ten years and then need to be replaced. Once the joints are jammed, it becomes impossible to extract and replace the neoprene seal. The nosing concrete will normally last longer than the neoprene seal, but generally suffers 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.

Later, the elimination of the deck joints at the abutments became possible with the development of integral abutments. The accommodation of deck thermal movements was shifted away from the superstructure ends to the end of the approach slab. Here, leakage through the control joints could 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 been mostly good. However, good understanding of the behaviour of integral abutments, and care in detailing and construction are required to ensure success.

In Alberta, many SCC type bridges with pin connected semi-integral abutments have been performing very well. These are prestressed concrete bridges up to 60 m in total length. More recently, AT has increased the limits on the length of new integral designs, and has encouraged converting existing bridges with conventional abutments into semi- integral bridges in rehabilitation projects where the costs can be justified. It should be noted that approximately 90% of all bridges in Alberta are shorter than 100 m which makes many of them potential candidates for conversion in to semi- integral.

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 underpassing roadways and streams.

A.2 TYPES OF INTEGRAL ABUTMENTS

1. Fully integral abutment on piles (Fig. 1 & 2): 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 (Fig. 3): 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 (Fig. 4): 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 (Fig. 5): Similar to 4 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.

A.3 CYCLIC MOVEMENT 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 temperature, this gap becomes iced over, effectively providing a natural seal over the joint. Over the summer months, the joints are normally closed.

Asphaltic pavement over control joints shall be saw cut to produce a good square edge, and 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 ends of approach slabs and control joints shall be located at least 1.125 m beyond the ends of the wing walls. The detailing shall be coordinated with the abutment drain trough and approach guardrail transition posts if present.

A.4 CYCLE CONTROL JOINT TYPES

The following table 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 the fixity point to the centerline of the integral abutment.

Table 1 - 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 (Fig. 6) - 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 asphaltic hot pour rubberized crack sealant.

Cycle Control Joint Type C2 (Fig. 7) - 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 as shown in Sketch SK-15 (Appendix D).

A.5 CYCLIC JOINT REPAIR

For longer bridges utilizing prestressed and post-tensioned concrete girders and C2 joints, the joint gap may open up excessively during the first two years after construction, due to shrinkage and creep shortening of the prestressed girders. However, this is a onetime permanent irreversible opening. The opened gap should 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 (see Sketch SK-13 in Appendix D);
- 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.

A.6 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 for as far as practical. 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.

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 de-icing salt away from the straps.

On structures using concrete approach transitions, details for maintaining water tightness between the thermally cycling barrier on the wing wall and the stationary concrete approach transition shall be provided.

A.7 CONSTRUCTION SEQUENCE CONSIDERATIONS

Since the girders are erected on abutments with a single line of flexible piles, the stability of the abutment should be checked through different stages of construction, and temporary support may be required if necessary. Depending on the temporary bearing type used during girder erection (e.g. plain neoprene pad), there may be sufficient friction between the girders and the abutment seat before the concrete diaphragm is poured, that may introduce bending into the piles from thermal cycling of the girders. For long precast girders with post-tensioning, sliding bearings are provided and the abutment seat is braced to prevent lateral movement during post-tensioning.

A.8 DESIGN REQUIREMENTS

In accordance with *BPG 3 - Protection Systems for New Concrete Bridge Decks*, integral abutments shall be used wherever practicable.

Integral abutments shall include both fully integral and semi-integral abutments. Integral abutments shall not be used for bridge spans greater than specified in “Table 1- Maximum Thermal Spans”.

In addition to the general requirements for abutments stipulated elsewhere in the *BSDC*, integral abutments shall be designed to meet the following requirements:

1. The effects of skew, including the potential for twisting of superstructure on plan and bi-axial bending of piles, shall be analyzed by a 3D model and accounted for in the design, especially for skew angles greater than 20°;
2. The amount of structure and earth that have to move with the abutment during thermal movement of the superstructure shall be minimized. Abutment seat heights above grade shall not be greater than 1.5 m. Wingwalls shall be turned back and orientated parallel to the overpassing 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;
3. Integral abutments shall not be used with turn back retaining walls. In this situation, salt laden runoff can migrate through the cycle control joint and along the transverse wick drain to the back of the retaining wall causing damage to wall concrete and steel reinforcing straps, with no access for inspection and repair. See Sketch SK-17 (Appendix D) for more information.
4. Additional deck reinforcement shall be provided at the abutments for negative restraint moments due to torsional restraints provided by the stiff abutments diaphragms and adjacent girders, and earth pressure;
5. For fully integral abutments the abutment foundation shall be a single row of H-piles oriented for weak axis bending wherever possible. For large movements exceeding the movement range of Type C1 cycle control joints or when surrounding soils will restrict pile movement, piles shall be installed in permanent steel casings. The casings shall be filled with Styrofoam beads. Styrofoam beads shall be “Storopack” virgin polystyrene 14.4 kg/m^3 (0.9 pounds per cubic foot) filler bead with a nominal diameter of 5 mm, or approved equivalent. Steel casings shall be designed to last the same life as the bridge, and an appropriate sacrificial corrosion thickness or galvanizing shall be provided. The H-piles shall be embedded a minimum of two pile widths into the abutment seat;
6. Cycle Control Joint Types C1 and C2 shall be designed as shown on Sketch SK-13 (Appendix D). The control joints shall be located at least 1.125 m beyond the ends of the wing walls by extending the length of the approach slab and shall be coordinated with the approach rail transition post and drain trough locations. Wick drains draining the deck wearing surface shall be terminated and connected to a washed rock trench as shown on Sketch SK-15 (Appendix D), or other positive drainage;
7. For 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 compacted granular base shall be extended across the full width of the road embankment and be daylighted at the sideslope for drainage. The transverse wick drain in the control joint shall be integrated with the washed rock trench as shown in Sketch SK-15 (Appendix D), while the perforated drain pipe at the bottom of the trench shall be integrated with the abutment drain troughs if they are present.
8. 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;

9. Integral approach slabs shall not be designed to move longitudinally in and out between stationary and parallel non-integral wingwalls;
10. Two layers of polyethylene sheet shall be provided under the approach slab to minimize frictional forces due to horizontal movement. The connection between the approach slab and the superstructure shall be designed to resist these forces;
11. The thriebeam transition shall be rigidly attached to the ends of barriers, regardless of whether they are stationary or moving;
12. 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 Q on Sketch SK-9 (Appendix D);
13. Provision shall be made to accommodate thermal movement between integral abutments and slope protection or inspection walkways. Gaps shall be protected against moisture ingress;
14. Effects of construction sequence shall be duly considered. Deck pour causing girder end rotation and post-tensioning causing girder shortening shall be completed before making permanent connection between girder end and abutment. Provide temporary sliding bearing and anchoring of the abutment seat for post-tensioning where necessary. Stability of structure shall be checked for all stages. Backfilling behind abutments shall remain balanced.

A.9 REFERENCES

1. England, G., Tsang, N. and Bush, D., *"Integral bridges: A fundamental approach to the time-temperature loading problem".* 2000. Imperial College & Highways Agency, London, UK.
2. *The 2005 – FHWA Conference: Integral Abutment and Jointless Bridges (IAJB 2005)*. 2005. U.S. Department of Transportation, Federal Highway Administration, Washington, DC.
3. Hussain, I. and Bagnariol, D., *"Integral Abutment Bridges"*. Report SO-96-01. 1996. Ministry of Transportation of Ontario, St. Catharines, ON.
4. Hussain, I. and Bagnariol, D., *"Performance of Integral Abutment Bridges"*. Report BO-99-04. 1999. Ministry of Transportation of Ontario, St. Catharines, ON.
5. Paraschos, A. and Made, A., *"A survey on the status of use, problems, and costs associated with Integral Abutment Bridges".* 2011. A Better Roads Magazine Contributed Case Study, Tuscaloosa, AL.

A.10 LIST OF FIGURES

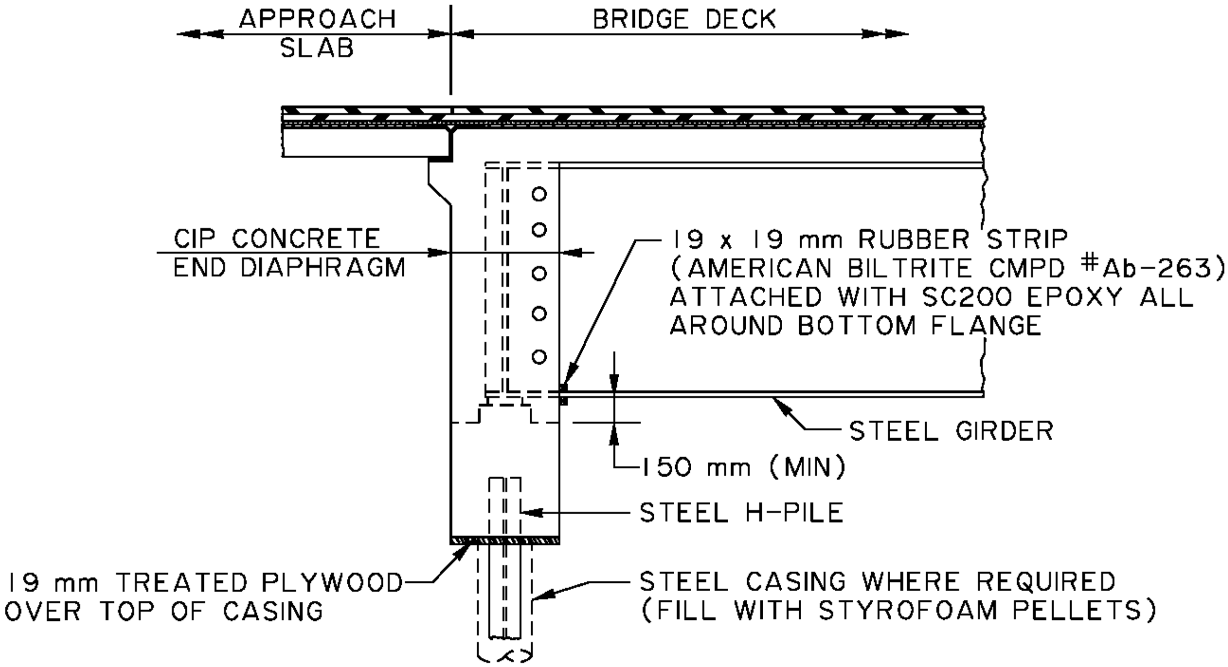


FIG 1 FULLY INTEGRAL ABUTMENT ON PILES – STEEL GIRDERS

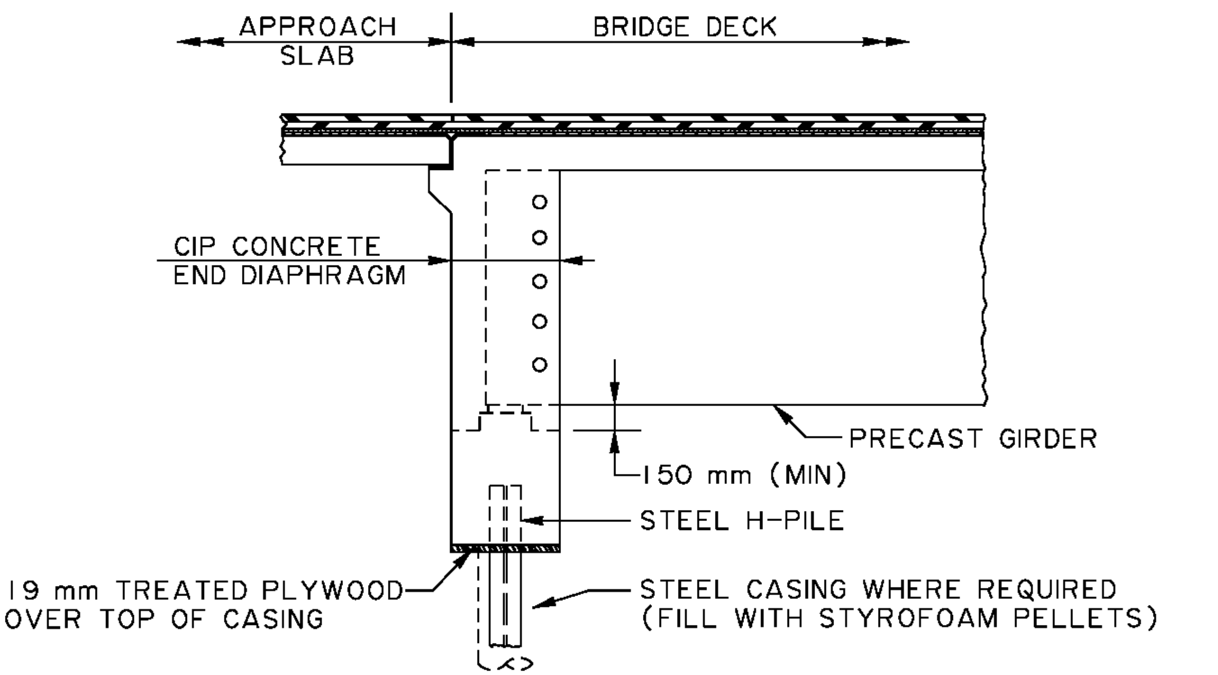


FIG 2 FULLY INTEGRAL ABUTMENT ON PILES – PRECAST GIRDER

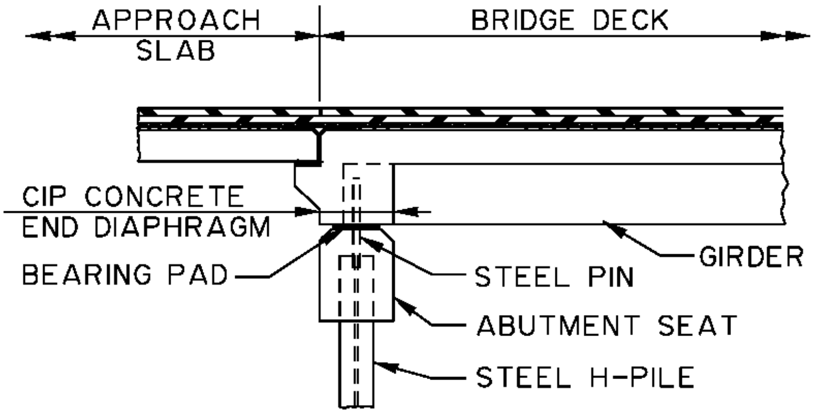


FIG 3 SEMI-INTEGRAL ABUTMENT WITH PINNED CONNECTION

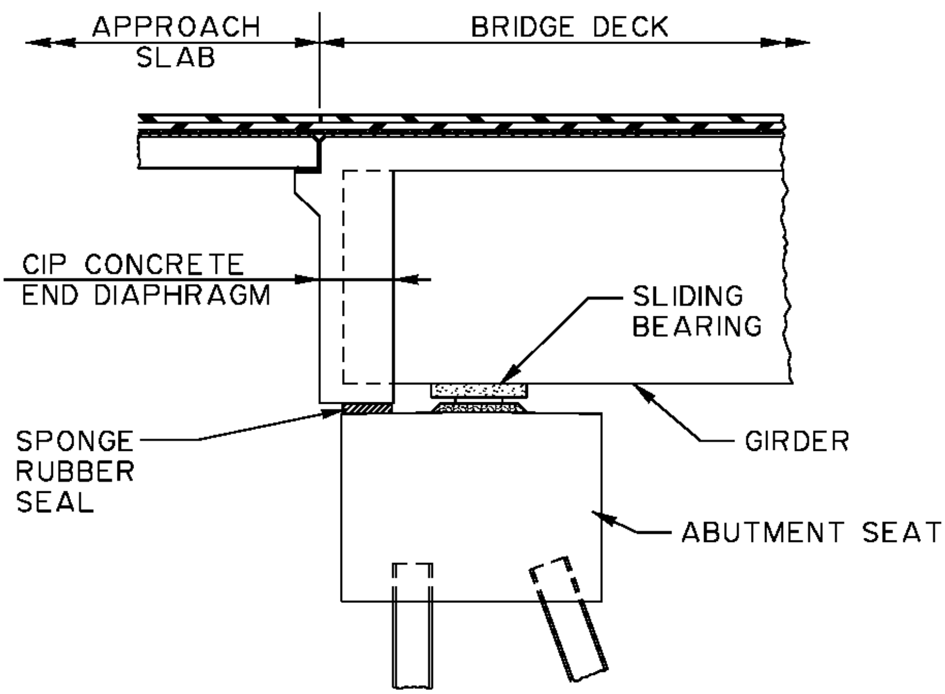


FIG 4 SEMI - INTEGRAL ABUTMENT WITH SLIDING BEARINGS

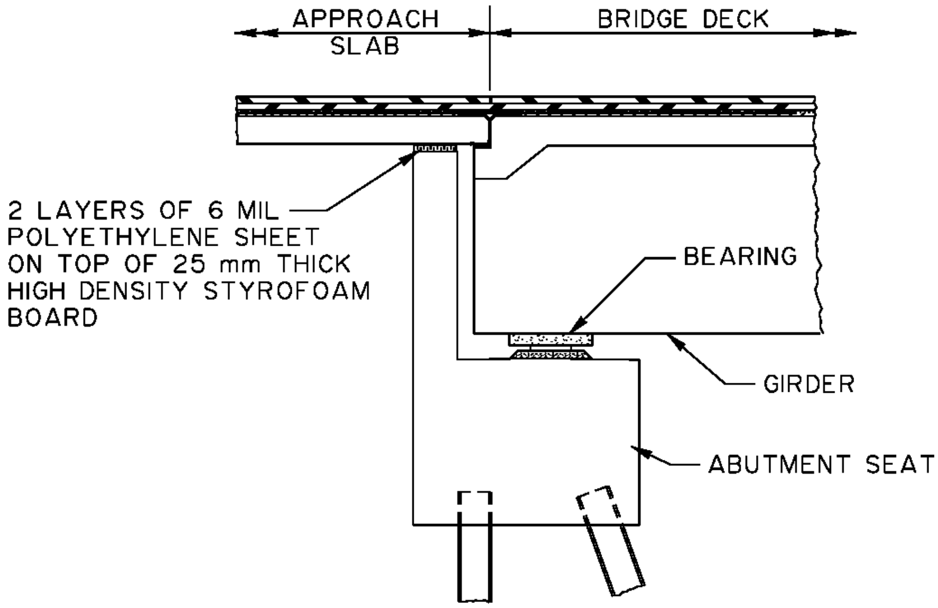


FIG 5 SEMI – INTEGRAL ABUTMENT WITH PARTIAL BACKWALL

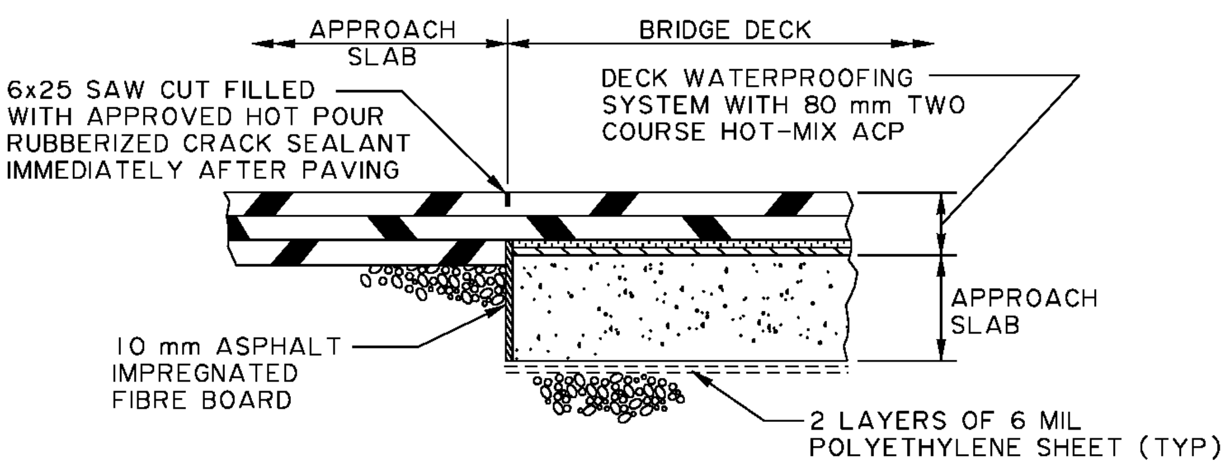


FIG 6 CYCLE CONTROL JOINT TYPE C1

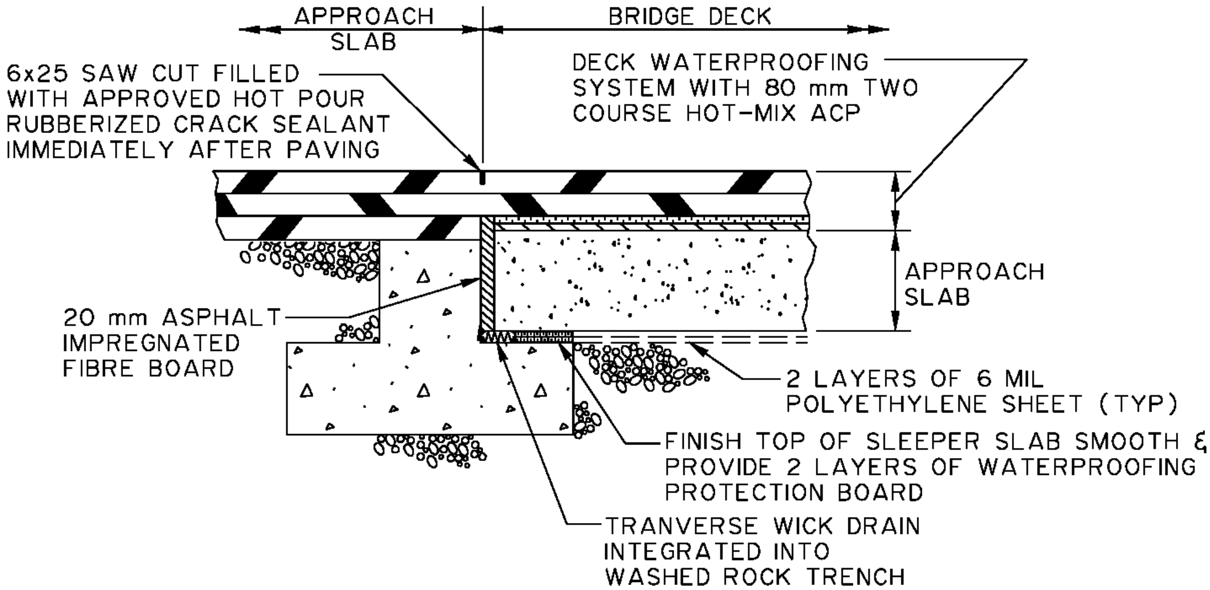


FIG 7 CYCLE CONTROL JOINT TYPE C2

Superceded by Version 8.0
April 31, 2017

Alberta Transportation Bridge Structures Design Criteria v. 7.0

Appendix C

PROTECTION SYSTEM STANDARDS FOR BRIDGE COMPONENTS

Date Published: May 31, 2012

Alberta Government

Table of Contents

C.1 INTRODUCTION

Alberta's geographic location dictates that highway maintenance contractors (HMCs) use 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 pre-mature deterioration of bridge components and often results in the need for bridge rehabilitation. Bridge rehabilitation is most often completed using staged construction and can have significant impact on the travelling public. Over the past decades Alberta Transportation (AT) has refined its protection system standards and maintenance policies to minimize the impact that winter roadway maintenance materials have on its bridge inventory.

Since the mid 1980's, AT's standard protection system for bridge components exposed to de-icing materials has consisted of epoxy coated reinforcing steel and several iterations of a high performance concrete mix. A hot rubberized membrane, protection board, and ACP system (standard drawing *S-1443*) has also been specified for the additional protection of bridge decks, roof slabs and approach slabs. Epoxy coated reinforcing steel was thought to provide improved corrosion resistance in aggressive environments, but research and field performance over the past several years has shown that epoxy coated reinforcing does not provide the improved corrosion resistance that was expected. Alternative reinforcing steel materials have evolved significantly since the mid 1980's and can provide a substantial reduction in life cycle and user costs with minimal impact on overall construction costs.

C.2 NEW STANDARD PROTECTION SYSTEM

The standard AT protection system shall consist of:

- Reinforcing steel with improved resistance to corrosion – herein referred to as corrosion resistant reinforcing (CRR);
- Class HPC concrete;
- Hot rubberized membrane with protection board and ACP for bridge decks, roof slabs, and approach slabs (refer to standard drawing *S-1443*).

The Department will not consider elimination of the hot rubberized membrane protection system when CRR materials are used. Bridge sealing programs will be maintained for concrete components directly exposed to anti-icing and de-icing chemicals.

CRR materials shall be used in the bridge components noted in *Section 6: Materials* of the *BSDC*.

C.3 CRR MATERIALS

CRR materials shall be one of the following types:

1. Solid stainless reinforcing steel of Unified Numbering System (UNS) designations S31653, S31603, S31803, S30400, 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 minimum yield strength based on 0.2% offset method shall be equal to 690 MPa. The common trademark name for ASTM A1035 compliant material is MMFX.

The Natural 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.

C.4 SELECTION OF CRR MATERIALS

Selection of CRR materials shall be based on a structure's exposure class. The exposure class shall be determined using Figure 1 for undivided highways and Figure 2 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 1
Reinforcing Steel Exposure Class Selection Chart for Undivided Highways

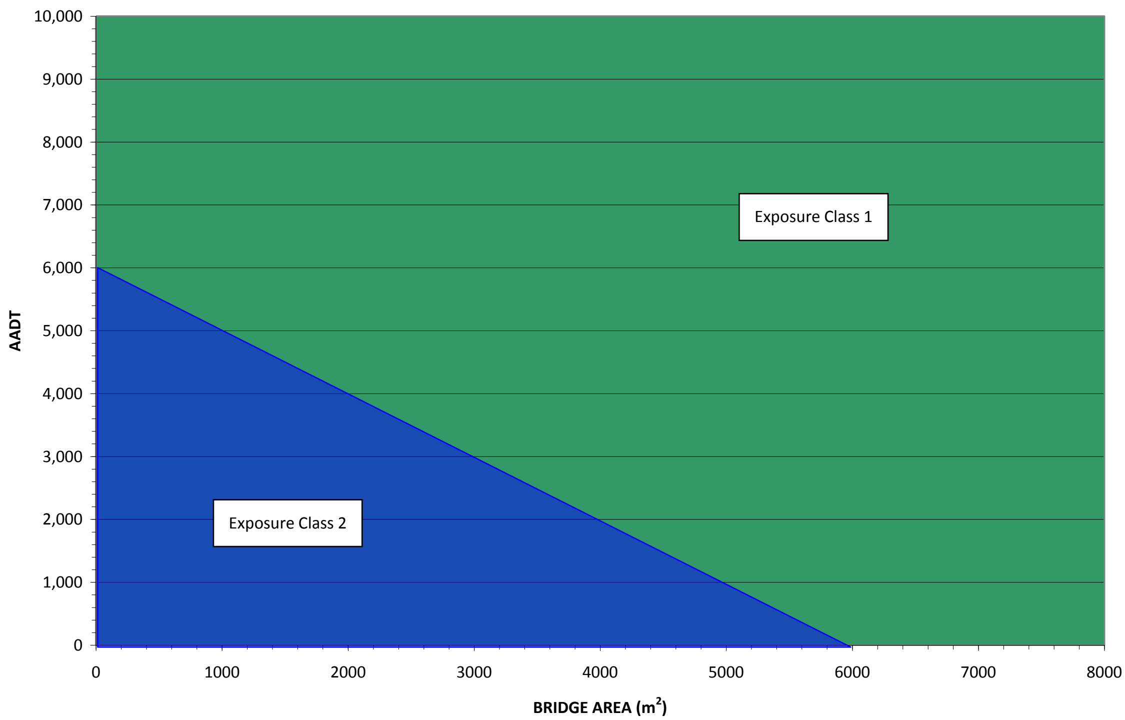
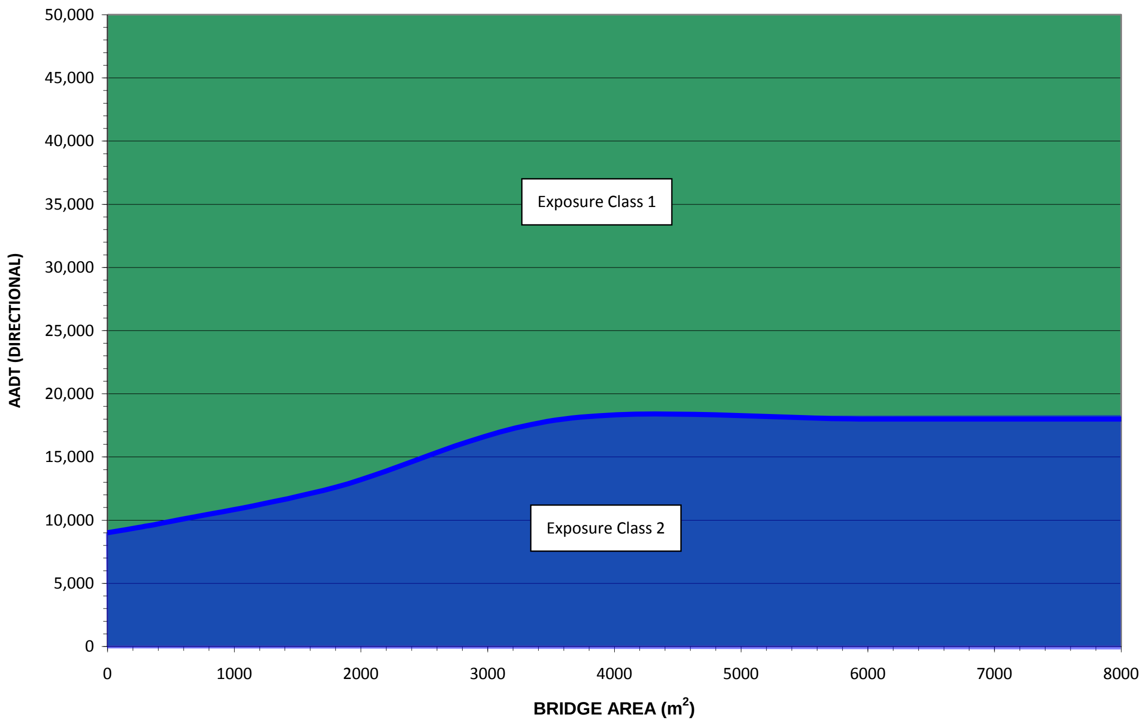


Figure 2
Reinforcing Steel Exposure Class Selection Chart for Divided Highways



Allowable CRR material types and grades for the applicable exposure class are listed in Table 1.

Table 1 – List of CRR Materials

Exposure Class	Allowable CRR Materials	Grades
1	*Solid Stainless Reinforcing Steel*	*- UNS S31653 - UNS S31603 - UNS S31803 - UNS S32304 - UNS S30400*
2	*Corrosion Resistant Reinforcing Steel*	*Solid stainless reinforcing steel (as noted in exposure class 1) or ASTM A1035 (Low carbon/ chromium reinforcing steel)*

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 <u>not</u> incorporated in 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 project sponsor.

C.5 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 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 2. Any proposed substitution of metric for imperial bars must be accepted by the Consultant.

Table 2 – 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*

C.6 DETAILING CONSIDERATIONS

The available sizes and lengths of CRR materials are listed in Table 3.

Table 3 – 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 12192 mm (40') for all 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*.

C.7 DESIGN DRAWING CONSIDERATIONS

Bridges in Exposure Class 1 require stainless steel. Bar marks shall be denoted with a 'SS' suffix on the drawings. The following as-constructed note shall be added to the applicable design drawings:

"BARS DENOTED AS 'SS' ARE UNS S##### WITH A YIELD STRENGTH OF ### MPA"

Bridges in Exposure Class 2 require corrosion resistant reinforcing (stainless steel or ASTM A1035). Bar marks shall be denoted with a 'CR' suffix on the drawings. The following as-constructed note shall be added to the applicable design drawings:

"BARS DENOTED AS 'CR' ARE *STAINLESS STEEL UNS S#####* WITH A YIELD STRENGTH OF ### MPA"

or

"BARS DENOTED AS 'CR' ARE *ASTM A1035* WITH A YIELD STRENGTH OF ### MPA"

C.8 CONTRACT AND SPECIFICATION CONSIDERATIONS

A supplementary specification for *SBC Section 5: Reinforcing Steel* has been published and is available on the Department's website. The supplemental specification should be reviewed and incorporated into all tender documents. Bid items in the unit price schedule shall follow the payment structure outlined in the supplemental specification and be consistent with design drawing bar marks and bar lists. The supplemental specification will be incorporated into future editions of the standard *SBC*.

The Department will be revising the applicable standard bridge drawings to include corrosion resistant reinforcing materials. During the revision period, standard drawings shall be reviewed and modifications addressed through contract Special Provisions such that the requirements of the *BSDC* are met.

C.9 REFERENCES

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

2. *ASTM A955/A955M-12* (2012). *Standard Specification for Deformed and Plain Stainless-Steel Bars for Concrete Reinforcement*. ASTM International, West Conshohocken, PA.

3. *ASTM A1035/A1035M-11* (2011). *Standard Specification for Deformed and Plain, Low-carbon, Chromium, Steel Bars for Concrete Reinforcement*. ASTM International, West Conshohocken, PA.

4. AT. *Specifications for Bridge Construction*. Alberta Transportation, Edmonton, AB (2010).

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

Superceded by Version 8.0
April 31, 2017

Alberta Transportation Bridge Structures Design Criteria v. 7.0

Appendix D

TYPICAL ABUTMENT DETAILS

Date Published: May 31, 2012

Alberta Government

D.1 INTRODUCTION

The following sketches show typical abutment details. These drawings were originally developed as part of the Department's Technical Requirements for P3 projects and are included for reference.

LIST OF SKETCHES:

SK-9 Standard Details for Fully-Integral Abutment

SK-10 Standard Details for Semi-Integral Abutment

SK-11 Standard Details for Conventional Abutment

SK-12 Standard Details for Conventional Abutment with Roof Slab

SK-13 Cycle Control Joint Details for Integral Abutments

SK-14 Drain Trough Details for Conventional Abutments

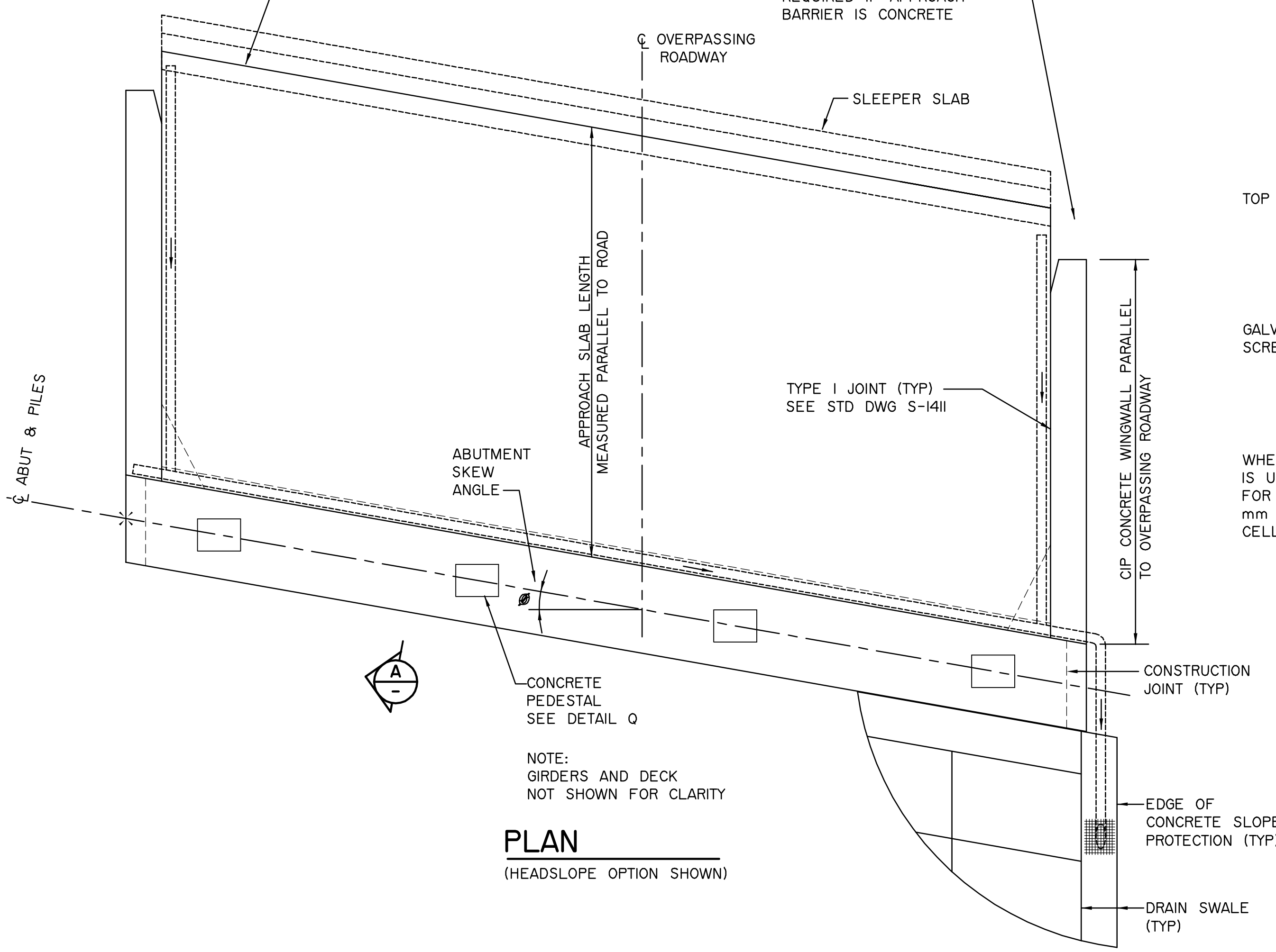
SK-15 Drain Trough Details for Integral Abutments

SK-16 Wall Layout and Site Drainage For Bridge Skew Angles ≤ 45 Degrees

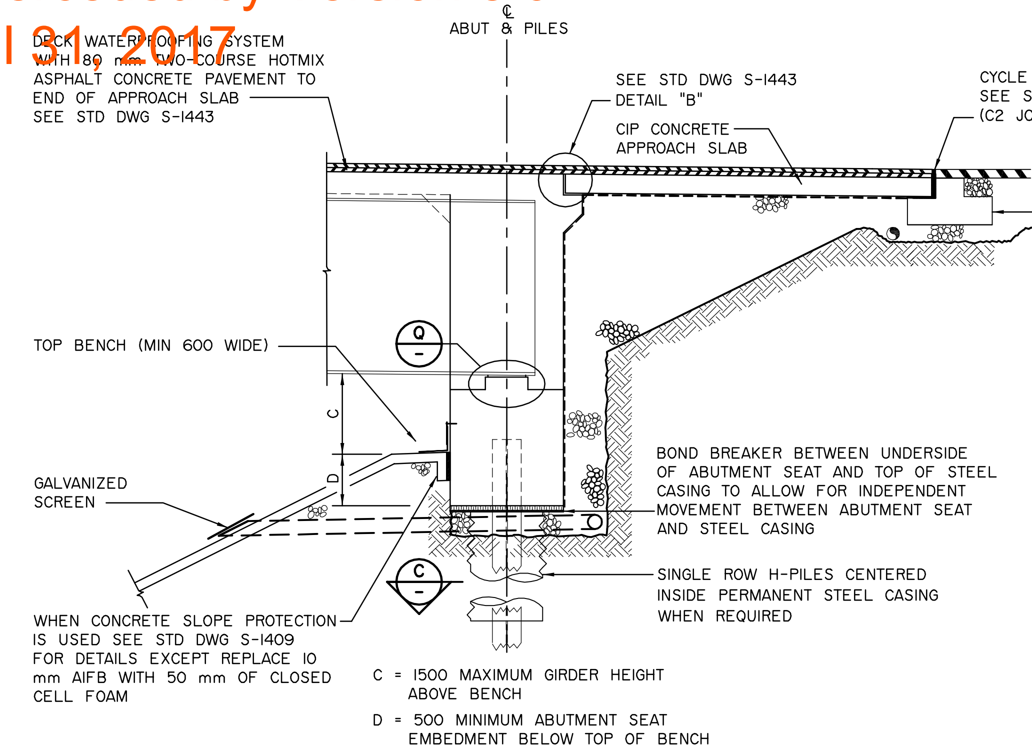
SK-17 Wall Layout and Site Drainage For Bridge Skew Angles > 45 Degrees

SK-18 Turned Back Wingwall Details For Bridge Skew Angles > 45 Degrees

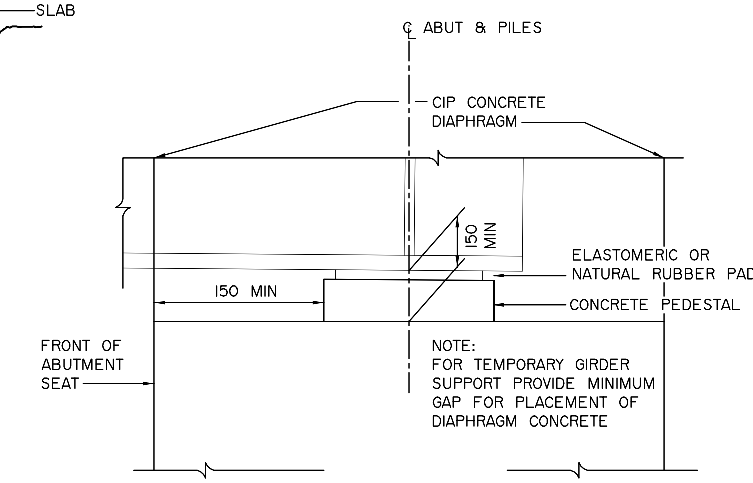
SK-19 Standard Details Associated with MSE Walls



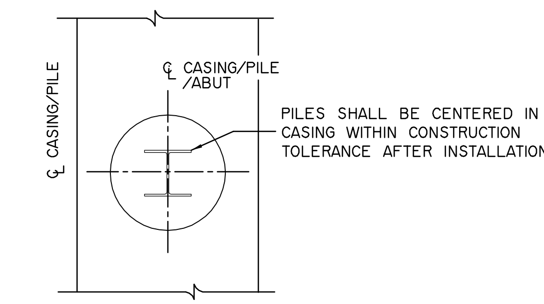
PLAN
(HEADSLOPE OPTION SHOWN)



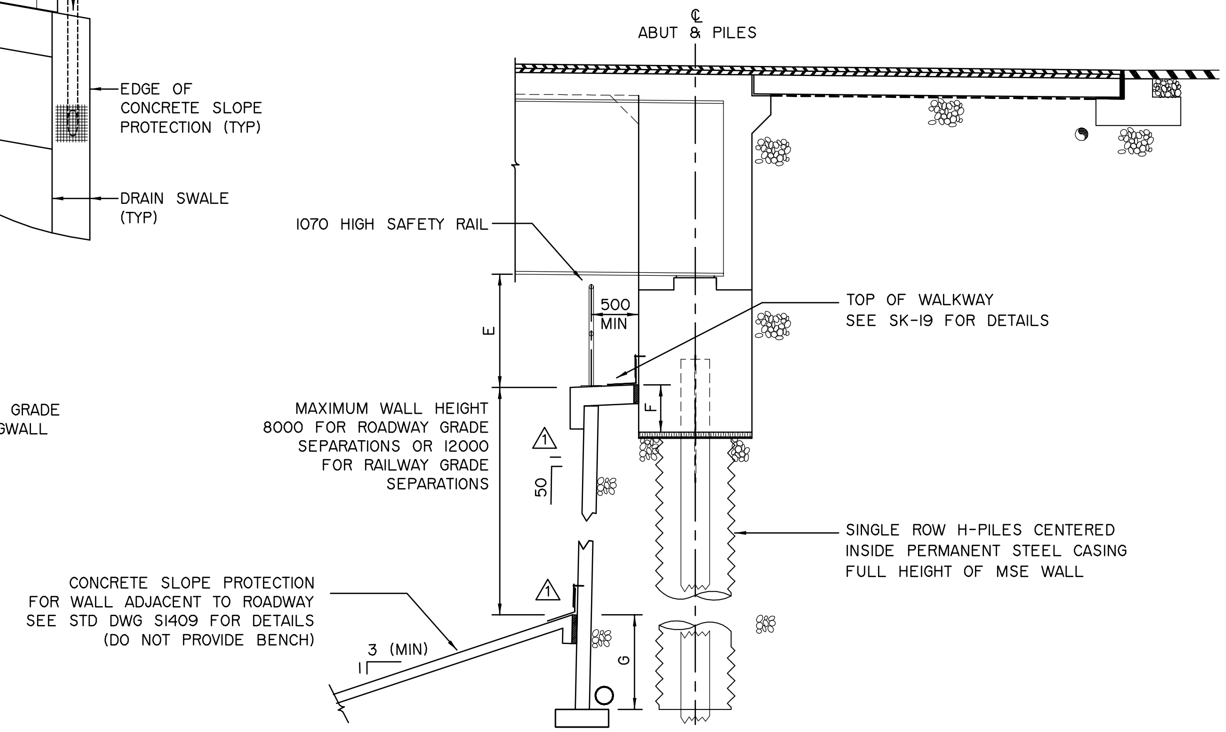
A – SECTION
HEADSLOPE OPTION



Q – PEDESTAL DETAIL



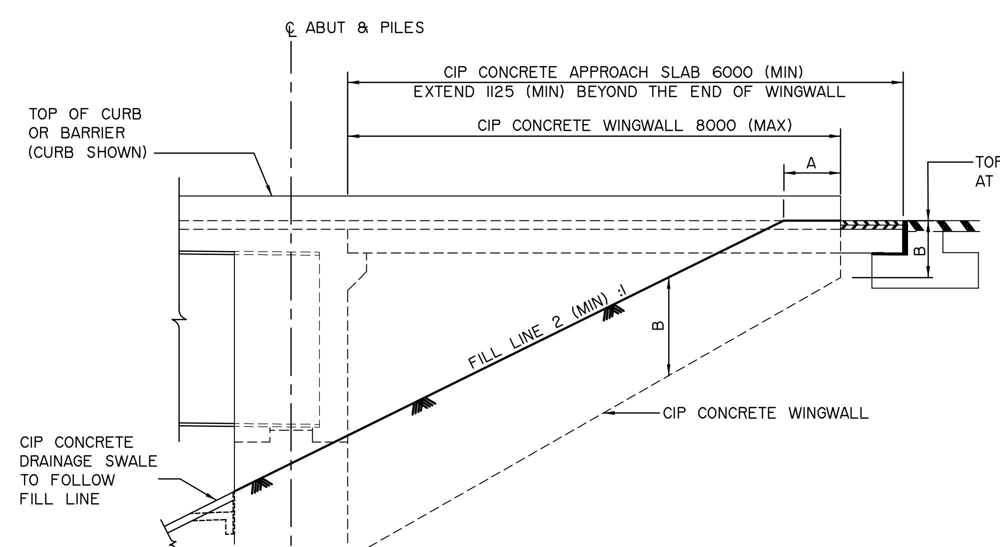
C – PILE POSITON INSIDE CASING



E = 1500 MAXIMUM GIRDER HEIGHT ABOVE TOP OF WALKWAY
F = 500 MINIMUM ABUTMENT SEAT EMBEDMENT BELOW TOP OF WALKWAY
G = 1000 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



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)

NOTES

- ALL DIMENSIONS ARE IN MILLIMETERS
- SKETCHES ARE PROVIDED WITH THE SOLE PURPOSE OF ILLUSTRATING SELECTED SCHEDULE 18 REQUIREMENTS. THEY INTENTIONALLY OMIT DETAILS THAT MIGHT DETRACT FROM THIS PURPOSE

REV	DATE	REVISIONS	BY
4	.	.	.
3	.	.	.
2	.	.	.
1	2011.10.31	WALL SLOPE & FLASHING	TL

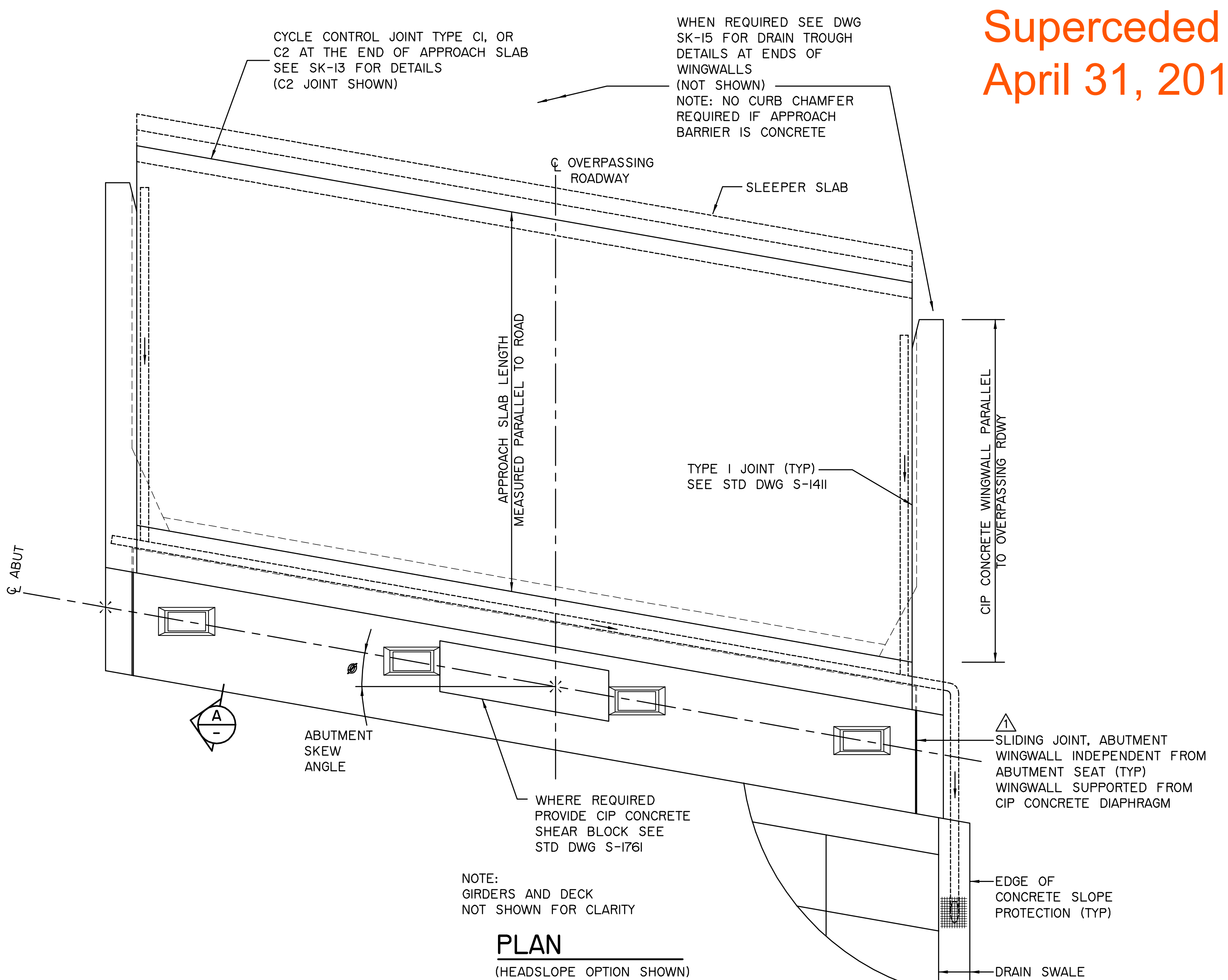
Government of Alberta ■ Transportation

NORTHEAST ANTHONY HENDAY DRIVE
STANDARD DETAILS FOR
FULLY-INTEGRAL ABUTMENT

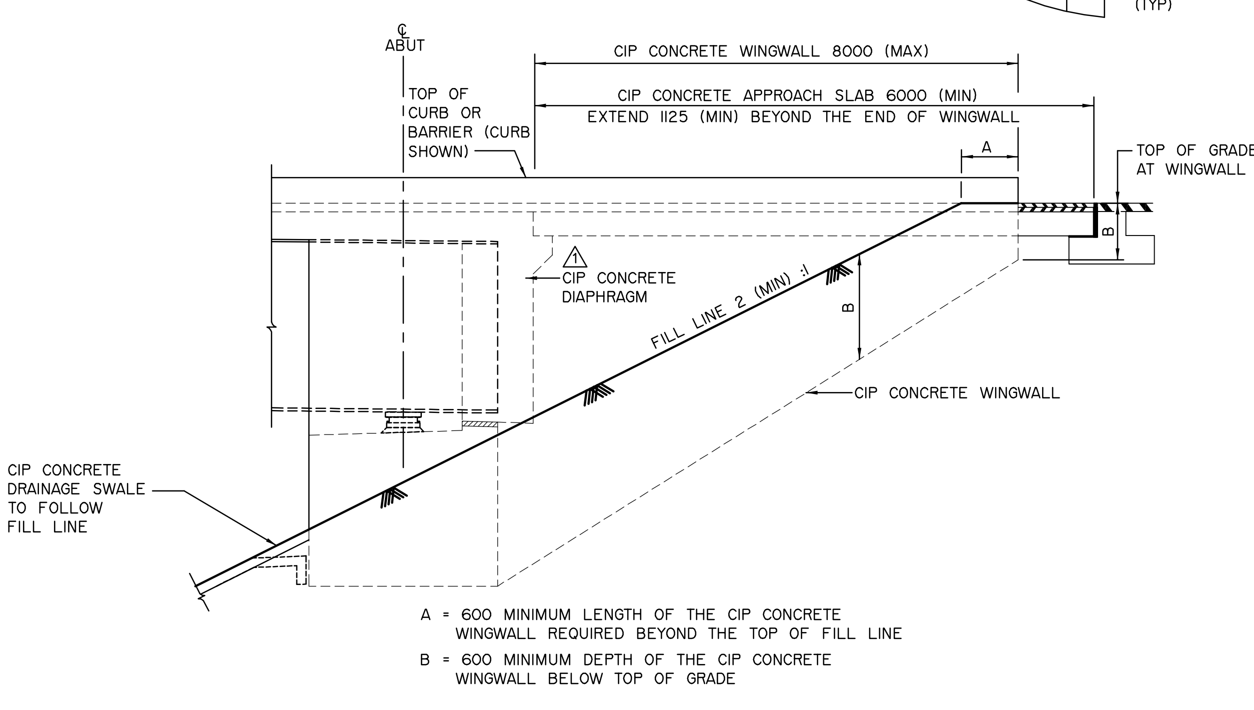
AT FILE NUMBER	DATE DRAWN	SHEET	DRAWING
–	2011-05-11	1 of 11	SK-9

RWK NOV.01, 2011 13010-SK09-RV1.DWG

Superceded by Version 8.0
April 31, 2017



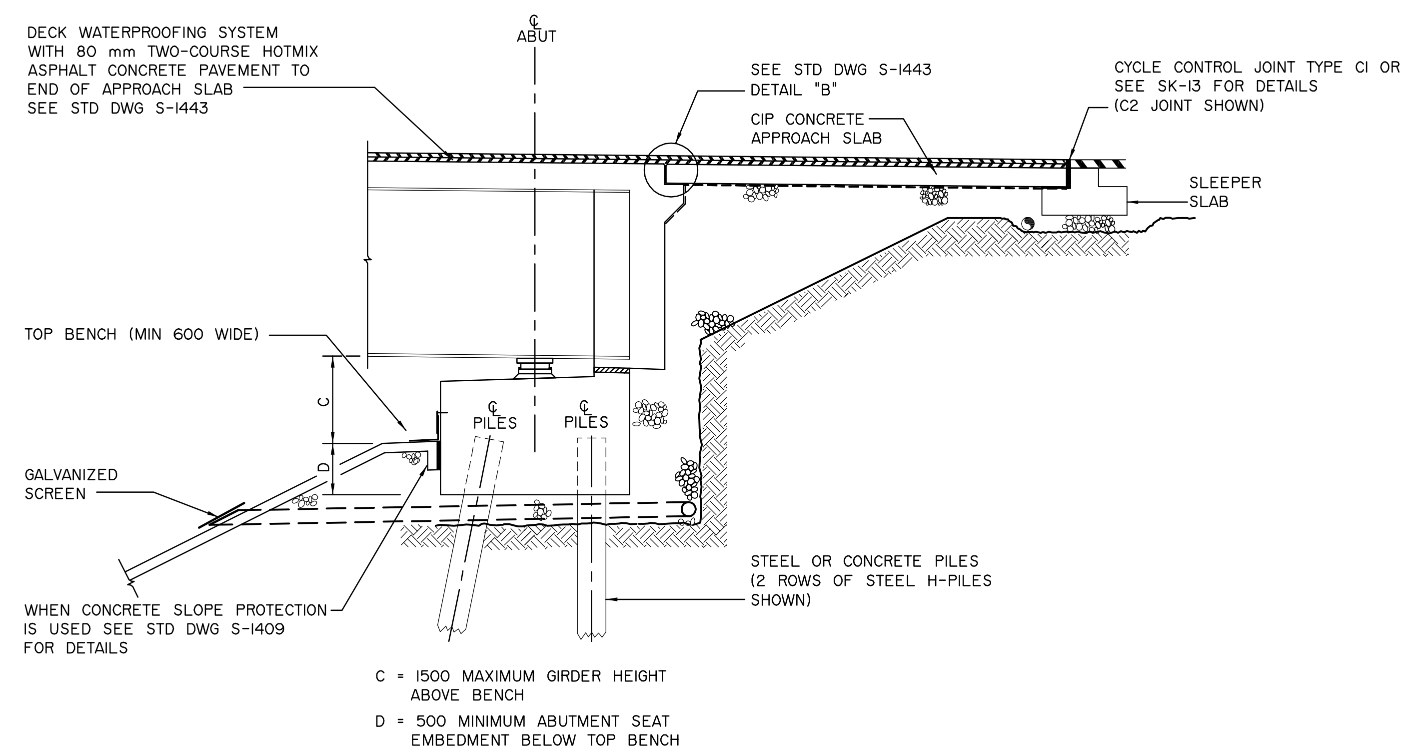
PLAN
(HEADSLOPE OPTION SHOWN)



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

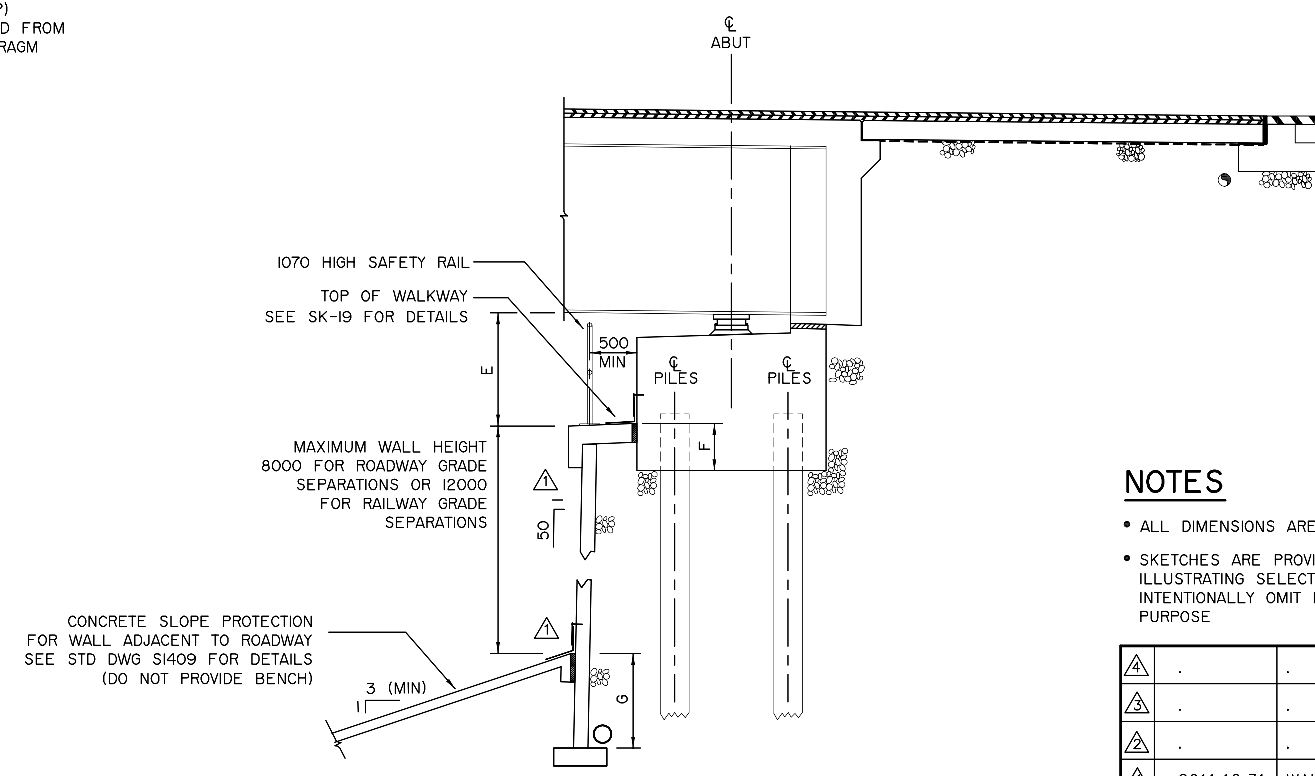
ELEVATION
(HEADSLOPE OPTION SHOWN)



C = 1500 MAXIMUM GIRDER HEIGHT ABOVE BENCH

D = 500 MINIMUM ABUTMENT SEAT EMBEDMENT BELOW TOP BENCH

A SECTION
HEADSLOPE OPTION



E = 1500 MAXIMUM GIRDER HEIGHT ABOVE TOP OF WALKWAY

F = 500 MINIMUM ABUTMENT SEAT EMBEDMENT BELOW TOP OF WALKWAY

G = 1000 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 MILLIMETERS
- SKETCHES ARE PROVIDED WITH THE SOLE PURPOSE OF ILLUSTRATING SELECTED SCHEDULE 18 REQUIREMENTS. THEY INTENTIONALLY OMIT DETAILS THAT MIGHT DETRACT FROM THIS PURPOSE

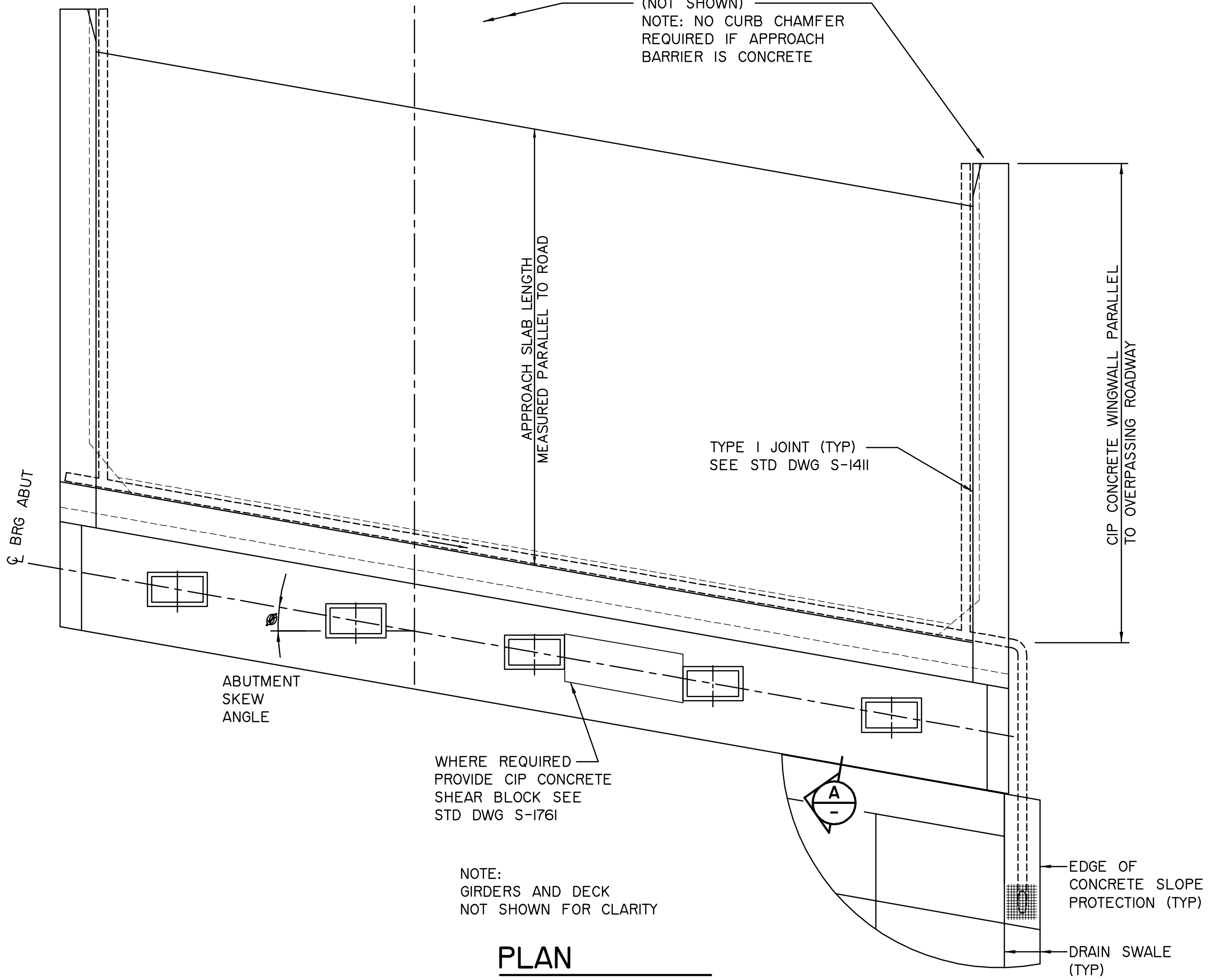
REV	DATE	REVISIONS	BY
4	.	.	.
3	.	.	.
2	.	.	.
1	2011.10.31	WALL SLOPE, FLASHING & SLIDING JOINT	TL

Government of Alberta ■ Transportation

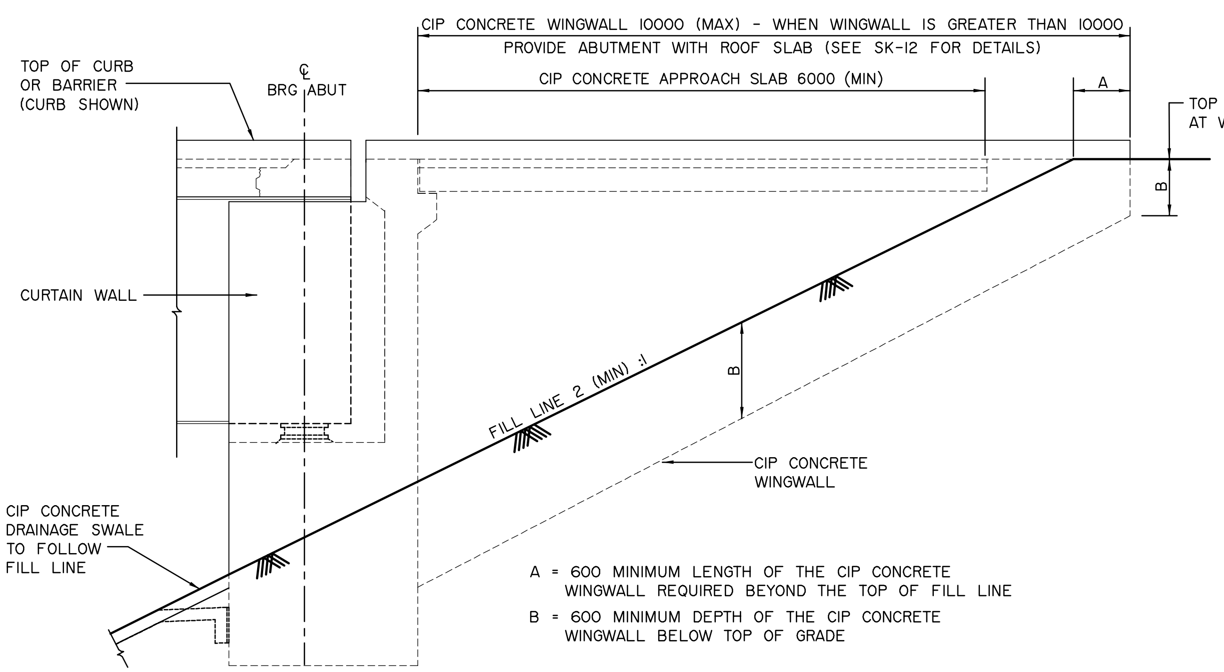
NORTHEAST ANTHONY HENDAY DRIVE
STANDARD DETAILS FOR
SEMI-INTEGRAL ABUTMENT

AT FILE NUMBER	DATE DRAWN	SHEET	DRAWING
–	2011-05-11	2 of 11	SK-10

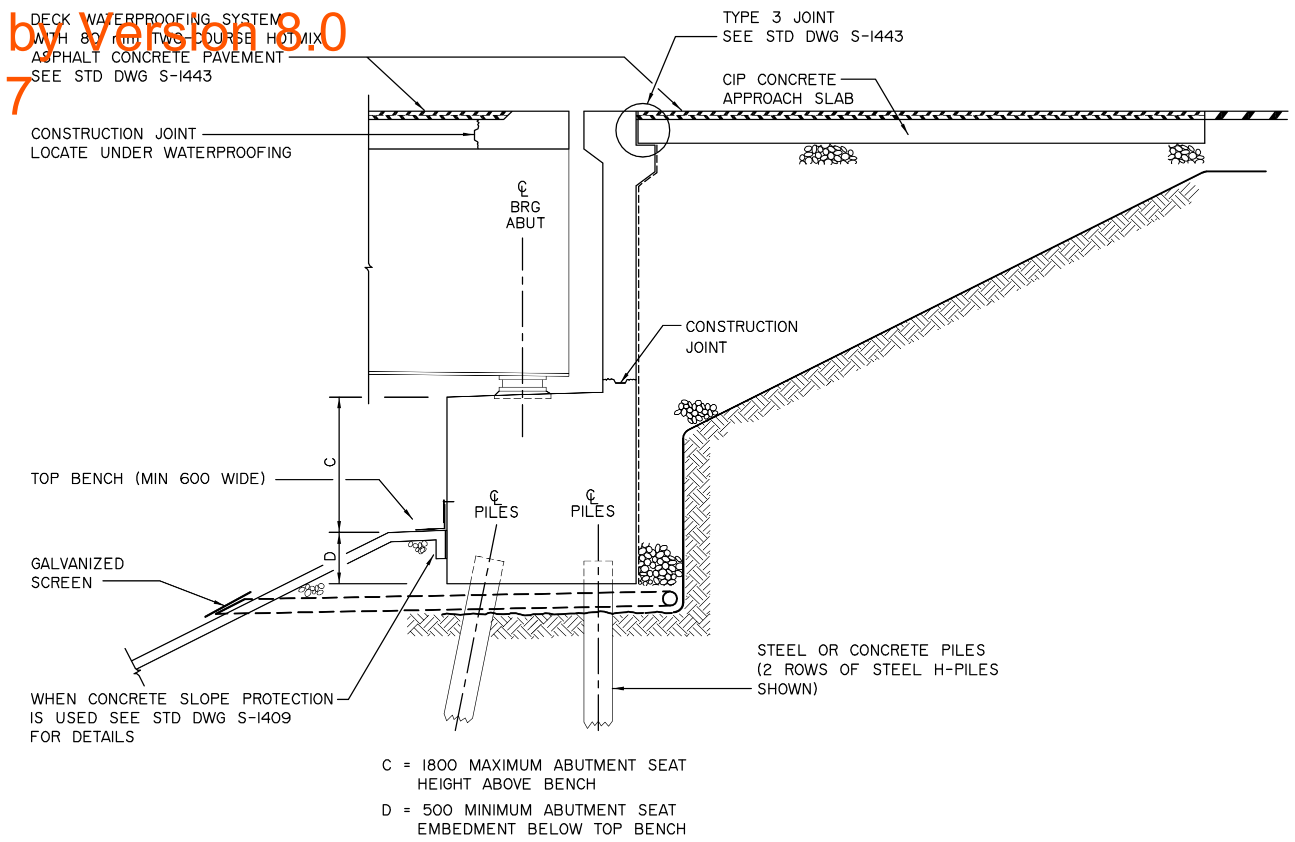
RWK NOV.01, 2011 13010-SK10-RV1.DWG



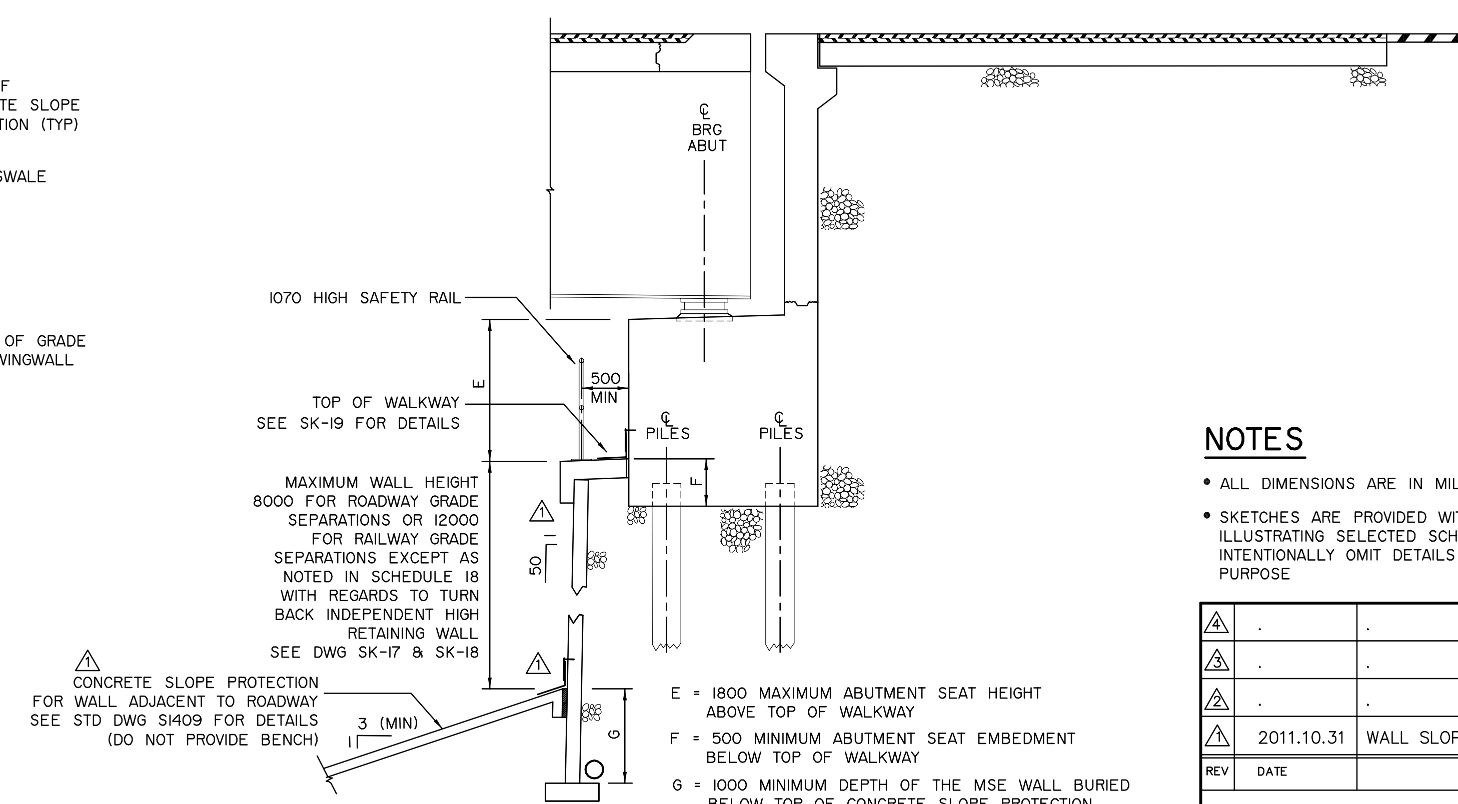
PLAN
(HEADSLOPE OPTION SHOWN)



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 MILLIMETERS
- SKETCHES ARE PROVIDED WITH THE SOLE PURPOSE OF ILLUSTRATING SELECTED SCHEDULE 18 REQUIREMENTS. THEY INTENTIONALLY OMIT DETAILS THAT MIGHT DETRACT FROM THIS PURPOSE

REV	DATE	REVISIONS	BY
4	.	.	.
3	.	.	.
2	.	.	.
1	2011.10.31	WALL SLOPE, FLASHING & SLOPE NOTE	TL

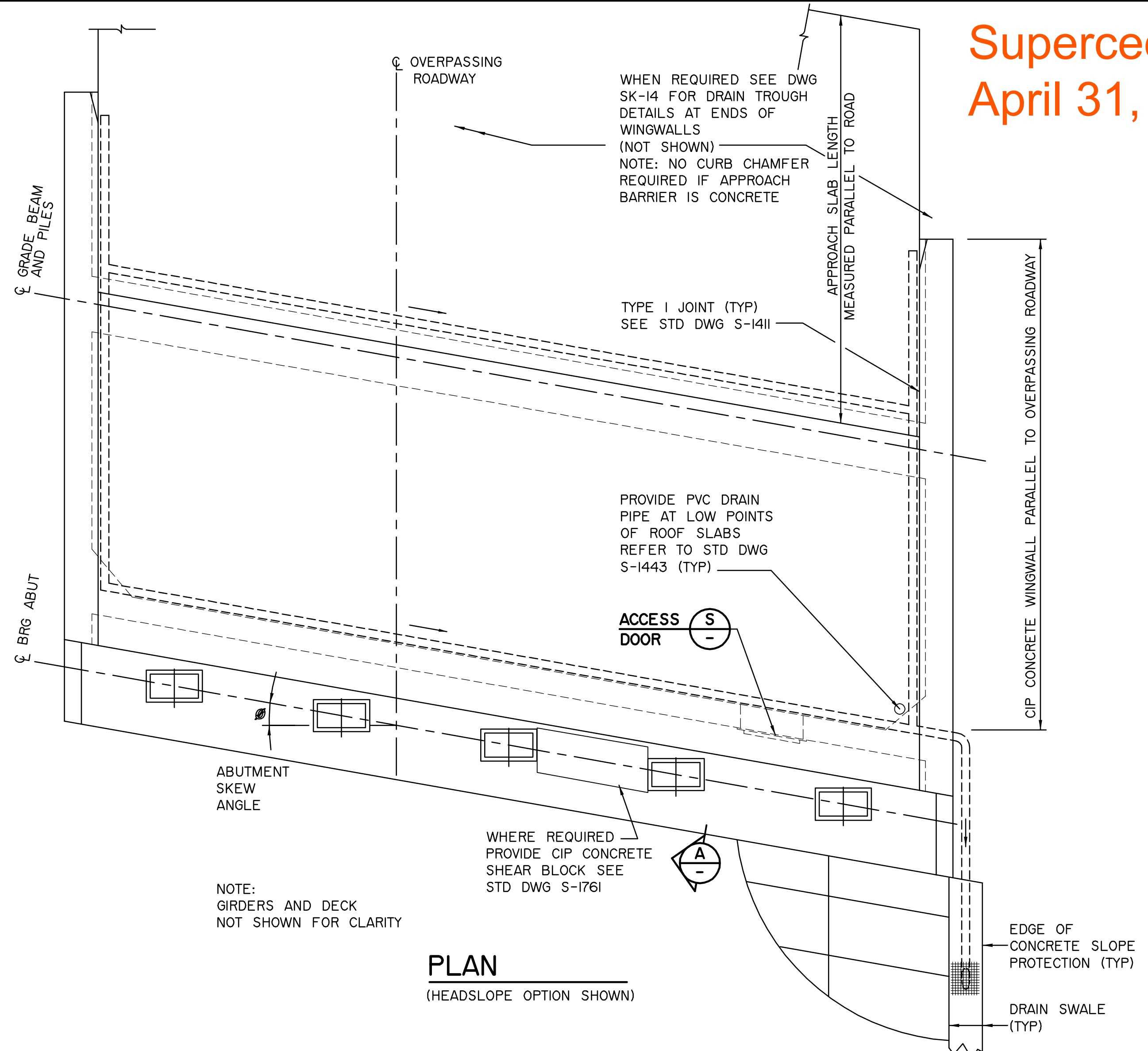
Government of Alberta ■ Transportation

NORTHEAST ANTHONY HENDAY DRIVE
STANDARD DETAILS FOR
CONVENTIONAL ABUTMENT

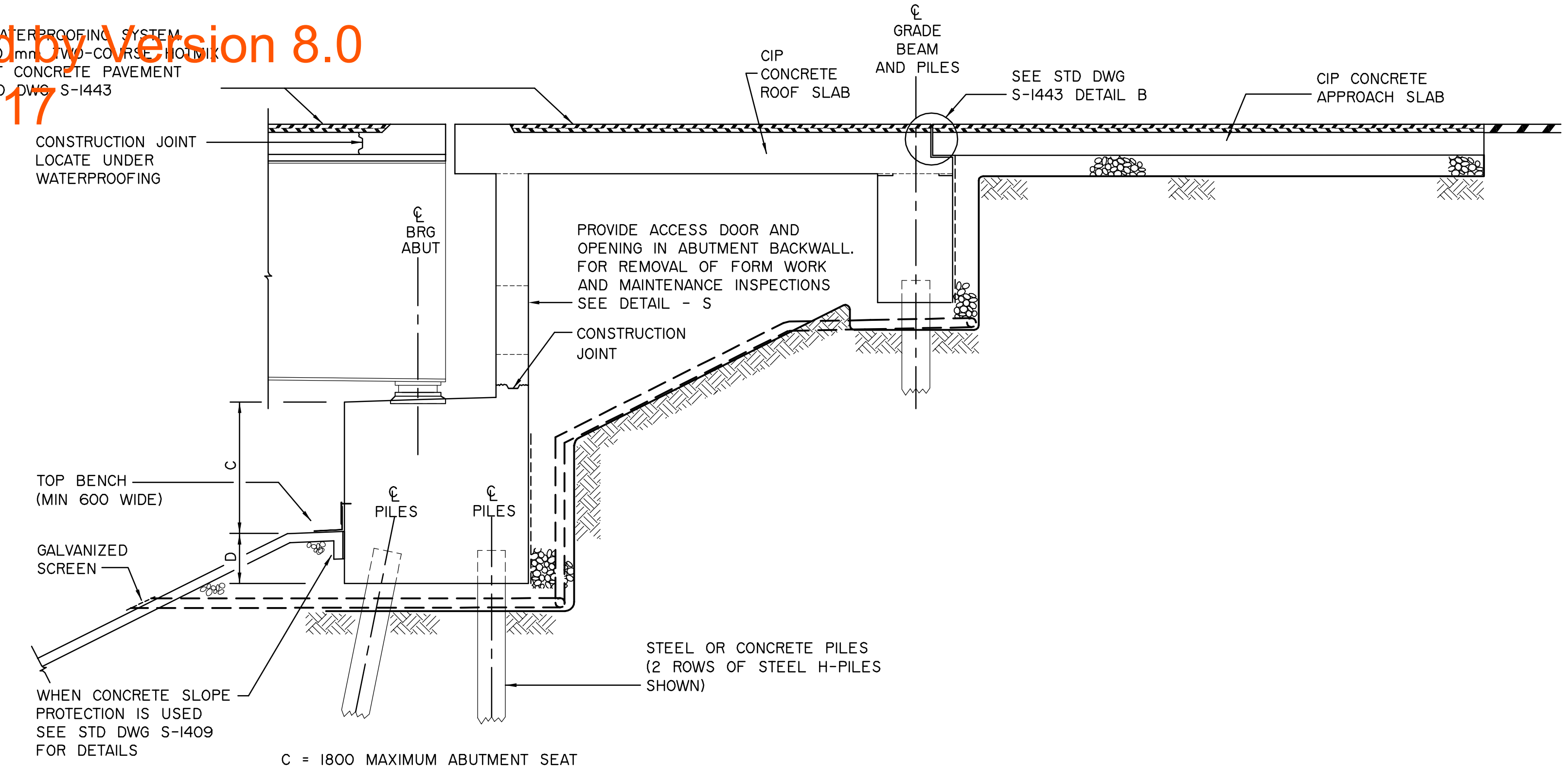
AT FILE NUMBER	DATE DRAWN	SHEET	DRAWING
–	2011–05–11	3 of 11	SK–11

RWK NOV.01, 2011 13010-SK11-RV1.DWG

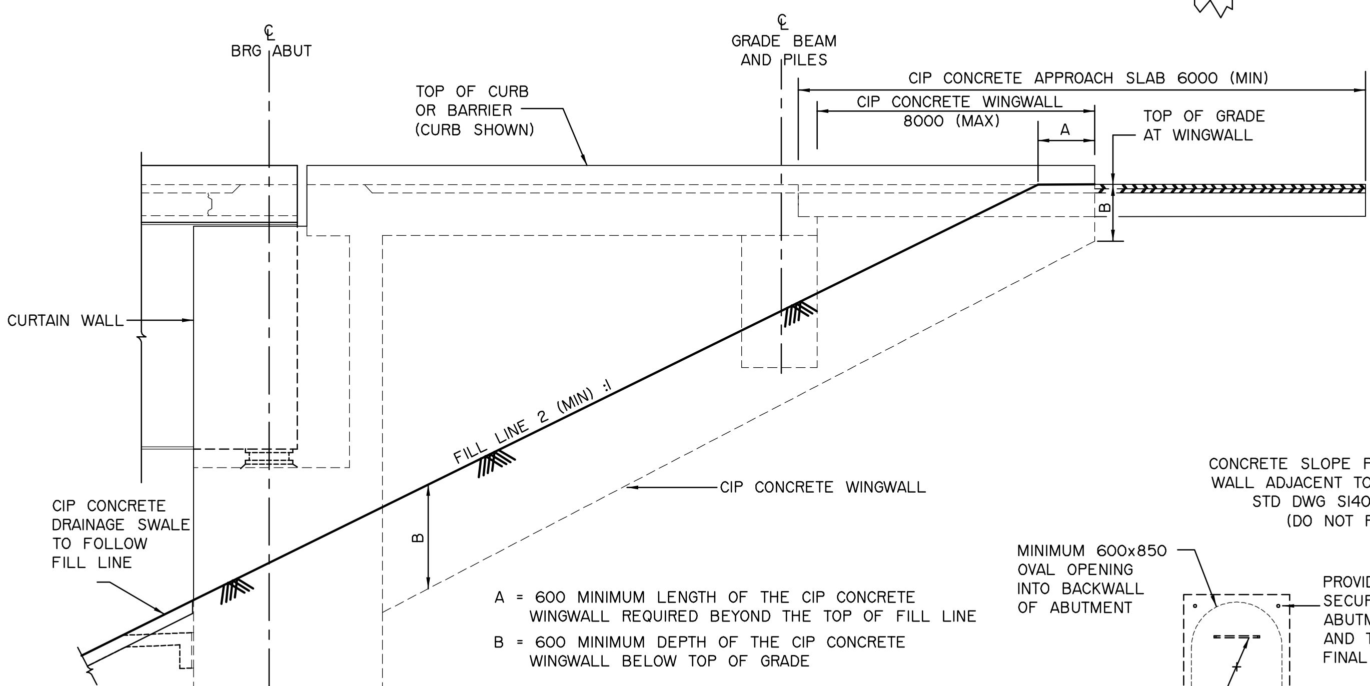
Superceded by Version 8.0
April 31, 2017



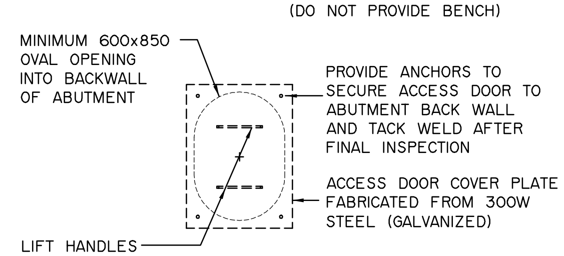
PLAN
(HEADSLOPE OPTION SHOWN)



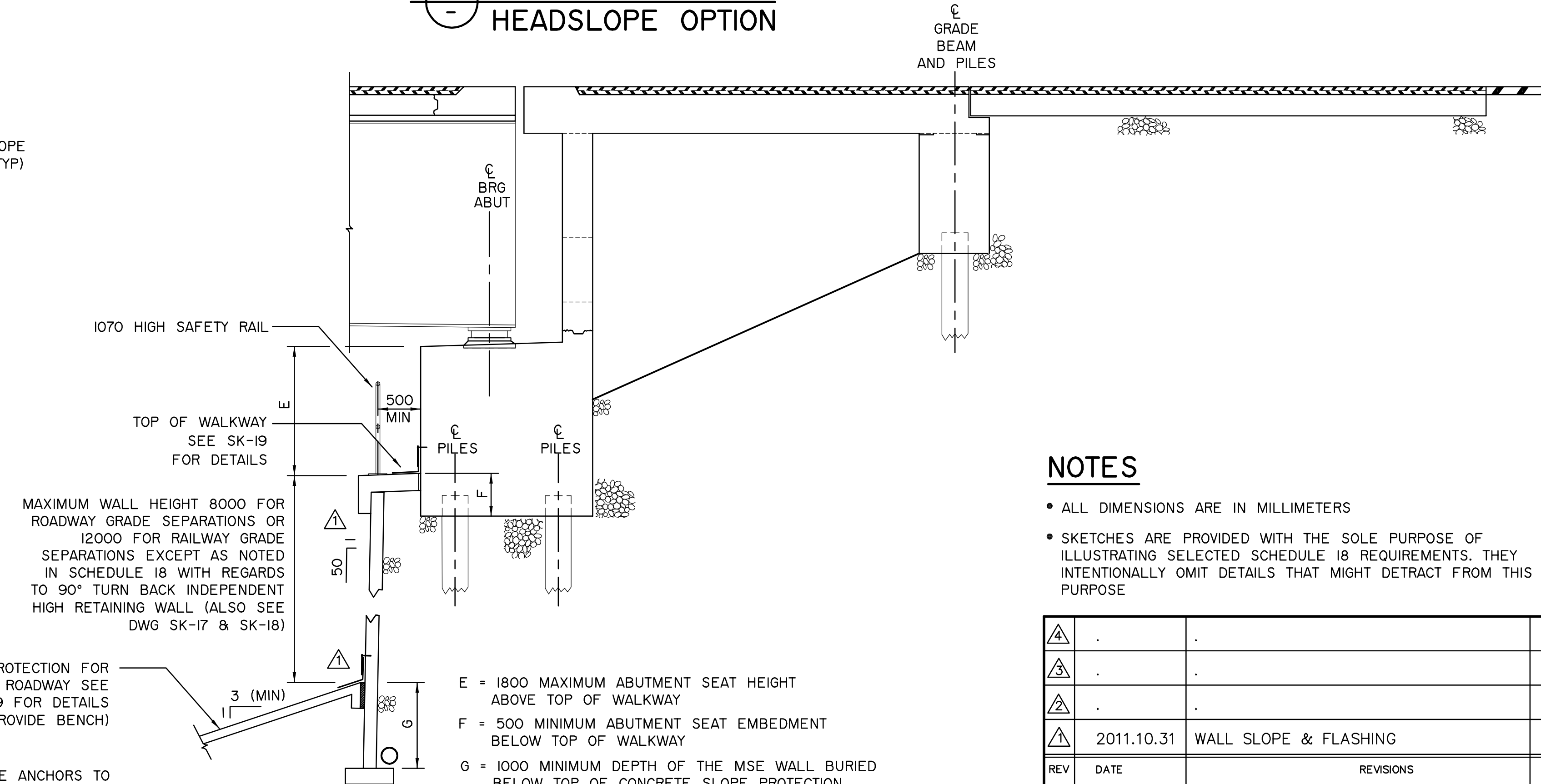
A SECTION HEADSLOPE OPTION



ELEVATION
(HEADSLOPE OPTION SHOWN)



S ACCESS DOOR



NOTE: MSE WALL SIMILAR TO HEADSLOPE OPTION EXCEPT AS NOTED

A SECTION MSE WALL OPTION

NOTES

- ALL DIMENSIONS ARE IN MILLIMETERS
- SKETCHES ARE PROVIDED WITH THE SOLE PURPOSE OF ILLUSTRATING SELECTED SCHEDULE 18 REQUIREMENTS. THEY INTENTIONALLY OMIT DETAILS THAT MIGHT DETRACT FROM THIS PURPOSE

REV	DATE	REVISIONS	BY
4	.	.	.
3	.	.	.
2	.	.	.
1	2011.10.31	WALL SLOPE & FLASHING	TL

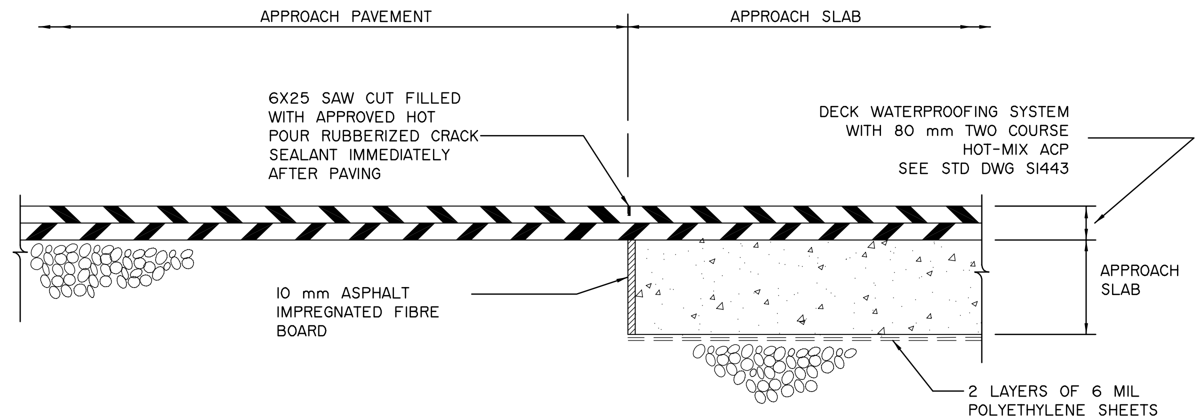
Government of Alberta ■ Transportation

NORTHEAST ANTHONY HENDAY DRIVE
STANDARD DETAILS FOR CONVENTIONAL ABUTMENT WITH ROOF SLAB

AT FILE NUMBER	DATE DRAWN	SHEET	DRAWING
–	2011–05–11	4 OF 11	SK–12

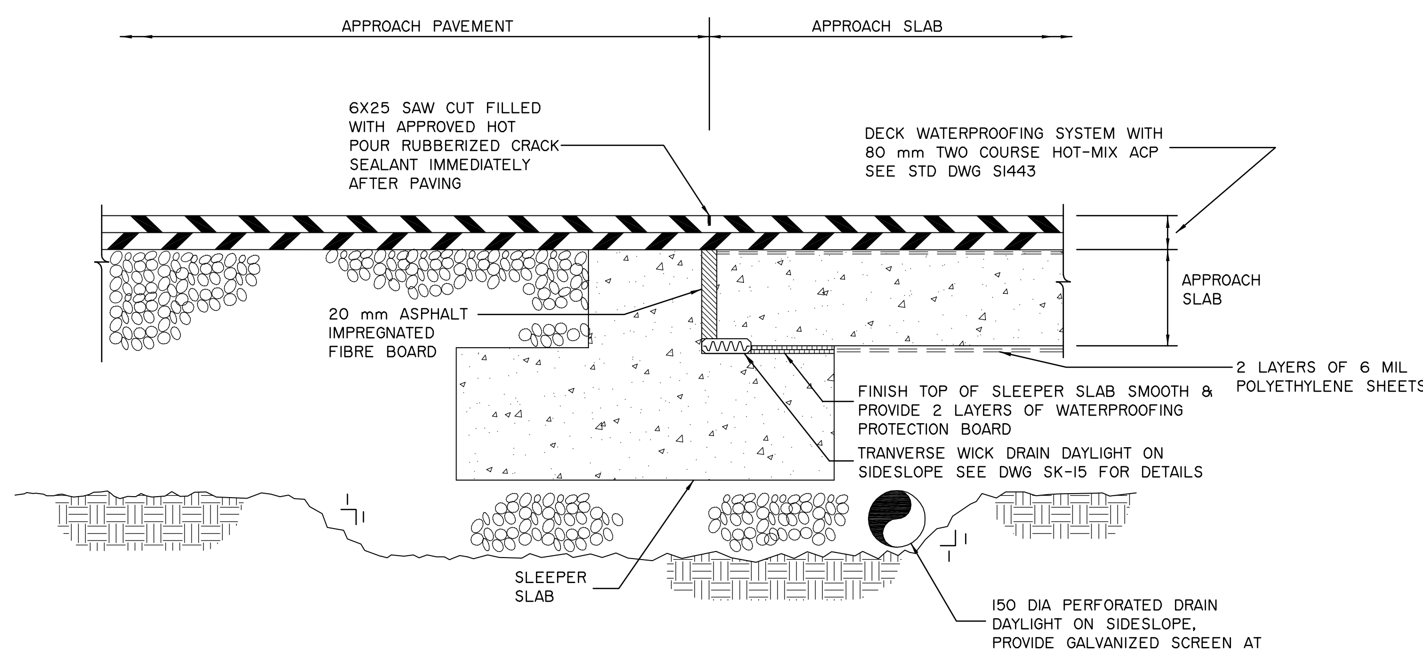
RWK NOV.01, 2011 13010-SK12-RV1.DWG

Superceded by Version 8.0
April 31, 2017



CYCLE CONTROL JOINT TYPE 1

1:10



CYCLE CONTROL JOINT TYPE 2

1:10

NOTES

- STANDARD CYCLE CONTROL JOINT TYPE C1 AND C2 DETAILS AT THE END OF APPROACH SLABS FOR INTEGRAL BRIDGES
- SEE DRAWING SK-9 AND SK-10 FOR ILLUSTRATIONS

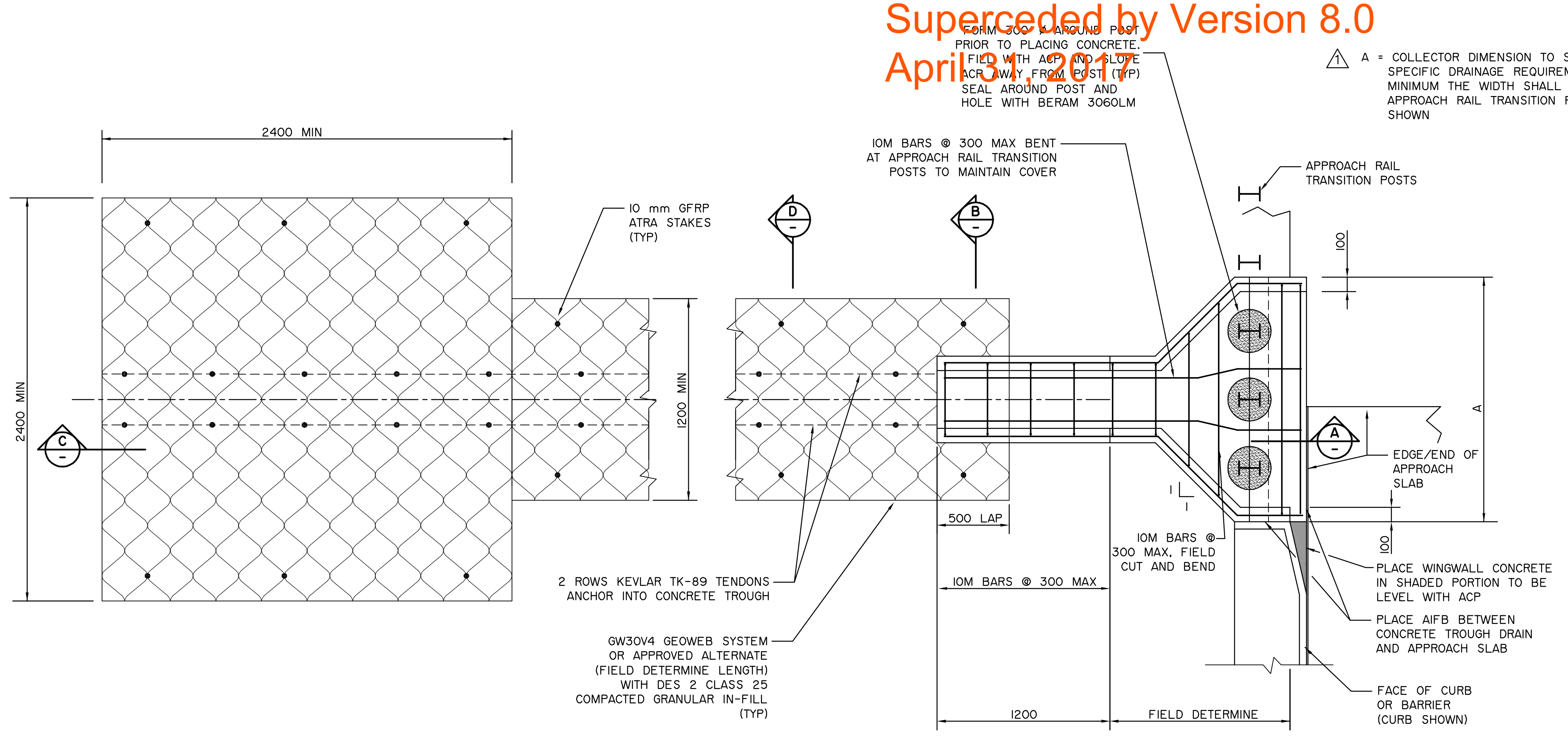
REV	DATE	REVISIONS	BY
4	.	.	.
3	.	.	.
2	.	.	.
1	.	.	.

Government of Alberta ■ Transportation

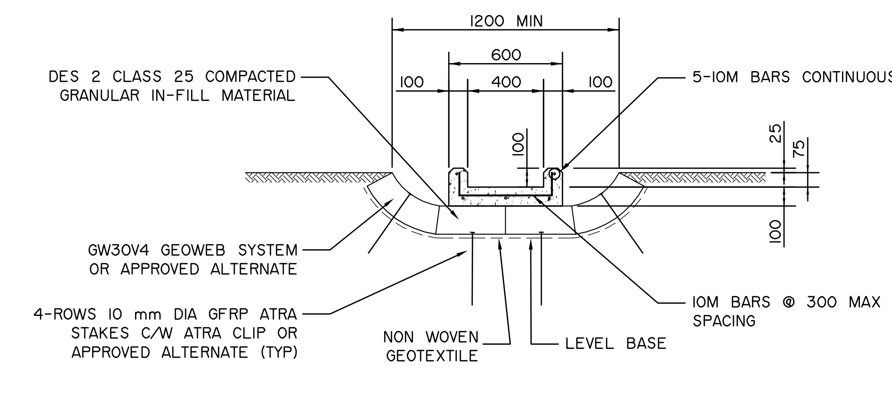
NORTHEAST ANTHONY HENDAY DRIVE
CYCLE CONTROL JOINT DETAILS
FOR INTEGRAL ABUTMENTS

AT FILE NUMBER	DATE DRAWN	SHEET	DRAWING
–	2010-09-13	5 of 11	SK-13

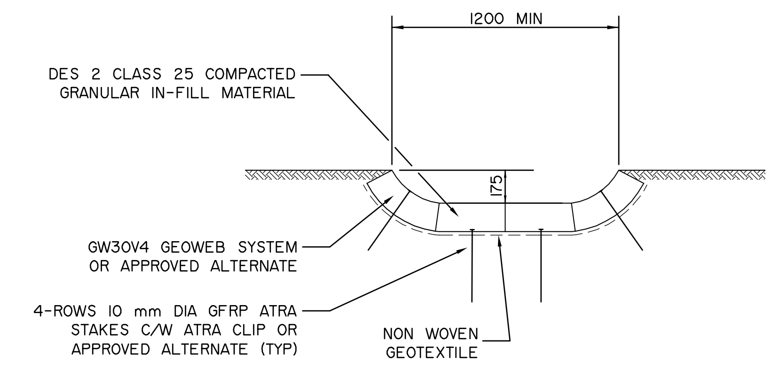
TL MAY 31, 2011 13010-SK13.DWG



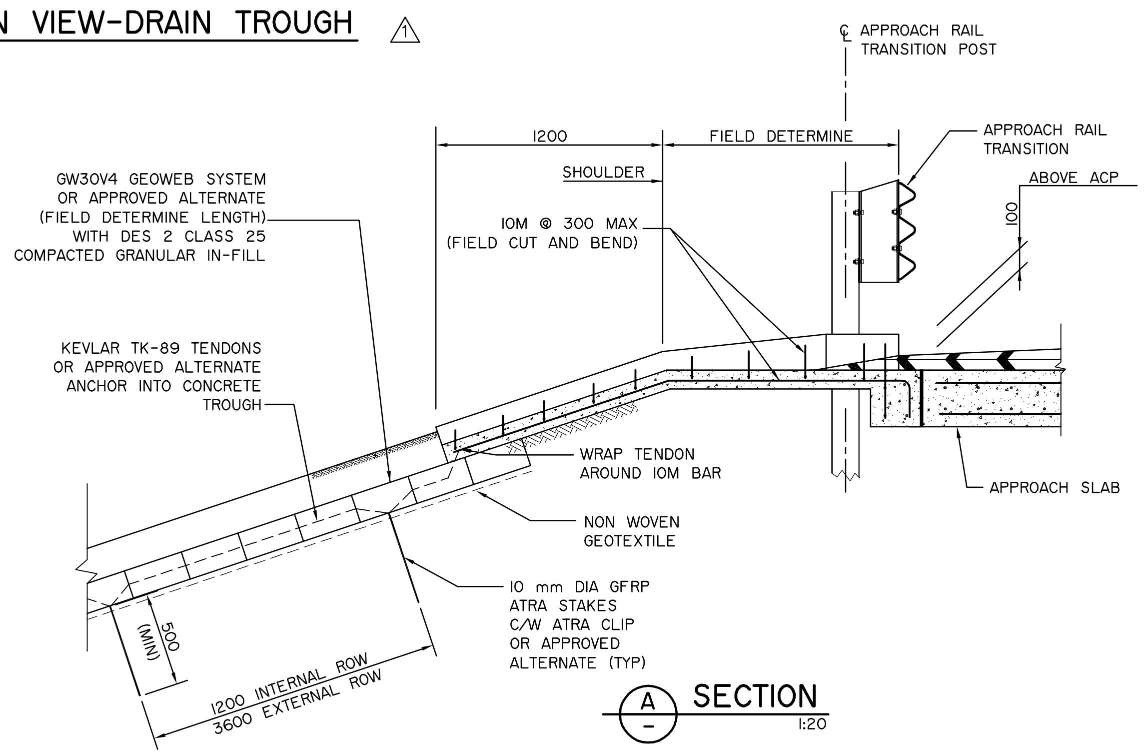
PLAN VIEW-DRAIN TROUGH 1



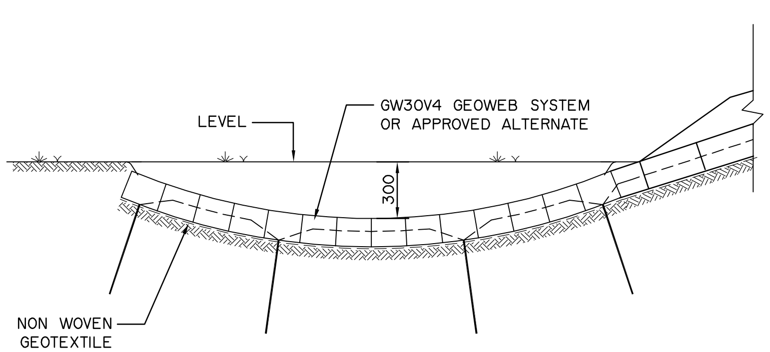
B SECTION 1:10



D SECTION 1:10



A SECTION 1:20



C SECTION NTS

NOTES

- ALL DIMENSIONS ARE MILLIMETERS
- STANDARD DRAIN TROUGH DETAILS AT THE END OF WINGWALLS FOR CONVENTIONAL ABUTMENTS

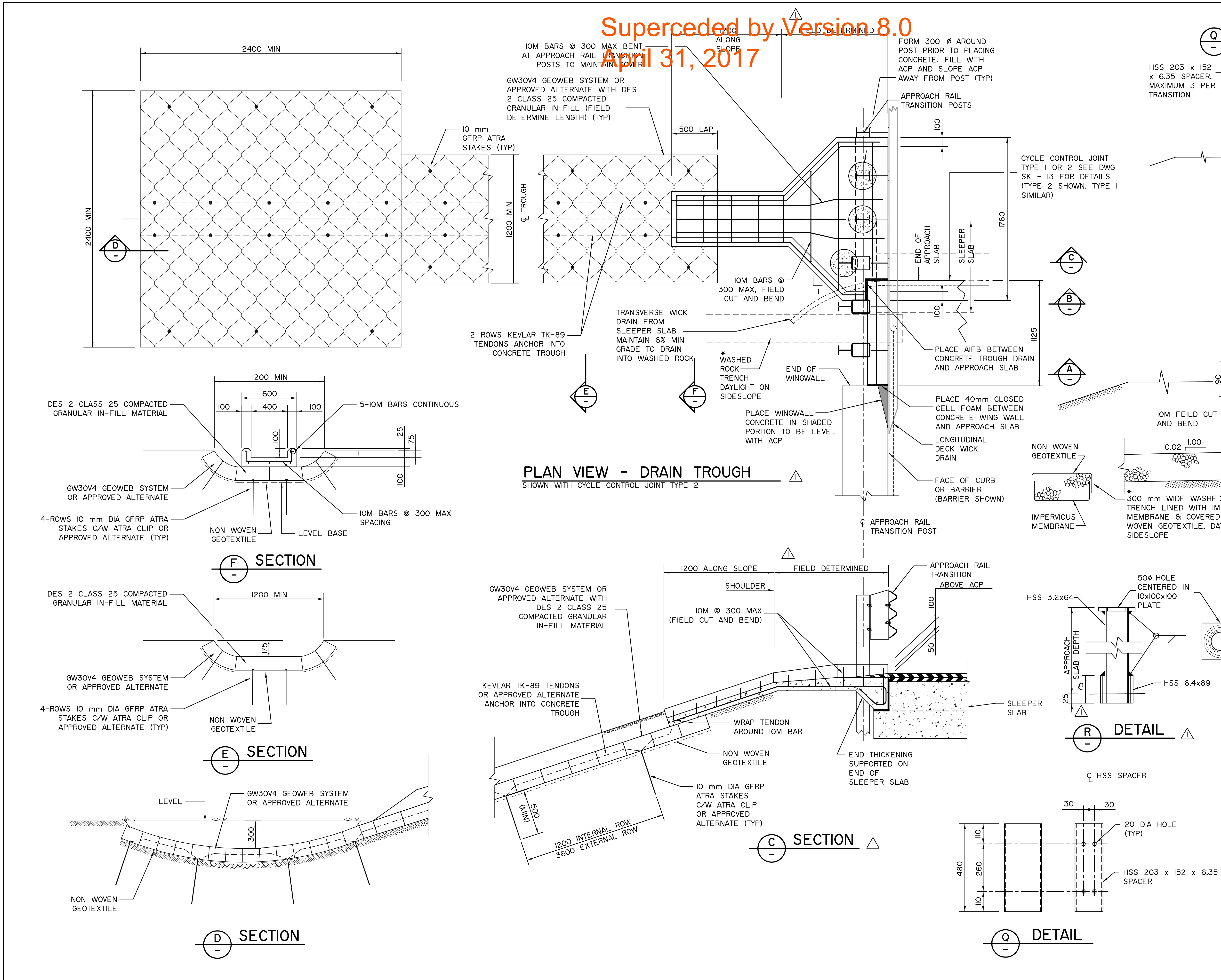
REV	DATE	REVISIONS	BY
4	.	.	.
3	.	.	.
2	.	.	.
1	2011-08-03	NOTE ADDITION	TL

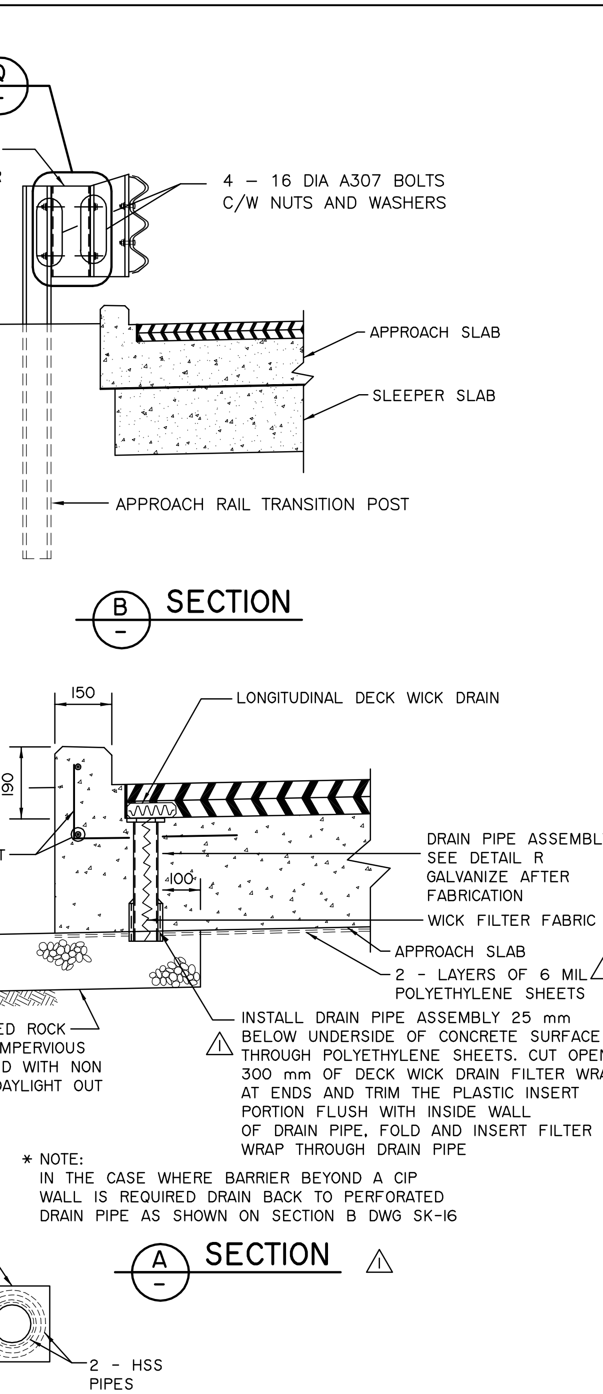
Government of Alberta ■ Transportation

NORTHEAST ANTHONY HENDAY DRIVE
DRAIN TROUGH DETAILS
FOR CONVENTIONAL ABUTMENTS

AT FILE NUMBER	DATE DRAWN	SHEET	DRAWING
–	2010-08-04	6 of 11	SK-14

WTS AUG 03, 2011 13010SK-14.DWG





NOTES

- ALL DIMENSIONS ARE MILLIMETERS
- STANDARD DRAIN TROUGH DETAILS AT THE END OF WINGWALLS FOR INTEGRAL ABUTMENTS
- WHEN BARRIERS BEYOND WINGWALLS ARE USED SEE DWG SK-14 FOR DETAILS

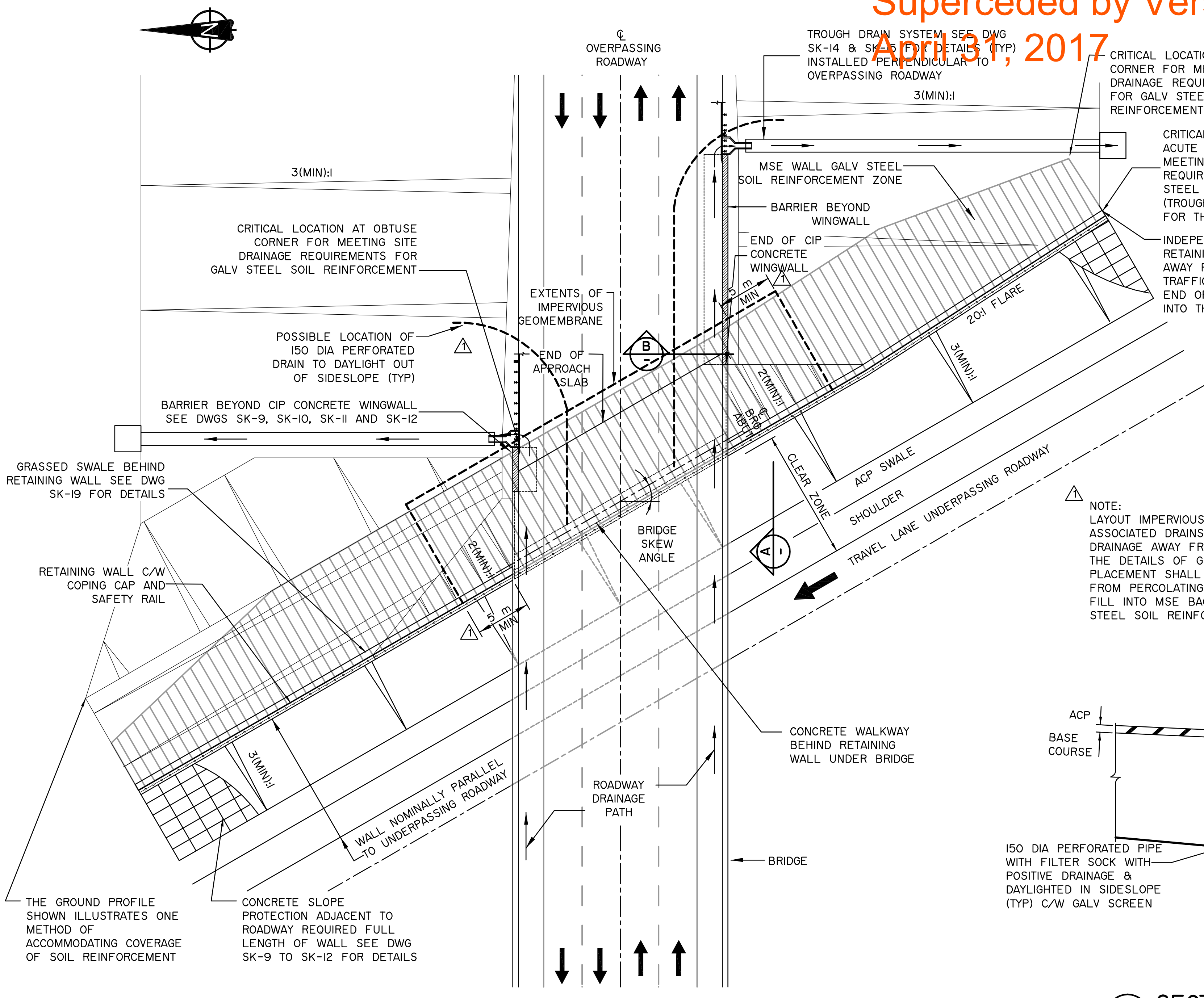
REV	DATE	REVISIONS	BY
4	.	.	.
3	.	.	.
2	.	.	.
1	2011-08-03	DRAIN PIPE ASSEMBLY, TROUGH LENGTH	TL

Government of Alberta ■ Transportation

NORTHEAST ANTHONY HENDAY DRIVE
DRAIN TROUGH DETAILS
FOR INTEGRAL ABUTMENTS

AT FILE NUMBER	DATE DRAWN	SHEET	DRAWING
-	2011-05-19	7 of 11	SK-15

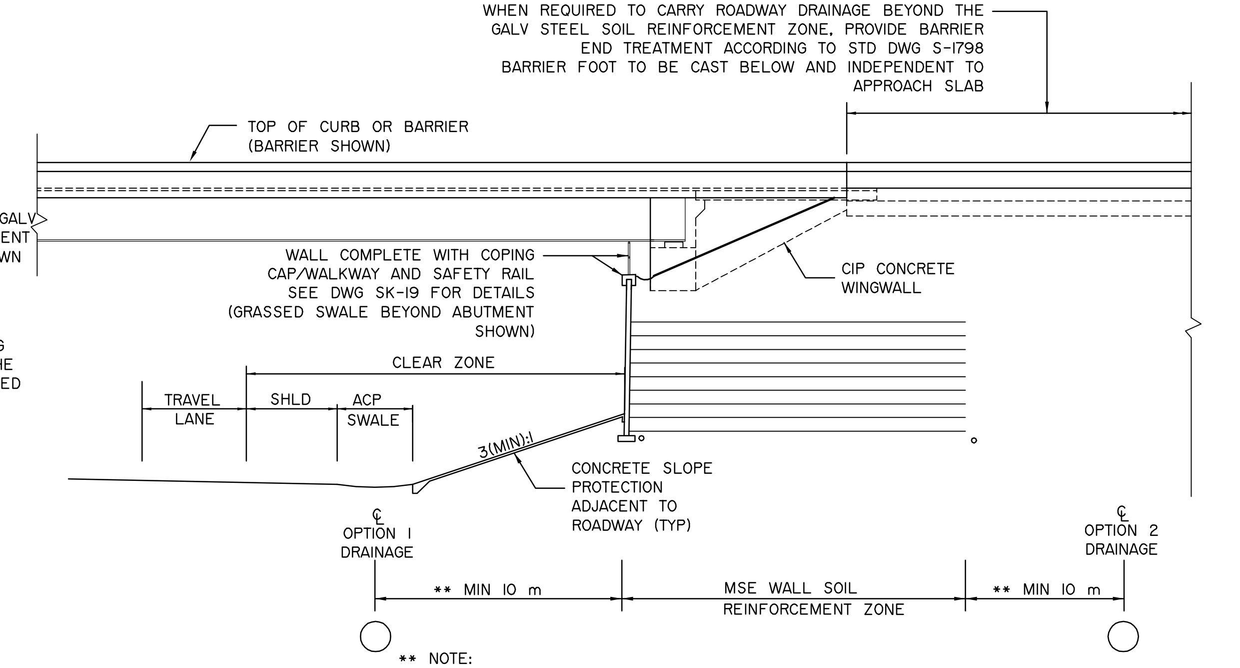
WTS AUG 03, 2011 13010-SK15_RV1.DWG



NOTE:
LAYOUT IMPERVIOUS 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

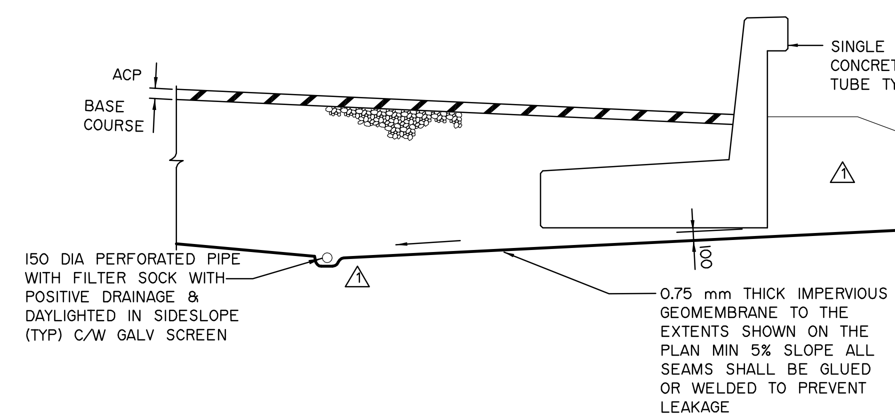
PLAN

NOTE: DITCH DRAINAGE AROUND INDEPENDENT WALLS NOT SHOWN FOR CLARITY

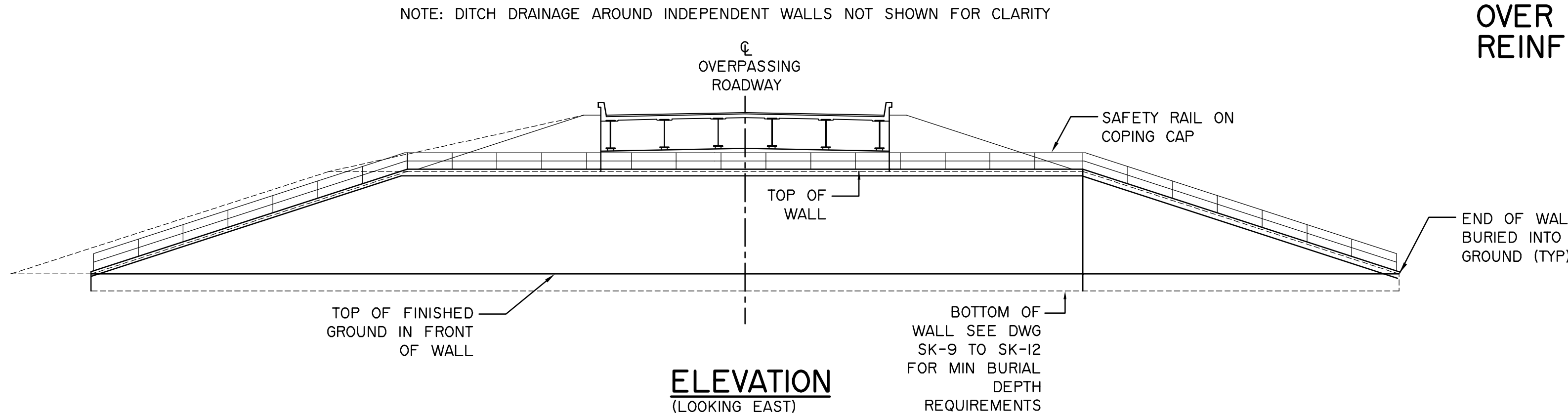


** NOTE:
- DITCH DRAINAGE TO BE LOCATED A MINIMUM 10 m BEYOND MSE WALL SOIL REINFORCEMENT ZONE
- DISTANCES ARE MEASURED PERPENDICULAR TO THE MSE WALL AND/OR END OF SOIL REINFORCEMENT ZONE

A SECTION



B SECTION
(SHOWING GEOMEMBRANE REQUIREMENT OVER GALVANIZED STEEL SOIL REINFORCEMENT)



ELEVATION
(LOOKING EAST)

GENERAL NOTES:

- SKETCHES ARE PROVIDED WITH THE SOLE PURPOSE OF ILLUSTRATING SELECTED SCHEDULE 18 REQUIREMENTS. 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 BRIDGE SKEW ANGLES GREATER THAN 45° SEE DRAWING SK-17 AND SK-18
- ROADWAY DRAINAGE TO BE DIRECTED PAST END OF WALL AND END OF STEEL SOIL REINFORCEMENT
- DRAIN TROUGHS SHALL NOT DRAIN ACROSS SLOPE

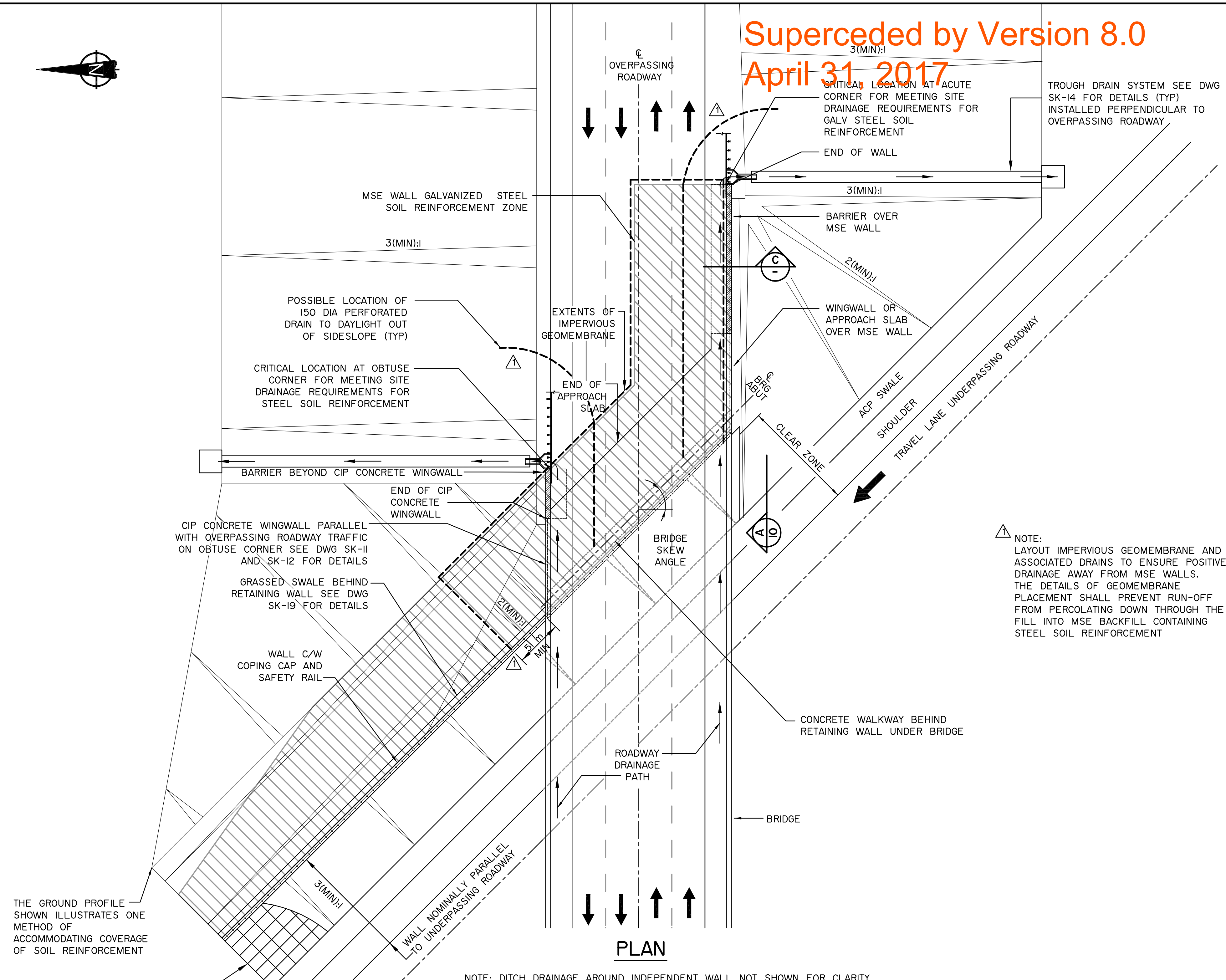
REV	DATE	REVISIONS	BY
4	.	.	.
3	.	.	.
2	.	.	.
1	2011.11.31	GEOMEMBRANE, DRAIN PIPE, NOTE	TL

Government of Alberta ■ Transportation

NORTHEAST ANTHONY HENDAY DRIVE
WALL LAYOUT AND SITE DRAINAGE
FOR BRIDGE SKEW ANGLES ≤ 45°

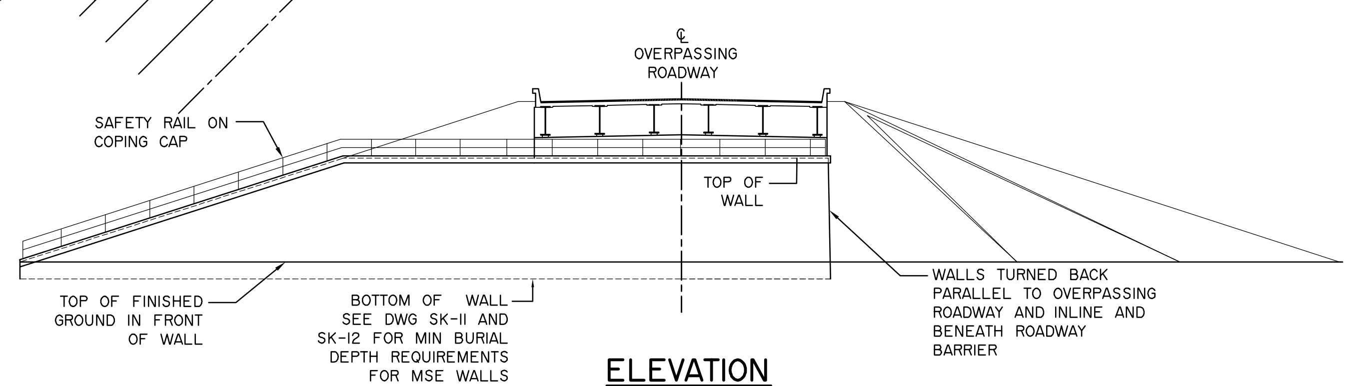
AT FILE NUMBER	DATE DRAWN	SHEET	DRAWING
–	2011-05-13	8 of 11	SK-16

TL NOV. 1, 2011 13000-SK16-RVI.DWG



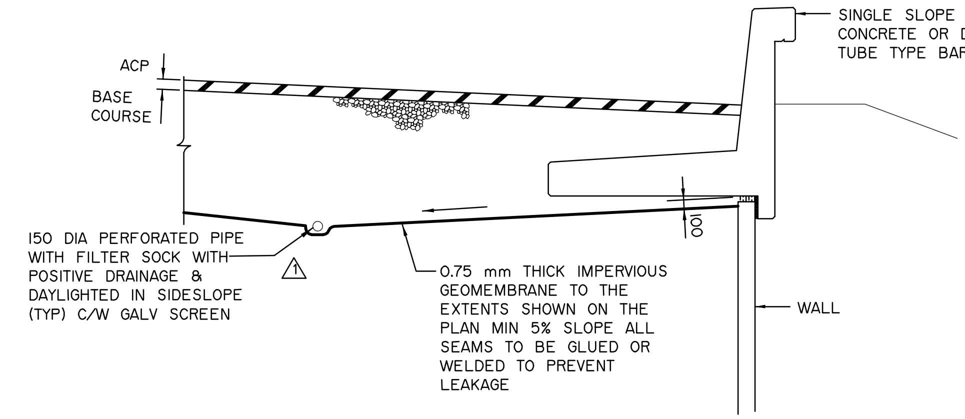
PLAN

NOTE: DITCH DRAINAGE AROUND INDEPENDENT WALL NOT SHOWN FOR CLARITY



ELEVATION
(LOOKING EAST ON SQ)

NOTE:
LAYOUT IMPERVIOUS 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



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

GENERAL NOTES:

- SKETCHES ARE PROVIDED WITH THE SOLE PURPOSE OF ILLUSTRATING SELECTED SCHEDULE 18 REQUIREMENTS. 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-16
- 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 TURNBACK WALLS

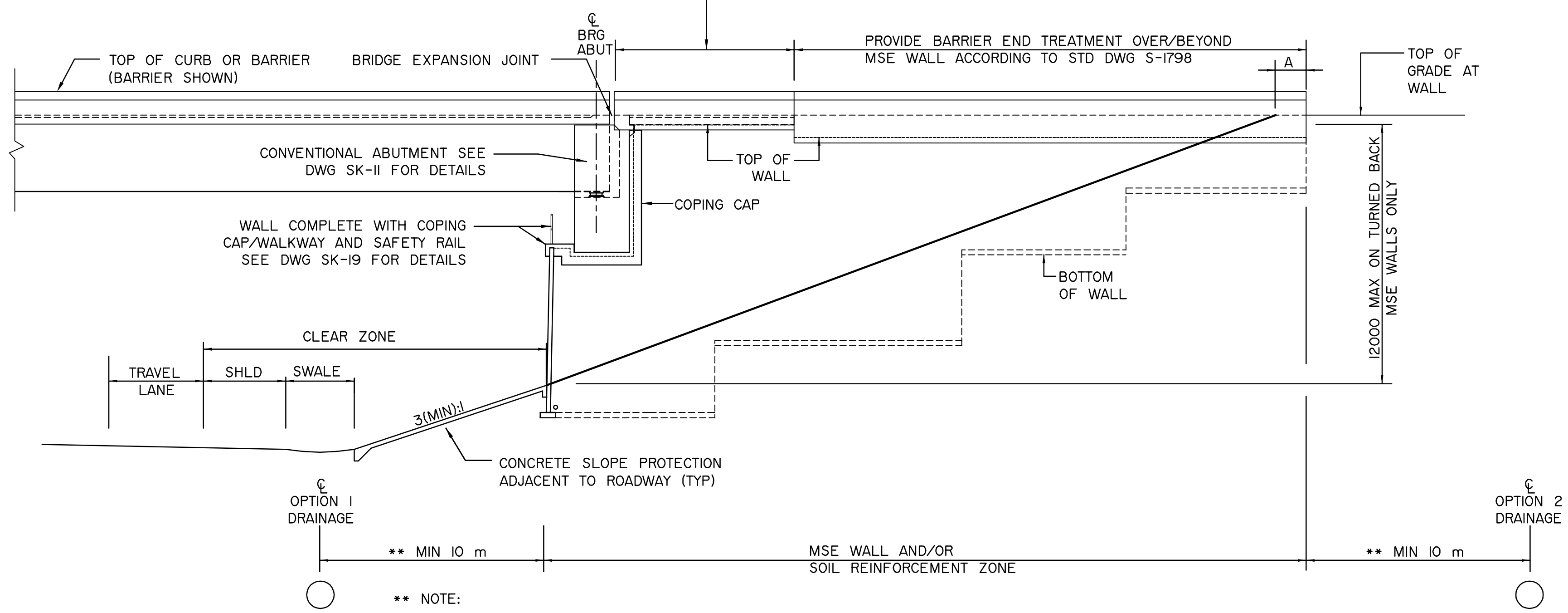
REV	DATE	REVISIONS	BY
4	.	.	.
3	.	.	.
2	.	.	.
1	11.10.31	GEOMEMBRANE, DRAIN PIPE, & NOTE	TL

Government of Alberta ■ Transportation

NORTHEAST ANTHONY HENDAY DRIVE
WALL LAYOUT AND SITE DRAINAGE
FOR BRIDGE SKEW ANGLES > 45°

AT FILE NUMBER	DATE DRAWN	SHEET	DRAWING
–	2011-05-13	9 of 11	SK-17

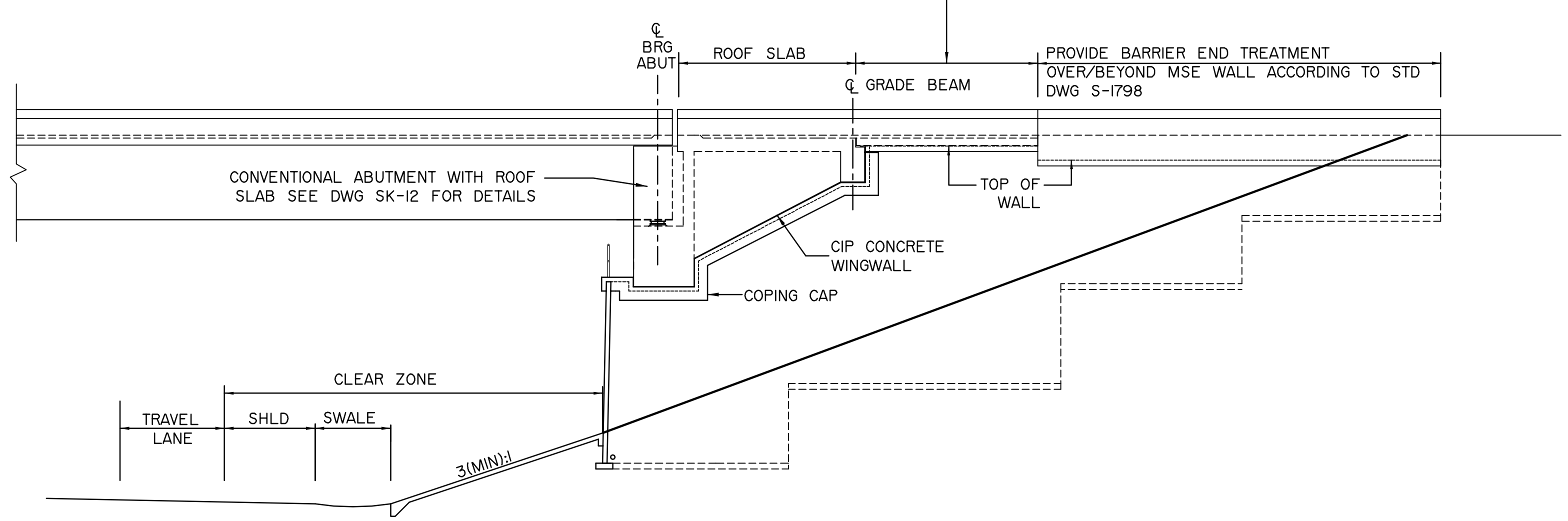
TAL NOV 1, 2011 13090-SK17-RV1.DWG



** 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/9 SECTION
MSE WALL OPTION FOR CONVENTIONAL ABUMTENT



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

A/9 SECTION
MSE WALL OPTION FOR CONVENTIONAL ABUTMENT WITH ROOF SLAB

NOTES

- DRAWING TO BE READ IN CONJUNCTION WITH DWG SK-17
- SEE GENERAL NOTES ON DRAWING SK-17

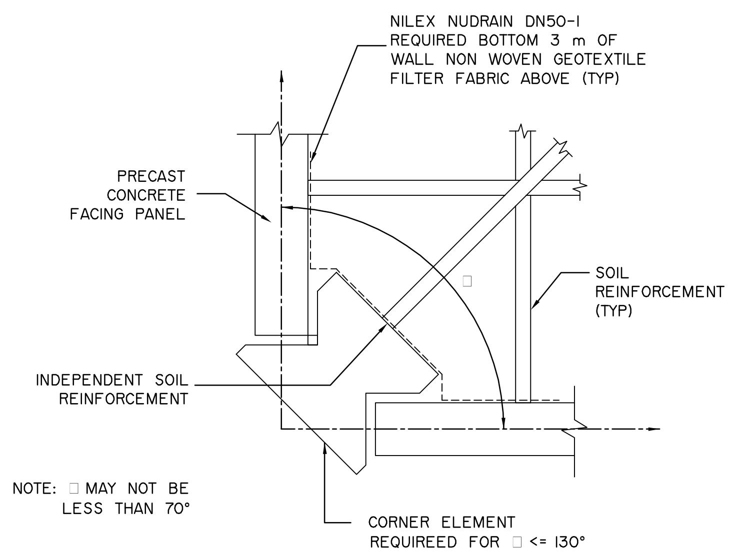
REV	DATE	REVISIONS	BY
4	.	.	.
3	.	.	.
2	.	.	.
1	.	.	.

Government of Alberta ■ Transportation

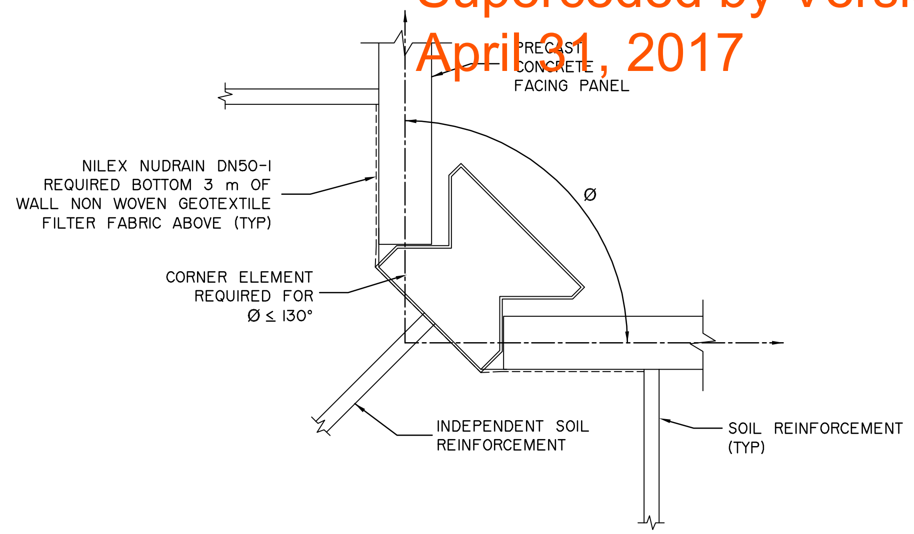
NORTHEAST ANTHONY HENDAY DRIVE
TURNED BACK WINGWALL DETAILS
FOR BRIDGE SKEW ANGLES > 45°

AT FILE NUMBER	DATE DRAWN	SHEET	DRAWING
–	2011–05–13	10 of 11	SK–18

RWK JUN.02, 2011 13010-SK17.DWG

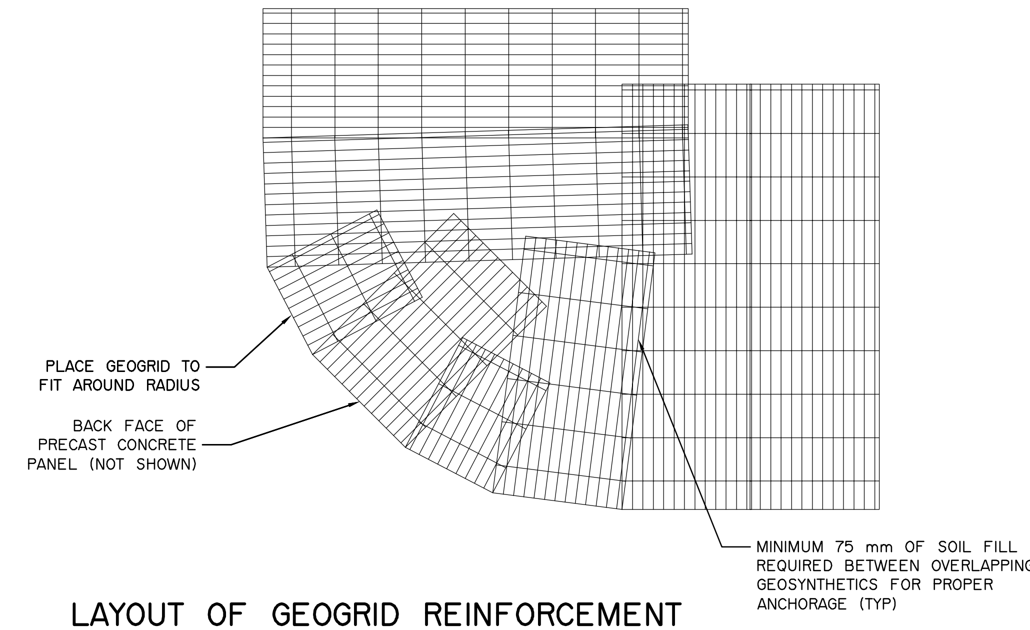


EXTERNAL CORNER

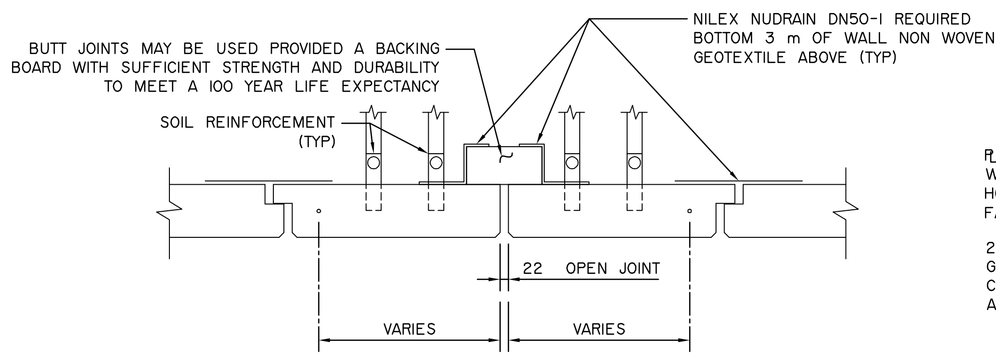


INTERNAL CORNER

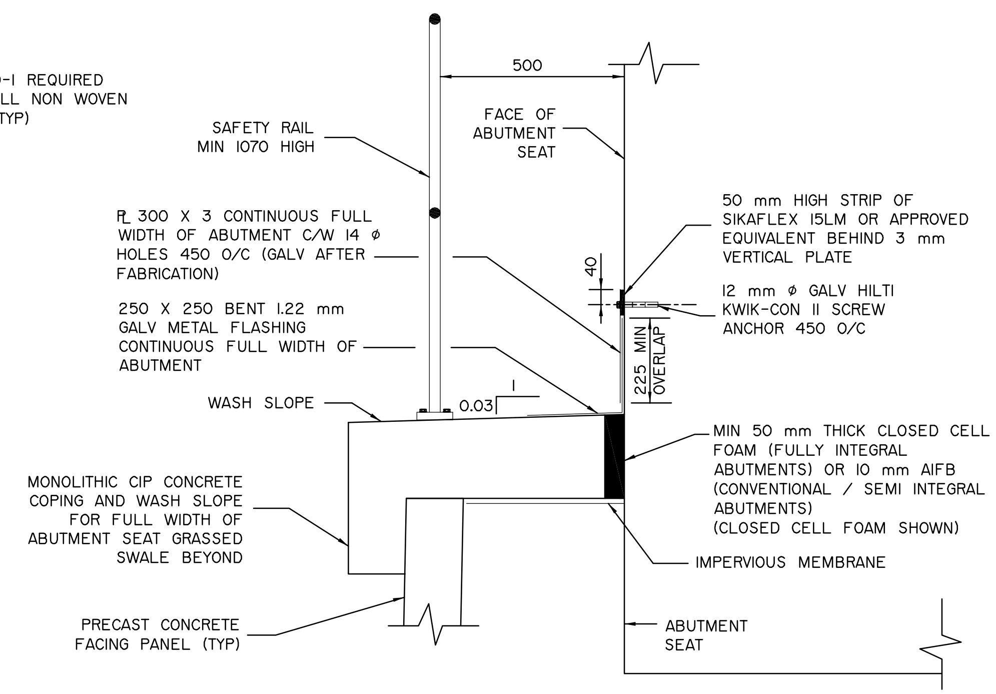
MSE WALL CORNER DETAILS



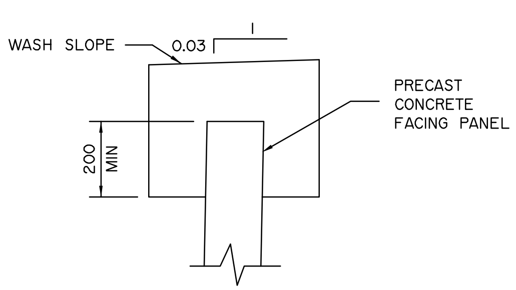
LAYOUT OF GEOGRID REINFORCEMENT FOR CURVED WALLS



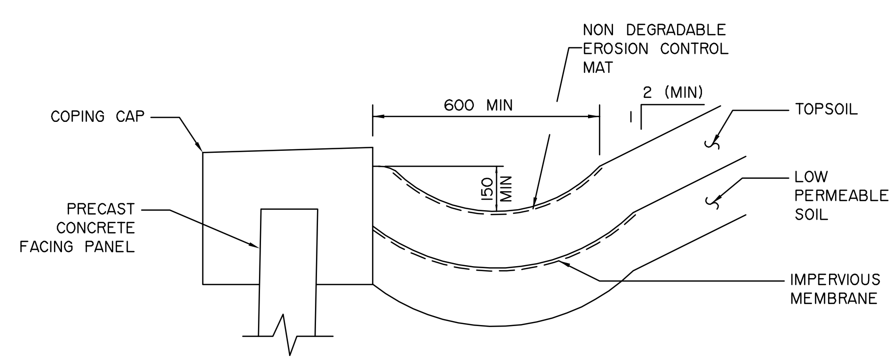
TYPICAL BUTT JOINT AND LIP AND RECESS (SHIP LAP) DETAIL FOR SEGMENTAL PRECAST PANEL FACINGS



WALKWAY BEHIND MSE WALL AT ABUTMENT



P/- COPING CAP DETAIL



GRASSED SWALE TOP OF MSE WALL OUTSIDE OF BRIDGE ABUTMENTS

GENERAL NOTES:

- SKETCHES ARE PROVIDED WITH THE SOLE PURPOSE OF ILLUSTRATING SELECTED SCHEDULE 18 REQUIREMENTS. THEY INTENTIONALLY OMIT DETAILS THAT MIGHT DETRACT FROM THIS PURPOSE

REV	DATE	REVISIONS	BY
4	.	.	.
3	.	.	.
2	.	.	.
1	.	.	.

Government of Alberta ■ Transportation

NORTHEAST ANTHONY HENDAY DRIVE STANDARD DETAILS ASSOCIATED WITH MSE WALLS

AT FILE NUMBER	DATE DRAWN	SHEET	DRAWING
–	2011–05–13	11 OF 11	SK–19

TGK MAY 30, 2011 13010-SK19.DWG

Superceded by Version 8.0
April 31, 2017

Alberta Transportation Bridge Structures Design Criteria v. 7.0

Appendix E

DESIGN-BUILD SPECIFICATIONS FOR PARTIAL DEPTH PRECAST CONCRETE DECK PANELS

Date Published: May 31, 2012

Alberta Government

Table of Contents

E.1 INTRODUCTION

Partial depth precast concrete deck panel construction is not a preferred deck system due to the potential for cracking around the perimeter of the precast panels that may affect long term performance. 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 partial depth precast concrete deck panels shall meet the following design, fabrication and construction specifications.

E.2 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 partial depth precast panels (precast panels);

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

3. The minimum composite deck slab system thickness shall be the greater of the girder spacing divided by 15.0 or 225 mm. In addition, the following shall be satisfied:

    a. The precast deck panel 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;

4. The precast panels shall be fully prestressed and the stresses in the precast panel shall not exceed the following:

    a. From transfer until the 28 day strength is attained:

        - Compression: 0.6 f’ci;

        - Tension: 0.5 fcri;

    b. After the 28 day strength is attained and at serviceability limit states:

        - Tension: fcr;

    c. The average compressive stress in the precast panel at prestress strand release shall be ≤ 7.0 MPa;

5. The empirical method in accordance with *CAN/CSA-S6 CODE Clause 8.18.4* shall not be permitted for design of the composite deck slab system using partial depth precast deck panels;

6. The composite deck slab system shall be designed using flexural design methods based on elastic moments:

a. For square 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 *CAN/CSA-S6 CODE Clause 5.7.1.7.1*, 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 panels and the composite slab as well as the transverse maximum negative moment reinforcing requirements in the cast-in-place 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 prestressing strands, transverse stainless steel reinforcing bars, with a minimum reinforcement ratio "*ρ*" of 0.003, shall be provided throughout the precast panel 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 panel edges and over the girder flanges as required to provide a lap splice with the bars projecting from opposing precast 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 panels shall consist of Class HPC Concrete.

8. The composite deck slab system shall conform to the following:

a. The precast 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 300 mm on centre placed directly on top of the precast panels. Where conflict with the transverse positive moment reinforcing bars exists, these longitudinal reinforcing bars shall be placed directly on top of the transverse reinforcing bars;

9. Prestressing strands shall be 9.5 mm diameter;

10. Prestressing strands shall not project beyond the edges of the precast panel;

11. Prestressing strand cast into the panels shall not be coated steel;

12. With a steel girder superstructure, the following additional provisions shall apply:

   a. The precast panel length shall be set to provide a minimum 75 mm bearing (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 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 stud head and the transverse reinforcing bars projecting out of the precast panels;

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 panel length shall be set to provide a minimum 200 mm bearing (as measured perpendicular to the girder line) on the haunch concrete. For all other girders, the precast panel length shall be set to provide a minimum 75 mm bearing (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 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 transverse reinforcing bars projecting out of the precast panels;

14. Vertical bleed holes shall be provided through the panels along the two supported panel edges at a maximum spacing of 200 mm on centre. The holes shall be not less than 25 mm diameter, and shall be located adjacent to the formed edge of haunch to facilitate the escape of entrapped air;

15. 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 panels and project into the barrier; and

16. No portion of any hardware associated with deck formwork, including deck overhang formwork, shall be visible after removal of all formwork.

E.3 FABRICATION AND CONSTRUCTION

Unless otherwise noted in this Section, 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 *SBC 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 *SBC Section 4: Cast-in-place Concrete* shall apply to the construction of deck systems using partial depth precast concrete deck panels.

E.3.1 **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.

E.3.1.1 Stressing Strand

All strands shall be cut flush with the precast panel edges, and the ends of the strands shall be sealed with Sikadur-31 or an approved equivalent.

E.3.1.2 Surface Finish

The top surface of 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.

E.3.1.3 Tolerances for Panels

Precast concrete 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) 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.

E.3.1.4 Defects and Deficiencies Causing Rejection

A 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 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.

E.3.2 **Erection and Construction**

The precast panels shall be erected on the girders with temporary supports. The precast 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 haunches shall be formed to be flush with the edge of the girder flanges. All haunch forming material shall be completely removed after casting the deck to fully expose the haunch concrete.

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 panel, but shall not exceed the top surface of the precast panel, and shall not extend in front of the second stage pour by more than 10 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 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 by the Department.

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 hangers may be cast into the girder top flanges. For steel girder superstructures, anchors for the exterior hangers may be shop attached to the girder top flanges. Field drilling of the girders or precast panels shall not be permitted.

All lifting hooks for the precast 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.

E.4 REFERENCES

1. AT. *Specifications for Bridge Construction*. Alberta Transportation, Edmonton, AB (2010).

2. CSA. (2006). *CAN/CSA-S6-06 Canadian Highway Bridge Design Code,* including *S6S1-10, Supplement #1* and *S6S2-11, Supplement #2*. 2006. Canadian Standards Association, Toronto, ON.

Superceded by Version 8.0
April 31, 2017

Alberta Transportation Bridge Structures Design Criteria v. 7.0

Appendix F

DESIGN EXAMPLE FOR STRIP SEAL DECK JOINT MOVEMENT WITH SKEW

Date Published: May 31, 2012

Alberta Government

Superceded by Version 8.0
April 31, 2017

F.1 INTRODUCTION

Deck joints shall be selected and designed in accordance with *Section 20: Deck Joints* of the *BSDC* 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 Multi-web 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 values applicable to that joint type.

F.2 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 θ = 25º 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 = Δ;
vii. Movement perpendicular to deck joint = Δp = ΔCosθ;
viii. Movement parallel to deck joint = Δt = ΔSinθ.

See Fig 1 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 Δ = Δp / Cosθ   Δ = 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 Δ = Δt / Sinθ   Δ = 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.

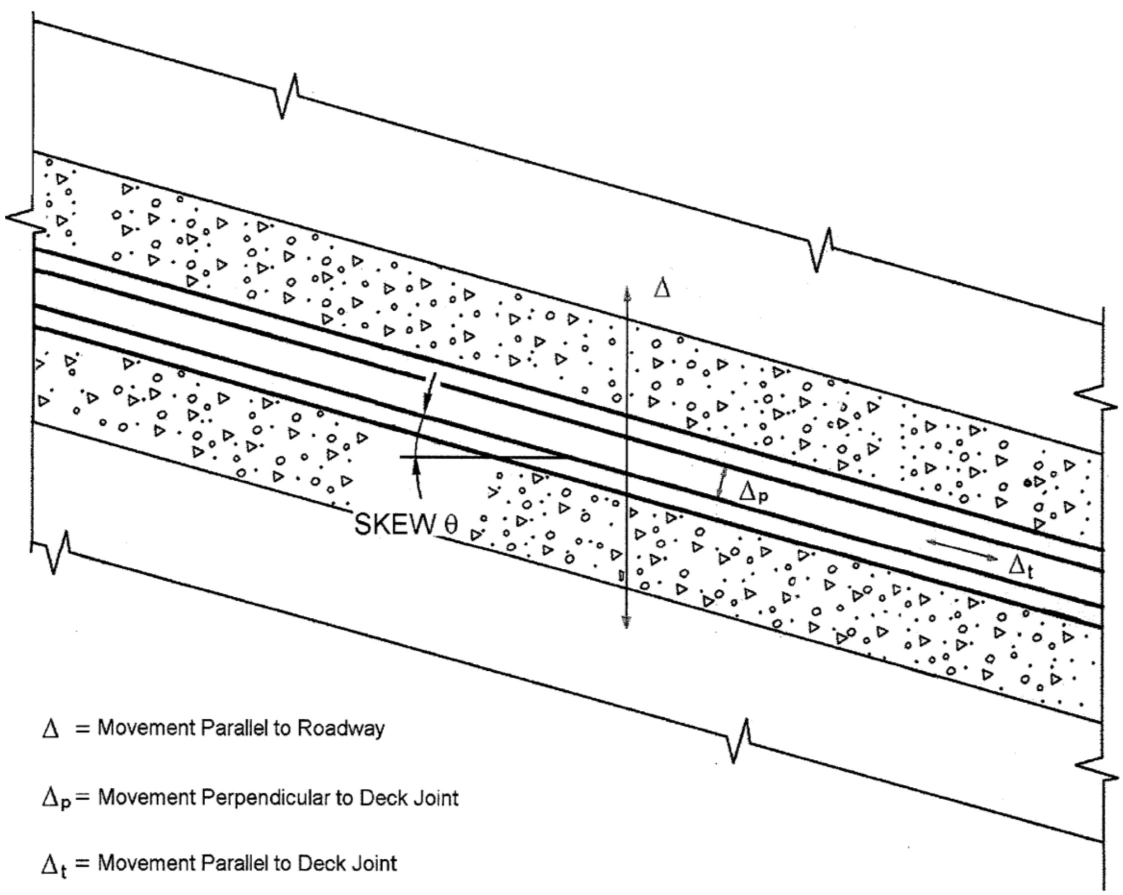


Fig 1

Superceded by Version 8.0
April 31, 2017

Alberta Transportation Bridge Structures Design Criteria v. 7.0

Appendix G

BRIDGE BEARING DESIGN GUIDELINES

Date Published: May 31, 2012

Government

Table of Contents

Superceded by Version 8.0
April 31, 2017

G.1 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.

The vast majority of bearings currently 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 standard SL, SLW and SLC bridges.

G.2 PREFERRED BEARING TYPES

G.2.1 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 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.

G.2.2 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.

G.2.3 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 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.

G.2.4 Reinforced Concrete Shear Blocks

Reinforced concrete shear blocks have proven to be more effective than anchor rods for transferring horizontal forces into the sub-structure. In the past, when resistance to lateral loads was provided by anchor rods cantilevered out from a concrete base, many anchor rods were bent or broken. Damaged rods often extend through holes in heavy steel sole plates or extension plates to girder bottom flanges, and are inaccessible for repair or replacement. Independent concrete shear blocks between girders provide higher resistance to lateral forces, and there is good access between girders for any required repair. 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.

G.3 SELECTION OF BEARING TYPE

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

1. Reinforced elastomeric bearings

2. Fixed plate rocker bearings

3. Pot bearings

Alternative bearing types shall require department approval of details and specifications. Bearings for high skew and curved bridges may require special considerations or special design features.

No uplift is permitted for pot bearings for the FLS limit state. Where uplift for the FLS limit states cannot be avoided, a hold-down device accessible for inspection and service shall be provided independent of the bearings.

Steel reinforced elastomeric bearings are the department's preferred choice and shall be specified whenever possible.

Reinforced concrete shear blocks between girders shall be used to provide lateral restraint whenever possible. For typical details of reinforced elastomeric bearings and reinforced concrete shear blocks, refer to standard drawing *S-1761*.

Continuous unreinforced elastomer sheets are used with standard SL, SLW and SLC bridges. Unreinforced elastomer sheets shall be as detailed on standard bridge drawings and shall be laid directly on top of the sub-structure.

Table 1 summarizes approximate design limits on the design loads, displacements, and rotations for the preferred bearing types.

Table 1 – 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, special bearings and appropriate specifications may be required. Approval shall be obtained from the Department for special bearings.

G.4 BEARING COMPONENT DESIGN REQUIREMENTS

G.4.1 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 *SBC 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 *SBC Section 8*: *Bridge Bearings*. Any adjustment required for sole plates and base plates shall be approved by the consultant. Requirements of *SBC Section 8*: *Bridge Bearings* will not be repeated here except for a few highlighted items. The Consultant shall familiarize himself with the requirements of *SBC Section 8*: *Bridge Bearings*.

Consultant designed components shall be in accordance with this Bridge Bearing Design Guidelines and standard drawing *S-1761*. Materials for bearings shall meet the requirements of the *BSDC Section 6*.

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 *CAN/CSA-S6 CODE Clause 4.4.10*. Lateral forces shall be transferred through girder bottom flanges to reinforced concrete shear blocks whenever practical.

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

G.4.2 Information to be Included on Design Drawings

The Consultant shall provide the following on the 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 1 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 1 – 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 CHBDC Table 3.1.

G.4.3 Tolerance and Uncertainties for Bearing Rotation

G.4.3.1 Current Code and Specification Requirements

There is considerable disagreement between *CAN/CSA-S6 CODE*, 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.

CAN/CSA-S6 CODE 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.

G.4.3.2 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.

CAN/CSA-S6 CODE 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, *SBC Section 6.2.6.11(b)* 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, *SBC Section 7.2.5.16* 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 *SBC* cannot account for these variables, so additional allowances are required.

SBC Section 6.3.2.4 specifies that bearings are to be set to the exact elevation specified on the design drawings, but poor quality control could still result in some discrepancy.

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

For elastomeric bearings, the Department's practice of using self-rocking pintels 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 conservative all inclusive tolerance requirement of 0.02 radians at ULS shall be used.

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

G.4.4 Tapered Sole 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 prestressed concrete girders. At abutments with sliding plate deck joints, such as cover plated joints or finger plate joints, tapered sole plates shall be used to bring the sliding plane parallel to the roadway grade so the sliding plates or finger plates can function properly. Tapered sole plates are not required for fixed steel plate rocker bearings.

The taper rate 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.

When the design taper rate 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.

The thickness of the tapered sole plate shall be sufficient to transfer 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 by welding shall be in the longitudinal direction along the edge of the shoe plate. 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 standard drawing S-1761.

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 *CAN/CSA-S6 CODE Table 10.9* shall be taken as 0.4.

G.4.5 Base Plates and Grout Pads

After girder erection and immediately before grouting, the longitudinal location of bearing base plates is adjusted in accordance with the bearing setting charts.

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.

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 sub-structure 40 mm and project above the sub-structure 40 mm. This will raise the bearings above the top of the sub-structure 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 *SBC Section 12.2.6.8*.

Shim plates used for shim stacks shall be Grade 300W steel and shall be hot-dip galvanized.

Base plates and anchor rods shall be grouted after girders are erected, elevations are checked and confirmed, and before the deck is poured.

G.4.6 Steel Reinforced Elastomeric Bearings

The design of steel reinforced elastomeric bearings shall meet the requirements of *SBC Section 8*: *Bridge Bearings*. Typical details for steel reinforced elastomeric bearings are provided on standard drawing *S-1761*.

Steel reinforced elastomeric bearings shall be designed for rotations resulting from factored loads at SLS plus a tolerance allowance of 0.005 radians. Rotations need not be considered at ULS.

A self rocking pintel welded under the base plate shall be used to ensure uniform contact between the elastomeric pad and the girder bottom flange at erection. A single pintel 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, an additional pintel is required, and the pintels shall be centred beneath the bearing along a line perpendicular to the longitudinal axis of the girder. Typical pintel details are provided on standard drawing *S-1761*.

At the time of erection, the self rocking pintel will bring the elastomeric pad into uniform contact with the girder underside, or the sole plate if one is provided. The bearing base plate is then 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 standard drawing *S-1761* is generally adequate. *AASHTO LRFD Section 14.7.5.3.5* provides guidance on checking shim plate thickness.

Notwithstanding *CAN/CSA-S6 CODE Clause 11.6.6.2.2*, material requirements for elastomers shall conform to *Section 18 "Bearings" Division II of AASHTO Standard Specifications for Highway Bridges*. Elastomeric material shall meet the requirements of AASHTO Grade 5 for cold temperature performance and a Shore A durometer hardness of 60.

For elastomeric bearings, the PTFE sliding surfaces shall be unfilled PTFE sheets.

Bearing base plates shall have removable keeper bars to restrain the bearings from walking out under the girders.

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 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 box beams, two separate reinforced elastomeric bearing pads shall be provided at each end of each girder. The bearing pads are installed directly on top of the sub-structure 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 *BSDC Section 15*.

G.4.7 Pot Bearings

The design of pot bearings shall meet the requirements of *SBC Section 8: Bridge Bearings*.

Notwithstanding *CAN/CSA-S6 CODE Clause 11.6.5.4*, *SBC Section 8: Bridge Bearings* requires that the average pressure on the elastomer at SLS shall not exceed 30 MPa.

Pot bearings shall be designed to accommodate rotations resulting from factored loads at ULS, plus a tolerance allowance of 0.02 radians.

Except where an inclined sliding plane is specified, girders are erected on the bearing supported on a level base plate sitting on four galvanized shim stacks. Initial rotation will be forced into the bearing elastomer due to initial girder camber at the time of erection. This rotation will be negative and will be cancelled out as the girder end rotates under additional load.

Top PTFE sliding surfaces shall be unfilled dimpled sheets permanently lubricated with silicone grease. PTFE sliding surfaces for lateral guides, if required, can be filled with up to 15% by mass of fibre glass.

Allowable contact pressures for PTFE sliding surfaces are less than those allowed by *CAN/CSA-S6 CODE*. Maximum average contact pressures for confined and unfilled PTFE are provided in *SBC 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.

Notwithstanding *CAN/CSA-S6 CODE Clause 11.6.5.2*, material requirements for elastomers shall conform to *Section 18 “Bearings” Division II of AASHTO Standard Specifications for Highway Bridges*. Elastomeric material shall meet the requirements of AASHTO Grade 5 for cold temperature performance.

G.4.8 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 *CAN/CSA-S6 CODE 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, plus 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 *CAN/CSA-S6 CODE Clause 3.9* shall be considered, including elastic shortening due to post-tensioning, and long term movements resulting from creep, shrinkage, relaxation, sub-structure movement and any other internal or external cause.

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

G.4.9 Fixed Steel Plate Rocker Bearings

The design of steel plate rocker bearings shall meet the requirements of *CAN/CSA-S6 CODE Clause 11.6.2*.

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 length of load bearing contact. Base plates shall be installed level on galvanized steel shim stacks. Due to the large rotational capacity, there is normally no need for tapered sole plates.

Horizontal loads are transferred through shear in steel anchor rods. Holes for anchor rods through the bottom half of rocker plates shall have the standard tolerance. Holes through the top half should be oversized to allow rotation of the rocker plate without bending the anchor rods. 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.

G.4.10 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.

G.4.11 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 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.

G.5 REFERENCES

1. AT. *Specifications for Bridge Construction*. Alberta Transportation, Edmonton, AB (2010).

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. (2006). *CAN/CSA-S6-06 Canadian Highway Bridge Design Code,* including *S6S1-10, Supplement #1* and *S6S2-11, Supplement #2*. 2006. 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).

8. Stanton, J.F., Roeder, C.W. and Campbell, I., *NCHRP Report 432: High-Load Multi-Rotational Bridge Bearings*. Transportation Research Board, National Research Council, Washington, DC (1999).