BRIDGE DESIGN CRITERIA

KING’S RIVER BRIDGE – Structure # 544

RG3151

PROJECT: AK DOT 135(1)

GLENN HIGHWAY MP 66.5
ANCHORAGE BOROUGH, ALASKA

STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION AND PUBLIC FACILITIES

APRIL 11, 2019

Prepared by: Central Federal Lands Highway Bridge Office (Lakewood, CO)

LATEST DRAFT (NOT FINAL)
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1. TECHNICAL POLICY GUIDELINES

The following design criteria identify the particular standards and procedures, which are used for the bridge design:


1.4 *Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects*, FP-14, Dual Units, Publication No. FHWA-FLH-14-001. This reference is hereby referred to as “FP-14”.


1.8 *Manual of Standard Practice*, Concrete Reinforcing Steel Institute (CRSI), current edition. This reference is hereby referred to as “CRSI”.

1.9 *Alaska Bridges and Structures Manual*, Alaska Department of Transportation, September 2017. This reference is hereby referred to as “BSM”.

1.10 Geotechnical Investigation: TBD. This reference is hereby referred to as “Geotechnical Report”.

1.11 Geotechnical Memo: TBD

1.12 Hydraulic Recommendations: TBD. This reference is hereby referred to as “Hydraulics Report”.

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2. ADDITIONAL EXISTING DOCUMENTS

2.1 Preliminary Plans, King’s River Bridge, dated September 21, 2018, prepared by AKDOT & PF.

2.2 King’s River Bridge Concepts, dated November 23, 2016, prepared by WFLHD.


3. GEOMETRIC LAYOUT

3.1 The bridge is located at Latitude 61° 43’ 55.33” North, Longitude 148° 45’ West.

3.2 The bridge spans, horizontal and vertical alignment, and general arrangement of the structure is as shown on the Preliminary Plans dated September 21, 2018 by the AKDOT & PF. Superelevation transitions shown on these plans have been relocated off the bridge by WFL.

3.3 Bridge width (out-to-out) is 39’-0” with a roadway width of 36’-0” between bridge rails. Bridge type is a deck, bulb tee with spray-applied waterproofing membrane and asphalt overlay.

3.4 Alaska multi-state bridge rail mounted on concrete curbs is provided on the bridge with corresponding transition rails, (AKDOT & PF Standard G-31.01). The updated Mash-tested TL-4 two-tube railing details will be used as shown on the WFL Purinton Creek bridge plans. The MASH crash-tested rail is taller and has different tubes, the crash tests were designed for the Deck bulb tee slab configuration/connection that is shown in the AKDOT bridge manual table. Therefore, no need to design the deck bulb tee slab other than using the standard table in the BSM. The transition railing is still being evaluated and tested for MASH standards and TTI expects it to pass TL-3. If new details for the transition railing are developed before the end of design, they will be incorporated into the plans, otherwise, the current transition railing details will be used. According to AKDOT, TTI reports that it is typical around the country to have a TL-3 transition to a TL-4 bridge rail.

3.5 Roadway design speed at the bridge is 60 mph.

4. DESIGN LOADS

4.1 Load Factors and Load Combinations (AASHTO 3.4)

4.1.1 Load combinations and load factors are in accordance with AASHTO Tables 3.4.1-1 and 3.4.1-2.
4.2 Permanent Loads (AASHTO 3.5)

4.2.1 Components and Attached Dead Loads (DC)

(1) Concrete with reinforcing steel (CIP) = 0.150 kcf  
(BSM Section 14.1.1 used for design DL. Use 0.145 kcf as unreinforced)

(2) Concrete with reinforcing steel (precast) = 0.158 kcf  
(BSM Section 14.1.1 used for design DL. Use 0.1525 kcf as unreinforced for Eci))

(3) Structural steel = 0.490 kcf

(4) Bridge railing = 0.253 klf (each)

4.2.2 Wearing Surface and Utilities (DW)

(1) Wearing surface allowance (4” thick HACP) = 0.050 ksf

(2) Future Wearing Surface (AKDOT Policy) = 0 ksf

(3) Utilities allowance in each ext bay = 0.050 klf  
(Per 01/08/19 AKDOT meeting)

4.2.3 For bridges with more than six girders, assume the superimposed dead load of the railing and utilities are distributed equally to the exterior three girders.

4.3 Live Load and Impact (AASHTO 3.6.1 & 3.6.2)

4.3.1 The maximum number of design lanes is as specified by AASHTO 3.6.1.1.1 with multiple presence factors in accordance with AASHTO 3.6.1.1.2. A maximum of three design lanes will be used in design.

4.3.2 The design live load is designated as HL-93, and consists of a combination of:
   1. Design truck or design tandem, and
   2. Design lane load.

4.3.3 Permit load to be considered is none.

4.3.4 Pairs of design tandems, as described in AASHTO C3.6.1.3.1, are not considered.

4.3.5 Deflection due to live load is investigated per AASHTO 3.6.1.3.2 and 2.5.2.6.2.
4.3.6 An (ADTT)SL value of 238 vpd is used for calculating number of fatigue cycles, assuming truck traffic composes 10% of projected 2030 ADT (2376 vpd). The fatigue load is one design truck or axles thereof with a constant spacing of 30'-0" between the 32.0-kip axles. The dynamic load allowance applied to the fatigue load shall be 15% per AASHTO 3.6.2.1.

4.3.7 The dynamic load allowance (impact) is applied in accordance with AASHTO 3.6.2 to superstructure and substructure elements. Impact is not applied to substructure units entirely below ground or to elastomeric bearings.

4.4 Centrifugal Forces (AASHTO 3.6.3)

4.4.1 No centrifugal forces are applied due to bridge being in a tangent.

4.5 Braking Forces (AASHTO 3.6.4)

4.6 Vehicular Collision Forces (AASHTO 3.6.5)

4.6.1 Vehicle collision with barriers shall be in accordance with AASHTO 3.6.5.2 and Section 13, based on TL-4 crash test level.

4.7 Water Loads (AASHTO 3.7)

4.7.1 Design water levels:

(1) The design flood for the structure at the strength and service limit states is taken to be the 100-year event.
(2) The check flood for analyzing structural stability at the extreme event limit state is taken to be the 500-year event.
(3) Design hydraulic capacity is based on 50-year event with 3 feet minimum freeboard, to account for drift passage. (BSM Section 11.2.2)

4.7.2 Buoyancy and stream pressure forces applied to the structure are computed based on the design flood event.

4.7.3 Applicable scour levels are used in conjunction with the respective design and check floods.

4.7.4 Place riprap to an elevation of 1 foot or more above the 50-year event water surface elevation (BSM Section 11.2.2)

4.8 Wind Loads (AASHTO 3.8)

4.8.1 Wind loads are computed based on a design 3-second wind speed of $V = 130$ mph.

4.8.2 The ground surface roughness category is Category C.
4.8.3 Wind exposure category is Category C.

4.9 Ice Loads (AASHTO 3.9)

4.9.1 Ice loading, both static and flowing, shall be considered in the hydraulic design. The AKDOT M&O office was unaware of historic ice thickness at this site. Ice thickness will be estimated using empirical method discussed in the Commentary of AASHTO Section 3.9.2.2. If thickness is greater than 1’, then ice loading applies. Apply ice load at the design high water level. Ice loading will be applied to the upstream pipe pile only. Ice loads may affect the gap region of the pile. (BSM Section 12.3.8)

4.9.2 Effects of frozen soil, see 2018 BSM Section 17.6.4)

4.10 Earthquake Effects (SGS)

4.10.1 The method used for seismic analysis is the “pushover analysis”.

4.10.2 Seismic coefficients to be used for design are:

(1) Horizontal Peak Ground Acceleration Coefficient on rock (site class B), 
PGA = 0.56 g.

(2) Horizontal response spectral acceleration coefficient at 0.2 second period on rock (site class B), Ss = 1.27 g.

(3) Horizontal response spectral acceleration coefficient at 1.0 second period on rock (site class B), S1 = 0.49 g.

4.10.3 The bridge is considered an “other” bridge importance category.

4.10.4 The bridge meets the “regular” bridge requirements of SGS 4.2

4.10.5 The Site Class used in design is D.

4.10.6 The Seismic Design Category used in design is SDC D.

The Seismic Design Strategy will be Type 1 (Ductile substructure, elastic superstructure) with an Earthquake-Resisting System (ERS) without longitudinal Abutment Contribution. Earthquake Resisting Elements (ERE) include in-ground plastic hinging of pipe piles and abutment curtain walls designed to fuse under transverse seismic loading. AKDOT neglects the passive soil pressure in both directions at the abutments, and expects plastic hinges below ground in pile extension piers.

4.10.7 For Extreme Event load cases, live loads will not be considered.

4.10.8 Liquefaction potential to be investigated.
4.11 Earth Forces (AASHTO 3.11)

4.11.1 Vertical Earth load (EV) is assumed to be 120 pcf for structural backfill.

4.11.2 For full active earth pressure conditions, the lateral equivalent fluid pressure shall be 36 pcf. For at-rest earth pressure conditions, the lateral equivalent fluid pressure shall be 57 pcf.

4.11.3 Live Load Surcharge (LS) on abutments is based on an equivalent height of soil = 3.5 ft. Retaining walls and wingwalls shall be designed based on an equivalent height of soil = 2.0 ft. Per BSM Section 12.3.6 for bridges with approach slabs, consider the reaction of the axle loads on the slab plus half of the live load surcharge for walls parallel or perpendicular to the roadway.

4.11.4 Downdrag (DD) on piles due to embankment loading will be investigated.

4.11.5 Uplift on piles due to frost jacking will be investigated.

4.12 Force Effects due to Superimposed Deformations (AASHTO 3.12)

4.12.1 Procedure A is used in determining the design thermal movement associated with uniform temperature change. Table 19-1 from BSM Section 19.1.1 will be utilized.

4.12.2 Forces and moments due to temperature rise and fall are calculated for the following temperature ranges:

(1) Concrete:
   coefficient of thermal expansion = 6.0 x 10^{-6} / °F
   Alaska temperature range = 140° F
   Construction at extreme temperatures (at or near the minimum of maximum temperatures) will be assumed. No 65% reduction in the design thermal movement range (per AASHTO 14.7.5.3.2) is allowed (BSM Section 19.2.3).

5. MATERIALS

Concrete

<table>
<thead>
<tr>
<th>Location</th>
<th>Class</th>
<th>f’c</th>
</tr>
</thead>
<tbody>
<tr>
<td>Superstructure (cast-in-place)</td>
<td>A(AE)</td>
<td>4.0 ksi</td>
</tr>
<tr>
<td>Barriers and Curbs</td>
<td>A(AE) or C(AE)</td>
<td>4.0 ksi</td>
</tr>
<tr>
<td>*Prestressed Beams (release)</td>
<td>P or P(AE)</td>
<td>6.5 ksi min.</td>
</tr>
<tr>
<td></td>
<td>(final)</td>
<td>8.0 ksi max.</td>
</tr>
</tbody>
</table>

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<table>
<thead>
<tr>
<th>Substructures (cast-in-place)</th>
<th>A(AE)</th>
<th>4.0 ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pipe Pile</td>
<td>A(AE)</td>
<td>4.0 ksi</td>
</tr>
<tr>
<td>Approach slab (BSM Table 14-1)</td>
<td>D(AE)</td>
<td>5.0 ksi</td>
</tr>
</tbody>
</table>

*For prestress items, identify maximum attainable $f'_c$ based on local fabricator. AK standard mix designs are available for $f'_c=4000$psi

5.2 Reinforcing Steel

5.2.1 Reinforcing steel shall be Grade 60 deformed bars conforming to ASTM A706, Grade 60 (BSM Section 14.1.2).

5.2.2 All reinforcing steel bends conform to CRSI Standards or as noted otherwise.

5.2.3 All reinforcing steel in the curbs, prestress girders, abutment diaphragms, wingwalls, approach slabs, and pier diaphragms is epoxy coated. Epoxy coated bars are denoted in the bar list sheets.

5.2.4 Reinforcing steel shall have a minimum concrete cover of 2 inches unless otherwise noted.

5.2.5 The maximum length for reinforcing bars is 40’-0” for #4 bars and 60’-0” for #5 bars and larger. Cut reinforcing bars to CRSI tolerances.

5.2.6 No allowance is made in bar length except for corrections associated with standard hooks and special bends.

5.2.7 All bent bar dimensions are taken as out-to-out.

5.2.8 Reinforcing splice lengths shall be determined according to AASHTO 5.10.8. Minimum splice lengths shall be shown in the plans and/or bar lists.

5.3 Prestressing Steel

5.3.1 Prestressing steel is 0.5-inch nominal diameter (area = 0.153 in²) per AKDOT & PF plans Grade 270 "Uncoated Seven-Wire Low Relaxation Strands for Prestressed Concrete", AASHTO M203. Minimum ultimate strength per strand is 41.3 kips.

5.3.2 Initial tensile force applied to each strand is 70 percent of ultimate strength or 28.9 kips. (BSM Section 14.4.2)

5.3.3 Modulus of elasticity, $E = 28,500$ ksi is assumed (AASHTO 5.4.4.2).

Structural Steel

5.4.1 Structural steel conforms to the following AASHTO (ASTM) requirements:
ASTM A36 (steel rail posts & plates) \( F_y = 36 \text{ ksi} \)
ASTM A709, Grade 36T3 (miscellaneous) \( F_y = 36 \text{ ksi} \)
ASTM A500, Grade B (steel rail tubes) \( F_y = 46 \text{ ksi} \)

Modulus of Elasticity = 29,000 ksi.

5.4.2 Structural steel piling conforms to the following:

ASTM A709, GR50T3 (one Helical seam weld) \( F_y = 50 \text{ ksi} \)
(Section 715-2.02 of the 2017 AKDOT Standard Specifications for Construction – will need an SCR to replace FP-14 715.05 (a))

Galvanize the pipe piles down to 10’ below scour elevation at piers.

6. SUPERSTRUCTURE DESIGN

6.1. Precast Decked Bulb-Tee Girder Top Slab

6.1.1. No allowance is made for a sacrificial wearing surface in the deck flange design.

6.1.2. Longitudinal bars shall meet minimum requirements for temperature and shrinkage reinforcement (AASHTO 5.10.6).

6.1.3. At exterior girder top slab, transverse bars shall be designed to include loads resulting from wheel load 1’ from face of barrier and vehicle collision with barriers based on TL-4 criteria (AASHTO A13.4). In the BSM, reinforcing steel tables for the deck bulb tee slab overhang account for the vehicle collision loads, therefore, the reinforcing steel in the table will be used, and no design is necessary. Interior girder top slab transverse bars shall be designed for a wheel load adjacent to the longitudinal joint, using equivalent strip method (AASHTO 4.6.2.1.3).

6.1.4. The top reinforcing steel cover is 2.25 inches. (AKDOT & PF plans)

6.1.5. Decked bulb-tee top flange will be provided with 6” minimum thickness.

6.2. Prestressed Concrete Girders

6.2.1. Temporary allowable stresses before losses (at release):

6.2.1.1. Concrete compression (AASHTO 5.9.2.3.1a) = \( 0.65f'_{ci} \text{ ksi} \)

6.2.1.2. Concrete tension outside precompressed tensile zone (AASHTO Table 5.9.2.3.1b-1)

Without bonded reinforcement = \( 0.0948 \lambda \cdot (f'_{ci})^{1/2} \text{ ksi} \leq 0.2 \text{ ksi} \)
With bonded reinforcement = \( 0.24 \lambda \cdot (f'_{ci})^{1/2} \text{ ksi} \)
6.2.1.3. Prestressing-strand stress (BSM Section 14.4.2) = 0.70\cdot f_{pu}

6.2.2. Allowable concrete stresses after losses have occurred:

6.2.2.1. Compression (AASHTO Table 5.9.2.3.2a-1)

\[
\text{PS+DL+LL} = 0.60 \cdot \phi_w \cdot f'c \text{ ksi} \\
\text{PS+DL} = 0.45 \cdot f'c \text{ ksi}
\]

6.2.2.2. Tension under Load Combination Service III (AASHTO Table 5.9.2.3.2b-1, bonded prestressing tendons

Moderate corrosion conditions = 0.19(f'_{ci})^{1/2} \leq 0.6 \text{ ksi}

6.2.2.3. Tension under Load Combination Service I with transformed section properties or Service III with gross-section properties (BSM Section 14.4.2): Zero tension allowed

6.2.3. Instantaneous losses shall be determined per AASHTO 5.9.3.2. Elastic gains due to superimposed loadings shall be considered.

Time dependent losses for decked bulb-tee girders shall be calculated in accordance with BSM Section 14.4.2.

\[
\Delta f_{plt} = 33 \left[ 1 - 0.15 \cdot \left( \frac{f'c - 6}{6} \right) \right] - 2
\]

6.2.4. Girders are designed as simple for live loads and composite dead loads. Provide adequate shear reinforcement in bridge girders so that the rating factors for moments and shears are approximately equal, within 20% (BSM Section 14.3.2).

6.2.5. Diaphragms shall be in accordance with AASHTO 5.12.4 and BSM Section 14.4.3. Diaphragms will be cast-in-place concrete placed perpendicular to the girders at midspan of the girders. Decked Bulb Tee sections will be tied together transversely with welded tie connections at 4'-0" maximum spacing along the length of the beams. Grout pockets shall be filled with non-shrink grout with a minimum compressive strength f'c = 9,000 psi. For the purposes of distribution factors, beams shall be considered connected only enough to prevent relative vertical displacement at the interface.

6.2.6. The top of the girders shall be finished with a light broom finish the entire length of the girder (AKDOT & PF plans).

6.2.7. Camber will be estimated using multipliers from PCI Design Handbook, 5th Edition, Section 4.8.5. Formwork for bulb-tee girders are cambered downward for the calculated prestressing camber minus self-weight deflections (AKDOT & PF plans).
6.2.8. The number of partially debonded strands should not exceed 25% of total number of strands, and the number of debonded strands in a 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 section. (AASHTO 5.9.4.3.3)

6.2.9. Two harp points, located 10’-0” from midspan, shall be used with strands bundled at midspan. The slope of the harped strands should not exceed 9 degrees (BSM Section 14.2.2).

6.2.10. All girders within a span shall be identically designed for the governing condition (interior or exterior girder). (BSM Section 11.4.5).

6.2.11. Per BDM Section 14.3.2, do not use the simplified procedure for prestressed and nonprestressed sections for determining shear resistance.
6.3. Approach Slabs

6.3.1. The bottom reinforcing steel cover shall be 3 inches for concrete cast against and permanently exposed to earth.

6.3.2. Concrete approach slabs will be used at each bridge end and anchored to the abutment diaphragm. Minimum length of approach slab shall be the greater of the length of the wingwall or 15’-0” (BSM Section 16.3.2).

6.3.3. The roadway end of the approach slab shall be supported directly upon the fill. The roadway approach pavement shall be constructed up to and against the end of the approach slab (per AKDOT & PF plans).

6.3.4. For approach slabs supported directly on fill, the end support is assumed to be a uniform soil reaction with a bearing length that is approximately 3’-0”. Design approach slabs greater than 20’-0” as longitudinally reinforced slabs using the Extreme Event II load combination (BSM Section 16.3.2). Since the approach slab is 20’ long, use the AKDOT standard details and there is no need for a special design.

6.4. Expansion Joints (not required)

7. SUBSTRUCTURE DESIGN

See BSM Sections 18.1 through 18.3 for abutment and pier design guidelines.

7.1. Piers

7.1.1. Piers are anticipated to be exposed concrete-filled steel pipe piles.

7.1.2. Piles shall conform to API 5L X52 PSL2, Fy = 52 ksi or API 2B using ASTM A709, GR50T3, Fy = 50 ksi (BSM Section 17.4.1) or ASTM A709, GR50T3 (one Helical seam weld) Fy = 50 ksi (Section 715-2.02 of the 2017 AKDOT Standard Specifications for Construction – will need an SCR to replace FP-14 715.05 (a)).

7.1.3. Piles shall not be battered (BSM Section 17.4.2).

7.1.4. Wall thickness shall not be less than 1:48 of the pipe diameter (BSM Section 17.4.1).

7.1.5. The effects of corrosion and deterioration from soil site conditions will not be considered in steel pile foundation design. Galvanize the pipe piles down to 10’ below scour elevation at piers. The AKDOT spec requires repair of any damaged galvanizing at any splices (due to welding) and above the ground. Two methods of repair are listed in the AKDOT Standard specs.

7.1.6. Driven pipe piles will not be embedded into pile caps, but will be left 2” minimum from top of steel pile to bottom of cap. Connections in these areas will be designed to develop plastic hinges resulting from extreme event seismic load case.
7.1.7. Maximum length of each pile section for determination of splices shall be 40’. Welds at splices will be full penetration groove welds and will require UT testing at all splices.

7.1.8. Pile design shall account for liquefiable soil layers at the pile bents in extreme event limit state analysis, with K=0 for soil stiffness modeling purposes at pile bents. Downdrag forces are to be included.

7.1.9. Resistance factors for structural capacity of steel pipe under combined axial and flexural resistance assume pile is undamaged after driving. (AASHTO 6.5.4.2)

7.1.10. Minimum pile spacing = 2.5 x pile diameter (AASHTO 10.7.1.2)

Edge distance to face of cap = 9” min./1”-0” preferred by AKDOT for pile driving tolerance (AASHTO 10.7.1.2 – 9” min)

7.1.11. Pipe piles shall be driven open-ended with the top xx’ filled with Class A(AE) concrete after removal of material from pipe. Soil within the pipe is not considered as an additional vertical load on piles. Pipe pile concrete fill limits: The depth of concrete fill in pipe piles is determined by the lateral load capacity taking into account scour/liquefaction depth. The controlling case typically occurs under seismic loading. The depth to bottom of concrete is taken as one-half Mmax. (Section 18.2.8 in the 2018 draft AKDOT BSM)

7.1.12. Per BSM Section 17.6.3, embed piles into the soil so that the deflected shape of the pile subjected to lateral loads crosses a zero deflection point in two places. For bridges in SDC B, C, or D, embed the pile sufficiently to achieve the overstrength plastic hinging moment of the pile. AKDOT allows below-ground plastic hinging of pile bents.

7.1.13. Use shear keys, dowels, or restrainers to transfer seismic load to the substructure from the superstructure (BSM Section 19.2.1).

7.1.14. The structural response due to effects from frozen soil shall be considered. (BSM Section 17.6.4) Pier models for ice loads and summer flow are recommended boundary conditions.

7.1.15. Pile/shaft design: AKDOT programs FB Multi-pier (Florida program), OpenSees v2 seismic pushover program (available for free on website) If OpenSees is used, need to develop forces in the cap by hand because the program has an error and does not correctly transfer the loads from the gap region in the piles into the cap.

7.2. Abutments

7.2.1. Provide semi-integral abutment diaphragms and wingwalls.
7.2.2. Bottom of abutment cap shall be placed 1’-6” minimum below berm elevation. AKDOT prefers 2 to 2.5’.

7.2.3. Bottom footing reinforcing steel is placed 3 inches clear of the bottom of the abutment cap.

8. MISCELLANEOUS

8.1. Drainage

8.1.1. Deck drains will not be provided on the bridge deck or approach slab unless hydraulic analysis proves otherwise.

8.1.2. AKDOT detail with coarse aggregate will be used as behind the abutments and wingwalls.

8.2. Bearings

8.2.1. Elastomeric Bearing Devices:

(1) AASHTO Design Method ‘B’ shall be used (BSM Section 19.2.3).

(2) The bearing loads and elastomer grade shall be shown on the plans. (BSM 19.2.3)

(3) The minimum length or width of the pad shall be 6 inches (BSM 19.2.3).

(4) No 65% reduction in the design thermal movement range (per AASHTO 14.7.5.3.2) is allowed (BSM Section 19.2.3).

(5) A rotation allowance for uncertainties (fabrication and installation tolerance) of 0.005 radians shall be included per AASHTO 14.4.2.1.

(6) Elastomeric pads shall only use steel reinforcement. Provide a minimum of 0.25” of elastomer cover to the edges of the steel reinforcement (BSM 19.2.3).

(7) Steel reinforced elastomeric bearing pads shall conform to AASHTO M251 with 50 Durometer hardness, elastomer Grade 5 or higher (BSM Section 19.2.3)

(8) The shear modulus (G) used for design shall be 115 +/- psi (0.95 min to 1.35 max). Design for both min and max shear modulus. (BSM Section 19.A.3).

(9) Beveled sole plates are not anticipated due to the shallow grade.
(10) Pad heights shall be limited to 6” (BSM Section 19.2.3)

(11) At fixed bearings, provide a horizontal restraint adequate for the full horizontal load.

8.3. Utilities

An 8” diameter openings per the AK plans in the exterior bays of the abutment and pier diaphragms for future utilities (BSM Section 16.5.4).

8.4. Lighting

No allowances for lighting shall be made at this bridge location.

8.5. Signing

No allowances for signing shall be made at this bridge location.

8.6. Aesthetic Treatments

8.6.1. Limits of aesthetic treatments shall be clearly detailed on the plans.

8.6.2. Details of the aesthetic treatments shall be specified in the Special Contract Requirements.

8.6.3. Structural steel components of bridge railing and transitions will be galvanized. No weathering steel components will be used.

9. LOAD RATINGS

The following design criteria identify the particular standards and procedures, which are used for the bridge rating:

9.1. Bridge rating calculations will be performed for both the interior and exterior girders. The reinforcing steel tables in the BSM will be followed for the top slab reinforcing, therefore no load rating for the top slab is required, but the justification will be documented. Compression reinforcement may be utilized for rating. Plan sheets or shop drawings will be provided for verifying input.

9.2. Bridge rating calculations will be in accordance with the LFR and LRFR methodologies. The LFR load rating will be checked per BSM Section 27.1.3. LRFR is a comparison load rating. Load rating information for both methods will be provided on separate AKDOT & PF forms. Inventory: multi-loaded lane inventory rating, single lane centered heavy vehicle load rating.

9.3. All loads and load combinations will be determined according to the LRFD methodology. The design vehicle will be the HL-93 for the LRFR rating and HS20 for the LFR rating. The intent of the design is to balance the moment and shear load rating to within 20%.
9.4. Overstresses determined from LFR bridge rating calculations will not be permitted. Any design, which yields a rating overstress, will be redesigned to satisfy the LFR rating requirements.

9.5. AKDOT prefers hand calculations to be submitted.
10. QUANTITIES

10.1. Calculate and report bridge quantities to the following accuracy:

10.1.1. Structural Concrete to the nearest 1 cubic yard.

10.1.2. Reinforcing Steel and Structural Steel to the nearest 100 pounds.

10.1.3. Piling and Bridge Railing to the nearest 1 foot.

10.1.4. Structure Excavation and Backfill to the nearest 10 cubic yards.

10.2. Independent check of quantities shall be performed, and discrepancies outside the following limits shall be resolved:

10.2.1. Structure Excavation and Backfill within 5%.

10.2.2. All other quantities within 1%.
11. SHEET LAYOUT

Sheets will be completed utilizing Microstation with WFL titleblock with full size scale for a 11 x 17 sheet.

Sheet layout for structural drawings for the bridge will generally be as follows:

RG3151-A.........................General Layout
RG3151-B.........................General Notes and Estimate
RG3151-C.........................Site Plan
RG3151-D.........................Riprap Layout
RG3151-E.........................Abutment 1
RG3151-F.........................Abutment 3
RG3151-G.........................Abutment Details
RG3151-H.........................Wingwalls
RG3151-I.........................Pier 2
RG3151-J.........................Pier 2 Details
RG3151-K.........................Framing Plan, Typical Section
RG3151-L.........................Girders
RG3151-M.........................Girder Details
RG3151-N.........................Approach Slabs
RG3151-O.........................Steel Bridge Railing
RG3151-P.........................Structure Transition Railing
RG3151-Q.........................Log of Test Holes
RG3151-R.........................Rebar List
RG3151-S.........................Rebar List
RG3151-T.........................Rebar List