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DESIGN SPECIFICATIONS

A. GOVERNING CRITERIA

1. AASHTO *LRFD Bridge Design Specifications,* Customary U.S. Units, Fourth Edition with Interims through 2008.
2. MoDOT Engineering Policy Guide.
3. Missouri Standard Plans for Highway Construction, English Version, April, 2008.
4. Construction Specifications
    a. 2004 *Missouri Standard Specification Book for Highway Construction*, August, 2008.
    b. Job Special Provisions, as required.
5. AASHTO "Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members."
6. AASHTO "Guide Specifications for Thermal Effects in Concrete Bridge Superstructures," 1989.
7. AASHTO "Guide Specifications for Highway Bridge Fabrication with HPS70W Steel", 2003.
8. AASHTO "Guide Specifications for LRFD Seismic Bridge Design, 1st Edition, 2009.
9. AASHTO/AWS D1.5M/D1.5:2008 Bridge Welding Code.
10. "Design of Bridge Deck Drainage," Publication No. FHWA-SA-92-010.
11. NSBA "Steel Bridge Design Handbook."

B. UNITS

1. The bridge shall be designed using English Units.
2. The units shown in the final plans shall be English Units.

C. LAYOUT

1. The spans and general arrangement of the structures are as shown on the preliminary plans.
2. Structure Width (out-to-out):
    a. MO Approach (Ramps 8 & 10): Varies.
    b. MO Approach: Varies, converging to 85'-8" near the Missouri anchor pier.
    c. IL Approach: two separate decks each with a 40-ft roadway and two 16" safety barrier curbs. There will be a 4" gap between the backs of the median safety barrier curbs between the decks.
3. Traffic Railings:
    a. ~~(Per MoDOT EPG 751.12.1)~~MoDOT Standard Safety

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Barrier Curb, 16" (TType DD, 42" tall. Shape)

b. MoDOT Standard Median Barrier Curb, 2'-10" (Modified Type C, 42" tall.

4. C Shape)Minimum Vertical Clearance

Formatted: Bullets and Numbering

a. Flood Wall (MO) – 10'-0"

b. Levee (IL) – 20'-0"

c. Ground level cross streets – 15'-06"

d. Illinois Route 3 – 16'-9"

e. Railroad Tracks (MO) – 23'-6" from T/R (final)

f. Railroad Tracks (IL) – 23'-6" from T/R (final)

Alternative Barriers may be selected through the ATC process. Any selected barrier shall be capable of slipform construction.

5. Minimum Horizontal Clearance

a. Flood Wall (MO) – 25'-0"

b. Levee (IL) – 25'-0"

c. Ground level cross streets – 5'-3" from shoulder line or 2'-0" from barrier curb

d. Existing Rrailroad tTracks (MO) - 14'-0" & 22'-0" from CL tracks (final); 12'-0" left and right of CL tracks (construction)

e. Existing Rrailroad tTracks (IL) - 14'-0" & 22'-0" from CL (final) ; 12'-0" left and right of CL tracks (construction)

6. Datum

a. Vertical – NAGVD 2988

7. Water Elevations (NGVD 29)
(NAVD 88 = NGVD 29 – 0.18 ft)

a. 5-yr Design High Water Elev. 416.4

b. 10-yr Design High Water Elev. 419.2

c. 50-yr Design High Water Elev. 425.1

d. 100-yr. Design High Water Elev. 427.1

e. Historic High Water Elev. 430.2 (1993 flood)

f. 2% Historic Flow Line Elev. 411.0

NAVD to NGVD conversion is per e-mail correspondence Brett Brooks (EDSI) to Hans Hutton (HNTB) 8/6/08
Water surface data provided by St. Louis District COE 8/5/08

D. DESIGN LOADS

1. Dead Load (DC, DW, & EV)

a. Self-weight: "A" DL

(1) Concrete (DC)

(a) Non-prestressed with reinforcing = 150 pcf

(b) Prestressed with reinforcing = 155 pcf

(c) Tremie concrete = 145 pcf

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(2) Steel = 490 pcf (DC)

(a) Misc. Steel = 15% of welded plate girder weight

(b) Earth = In accordance with AASHTO~~120 pcf (EV)~~

b. Superimposed Dead Load: "B" DL
(Equally distributed among all girders)

(1) An allowance of 35 psf shall be made for the weight of a future wearing surface on the approach spans. This load will not be included in dead load deflection computations. (DW)

(2) No provisions for future commercial utility attachments.

(3) Safety Barrier Curb = ~~5~~2per type. (DC)

(4) Median Barrier Curb = ~~1115~~ per type. (DC)

Design shall be for 16" wide by 42" high rails.

c. Total Dead Load = "A" DL + "B" DL

d. Erection Loads (EL):

(1) Construction loads shall be considered per AASHTO Article 6.10.3.4.

(2) Construction load cases shall be evaluated per AASHTO Article 3.4.2.

2. Live Load (LL, IM)

a. AASHTO HL-93 notional live load.

b. Design Speed = 60 mph.

c. HL-93 distribution factors per AASHTO~~ Article 4.6.2~~.

d. The maximum number of design lanes for superstructure and substructure design shall be based on a 12'-0" lane.

e. Dynamic load allowance, IM, shall be in accordance with AASHTO 3.6.2.

(1) Dynamic load allowance shall not be applied to pier foundations which are entirely below ground.

(2) Dynamic load allowance shall be applied to approach piers above top of footing.

f. Multiple presence factors per AASHTO 3.6.1.1.2.

g. Live load plus impact deflection criteria shall be in accordance with AASHTO 2.5.2.6.2~~. The optional deflection limits shall be employed.~~

h. Design Traffic (One Way):

(a) Design Year ADTT = 8150

3. Vehicular Collision Forces (CT)

a. Vehicular impact from railroads need only be considered per the requirements of AASHTO where substructures are adjacent to existing tracks.

4. Earthquake Effects (EQ)

a. Seismic Hazard

Higher levels of performance than specified in AASHTO have been mandated by the Owner for these bridges. A dual design philosophy shall be implemented for the approach spans:

(1) When subject to earthquake ground motions that have a 7% probability of exceedance in 75 years (975-year return period event – hereinafter referred to as the "design earthquake") all force effects will be in the near-elastic range (R=1.5 see below) with no associated cracking of prestressed concrete or yielding of structural steel under the action of the reduced force effects. For such event the seismic hazard parameters taken from the USGS map in the area at location:

Comment: Performance level immediately after such an event should be such that the bridge is fully accessible by both emergency vehicles and the general public. Level of repair after this event should be minor.

Latitude = 38.645864

Longitude = -90.178343

Comment: Taken at the centerline of the Main Span Bridge

PGA = 0.17 g

S_s = 0.35

S_1 = 0.10

(2) When subject to earthquake ground motions that have a 3% probability of exceedance in 75 years (2% in 50 years or 2500-year return period – hereinafter referred to as the "maximum credible earthquake") the structure shall resist force effects in the inelastic range (R = 2 - 3 as shown below). The structure would be expected to suffer significant damage but without collapse. For such event the seismic hazard parameters taken from the USGS maps in the same location as above shall be as follows:

Comment: Performance level immediately after such an event should be such that the bridge would be accessible to emergency vehicles with minimal shoring or repair, but would be open to public within a few days (5-7 days) with moderate repairs.

PGA = 0.23 g

S_s = 0.45

S_1 = 0.15

b. Site Effects

Subject to the geotechnical investigation report, site effect data on both sides of the rivers should be considered as follows.

(1) On the Missouri side of the river limestone is at relatively shallow depths. Available test data for the rock show average Standard Penetration Tests (SPT) blow counts consistently $\underline{N}$ > 50. However, above the limestone there are also other layers of soils with $\underline{N}$ < 50 in depths that vary from 15’ – 65’. As a result, it is expected that the average Site Class Definition will be classified as a Class C or better. Accordingly, the values of the Site Factors on the Missouri side shall be:

(a) For the 975-year event

$F_{pga} = 1.2$

$F_a = 1.2$

$F_v = 1.7$

(b) For the 2500-year event

$F_{pga} = 1.17$

$F_a = 1.2$

$F_v = 1.65$

(2) On the Illinois side of the river limestone is at much deeper elevations, more than 100' deep from surface. Available test data for the soils above rock show average Standard Penetration Tests (SPT) blow counts consistently 15>$\underline{N}$ > 50, and 600 <$\underline{v}_s$ < 1200. As a result, it is expected that the average Site Class Definition will be classified as a Class D. Accordingly, the values of the Site Factors on the Illinois side shall be:

(a) For the 975-year event

$F_{pga} = 1.456$

$F_a = 1.522$

$F_v = 2.4$

(b) For the 2500-year event

$F_{pga} = 1.348$

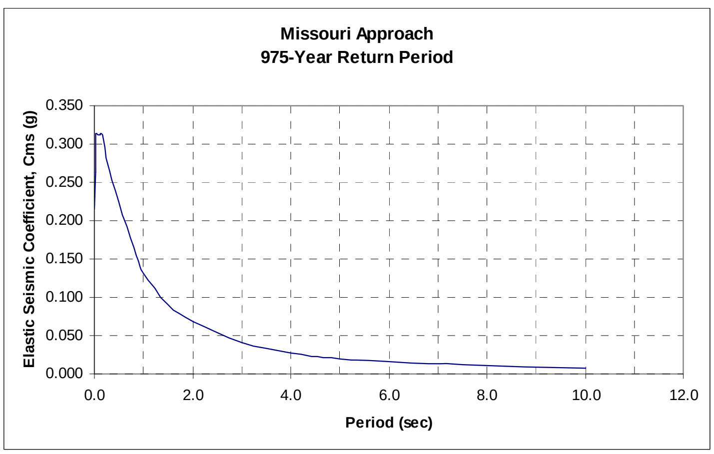
$F_a = 1.438$

$F_v = 2.188$

c. Design Response Spectra

(1) The Design Earthquake Response Spectrum for the Missouri side shall be as follows:

T	Csm
0.000	0.215
0.018	0.264
0.035	0.313
0.106	0.313
0.176	0.313
0.300	0.265
0.851	0.154
1.000	0.131
1.500	0.091
2.000	0.068
3.000	0.041
4.000	0.027
5.000	0.020
7.500	0.013
10.000	0.007



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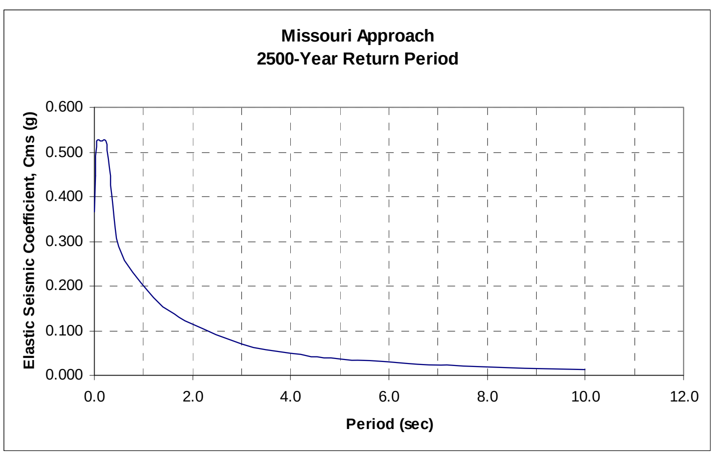
(2) The Maximum Credible Earthquake Response Spectrum for the Missouri side shall be as follows:

T	Csm
0.000	0.367
0.024	0.446
0.049	0.524
0.146	0.524
0.244	0.524
0.300	0.464
0.500	0.289
1.088	0.191
1.500	0.146
2.000	0.115
3.000	0.071
4.000	0.048
5.000	0.036
7.500	0.022
10.000	0.013



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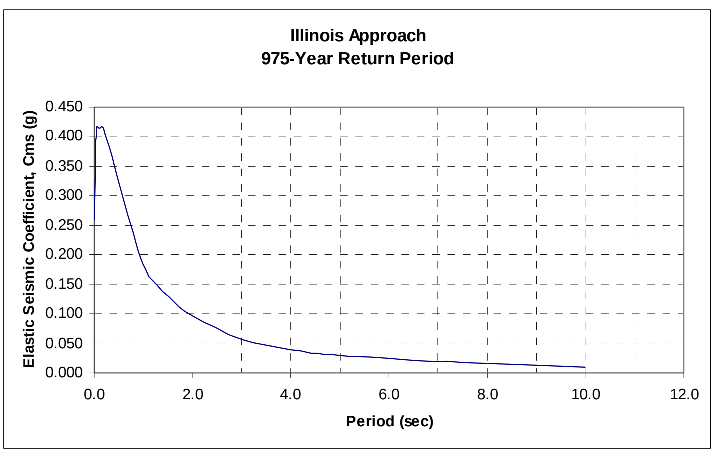
(3) The Design Earthquake Response Spectrum for the Illinois side shall be as follows:

T	Csm
0.000	0.259
0.019	0.337
0.037	0.415
0.112	0.415
0.187	0.415
1.000	0.185
1.500	0.129
2.000	0.096
3.000	0.058
4.000	0.039
5.000	0.029
7.500	0.018
10.000	0.010



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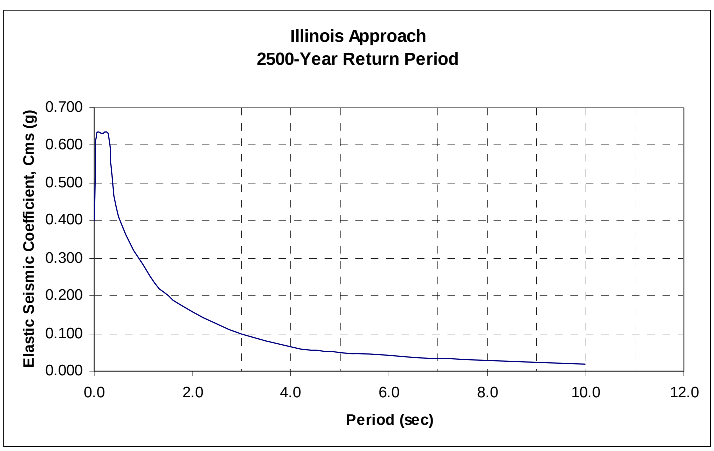
(4) The Maximum Credible Earthquake Response Spectrum for the Illinois side shall be as follows:

T	Csm
0.000	0.402
0.029	0.518
0.057	0.634
0.171	0.634
0.285	0.634
0.500	0.409
1.122	0.254
1.500	0.200
2.000	0.158
3.000	0.098
4.000	0.066
5.000	0.049
7.500	0.030
10.000	0.018



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d. Importance Categories

The approach bridges on each side of the river shall be classified as "Essential." The expected seismic performance shall be:

(1) For the 975-year event the approach structures perform in the "near-elastic" range.

After such an event the structure shall be readily accessible to both emergency/defense vehicles and the general public. The structures shall suffer minor repairable damage.

(2) For the 2500-year event the approach structures are allowed to yield yet not fail.

After such an event the structure shall be readily available to emergency and defense vehicles, following some immediate repair. More significant maintenance may be required before being open to public within 5-7 days. The structure may suffer significant damage without collapse, and partial or complete replacement may eventually be needed in the long term.

e. Seismic Performance Zones

(1) On the Missouri side for the 975-year event and the 2500-year event the S_{D1} acceleration coefficients are:

(a) S_{D1} = 0.167 for the 975-year event

(b) S_{D1} = 0.252 for the 2500-year event

(c) Therefore, the Missouri side approach

The SPZs are used to permit different requirement methods for methods of analysis, column details, and foundation and abutment design procedures. Therefore, the highest S_{D1} = 0.335 should control these different requirements.

structures should be assigned Seismic Performance Zone 2 (SPZ 2) ($0.15 < S_{D1} < 0.30$)

(2) On the Illinois side for the 975-year event and the 2500-year event the S_{D1} acceleration coefficients are:

(a) $S_{D1} = 0.235$ for the 975-year event

(b) $S_{D1} = 0.335$ for the 2500-year event

(c) Therefore, the Illinois side approach structures should be assigned Seismic Performance Zone 3 (SPZ 3) as $S_{D1} >$ 0.30 for the 2500-year event.

f. Response Modification Factors

(1) Response Modification factors are specified as follows:

Substructure	Earthquake Level	
	Design Earthquake	Maximum Credible Earthquake
Wall Type Piers – Larger dimension	1.5	1.5
Reinforced Concrete Pile Bents		
Vertical piles only	1.5	2.0
With batter piles	1.5	1.5
Single Columns	1.5	2.0
Steel or composite steel and concrete pile bents		
Vertical piles only	1.5	3.5
With batter piles	1.5	2.0
Multiple column bents	1.5	3.5

Connections	All Earthquakes
Superstructure to abutment	0.8
Expansion joints within a span of the superstructure	0.8
Columns, piers, or pile bents to cap beam or superstructure	1.0
Columns or piers to foundations	1.0

g. Combination of Seismic Force Effects

The elastic force effects on each of the principal axes of a component resulting from analyses in the two

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perpendicular directions shall be combined to form two load cases as follows:

(1) 100% of the absolute value of the force effects in one of the perpendicular directions combined with 30% of the absolute value of the force effects in the second perpendicular direction, and

(2) 100% of the absolute value of the force effects in the second perpendicular direction combined with 30% of the absolute value of the force effects in the first perpendicular direction.

h. Calculation of Design Forces

Earthquake loads for the approach structures shall be taken to be horizontal force effects to be determined as follows:

(1) On the Missouri side of the river, where the approach structures were assigned a SPZ 2, as a minimum all multispan structures shall be analyzed using the multimode elastic method. Seismic design force for all components shall be determined according to AASHTO 3.10.9.3, and

(2) On the Illinois side of the river, where the approach structures were assigned a SPZ 3, as a minimum all structures shall be analyzed using the multimode elastic method. However, for structures that exhibit different levels of irregularity, a time history method analysis may need to be performed. Regularity is a function of the number of spans and distribution of weight and stiffness. Regular, bridges have less than seven spans; no abrupt or unusual changes in weight, stiffness, or geometry; and no large changes in these parameters from span to span or support-to-support. The assessment of whether a time history method of analysis is to be carried out shall be made in a case-by-case basis in conjunction with the Owner and the Project Manager. If a time history analysis is deemed as the appropriate type of analysis for certain structures, time histories of ground acceleration that characterize the seismic hazard for the site shall be determined in accordance with AASHTO 4.7.4.3.4b. Seismic design forces for all components shall be determined according to AASHTO 3.10.9.4.1.

i. Miscellaneous

(1) Friction shall not be considered an adequate restrainer. Longitudinal restrainers are to be provided according to AASHTO 3.10.9.5.

(2) Hold-down devices shall be provided at support when required as discussed in AASHTO 3.10.9.6.

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(3) Minimum support lengths should be provided to all structures as required in AASHTO Guide Specifications for LRFD Seismic Bridge Design.

(4) No seismic consideration is to be applied to the approach bridges during the construction stage.

(5) Cracked section moment of inertia shall be used for the concrete sections of the substructures during the analyses.

(6) No live load shall be considered in the structure during the earthquake.

(7) Abutment contribution (where applicable) should be taken into account in the Earthquake Resisting System (ERS) both in the transverse and longitudinal direction.

(8) All the analyses recommended above for the global system models are to be elastic models.

(9) The design strategy to be implemented shall be to design a ductile substructure with essentially elastic superstructure. This will include conventional plastic hinging in the columns and walls and abutments that limit inertial forces by full mobilization of passive soil resistance. This strategy is implemented through the use of the R factors listed above.

(10) Damping shall be assumed to be 5 percent of critical.

5. Thermal Forces (TU, TG)

a. The design mean temperature shall be 60ºF.

b. Thermal effects due to temperature rise and fall (TU) for design of bearings, expansion devices, and substructure shall be calculated for the following ranges:

(1)	Steel:	Rise	60ºF
		Fall	90ºF
(2)	Concrete:	Rise	50ºF
		Fall	70ºF

Deck concrete temperature requirements shall be equitable with those for steel.

c. Design movement for bearings shall be in accordance with AASHTO.

6. Thermal Gradient, (TG):

a. Steel superstructure per AASHTO 3.12.3.

b. Concrete superstructure will use the Guide Specifications for Design and Construction of Segmental Concrete Bridges.

c. See AASHTO 3.4.1 for load factors to be used with TG.

7. Longitudinal Forces (BR)

a. Longitudinal forces, BR, shall be computed in accordance

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with AASHTO 3.6.4.

b. Forces at elastomeric bearings shall be in accordance with ~~MoDOT standard practice~~AASHTO.

c. Forces at PTFE bearings shall be based on a coefficient of friction of 0.06.

8. Earth Forces (EH, EV)

a. ~~An e~~Equivalent fluid pressure ~~of 60 lb/ft^3~~ (at rest condition) per AASHTO shall be used for design of the end bent wing walls.

b. ~~An e~~Equivalent fluid pressure ~~of 120 lb/ft^2~~ ((at-rest condition~~)~~) per AASHTO shall be used for live load surcharge.

c. Vertical earth loads shall be in accordance with AASHTO.

9. Wind Loads (WS, WL)

a. Wind loads shall be computed in accordance with AASHTO Article 3.8 for a base wind velocity of 100 mph.

10. Differential Settlement (SE)

a. Differential settlement between piers shall be considered in accordance with the Geotechnical Engineering Report.

E. LIMIT STATE COMBINATIONS

1. Limit state combination shall be in accordance with AASHTO Article 3.4, Table 3.4.1-1 except as noted below:

a. Shrinkage and creep of concrete are treated as dead loads for extreme events.

b. The factor γ_{EQ} for live load in combination with seismic loads for Extreme Event I shall be 0.0.

Comment [j1]: Should this be further investigated as this bridge will see significant rush hour traffic?

c. Load modifiers relating to Ductility, η_D, Redundancy, η_R, and Operational Importance, η_I, are to be applied to the design loading as follows:

Component	η_D	η_R	η_I
Deck Slab	1.00	1.00	1.05
Floor Beams	1.00	1.00	1.05
Drilled Shafts	1.00	1.00	1.05

d. All load modifiers are combined with individual load factors, η_i, as indicated in AASHTO Article 1.3.2.

e. η_I equal to 1.0 is applied to buoyancy force as a part of the total load WA.

f. Secondary effects due to post-tensioning (EL) shall have a load factor, η_p, of 1.0.

F. MATERIALS

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The properties listed shall be considered as minima for design and ATC consideration.

1. Concrete

   a. General

Location	Class	f'_c
Prestressed Deck Panels	A-1	6000 psi
Prestressed Beams	A-1	8000 psi
Safety Barrier Curbs	B-1	4000 psi
Median Barrier Curbs	B-1	4000 psi
Slabs	B-2	4000 psi
End Bents	B	3000 psi
Intermediate Bents	B	3000 psi
Footings	B	3000 psi
Drilled Shaft	B-2	4000 psi
Rock Sockets	B-2	4000 psi
Mass Concrete	B	3000 psi
Seal Concrete	Seal	3000 psi

   b. Initial Design Stresses

Location	Class	f'_c
Prestressed Deck Panels	A-1	4000 psi
Prestressed Beams	A-1	6500 psi

2. Prestressing Steel

   a. Strands

      (1) ASTM A 416, Grade 270 uncoated, seven-wire, low-relaxation strands

      (2) E = 28,500 ~~p~~ksi

      (3) Anchor Set = 0.375 in

      (4) Max. Jacking Stress (75% GUTS) = 202.5 ~~p~~ksi

      (5) Max. Anchorage Stress (70% GUTS) = 189.0 ~~p~~ksi

   b. Bars

      (1) ASTM A722, Grade 150 ksi (Type II)

      (2) E = 30,000 ~~p~~ksi

      (3) Anchor Set = 0.0625 in

      (4) Max. Jacking Stress (80% GUTS) = 120 ksi

      (5) Max. Anchorage Stress (70% GUTS) = 105 ksi

   c. Friction and Wobble Coefficients

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Duct	Friction Coefficient m($^1/_{RAD}$)	Strand Wobble Coef. K($^1/_{ft}$)	Bars Wobble Coef. K($^1/_{ft}$)
Galvanized	0.25	0.001	0.0002
Polyethylene	0.23	0.001	0.0002

3. Reinforcing Steel

   a. Reinforcing steel shall conform to the requirements of ASTM A 615, Grade 60, F_y = 60 ksi.

   b. Weldable reinforcing steel shall conform to the requirements of ASTM A706 Grade 60.

   c. All reinforcing steel within the deck, protruding into the deck and barrier curbs, in pier caps beneath deck expansion joints, and in other substructure elements within 25 ft. of roadways shall be epoxy coated.

   d. The maximum length for reinforcing bars shall be 60 feet.

   e. All bent bar dimensions shall be out-to-out.

   f. Clear cover shall be per MoDOT standard practice (MoDOT BDG 751.5.10) except as follows:

      (1) Pier caps – 2 inches

      (2) Pier columns – 2.5 inches

      (3) Drilled shafts and rock sockets – follow MoDOT EPG 751.37

4. Structural Steel

   a. Plate Girders: Structural steel shall be ASTM A709, Grade 50, and ASTM A709 Grade HPS 70W.

   b. Bearing Stiffeners: Structural steel shall be ASTM A709, Grade 50.

   c. Cross Frames, Diaphragms (including all connection plates and angles), and Intermediate Web Stiffeners: Structural steel shall conform to requirements of ASTM A709, Grade 36 or Grade 50.

   d. Field Splice Plates: Structural steel for flange and web splice plates shall be ASTM A709, Grades 50.

   e. Fasteners:

      (1) Bolts shall be ~~7/8" diameter~~ ASTM A325 high strength bolts.

      (2) Size

         (a) Primary member connections: 1-in. diameter

         (b) Secondary member connections: ¾" diameter, minimum

   f. Anchor Rods (Bearings): ASTM F1554 Grade 55

7/8" bolts may be proposed by contractors in the ATC process

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swedged anchor rods.

SUPERSTRUCTURE DESIGN

A. CONCRETE DECK SLABS

1. The slab shall be designed in accordance with AASHTO Article 4.6.2.1.
2. Prestressed concrete deck panels shall be used where applicable.
3. Top reinforcing steel cover shall be: 3-1/8" (Min. with #5 longitudinal bars); 2-3/4" (Min. with #8 longitudinal bars).
4. Bottom reinforcing steel (cantilevers and C.I.P. bays) cover shall be 1" (min.).
5. Slab details shall be based on 8-1/2" thick slab (min.), or 5 ½" of cast-in-place concrete over precast deck panels when utilized.
6. 1" reduction in the effective slab depth shall be made for the design of bottom transverse slab reinforcement.
7. Slab overhang design for Extreme Event I load case will be based on a TL-4 barrier curb rating and a 42" tall barrier..
8. A Class 2 exposure condition shall be assumed for crack control.
9. Stay-in-Place metal deck forms shall not be used.
10. Design of prestressed, precast panels (other than standard panels) will follow AASHTO 5.9 and 9.7.4.3, and MoDOT EPG 751.10.2.1. See section for "Prestressed Pecast Concrete Girders" below for applicable design parameters and criteria.

Update if age at deck pour differs from that shown for girders.

B. STRUCTURAL STEEL – APPROACH SPANS

1. Welded Plate Girders
   a. Plate girder design shall satisfy Strength, Fatigue, Serviceability, and Constructability Limit States in accordance with AASHTO LRFD.
   b. Composite section properties shall be used in the positive moment regions. Composite section properties, including deck reinforcement properties, may be used in the negative moment regions.
   c. A reduction of 1 inch in the effective depth of the slab shall be assumed for the design of the girders in the positive moment regions.
   d. A detail factor of 1.15 shall be applied to the girder self weight in BDGS.
   e. Shear connectors shall be 3/4 inch diameter by 5 inch long, unless larger studs, in accordance with MoDOT EPG 751.14.4, are required.
   f. Members requiring Charpy V-notch testing shall be called out as such in the plans. Members that require Charpy testing are:
      (1) All tension members or members subject to

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reversal of stress.

(2) All tension flanges of plate girders.

(3) Web plates of plate girders.

(4) Web splice plates.

(5) Top or bottom flange splice plates (tension) if notch toughness testing is required for the flanges on both sides of the splice.

g. Welded plate girders shall be cambered for dead load deflection and shall be computed at 1/20 points.

h. Plate girder intermediate stiffeners shall be single plates placed on one side of interior girders and the inside of exterior girders. They shall be fillet welded to the compression flange and tight fit to tension flange.

i. Cross frames or diaphragm connection plates for all girders shall be tight fit and fillet welded to the tension and compression flanges.

j. Bearing stiffeners shall be ground ~~or milled~~ to bear to the compression flange and tight fit to the tension flange. Bearing stiffeners which also function as connection plates for cross frames shall be fillet welded to the tension and compression flanges. Bearing stiffeners which also function as end diaphragm connection plates will be fillet welded to the top and bottom flanges, with the exception of cases in which finger plate type expansion joints are used. For the latter case, follow the standard finger plate type expansion joint details.

2. Field Splices

a. Field splices shall generally be located at points of dead load contraflexure.

b. The maximum length of member between field splices (that is, the maximum length of a fabricated girder section) shall be approximately 130 ft.

c. Field splices shall be designed in accordance with AASHTO 6.13.~~, except as modified in MoDOT EPG 751.14.3.~~

3. Fatigue

a. Per AASHTO 6.5.3.

b. Structural steel shall be designed for a fatigue life of 100 years.

c. Structural steel shall be designed for AASHTO Stress Categories A thru C' for redundant load path structures.

4. Barrier Curb

~~a.~~Provide joints in barrier curb ~~on either side of the CL of intermediate bents based on the following criteria:~~

~~C.~~

~~D. Span Length (ft) No. of Joints~~

~~E.~~

~~F. 0 to 80 1~~

~~G. 80 to 120 2~~

~~H. 120 to 160 3~~

~~I. 160 to 200 4~~

a. ~~200 to 240 5~~at 20 ft. (+/-) intervals along the entire length of structure.

~~J.~~C. PRESTRESSED PRECAST CONCRETE GIRDERS

1. Prestressed Concrete Stresses

a. Temp stresses at Service Limit State after release

(1) Compression: 3.9 ksi

(2) Tension:

Precompressed Tensile Zone, 0 ksi

In other areas without bonded reinforcement, 0.200 ksi

In other areas with bonded reinforcement, 0.612 ksi

b. Final stresses at Service Limit State after losses

(1) Compression: (Service Limit State I)

Due to the effective prestress plus permanent loads (Final 2), 3.6 ksi

Due to live loads plus one-half of the sum of compressive stresses due to the effective prestress and permanent loads (Final 3), 3.2 ksi

Due to the sum of effective prestress, permanent loads, and transient loads (Final 1), 4.8 ksi

(2) Tension: (Service Limit State III)

For component with unbonded prestressing tendons, 0 ksi

For components with bonded prestressing tendons or reinforcement (assuming moderate corrosion conditions), 0.537 ksi

2. Losses shall be calculated using the refined method per AASHTO 5.9.5.4. The following parameters will be used:

a. Age at release = 0.75 days

b. Age at deck placement = 56 days

c. Final Age = 3650 days (insignificant creep beyond 10 years)

d. Relative Humidity = 70%

3. Camber computations shall be based on MoDOT EPG 751.22.2.5.

4. The girders shall also satisfy the strength requirements for moment

COMMENTS

and shear due to Strength Limit State I.

5. Girders will be designed assuming simple span behavior for all loads, and checked for continuous behavior for composite dead loads and live load plus impact.

6. Camber and negative moment reinforcing steel will be determined assuming continuous behavior for composite dead loads and live load plus impact.

7. For composite design, allowance will be made for the difference in modulus of elasticity of the slab and girders.

8. The effective slab depth used for computing composite section properties and effective slab width shall be taken as 1 inch less than the total slab thickness.

K.

L.

M.

N.

SUBSTRUCTURE DESIGN

A. PIER

1. Pier shall be designed by the AASHTO LRFD method.

2. For designs not using a nonlinear analysis method (P-Δ), the effective length factors to be used for computing moment magnification factors shall be:

Transverse Design	K = 2.1 (single column)
Transverse Design	K = 1.2 (muli-column)
Longitudinal Design	K = 2.1

nns shall be designed and detailed with spiral reinforcement for confinement.

4.3. Pier caps at approach spans are assumed to be a minimum of 6" wider than the column diameter.

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B. FOUNDATIONS

1. Foundations shall be designed for all applicable limit states using factored loads according to AASHTO LRFD.

2. Concrete and steel design shall be per AASHTO LRFD LRFD Chapters 5 and 6, respectively.

3. Ultimate bearing capacity for footings, and ultimate pile bearing resistance and lateral load analysis parameters shall be per the geotechnical engineering report.

4. Drilled shafts may be designed for both side resistance and end bearing, as provided in the geotechnical engineering report.

COMMENTS

5.

a. Ultimate downward side resistance

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b. Ultimate uplift side resistance

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c. Ultimate end bearing resistance

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Lateral load analysis parameters

4.5. Minimum center-to-center spacing for drilled shafts shall in accordance with the geotechnical report.

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5.6. Drilled shafts shall be a minimum of 6" larger than the column diameter.

6.7. Rock sockets shall be 6" smaller than the drilled shaft diameter.

7.8. Moment magnification factors shall be applied for the design of pier foundations.

MISCELLANEOUS

A. DRAINAGE

1. Drainage shall be in accordance with FHWA Design of Bridge Deck Drainage.
2. Deck drainage system consisting of deck drains and collection system where necessary to prevent drainage directly on existing railroads or underpassing roadways shall be provided.
   a. Drainage and spread shall be based on the current proposed lane configuration (6 ft. and 10 ft. shoulders).

B. BEARINGS

1. Bearings shall be designed as specified in AASHTO and MoDOT EPG 751.11.
2. Forces at PTFE expansion bearings shall be based on a coefficient of friction of 6%.
3. Anchor Bolts shall be designed to resist 20% (minimum) of the DL reaction. Shear blocks shall be utilized as appropriate to prevent superstructure elements from losing support at beam seats.
4. The plans shall provide for future replacement of bearings, as appropriate.

C. EXPANSION DEVICES

2.1. Swivel modular expansion joints shall be used at deck joints.

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D. APPROACH SLABS

1. Plans for standard reinforced concrete approach slabs shall be provided.

E. UTILITIES & LIGHTING

1. No commercial utilities are to be placed on the bridge.

2. Roadway lighting by others.

F. SIGNING AND STRIPING

1. Signing and striping shall be by others.

G. STRUCTURAL STEEL PROTECTIVE COATING

~~2.~~1. The use of painted or weathering steel shall be determined by a cost analysis.

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~~H.~~

~~I.~~H. PROTECTIVE COATING FOR CONCRETE PIERS

1. Protective coating shall be applied to all concrete surfaces exposed to deck drainage. At end bents, seal the backwall, top of beam cap, front face of beam cap, and inside face of cheek walls with protective coating. At piers, seal top of beam and faces of beam.

2. The quantity of protective coating, in sq. ft., shall be included in the book of quantities but not shown on the plans.

3. The protective coating shall be epoxy.