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. Post Tensioning Institute "Recommendations for Stay Cable Design, Testing and Installation," 5th Edition, 2007.

6. AASHTO "Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members."

7. AASHTO "Guide Specifications for Thermal Effects in Concrete Bridge Superstructures," 1989.

8. AASHTO "Guide Specifications for Highway Bridge Fabrication with HPS70W Steel", 2003.

9. AASHTO/AWS D1.5M/D1.5:2008 Bridge Welding Code.

10. "CEB-FIP Model Code for Concrete Structures," 3rd Edition 1990, Appendix E Time Dependent Behavior of Concrete, Creep and Shrinkage.

11. ASCE "Guidelines for the Design of Cable-Stayed Bridges."

12. "Design of Bridge Deck Drainage," Publication No. FHWA-SA-92-010.

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.

COMMENTS

2. Structure Width:

a. Two 12'-0" lanes, one 10'-0" shoulder and one 6'-0" shoulder in each direction (i.e. 40'-0" between gutter lines in each direction). Two 16" safety barrier curbs and one 3'-0" median barrier curb. Total width back to back barrier rails is 85'-8".

b. Traffic Railings: Per MoDOT EPG 751.12.1

Modified Wyoming 2-Tube Steel Rail

Comment: The open rail is a desirable rail for wind performance and aesthetics.

MoDOT Standard Median Barrier Curb, 42" single slope

3. Channel – as designated by U.S. Coast Guard.

4. Minimum Vertical Clearance

a. Navigation Channel – 60 feet above 2% flow line

b. Flood Wall (MO) – 10'-0"

c. Levee (IL) – 20'-0"

d. Ground level cross streets – 15'-6"

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

f. Railroad Tracks – 21'-6" above T/R (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. Railroad Tracks (MO) - 14'-0" & 25'-0" from CL tracks (final); 12'-0" left and right of CL tracks (construction)

e. Railroad Tracks (IL) - 14'-0" & 25'-0" from CL (final); 12'-0" left and right of CL tracks (construction)

6. Datum

a. Vertical – NAVD 88

b. Horizontal – To be determined

c. Water Elevations (Based on NGVD 29)
(NAVD 88 = NGVD 29 – 0.18 ft)

Comment: NAVD to NGVD conversion is per e-mail correspondence Brett Brooks (EDSI) to Hans Hutton (HNTB) 8/6/08

(1) 5-yr Design High Water Elev. 416.4

Comment: Water surface data provided by St.

COMMENTS
Louis District COE 8/5/08

(2) 10-yr Design High Water Elev. 419.2

(3) 50-yr Design High Water Elev. 425.1

(4) 100-yr. Design High Water Elev. 427.1

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

(6) 2% Historic Flow Line Elev. 411.0

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

(2) Steel = 490 pcf (DC)

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

(3) Earth = in accordance with AASHTO LRFD (EV)

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

(1) Two inch minimum initial wearing surface on the ***main spans***. The wearing surface is nonstructural. (DC)

(2) No provisions for future wearing surface will be made for the main spans.

(3) No provisions for future commercial utility attachments.

(4) Safety Barrier Curb based on cross section (DC)

(5) Median Barrier Curb based on cross section (DC)

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

2. Erection Loads (EL):

a. Erection loads will be based on an assumed erection sequence.

COMMENTS

b. Construction load cases shall be evaluated per AASHTO Article 3.4.2.

3. Live Load (LL, IM)

a. AASHTO HL-93 notional live load.

b. Design Speed = 60 mph.

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

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

(1) Dynamic load allowance shall be applied to stay cables and anchors.

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

e. Multiple presence factors per AASHTO Article 3.6.1.1.2.

f. Design Traffic (One Way):

(1) Design Year ADT = To be determined

(2) Design Year ADTT = 8150

4. Earthquake Effects (EQ)

a. Seismic Hazard

Higher levels of performance than specified in AASHTO have been mandated by the Owner for the Main River Bridge. A single design philosophy shall be implemented for the main bridge.

Because the importance of the bridge is such that a lower probability of exceedance shall be considered, a site specific procedure specified in AASHTO 3.10.2.1 shall be used to obtain the acceleration spectrum.

(1) General Procedure

(a) When subject to earthquake ground motions that have a 3% probability of exceedance in 75 years (2500-year return period event) all force effects will be in the near-elastic range (R=1.5 see below in 6.) with no associated cracking of prestressed concrete or yielding of structural steel under the action of the reduced force effects.

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 shall be minor.

For such event the seismic hazard parameters taken from the USGS map in the area at the location:

COMMENTS
This location is taken at the centerline of the cable-stayed spans

Latitude = 38.65
Longitude = -90.18

shall be as follows:

PGA = 0.23
S_s = 0.45
S_1 = 0.15

b. Site Effects (General Procedure)

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

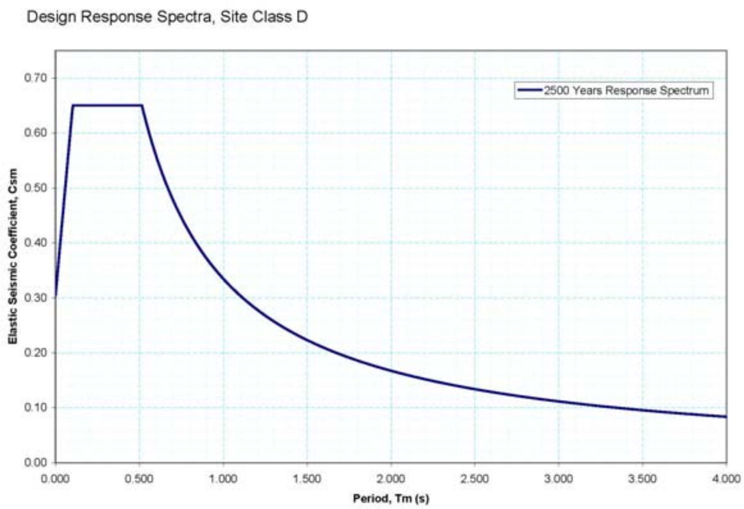
(1) On the river it at deep elevations 50’-70’ below ground line. Available test data for the soils above rock show average Standard Penetration Tests (SPT) blow counts consistently 15>$\underline{N}$ > 50, and 600 <$\underline{v}_s$ < 1200. As a result, it is expected that the average Site Class Definition will be classified as a Class D. Accordingly, the values of the Site Factors shall be:

(a) For the 2500-year event

F_{pga} = 1.35
F_a = 1.44
F_v = 2.19

c. Design Response Spectrum (General Procedure)

(1) Design Response Spectrum shall be as follows:



d. Importance Categories

(1) Cable-stayed spans shall be classified as follows:

For the 2500-year event the approach structures shall be classified as **Critical.**

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

e. Seismic Performance Zones

(1) The 2500-year event the S_{D1} acceleration coefficient is:

$S_{D1} = 0.335 > 0.3$

The cable-stayed spans shall be assigned Seismic Performance Zone 3 (SPZ 3).

f. Response Modification Factors

(1) Response Modification factors are specified as follows:

COMMENTS

Substructure	Importance Category Critical
Pylon Tower Legs (Above cap wall)	1.5
Single Columns (at top of footing)	1.5
Foundations	
• Drilled shafts	1.0
• Footings	1.0
• Caisson	1.0
Multiple column bents (at top of footing or bottom of pier cap)	1.5

Connection	
Superstructure to anchor piers, locking device at pylon	0.8
Columns, piers, to cap beam or superstructure	1.0
Columns or piers to foundations (Shafts, piles etc)	1.0

g. Seismic Force Effects

At least three response spectrum-compatible time histories shall be used for each component of motion in representing the design earthquake (ground motions having a 3% probability of exceedance in 75 years) during the analyses. Three orthogonal components (longitudinal, transverse and vertical) of design motion shall be input in the structure when conducting the analysis. The time histories will include modeling of spatial variations in terms of the differences between seismic wave arrival times at the different bridge piers.

The design actions shall be taken as the maximum or minimum response calculated for each combination of the three orthogonal time histories.

h. Miscellaneous

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

provided according to A 3.10.9.5.

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

(3) Minimum support lengths should be provided to all structures as required in A 4.7.4.4.

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

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

(6) Damping coefficient to be used in the analysis shall be 3% of critical damping.

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

(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 quasi-elastic substructure with essentially elastic superstructure. This will include conventional plastic hinging in the columns and walls. This strategy is implemented through the use of the R factors listed in the tables of section 6 above.

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

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

6. Thermal Gradient, TG, (main span):

a. Between cables and bridge ±18ºF

b. Between east and west face ±10ºF of tower and north and south side of bridge

COMMENTS

c. Superstructure per AASHTO 3.12.3.

7. Longitudinal Forces (BR)

a. Longitudinal forces, BR, shall be computed in accordance with AASHTO Article 3.6.4.

8. Earth Forces (EH)

a. Based on recommendations from project geotechnical engineer.

9. Wind Loads (WS, WL)

a. Wind forces shall be that provided as a result of wind tunnel test results.

b. Design wind speed for structural design of the completed bridge shall have a return period of 100 years. The design speed to be determined.

c. Design wind speed for flutter evaluation of the completed bridge to be determined.

d. Design wind speed for structural design during construction stages to be determined.

e. The wind speed for flutter evaluation during construction stages to be determined.

10. Stream Flow (WA)

a. Forces on substructure due to stream flow shall be computed in accordance with AASHTO 3.7.

b. Forces due to stream current shall be based on the 100-year event.

11. Ice Loads (IC)

a. Dynamic ice forces will not be considered due to the proximity of upstream locks and dams.

12. Vessel Collision (CV)

a. Vessel collision force shall be in accordance AASHTO 3.14

13. Scour

a. Design for the 100 year scour.

b. Assume ½ of 100-year scour for earthquake design.

c. For barge impact loading assume:

(1) Free floating barge – 100 year flood scour.

(2) Normal vessel operating conditions – 5-year flood scour.

d. Design for structural stability at the 500 year scour. Design loads shall be dead load (DC + DW), earth pressure (EH + EV), and water loads including buoyancy (WA).

14. Differential Settlement (SE)

a. Differential settlement between piers shall be considered in accordance with the recommendations of the geotechnical engineer.

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 effects of stay cable force adjustments are treated as dead load.

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

d. 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
Edge Girders	1.00	1.00	1.05
Cables	1.00	1.00	1.05
Tower Legs Above Deck	1.00	1.00	1.05
Towers Below Deck	1.00	1.00	1.05
Caissons	1.00	1.00	1.05
Drilled Shafts	1.00	1.00	1.05

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

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

COMMENTS

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

F. MATERIALS

1. Concrete Minimum Values

Location	Class	f'_c
Prestressed Deck Panels	A-1	6000 psi
Safety Barrier Curbs	B-1	4000 psi
Median Barrier Curbs	B-1	4000 psi
Deck (Cast-in-Place)	B-2	4000 psi
Intermediate Bents	B-1	4000 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
Tower	A-1	6000 psi
Caissons	B-2	4000 psi
Distribution Caps	B-2	4000 psi

Revised to Class A-1 per 501.3.8

2. Prestressing Steel

a. Strands

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

(2) E = 28,500 ksi

(3) Anchor Set = 0.375 in

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

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

b. Bars

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

(2) E = 30,000 ksi

COMMENTS

(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

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 in the deck and barrier curbs, caps at expansion joints and columns adjacent to roadways to a height of 25 ft shall be epoxy coated.

d. The maximum length for reinforcing bars shall be as 60 ft.

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

f. Clear cover shall be per MoDOT EPG except as follows:

(1) Top of deck slab (main spans) – 4 inches below top of concrete wearing surface

(2) Cable-stayed towers – 2 inches

(3) River piers below deck – 3 inches

(4) Caissons – 4 inches

4. Stay Cables

a. Steel for stay cables shall be 0.6 inch diameter low relaxation, weldless, seven-wire strand conforming to ASTM A416, Grade 270.

b. Steel strands shall be individually greased strands within a polyethylene sheath.

c. Acceptance testing of stay cables shall be in accordance with PTI Recommendations for Stay Cable Design,

COMMENTS

Testing and Installation.

5. Structural Steel

a. Plate Girders: Structural steel shall be ASTM A709, Grade 50, and ASTM A709 Grade HPS 70W. Tensile zones of edge girders will be considered fracture critical.

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.

6. Fasteners:

a. Bolts shall be ASTM A325 high strength bolts.

b. Size

(1) Primary member connections: 1-inch diameter

(2) Secondary member connections: 3/4" diameter, minimum

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

7. Anchor Rods (Bearings):

a. ASTM F1554 Grade 55 swedged anchor rods.

SUPERSTRUCTURE DESIGN

A. CONCRETE DECK SLABS

1. Prestressed concrete deck panels shall be used where applicable.

2. The 2" concrete wearing surface will be considered nonstructural. No reduction in the effective slab depth shall be made for the design of bottom transverse slab reinforcement.

3. Slab overhang design for Extreme Event I load case will be based on a TL-4 barrier curb rating.

4. Concrete panels shall be aged a minimum of 90 days before installation into the structure.

B. STRUCTURAL STEEL

1. Steel edge girders and floor beams

a. Design shall be by the AASHTO LRFD design method.

b. Composite design shall be used.

c. Effective width of deck slab will be determined in accordance with AASHTO. Edge girders shall be designed in accordance with Article 4.6.2.6.2 and floorbeams in accordance with Article 4.6.2.6.1. The wearing surface shall not be considered as effective in resisting axial or bending forces.

d. A value of $n=E_s/E_c$ shall be used.

e. A minimum age of the precast slab at time of placement shall be 90 days.

f. Tension in slab panels shall be limited to $0.0948\sqrt{f'c}$ (KSI) during handling, shipping, and storage.

2. Field Splices

a. Girder field splices shall be located between stay cables.

b. Floorbeams shall not have field splices.

c. Field splices shall be designed for 75 percent of the member strength or the average of the member strength and load in accordance with AASHTO Article 6.13.1.

3. Fatigue

a. Structural steel, cable stays and anchorages shall be designed for a fatigue life of 100 years.

b. Fatigue loading for floorbeam design shall be 6 lanes of HL-93 (truck load only) centered in future traffic lanes with impact and multiple presence factors in accordance with AASHTO.

C. Fatigue loading for edge girder and stay cable design shall be 1 lane of HL-93 (truck load only) plus impact positioned as close as possible to the girder or stay cable under consideration.

C. STAY CABLES

1. Stay cables shall be designed in accordance with PTI Recommendations for Stay Cable Design, Testing and Installation.

2. Provisions shall be made for the replacement of any individual cable while maintaining two lanes of traffic on the far lane and shoulder of the deck.

3. The design shall provide for the loss of any single stay cable with four lanes of traffic centered in traffic lanes with multiple lane

reduction factors.

4. Details will provide for the replacement of any individual stay cable by detensioning at the live end anchors.

5. After final adjustment the polyethylene pipe sheathing shall not be filled with grout.

6. Provisions shall be made for the adjustment of any individual cable during construction.

7. The following special design group loadings shall be used for the following two stay-cable conditions.

   a. Stay Cable Exchange

      Use a resistance factor, ϕ, of 0.8 for this limit state event:

      1.2DC + 1.4DW + 1.5(LL* + IM) + cable exchange forces

      Two lanes of live load on the far lanes and shoulder of the deck.

   b. Loss of One Stay-Cable

      Use a resistance factor, ϕ, of 0.9 for this limit state event:

      1.2DC + 1.4DW + 0.75(LL** + IM) + cable loss forces

      ** Four lanes of live load centered within the traffic lanes to maximize the reaction at the considered stay-cable.

SUBSTRUCTURE DESIGN

A. PIER

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

B. FOUNDATIONS

1. Foundations shall be designed for all applicable limit states using factored loads.

2. Concrete and steel design shall be per AASHTO LRFD.

3. Ultimate bearing capacity and lateral load analysis parameters for foundation elements shall be per the project geotechnical engineer report.

4. Drilled shafts shall be designed for side resistance and end bearing. The following items will be in accordance with the geotechnical engineering report:

   a. Ultimate downward side resistance

   b. Ultimate uplift side resistance

c. Ultimate end bearing resistance

d. Lateral load analysis parameters

5. Minimum center-to-center spacing for drilled shafts shall be in accordance with the project geotechnical engineering report.

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

7. Moment magnifications factors shall be applied for the design of pier foundations.

8. Resistance factors for foundation elements shall be per AASHTO.

C. SEALS

1. Cofferdams and seals shall be designed for a hydrostatic head from bottom of tremie seal to 5-years flood elevation.

2. The contractor shall be responsible for the design of the cofferdam.

3. Minimum tremie seal thickness shall be 3 feet.

MISCELLANEOUS

A. DRAINAGE

1. Drainage shall be in accordance with FHWA "Design of Bridge Deck Drainage", Publication No. FHWA-SA-92-010.

2. Deck drainage system consisting of deck drains and collection system where necessary to prevent drainage directly on railroads or underpassing roadways shall be provided.

B. BEARINGS

1. Anchor Bolts shall be designed to resist the horizontal forces determined from the seismic analysis.

2. The plans shall provide for future replacement of bearings.

C. EXPANSION DEVICES

1. Modular expansion joints shall be used at the end of the main spans. Swivel joints may be used if necessary for seismic considerations.

D. UTILITIES & LIGHTING

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

2. Obstruction lighting shall be in accordance with FAA requirements.

3. Navigation lighting shall be in accordance with USCG requirements.

4. Inspection lighting shall be provided within the towers.

E. SIGNING AND STRIPING

Signing and striping shall be by others

F. INSPECTION ACCESS

1. Main Spans

    a. A permanently mounted traveling inspection platform will be installed.

    b. Design live load for inspection platforms will be 50 psf plus two 200-lb concentrated loads positioned to produce maximum stress in the platform members.

    c. The length shall permit emergency exit from either end via an aluminum extension ladder stored onboard.

    d. The platform shall provide 5 feet of walkway width.

    e. Mechanical propulsion drives will be powered by an electric motor. Power source will be from the local utility.

    f. Trolley rails (wide-flange beam sections) will be attached to the underside of each floorbeam.

    g. Platform storage will be adjacent to the tower piers. Platform access from pier tops will be via permanently mounted gangplanks.

2. Towers

    a. Permanently mounted ladders with platforms at 20 to 30-foot intervals shall be provided inside each tower leg from top of pier cap to top of tower.

    b. The access door shall be placed at the bottom and at the top of each tower leg.

    c. Adequate ventilation shall be provided in each tower leg.

G. STRUCTURAL STEEL PROTECTIVE COATING

1. The intermediate field coat and finish field coat to be determined.

H. PROTECTIVE COATING FOR CONCRETE PIERS

1. Protective coating shall be applied to all concrete surfaces. Type of coating to be determined.