Design Challenges of BNSF Bridge 3.8 over the Missouri River near Plattsmouth, NE

Principal Author
Nick Staroski, PE
Senior Bridge Engineer
TranSystems
2400 Pershing Road, Suite 400
Kansas City, MO 64108
816-329-8837
njstaroski@transystems.com

Scott M. Mackiewicz, PhD, PE, D.GE
Senior Principal, Geotechnical Engineer
Kleinfelder
11529 W 79th Street, Suite 21
Lenexa, KS 66214
913-962-0909
smackiewicz@kleinfelder.com

Larry D. Woodley, PE
Director Bridge Construction
BNSF Railway
4515 Kansas Avenue
Kansas City, KS 66106
913-551-4131
Larry.Woodley@bnsf.com

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ABSTRACT

To meet increasing shipping needs and relieve the aging bridge built in 1879/1902 over the Missouri River near Plattsmouth, NE, construction of the $46-million BNSF project began in 2012. The new single-tracked, 11-span, 1,682-ft. bridge includes a 400-ft. main span camelback Warren truss, five 200-ft. steel deck plate girder spans and five prestressed concrete spans. The west approach cut includes nearly 85,000-ft.² of soil-nail wall with near vertical benched loess soil to reduce right-of-way requirements. The east approach consists of embankment within the Missouri River floodplain. The project includes nearly 2.3-miles of new track construction.
Key bridge design challenges included barge impact on river piers, longitudinal force distribution, soil-structure interaction analysis and design of the 200-ft. steel deck plate girder spans. The 200-ft. deck plate girder spans with a composite concrete deck are the longest of their type on the BNSF system.

Key geotechnical design challenges included development of a combined near vertical loess cut and soil-nail wall configuration to provide the proposed 100-ft. cut within the current BNSF right-of-way and constructing high embankments up to 52 ft. upon soft alluvial soils. Other geotechnical issues addressed during the design included evaluation of rock scour, modification to existing USACE levees, and driven pile/drilled shaft foundations for bridge support.

This paper will discuss the project approach and highlight: 1) the bridge design, 2) evaluation of potential scour issues, and 3) design of the 100-ft. vertical loess cut with soil-nail wall.

INTRODUCTION

Located 20 miles south of Omaha, NE., the original 1673-ft. single-track Plattsmouth Bridge over the Missouri River has passed the test of time by safely providing mainline rail service for over 130 years.

The original bridge built in 1880 was designed by George S. Morison and was his first major bridge commission as a consulting engineer for The Chicago, Burlington and Quincy (CB&Q), one of several predecessor railroads to BNSF Railway. The bridge underwent major structural revision in 1903 and currently consists of a 402-ft. pin-connected through truss, five 200-ft. pin-connected deck trusses (one truss made from two of the 1880 iron trusses), and four other shorter girder spans.

As a critical link in the BNSF Railway network, the Plattsmouth Bridge is one of nine BNSF Missouri River crossings from Bismarck, ND to St. Louis, MO. An average of 50 trains per day (131 Million Gross Tons (MGT)) comprised of coal, grain, intermodal, two Amtrak trains, and mixed freight trains utilize this single track bridge.

The mainline route at the Plattsmouth Bridge has seen a significant growth in rail traffic over the past 30 years with an increase of more than 200 percent in annual MGT. This fact coupled with the aging structure resulted in increasing maintenance demands and associated costs. Significant repairs, component replacement, and member reinforcing were performed during this time period for continued safe operations over the bridge. Consequently in 2010, the BNSF Railway authorized an engineering evaluation to analyze replacement alternatives to the existing bridge. Five routes consisting of two adjacent alignments and three major re-alignments were evaluated. Based on this evaluation, the alternative of constructing a new bridge 60-ft. downstream of the existing bridge was selected based on cost, constructability, and operational needs.
After the new bridge is placed in service late 2013, BNSF Railway will utilize the existing bridge for empty train traffic for the near term. In the future, the existing bridge will either be substantially rehabilitated, replaced, or simply removed based on operational demands.

The new Plattsmouth Bridge consists of a single-tracked, 11-span, 1,682-ft. bridge including a 400-ft. main span camelback Warren truss, five 200-ft. steel deck plate girder spans and five prestressed concrete approach spans. The truss and typical deck plate girder span are shown in Figure 1 and the general elevation of the bridge is shown in Figure 2. The west approach cut includes nearly 85,000-ft.$^2$ of soil-nail wall with benched upper slopes to reduce right-of-way requirements. The east approach consists of embankment within the Missouri River floodplain. The project includes nearly 2.3-miles of new track construction.

![Completed Truss and Deck Plate Girder Span](image-url)
BRIDGE DESIGN CHALLENGES

With many aspects to the design of this 1682-ft bridge, this paper focuses on the barge impact loading, longitudinal force distribution to the substructure and design of the 200-
ft steel deck plate girder spans. The structure was designed in accordance with the 2010 AREMA Manual for Railway Engineering.

**Barge Impact**

Vessel traffic in the Missouri River near Plattsmouth, NE is low seeing approximately 50 vessels per year. Even with this low traffic and probability of vessel impact, the bridge importance and structure type warranted vessel collision force to be considered in the design. AREMA gives recommendations for design of pier protection systems in Chapter 8 Part 23 but does not give specific design guidelines for impact to permanent main substructure elements. For the main channel river piers (Pier 7, 8 & 9), the vessel impact loading was determined according to the *AASHTO 1991 Guide Specifications for Vessel Collision of Highway Bridges (AASHTO Guide Spec)*. This specification was chosen for its consistency to AREMA load factor design methodology and is intended to provide bridge components with a “reasonable” resistance capacity against ship and barge collisions.

Without detailed knowledge of the vessel types and frequency in this area, Method I of the AASHTO Guide Spec was used to provide a simple, conservative procedure for determining the barge impact force. Using this method, first the vessel collision energy is determined. With input from the United States Army Corps of Engineers (USACE), the design barge configuration selected was the Jumbo Hopper, with slight modifications. The tow configuration selected was two barges wide by three barges long.

With the vessel velocity and weight known, the last factor in determining the vessel collision energy is the hydrodynamic mass coefficient. This accounts for the mass of water surrounding and moving with the vessel which depends on many factors. The next step in determining the barge impact force is to calculate the barge bow impact damage depth. This variable describes the distribution of energy from the vessel to the pier on impact. With the barge impact damage depth known, the equivalent static barge impact was calculated to be 2,500-kips.

Per the AASHTO Guide Spec, the full force is applied on the pier in the direction of stream flow and 50 percent of the force is applied transverse to stream flow, but not concurrently. The force is applied at a distance equal to three-ft above the 50-year flood water surface elevation. The design water level selected was the 50-year (2 percent) flood event water surface elevation.

The substructure was designed using load factor design (LFD) methodology for concrete elements and foundations. The load combination for vessel collision was determined according to AASHTO Guide Spec:

$$D + E + B + SF + Vessel$$

where:  
D = Dead Load  
E = Earth Pressure
Rail live load is not considered in this combination primarily due to the low probability of a vessel collision with a train on the structure.

**Longitudinal Force Distribution**

The current AREMA design longitudinal force revised in 1993 was used for the design of each substructure element. The total longitudinal force is distributed to each substructure element based on the relative stiffness of each. The magnitude and distribution method of this longitudinal force is relatively new to the design code. The following is meant as a case study to showcase the impacts of the design code to the structure design and detailing.

The longitudinal force applied to the substructure is specified in AREMA Chapter 8 Part 2.2.3(j). The design longitudinal force is taken as the larger of the braking force \((45 + 1.2L)\) or traction force \((25 \cdot L)\) where \(L\) is the total length of the bridge for continuously welded rail. The braking force controls the design and was calculated to be 2064-kip. This force is distributed to the substructure elements based on the relative stiffness of each to the total stiffness of the entire bridge, including the abutments. The longitudinal deflection of the entire bridge acting as a unit is also limited to a maximum of one inch. This limit is set to increase serviceability of the structure and reduce potential track problems just beyond the end of the bridge.

With the load distributed to each substructure element based on its stiffness, the process of determining the load becomes iterative in that with each change in pier geometry, the load applied to all substructure elements changes. For this reason, it is extremely beneficial to use computerized methods to simplify the calculations. The next challenge is to determine the actual stiffness value (kips/inch) of each substructure element. Another challenge is determining the depth of fixity of each pile or drilled shaft. This is typically an assumed value based on experience or empirical equations. For this project, a more in-depth approach was used taking into account the soil-structure interaction to determine the theoretical point of fixity.

To determine the pier stiffness values, FB-Multipier v4.17 software developed by the Florida Bridge Software Institute was used. This software has the ability to model the effect of vertical pile displacement on the rotation and lateral deflection at the top of the pier cap. This software allows nonlinear structural finite element analysis along with nonlinear static soil models for axial, lateral and torsional soil behavior. The software also allows the entire pier to be modeled, from top of cap to bottom of rock socket, in one analysis. This ensures rotational and displacement compatibility between the footing and drilled shafts.

A structural model was created for each different pier and abutment configuration to determine the stiffness of the substructure elements. Piers 2, 3, 4, 5, 6 and 10 consist of...
a rectangular pier cap supported on two round columns transitioning into drilled shafts with rock sockets. The longitudinal stiffness constant for these types of piers are easily calculated assuming they act as a cantilevered beam with a rigid support at a specified point below the top of soil. For simplicity, the weighted moment of inertia of the pier column and drilled shaft was used. The gross moment of inertia was assumed for concrete members under service loads.

A FB-Multipier model was created to verify the depth to fixity for Pier 6 by imposing a one inch displacement at the top of the pier. The corresponding lateral reaction becomes the stiffness constant. The cantilevered length of the pier and drilled shaft was then calibrated to obtain the same stiffness constant as the model. This depth to fixity below the groundline was then used for other similar types of piers.

Structural models were created for Piers 7, 8 and 9 taking into account the unique geometry of each. Similar to the two column piers, a deflection of one inch was imposed at the top of the pier cap and the lateral reaction was used as the stiffness constant. The concrete seal below the footing was ignored for lateral stiffness calculations. Abutments 1 and 12 and Bent 11 consist of precast concrete caps supported on a single row of steel h-piles. A model was created for each accounting for pile batter and eccentric dead load reactions from the superstructure. The abutment backwall and passive soil resistance were not modeled due to program limitations. The lateral stiffness of the abutment is then taken in the direction towards the main river channel only, with no passive soil support. This abutment stiffness was conservatively assumed to be the same for each abutment. Once the lateral stiffness of each pier and abutment was determined, the longitudinal force was calculated for each. Through this analysis, it was shown that the majority of the longitudinal force is transferred through the abutments, as would be expected.

The standard concrete end spans are typically supported on plain neoprene bearing pads with no means of longitudinal force transfer other than the pads themselves and preformed joint filler between the backwall and end of beams. The shear resistance of the bearing pads was determined not to be sufficient to transfer the force. Thus, anchor bolts were placed in precast holes and grouted into place to transfer the longitudinal force. The detail at each abutment is shown in Figure 3. The concrete bursting capacity of the precast slab and double cell beams was also checked.
**200-ft. Span Deck Plate Girder Design**

The existing east approach spans leading up to the main span truss are comprised of five 200-ft pin connected deck truss spans. To minimize fabrication and erection costs, a welded steel, four-girder system with a composite concrete deck was utilized for the new bridge as shown in Figure 4. This system increases the vertical clearance while providing a redundant girder system. The girder elevation and framing plan is shown in Figure 5. The girders are spliced near the quarter points with bottom lateral bracing throughout.
Figure 4 – Girder Cross Section
Design of the composite girder section was in accordance with AREMA Section 15.1.7.9. Allowable stress design (ASD) was used for the composite steel and concrete section with stresses calculated based on the moment of inertia method. Provisions for the depth of web in compression specified in the *AASTHO LRFD Bridge Design Specifications, Fifth Edition 2010*, were also satisfied to ensure ductility at the ultimate strength condition state. The non-composite steel section was checked for construction loading during the deck placement. In addition, the non-composite section was checked for E65 live load assuming the deck has lost composite action and only provides continuous bracing of the top flange. The girders satisfied the live load distribution requirements such that each girder shares the live load equally. The exterior girder was found to control the design due to a lesser effective deck width.

The concrete stress at the top of the deck was checked versus the limit of 0.4$f'_c$ as specified in AREMA 8.2.26.1(a). This limit on the concrete compressive stress was reached before the tensile stress limit in the bottom flange of the steel section. For this reason, the concrete compressive stress in the deck was increased to 5500 psi. Table 1 shows the utilization ratios for the different limit states checked. The utilization ratio is the load effect divided by the stress limit. A value greater than 1.0 violates the allowable stress limit.
TABLE 1 – Girder Design Summary

<table>
<thead>
<tr>
<th>Design Summary - Exterior Girder @ Midspan</th>
<th>Utilization Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flange Bending Stress - NonComposite Construction</td>
<td>0.52</td>
</tr>
<tr>
<td>Top Flange Bending Stress - NonComposite (E65)</td>
<td>0.96</td>
</tr>
<tr>
<td>Bot. Flange Bending Stress - NonComposite (E65)</td>
<td>0.96</td>
</tr>
<tr>
<td>Concrete Deck Compressive Stress - Composite</td>
<td>0.99</td>
</tr>
<tr>
<td>Top Flange Bending Stress - Composite</td>
<td>0.77</td>
</tr>
<tr>
<td>Bot. Flange Bending Stress - Composite</td>
<td>0.98</td>
</tr>
<tr>
<td>Web Shear Stress</td>
<td>0.23</td>
</tr>
<tr>
<td>Live Load + Impact Deflection</td>
<td>0.82</td>
</tr>
<tr>
<td>Flange-to-Web Weld Stress</td>
<td>0.50</td>
</tr>
<tr>
<td>Fatigue Tensile Stress - Bot. Flange</td>
<td>0.60</td>
</tr>
<tr>
<td>Fatigue Tensile Stress - Intermediate Stiffener</td>
<td>0.81</td>
</tr>
</tbody>
</table>

The depth of the web plus camber was limited to 144-in. to fit within the limitations of more fabrication shops. A non-slender web section was utilized to eliminate the need for longitudinal stiffeners and to provide greater out-of-plane stiffness during handling and erection. These factors along with added long term durability were considered to outweigh the efficiency and perceived cost savings gained by using a slender web section.

The design process described herein reiterates that serviceability and constructability often control the steel section sizes. Using a non-slender web eliminates longitudinal stiffeners and allows the intermediate transverse stiffeners to be spaced farther apart, further simplifying fabrication and cost. A thicker web also provides higher out-of-plane stiffness during shipping and handling and provides higher ductility at the ultimate strength limit state. For this project the benefits of using a thicker, non-slender web proved to outweigh the reduced weight of a slender web.

GEOTECHNICAL DESIGN CHALLENGES

The project alignment transects the Missouri River floodplain on the eastern approach and loess bluffs underlain by glacial till along the western approach. The primary geotechnical concern for the project was the construction of a 100-ft. high cut within the loess bluff at the western approach. Other geotechnical challenges included construction of 50-ft. high eastern approach embankments over soft clayey alluvium, potential scour of river alluvium and bedrock, and variable depth to bedrock (20 to 70 ft.) for support of bridge foundations.
Geologic Conditions

The Missouri River alluvium is comprised of about 10 to 20 ft. of clay mantling the underlying sandy silts, silty sands and sands that extend to the underlying Pennsylvanian bedrock comprised of interbedded limestone and shales. The loess bluffs were created by loess (wind-blown silt) that was carried by the wind from the adjacent Missouri floodplain following periods of glacial meltwater flooding. The underlying glacial till was placed during the glacial advances and recessions. Groundwater was observed near the Missouri River elevation within the alluvial floodplain and near the glacial till/loess interface along the western approach.

Loess Slope Design

The loess slope was designed using the observational approach, sliding wedge theory and the WASHDOT (2005) loess slope design guidance. Material properties of the loess are shown in Table 2. The loess is classified as a “silty loess” in accordance with WASHDOT guidance and could be constructed as a near vertical cut (0.25H:1V) provided that moisture contents are maintained at relatively low levels (< 17 percent) and drainage facilities are constructed to prevent increase of moisture content due to surface infiltration. Lohnes and Handy (1968) performed research on the Nebraskan loess and concluded that maximum stable loess cut slopes are related to soil density and shear strength in accord with “sliding wedge theory”. In addition, they developed a hypothetical sequence of loess slope failures when slough material is removed from the toe of a cut, and as removal of material progresses until the natural angle of repose is reached, which was reported to be about 38 degrees.

Existing Loess Cut Observations

The original loess cut was constructed in 1976 during the initial realignment of the west approach. Existing benches are about 35-ft. in height and have a remaining bench width of about 6 to 24 ft.

TABLE 2 – Loess Material Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Range of Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material Type</td>
<td>Lean Clay, Clayey Silt, Silt</td>
</tr>
<tr>
<td>Unit Weight, pcf</td>
<td>87 to 105</td>
</tr>
<tr>
<td>Moisture Content, %</td>
<td>11 to 26</td>
</tr>
<tr>
<td>Plasticity Index, %</td>
<td>6 to 20</td>
</tr>
<tr>
<td>Sand Content, %</td>
<td>1 to 3</td>
</tr>
<tr>
<td>Silt Content, %</td>
<td>70 to 83</td>
</tr>
<tr>
<td>Clay Content, %</td>
<td>15 to 27</td>
</tr>
<tr>
<td>Shear Strength</td>
<td>c = 684 to 800 psf phi = 26 to 30 degrees</td>
</tr>
</tbody>
</table>

Figure 6 – Existing Loess Slope (2010)
ft. (see Figure 6). In addition, the concrete lined ditch constructed originally on each bench to aid in transport of surface water has been covered by talus (i.e., slough material), broken, or removed by erosion. Based on topographic data and observations, the current angle of repose for the bench slopes ranges from about 38 degrees (i.e., angle of repose) to about 56 degrees where benches were relatively intact.

Slope Configuration Design

The slope configuration was designed using “sliding wedge theory” and the subsurface information, observational method, and the laboratory testing performed on the loess materials. The results of our evaluation indicated that the slope could be configured with (see Figure 7):

- Max. Bench Height = 35-ft. (from wedge theory)
- Bench Width = 0.55H₂ + 0.5H₁
  (H₁-height of cut face 1, H₂-Height of cut face 2)
- Residual slope = 1.3H:1V (38 degrees)

![Figure 7 – Loess Cut Slope Configuration](image)

The stability of the proposed loess cut was evaluated using slope stability analyses for the different potential failure modes: 1) local wedge failure (no tension crack), 2) local wedge failure (w/ dry tension crack), 3) local wedge failure (w/ wet tension crack), 4) global failure (no tension crack), 5) global failure (w/ dry tension crack), 6) global failure
(w/ wet tension crack) and 7) residual slope. Slope stability calculations were performed using the computer program "SlopeW" which calculates the Factor of Safety (FOS) of a slope using the Spencer limit equilibrium method of slices and block/wedge failure theory. All of the designed loess slopes meet or exceed the minimum slope stability FOS of 1.3 established for the project, since the slope is designed to accommodate slope sloughing to the residual slope angle of 38 degrees (1.3H:1V) over time. Drainage measures incorporated into the design to improve the loess cut slope performance included ditches above upper bench to divert surface water, an EPDM geo-membrane/geotextile layer placed on the bench surfaces to reduce water infiltration, berms and pipes used in areas where surface drainage is going to be directed over the slope, and seeding of the bench surfaces to reduce erosion. Loess slope construction is shown in Figure 8.

Figure 8 – Loess Cut Slope Construction
Potential Conflict Areas

Three potential conflict areas were identified during design that would not allow the near vertical loess slope to be constructed along the western approach. The conflicts included: 1) a high pressure gas line crossing, 2) proximity of the US 34 right-of-way, and 3) proximity of adjacent property structures. In the conflict areas, a soil nail wall was constructed in order to maintain the loess cut and residual slope configuration (see Figure 9) within BNSF Railway right-of-way and existing pipe backfill materials.

Figure 9 – Soil Nail Wall – Loess Slope
The soil nail retaining wall system was contracted as a design-build structure. About 85,000 sq. ft. (2,125 ln. ft. x 40-ft. average height) of soil nail wall construction was required to develop the design grades within the western approach right-of-way.

The soil nail wall was designed using ultimate soil-grout bond strength of 15 psi. Soil nails were installed at lengths between 25 and 50 ft. on a four-ft. by four-ft. or six-ft. by six-ft. square nail spacing pattern. The soil nail wall facing was comprised of five inch thick construction shotcrete, one inch thick insulating foam, and a seven inch thick permanent shotcrete. Strip drains extending to weeps at the base of the wall were installed behind the construction shotcrete at a center-to-center spacing of six-ft. Typical soil nail wall construction is shown in Figure 10.

**Rock Scour Potential**

The potential scour of the underlying rock units in relation to the bridge foundations was evaluated using the Annandale method that empirically compares rock characteristic data to the potential for scour under varied flow conditions and stream power. According to Annandale (1995), the erodibility index (K) of a granular material or rock mass is directly related to the erosive power of the water flowing (i.e, available stream power, ASP) in contact with the rock mass.

The erodibility index (K) of the shale and limestone materials estimated from the rock mass and strength data ranged from 13 to 2975. Based on HECRAS modeling, the estimated available stream power (ASP) at the piers within the bridge opening is 0.525 kW/m² (36 lb/ft-s) for the 100-year event and 0.978 Kw/m² (67 lb/ft-s) for the 500-year event. Based on our erodibility index estimates and the estimated ASP for the 100-year and 500-year event, the potential for scour in the observed limestone and shale is very low (see Figure 11).
CONCLUSION

Coordination, effective decision making, trust, flexibility, innovation, determination, and perseverance are just a few of the key words that describe the building of an effective team resulting in the successful completion of the Plattsmouth Bridge project on time and budget. The bridge design challenges were tackled by using simple, practical approaches coupled with sound engineering principles. The unique soil conditions of this area created the opportunity for distinctive design and construction. In the end, collaboration and effective communication proved to be keys to a successful project.
REFERENCES

AASHTO LRFD Bridge Design Specifications, Fifth Edition 2010
AASHTO 1991 Guide Specifications for Vessel Collision of Highway Bridges
AREMA 2010 Manual for Railway Engineering, Volume 2 - Structures
WASHDOT, Washington DOT, Geotechnical Design Manual, M46-03,

Geology, May 1968.

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Design Challenges of BNSF Bridge 3.8 over the Missouri River near Plattsmouth, NE

Nick Staroski, PE
Scott M. Mackiewicz, PhD, PE, D.GE
Larry D. Woodley, PE
Site History

- Built in 1880 for the CB&Q
- George S. Morison Design
- Revisions in 1903

Existing Bridge
- 400’ Main Span Through Truss
- Five 200’ Deck Truss Spans

Project Overview
- New single track structure offset 60’
- 11-Span, 1682’ total bridge length
  - 400’ main span truss
  - 5-200’ steel deck plate girder spans w/ comp. conc. deck
  - 2-82’ prestressed concrete I-girder spans
  - 3-prestressed concrete standard spans

Project Timeline
- Dec 2010: Start Design
- 1 April 2012: Full NTP
- 1 Sept 2013: Bridge Subst. Complete
- 31 Dec 2013: Contract Complete

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Structural Design Challenges

- Pier Design
  - Barge impact on river piers
  - Longitudinal force distribution
  - Soil-structure interaction

- 200’ Deck Plate Girder Spans
  - Composite design
  - Lateral Stability

- 400’ Truss Span

Pier Design - Geometry

- Supports 400’ truss span & 200’ DPG span (6,700 k total reaction D+L+I)
- Cap 9’ x 37’ x 11.5’
- Column 8’ x 28’ x 21’
- Pier Wall 9’ x 37’ x 47’
- 79.5’ T/Cap to T/Footing

Pier Design - Barge Impact

- Design per 1991 AASHTO Guide for Vessel Collision
- Barge configuration provided by USACE
  - 6 box type barges 35’ x 195’ with 9’ draft
  - travel 2 wide x 3 long
- Design speed 5 miles/hour
- Design water elevation is 50-year (2%) flowline

- Equivalent Static Barge Impact $P_B = 2500$ kips
- 100 % impact force - longitudinal to pier
- 50% impact force - transverse to pier
**Pier Design - Longitudinal Force**

- Longitudinal force calculated based on total length (1682')
- Max. of braking and traction force ($F_{BR} = 2,064k$)
- Stiffness (spring constant) of each substructure element determined
- Longitudinal force proportioned based on relative stiffness of each pier
- Longitudinal deflection limited to 1”

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**200’ DPG Design**

- 200’ Brg-to-Brg
- 3 Field sections
- 12’ Deep Girders
- 140” x 1.125” Web
- 26” x 2.5” Flanges
- Composite Design
- Lateral Stability

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**Modified Abutment Details**

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**200’ DPG Design**

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**400’ Truss Design**

- 400’ Brg-to-Brg
- Camelback Warren Type Truss
- 12 Panels (33’-4”)
- Stringers composite with concrete deck
Truss Erection

Geotechnical Design Considerations

- East Approach: Large fills (~50’) placed over Missouri River alluvial deposits
- West Approach - Loess Bluff in excess of 100’
- Bridge over Missouri River - Scour and Deep Foundations

Project Alignments

- Pre-1976 alignment
- Post-1976 alignment

Loess

- Loess (wind blown silt) carried by the wind from the floor of the Missouri Valley
- Loess later carved by erosion into narrow ridges and steep sideslopes

Project Details - Existing Cut 1976

- Concrete Lined Ditch
- Pre-1976 alignment
- Post-1976 alignment
Existing Conditions 2010 - Loess Slope

Sequence of Loess Slope Failures

When the depth of vertical downcutting exceeds $H_c$, tension crack, $y$, results in slab failure (stage 1). Removal of talus and continued downcutting (stage 2) will result in shear failure (stage 3). (Lohnes, et.al.)

Vertical Loess Cut - Design

- Observational Approach
  - Measured existing residual angle
  - Observed failed vertical faces in areas where surface water was not controlled

- Two most important factors affecting performance of loess cut slopes in loess
  - Gradation (if silty, can be cut near vertical)
  - Moisture Content

Loess Slope based on Wedge Theory

Loess Slope - Construction

Soil Nail Wall - Why it was needed.

New RR alignment

Conflict 1: 30 in. Gas lines

Conflict 2: Adj. Property

Conflict 3: Adj. US34 ROW
Soil Nail Wall - Preliminary Design

- Residual Slope Angle 1.3H:1V (38 degrees)
- Loess Cut Configuration

Soil Nail Wall Construction

- 85,000 SF of wall, 2125 ln ft x 40’ avg. height
- 4x4 ft to 6x6 ft nail spacing
- 25 to 50 ft, 6 inch diameter nails, installed at 15 degrees
- Max. allowable nail force of 1.7 kips/ft
- Nails installed using air rotary and conventional augers
- 7 in thick permanent facing, 1 inch of insulation

Soil Nail Wall Cross Section

Soil Nail Wall Construction

- Temporary Shotcrete facing with strip drains

Soil Nail Wall Construction

- Permanent Shotcrete facing with embedded insulation layer

Summary - Design Challenges/Solutions

- Bridge
  - Barge impact on river piers
  - Longitudinal force distribution
  - Soil-structure interaction analysis
  - 200-ft. steel deck plate girder span
  - 400-ft. camelback warren truss

- Geotechnical
  - Large loess cut-west approach, used near vertical benched slope/soil nail wall to provide grade transitions within ROW
  - 50 ft embankments on soft alluvial soils, no special construction methods required
  - Bridge foundations, used 60 to 140 ft long, rock socketed drilled shafts and driven piles