This project includes replacement of the historic vertical-lift span Meridian Bridge (U.S. Highway 81 over the Missouri River) that connects Yankton, South Dakota with rural Cedar County, Nebraska. The new 76-foot wide, 1,590-foot long steel plate girder structure required the use of drilled shaft foundations that extended into the underlying Carlile Shale formation. Designers indicated that an individual drilled shaft capacity of about 2,000 kips will be needed at each of the river piers, and based on the subsurface information, 8 to 10-foot diameter drilled shafts extending to about 115-foot would be required for support of the bridge structure. An Osterberg Load Test was performed on one test shaft to determine the ultimate skin friction and end bearing of the Carlile Shale. This paper describes the design and installation of the load test drilled shaft as well as presents the load test results as compared with predicted design values determined using FHWA methods.

**Existing Bridge Details**

The Meridian Bridge built in 1924 was the first permanent crossing of the Missouri River. The 1,668-foot long, double decked, steel truss vertical lift bridge was constructed for the cost of $1.1 million. The bridge decks were configured to carry highway traffic on the top chord and rail traffic on the lower chord. However, the bridge was never used for rail traffic and the lower chord was converted to highway traffic in 1953. The bridge has kept its original configuration of seven steel river spans (see Figure 2) and will be used as a pedestrian crossing once the new bridge is completed.
Proposed Bridge Details
The new “Discovery” Bridge (see Figure 3) will extend from a high, 55-foot terrace at the north bank of the Missouri River (Yankton, SD) to broad Missouri River floodplain on the south (Nebraska) side. The 1,590-foot, 76-foot wide, four lane bridge is being constructed by Jensen Construction of Des Moines, Iowa and is expected to be completed by the end of 2008. The cost of the new bridge will be about $24 million. The alignment of the new bridge will be 2 blocks west of the existing bridge as shown in Figure 4.

Figure 3. New “Discovery” Bridge (artist view)

Figure 4. Proposed Bridge Alignment

Project Geology
The project site is located in an area with two distinct geologic histories. The northern approach will be located in a high bluff (50 ft) area that has aeolian loess mantling glacial till and outwash deposits. The south approach will be located within the Missouri River floodplain comprised of silt and sand alluvium. The glacial and alluvial deposits are in turn underlain by the Cretaceous Age, Carlile Shale.

The Carlile Shale was encountered at depths of 30 to 55 feet below ground surface and is comprised of gray to dark gray, non-calcareous shale, with some calcareous seams. The Carlile Shale has been known to be bentonitic at lower depths. In general, the Carlile Shale ranges in thickness in this region from 250 to 300 feet. A generalized subsurface profile is shown in Figure 5.

Figure 5. Generalized Subsurface Profile

Groundwater was encountered during our subsurface exploration at depths of about 50 feet on the northern bluff and 6 to 10 feet within the floodplain.

Samples of the Carlile Shale were obtained using NX-coring procedures. The core samples were sealed and returned to the laboratory for unconfined compression testing per American Society of Testing Materials (ASTM) D2938. The results of the laboratory testing indicated unconfined compressive (UC) strength of the Carlile Shale to range from 2.4 to 16.6 kips per square foot (ksf), dry densities of 124 to 133 pounds per cubic foot (pcf) and moisture contents of 19 to 24 percent. A summary of the UC test results with respect to elevation is shown in Figure 6.

Figure 6. Unconfined Compression Strength vs. Elevation, Carlile Shale
Predicted Drilled Shaft Capacities

The capacities of the drilled shafts were predicted using the design methodology outlined in the FHWA publication “Drilled Shafts: Construction Procedures and Design Methods”. The ultimate skin friction of the drilled shafts in shale was estimated using the “α” method for cohesive materials on samples that had unconfined compression results of less than 10 ksf and using the Intermediate GeoMaterial (IGM) method with an interface friction (concrete/shale) of 30 degrees for remaining samples. It should be noted that none of the shale samples had unconfined compressive strengths great enough to be considered rock. The above noted design methods are described in the FHWA document and will not be covered herein.

Based on the above noted design methods, we predicted ultimate skin friction ($f_{\text{max}}$) values of 0.66 to 2.32 ksf between elevations 1130 and 1100 feet and 1.22 and 6.2 ksf between elevations 1100 to 1065 feet. For design purposes, we used an ultimate skin friction of 1.8 ksf (elev. 1130 to 1100 feet) and 2.4 ksf for below elevation 1100 feet.

It appears that the ultimate skin friction computed for shale defined as cohesive materials are lower than those computed for shale classified as IGM in the zones. The distribution of ultimate skin friction values and provided design values are shown in Figure 7.

Based on the FHWA methods, we predicted an ultimate end bearing ranging from 11 to 46 ksf in the Carlile Shale. For design purposes, we used an ultimate end bearing resistance of 42 ksf. Drilled shaft lengths were estimated based on the following parameters for the Carlile Shale:

- **Allowable Side Friction (Factor of Safety = 2.0):**
  - Elev. > 1100 ft: 0.9 ksf
  - Elev. < 1100 ft: 1.2 ksf

- **Allowable End Bearing (Factor of Safety = 2.5)**
  - Elev. <1100 ft: 17 ksf

For the working load of 2,000 kips plus the shaft weight and a top of shale elevation of 1130 feet, we estimated that an 10-foot diameter shaft with a 52-foot shale socket or a 8-foot diameter shaft with a 75-foot shale socket (which extended below the borings) would be required to develop the capacity.

![Figure 7. Estimated Ultimate $f_{\text{max}}$ Values](image-url)
Drilled Shaft Load Test Program

The 8-foot diameter, 115.1-foot, load test drilled shaft was constructed by Jensen Construction between May 23 and June 20, 2007 on the Nebraska side of the project within the Missouri River floodplain. The subsurface conditions at the drilled shaft load test site were comprised of about 32 feet of silty sand/sand underlain by the Carlile Shale. Groundwater was encountered at about 6 feet below the ground surface at the time of the load test shaft construction.

The drilled shaft was constructed primarily in the wet, using water as the fluid and a steel casing driven to about 35 feet below the existing grade (i.e., shale penetration of about 3 feet). Prior to final cleaning, the sidewalls of the shaft were brushed with a wire cable connected to the auger. After cleaning the base with a clean out bucket, the reinforcing cage and O-cell assembly was inserted into the shaft and temporarily supported. As shown in Figure 8, the reinforcing cage was not kept in alignment and repairs to the cross hole sonic logging tubes and cage had to be performed prior to final placement. Concrete was placed by tremie methods through a 12-inch O.D. pipe into the base of the shaft until the concrete reached the cutoff elevation.

Load testing of the drilled shaft was performed by LOADTEST, Inc. The configuration of the test shaft included an O-Cell located at 29.3 feet above the shaft tip and three levels of strain gauges to assess side shear load transfer. The configuration of the test shaft is shown in Figure 9.

The load was applied to the test shaft using the Quick Load Test Method for Individual Piles (ASTM 1143), holding each successive load increment constant for eight minutes by manually adjusting the O-cell pressure. The data logger recorded the strain gauge readings, LVDT's and other instruments on a 30 second interval. The load test was performed by pressurizing the O-cell to assess the combined end and lower side shear resistance as well as the upper side shear resistance. At a maximum (bi-directional) loading of 4,522 kips the load test was halted due to an increase in the rate of displacement in the upper side shear zone.
In order to assess the side shear resistance of the test shaft, loads along the shaft are estimated using the strain gage data and the shaft stiffness (AE). Using this method, the distribution of load along the shaft can be determined for each load increment as shown in Figure 10.

![Figure 10. Strain Gage Load Distribution Curves](image)

Using the load distribution curves, the net unit shear values for each segment and end bearing can be determined for the maximum loading. As indicated by extrapolation of the load distribution curve, it appears that none of the applied load was transferred to end bearing at the noted displacement. A summary of the average net unit side shear values are presented in Table 1.

<table>
<thead>
<tr>
<th>Load Transfer Zone</th>
<th>Elevation (ft)</th>
<th>Ultimate Skin Friction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top of Shaft to SG Level 3</td>
<td>1156.86 to 1121.69 ft</td>
<td>0.1 ksf</td>
</tr>
<tr>
<td>SG Level 3 to SG Level 2</td>
<td>1118.02 to 1091.02 ft</td>
<td>2.4 ksf</td>
</tr>
<tr>
<td>SG Level 2 to O-cell</td>
<td>1091.02 to 1071.02 ft</td>
<td>4.9 ksf</td>
</tr>
<tr>
<td>O-cell to SG Level 1</td>
<td>1071.02 to 1051.02 ft</td>
<td>6.4 ksf</td>
</tr>
<tr>
<td>SG Level 1 to Tip Elevation</td>
<td>1051.02 to 1041.76 ft</td>
<td>5.2 ksf</td>
</tr>
<tr>
<td>End Bearing</td>
<td>1041.76 ft</td>
<td>No load transferred to tip</td>
</tr>
</tbody>
</table>

Figure 11 presents an equivalent top-loaded load settlement curve. For a top loading of 2,000 kips (working load), the adjusted data indicates that the shaft would settle less than 0.1 inches.

At completion of the load testing, the O-cell installed in the shaft was post-grouted since the test shaft was planned to be used as a production shaft.

![Figure 11. Equivalent Top Load-Movement Curve](image)

**Findings**

The ultimate skin friction measured during load testing was about 30 to 200 percent greater than the provided design values, which were estimated using FHWA methods and the laboratory test results (see Figure 12).

![Figure 12. Estimated vs. Load Test fmax Values](image)
Using the information obtained from the load test program, the final drilled shaft lengths were estimated based on the following parameters for the Carlile Shale:

**Allowable Side Friction (Factor of Safety = 2.0):**
- Elev. > 1118 ft: 0.05 ksf
- Elev. 1100-1118 ft: 1.2 ksf
- Elev. < 1100 ft: 2.45 ksf

**Allowable End Bearing (Factor of Safety = 2.5)**
- Elev. <1100 ft: 17 ksf

For the working load of 2,000 kips plus the shaft weight and a top of shale elevation of 1130 feet, we estimated that an 8-foot diameter shaft with a 41-foot shale socket would be required to develop the capacity.

As shown in Figure 13, it appears that the skin friction values predicted using the methods outlined for cohesive materials is underestimated in shale and to a lesser degree when classified as an IGM.

**Conclusions**

Based on the subsurface information and laboratory test results, the ultimate skin friction values predicted using FHWA methods were 50 to 75% of measured full scale load testing ultimate skin friction values. Thus, it appears that, in this case, the cost of full scale load testing would be offset by the project cost and time savings.

For this load test program, the cost incurred totaled about $200,000 for the shaft construction and load test. Since the improved skin friction values reduced the length of each shaft by an average of 22 feet, we saved the cost of 528 lineal feet of shale socket, which resulted in a foundation cost and time savings totaling about $500,000.

Thus, we have demonstrated that the skin friction in shale can sometimes be under-predicted by the FHWA methods and that full scale load testing can provide better design information that, in turn, can be used to reduce project costs.

**References**


![Figure 13. Estimated vs. Load Test f_{max} Values](image-url)