

DRILLED SHAFT FOUNDATIONS FOR THE kcICON MISSOURI RIVER BRIDGE

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The design-build delivery system is being used to erect the landmark Christopher Bond Bridge as part of the kcICON project. The cable-stayed bridge is currently under construction and will span the Missouri River on I-29/35 in Kansas City, Missouri. The design-build process for the foundation work on this project has been successfully implemented as a result of cooperative efforts between the foundation designers and the constructors to develop a constructible design.

INTRODUCTION

The Christopher Bond Bridge is a cable-stayed structure that will span the Missouri River, replacing the existing Paseo Bridge on I-29/35, located immediately to the west of the new bridge. This paper provides a summary of the design and construction of the drilled shaft foundations supporting the main pylon and the approach piers.

Various configurations were considered for the bridge layout, which is illustrated on Figure 1. The bridge consists of seven spans totaling 1715 feet, with a 550 ft main span and a 452 ft back span. The main pylon foundation is located in the river near the navigation channel, and is the only pier subject to significant vessel impact forces. Although this layout required marine construction for the largest foundation unit, design efficiency was realized because the design of the robust main pylon support was not significantly altered by the consideration of barge impact as would be the case for more

lightly loaded approach pier foundations. The approach piers utilized individual drilled shafts under each column.

The main pylon for the bridge is supported on large diameter cast-in-place drilled shaft foundations that are embedded into shale bedrock. The shafts beneath the approach bents are embedded into bedrock at Bents 1 through 4, and founded in overburden soils above the deeper rock at Bent 5. Base grouting was utilized on drilled shafts at the approach piers.

SUBSURFACE CONDITIONS

The overburden soils are predominantly loose to medium-dense, poorly graded, rounded sand with gravel. Some thin, low-plasticity clay layers were also encountered. Cobbles and boulders also exist, particularly in the 15 to 20 feet above the top of bedrock. The soil overburden is approximately 55-feet thick.

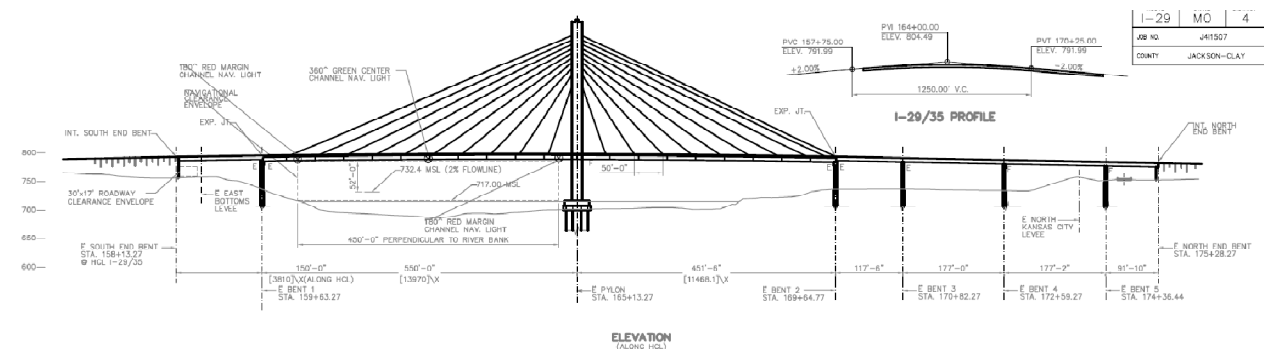


Figure 1: kcICON River Bridge Profile.

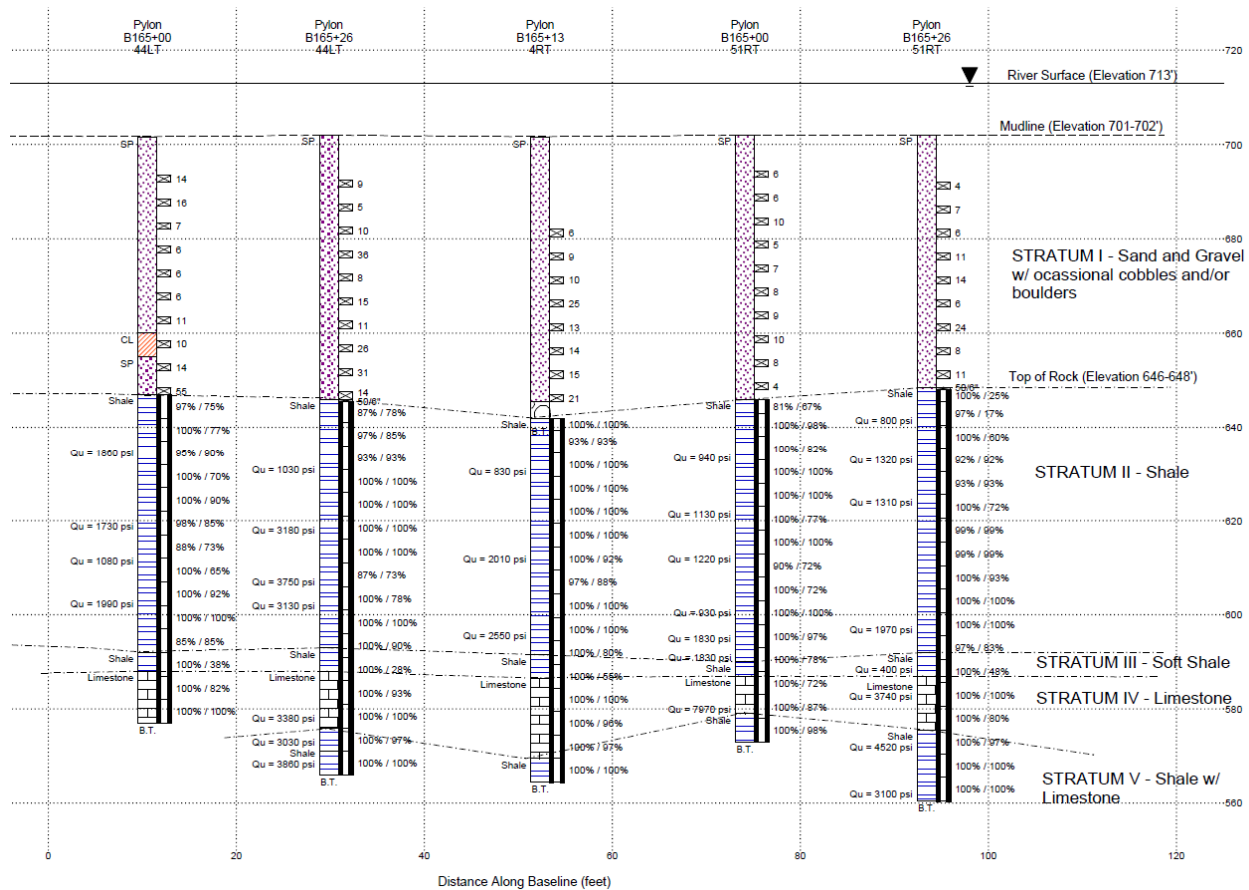


Figure 2: Subsurface Profile at Main Pylon.

The majority of the bedrock encountered was shale with lesser amounts of limestone encountered at depth. The bedrock is from the Pleasanton Group of Pennsylvanian Age. Bedrock was cored to a maximum depth of 88 feet below the bedrock surface in a boring at the location of the Main Pylon. The strike of the bedrock is northeast and the dip is down to the northwest at 1 to 2 degrees. Jointing is commonly orientated to the northeast and northwest and is nearly vertical.

Five borings were drilled at the location of the Main Pylon during the design phase. A subsurface profile containing those five borings is shown in Figure 2. The numerical values presented in the soil overburden are the uncorrected Standard Penetration Test blowcounts. The values presented in the bedrock are unconfined compressive strength of core samples (denoted Q_u), percent recovery, and Rock Quality Designation (RQD). All of the drilled shafts socketed into bedrock terminate in the layer labeled Stratum II in Figure 2.

The top of rock was about Elevation 648 to 646 feet at the location of the Main Pylon, and weathered in the upper 3 to 5 feet. Limestone laminations were observed beneath Elevation 607 feet. Occasional 1 to 2 inch coal seams were observed below Elevation 616 feet. Most runs had full recovery and, excluding the weathered portion near the surface of the bedrock, the majority of the RQD's measured in the bearing stratum exceed 70 percent, with only two exceptions that measured 60 and 65 percent. Some pieces of the shale recovered during excavation of the test shaft are shown in Figure 3. Unconfined compression results in the bearing stratum ranged from 800 to 3750 psi.

A highly weathered, relatively soft shale layer was observed in all fifteen borings where coring was performed. This six foot soft shale layer exists beneath the bearing stratum between approximate elevations 592 to 586 feet. While the recovery was high, the RQD's were very low, as were the unconfined compression strength test results. In addition, some layers within this zone oozed hydrocarbons.

Conditions were similar for the approach bents. However, the top of rock elevation varied, declining towards the north (Bents 2 through 5). Boulders were also present atop the rock, and more prevalent at locations where top of rock elevation was lower. Some of the boulders were observed to be hard granitic rock, likely as a result of glacial deposition.

Liquefaction of the overburden was not a concern based on the geographic location of the project. In Kansas City, considering a probability of exceedance of 2 percent in 50 years, the peak ground acceleration of bedrock is estimated to be about 0.05g's based on the 2008 USGS maps. Also, the extreme degree of scour predicted at the river pier locations was considered (e.g., any resistance provided by the soil was neglected).

All of the soil overburden was neglected for design of the main pylon foundations to account for scour. Based on the hydraulic report for the project, various depths of scour were considered at Bents 1 through 4, generally extending to the shale bedrock in the river and shallower to the north. Bent 5 is located on the opposite side of a federal flood control levee, so scour at that location is not predicted to occur.



Figure 3: Shale recovered from drilling bucket at tip of test shaft.

MAIN PYLON

The main pylon foundation consists of a single footing, approximately 116 feet by 48 feet in plan, supported by a group of eight drilled shafts, as shown in Figure 4. The drilled shafts were constructed with a permanent steel casing extending into the top of the shale bedrock, with a 10.5-foot diameter socket extending into the

shale formation. Each shaft is designed to provide a required axial resistance of approximately 10,000 kips.

This single large pile cap with multiple shafts was selected to provide a robust and reliable foundation which is not sensitive to scour and which has strength that substantially exceeds potential vessel impact or lateral load demand. The permanent steel casings provide additional strength, ductility, and confinement for the bending stresses in the drilled shafts and facilitated construction by providing a stable environment in which to construct the rock socket. The multiple shafts provide reliability as a redundant foundation system.

Although somewhat smaller diameter shafts could have satisfied the flexural strength demands, the large diameter shafts were selected to the necessary axial resistance within the rock of stratum II and thereby avoid the softer deeper strata. In addition, the use of fewer larger shafts provided a minimum footprint dimension so that the required navigation clearance could be maintained with the minimum span length.

A 6-foot diameter test shaft was constructed and tested in the center of the main pylon, as depicted in Figure 4. The test shaft was proportioned to evaluate the design values of side shear and end bearing in the rock socket using the O-cell load test method. The test shaft rock socket was drilled using similar tools, installation techniques, and inspection techniques as was used on the production shafts.

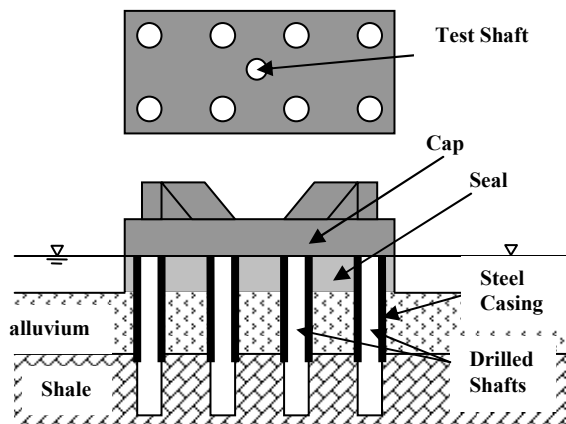


Figure 4: Schematic Diagram of Main Pylon Foundation

Slake Durability Testing

Deterioration of the shale in the presence of various drilling fluids was a concern during the design. In order to evaluate potential deterioration, slake durability tests (ASTM D 4644) were performed on core samples of rock from the bearing formation. The intent of the testing was to evaluate the relative susceptibility of the shale to deteriorate under agitated conditions similar to drilled shaft construction in the presence of drilling fluids.

One test was conducted on shale exposed to Missouri River water and the second using a polymer drilling fluid. The polymer was POLY-BORE™ Borehole Stabilizing Agent mixed with Missouri River water. The POLY-BORE™ modified slurry was mixed per instructions provided by the manufacturer at the target density and viscosity. Soda ash was used to achieve the proper pH in the mixing water.

The shale bedrock used in the tests was from a depth of 109 feet to 110 feet (Elevation 628.5 feet to 627.5 feet) from a boring located near the west side of Bent 3. Photographs of this core are shown in Figure 5. Testing started the day immediately following recovery of the cores.



Figure 5: Photographs of the tested rock in the core box.

The slake durability test provides an index for rocks that will weather and degrade rapidly by measurement of the physical breakdown of a rock sample after a series of wet/dry cycles with mechanical agitation by tumbling in a drum. The Slake Durability Index (second cycle), $I_d(2)$, corresponds to the quantity retained on a No. 10 sieve after two cycles of wetting and agitation; a higher value represents a more durable specimen (100% is the upper-bound). Both of the specimens were visually classified as Type II, indicating that the retained specimen consists

of large and small fragments as shown in the photographs provided in Figures 6 and 7.

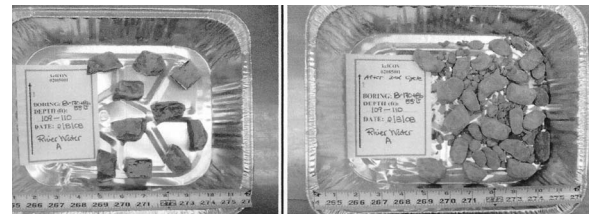


Figure 6: River Water – Oven dried before 1st cycle and after 2nd cycle.



Figure 7: Polymer Slurry – Oven dried before 1st cycle and after 2nd cycle.

The results of the slake durability tests are summarized below in Table 1. Rocks with $I_d(2) < 60\%$ are considered prone to rapid degradation and may indicate a susceptibility to degradation and formation of “smear zones” when the borehole wall is exposed to water (Turner, 2006). Both tests in water and polymer slurry had values of $I_d(2) > 60\%$.

Also included in Table 1 are the results of an index durability rating system known as the Durability Rating Based on Shear Strength Loss, DR_s (Richardson and Wiles, 1990). Based on this rating system, the shale is classified as “intermediate” when tested in river water and “hard, more durable” when tested in polymer slurry. While this method of evaluation is more applicable to shale used as compacted fill in embankment construction, the difference in DR_s between the two tests is another indicator of the relative performance of shale in water and polymer slurry.

Table 1: Slake Durability Test Results.

Sample	Natural Moisture Content	Slake Durability Index		Durability Rating Based on Shear Strength Loss	
	(%)	Type	$I_d(2)$ (%)	Type	DR_s
River Water	8.3	II	72.2	Intermediate	61.9
Polymer Slurry	8.3	II	98.2	Hard, more durable	78.6

The results of the slake durability tests suggest that, while the shale may not be prone to rapid deterioration in the presence of river water, the polymer slurry appears to preserve the integrity of the shale better than river water alone. Based on the results, the shale was not estimated to experience significant decomposition if polymer slurry is used to drill the rock socket.

Development of Design and Installation Plan for Main Pylon Foundations

Because of the time required to install large diameter shafts in the river, the exposure time of the excavation and potential effects on performance were a critical consideration for the design-build team. An installation and testing plan was developed which included the following components:

1. Install a temporary casing through water and very soft overburden soils
2. Drill through granular (and boulder-filled) overburden using mineral slurry
3. Seat permanent casing into shale
4. Exchange the mineral slurry with a polymer drilling fluid
5. Excavate the rock socket under polymer to the target tip elevation
6. Scarify the sidewall surface to remove any slick or decomposed material
7. Perform final cleanout of the shaft base
8. Place reinforcement and concrete

As a means to demonstrate the installation plan and to provide site-specific measurement of axial performance under the as-built conditions, a load test shaft was designed and constructed following the specific details of the plan. In addition, the exposure time of the excavation used in the test shaft was intentionally extended to 4 days in order to simulate the worst possible conditions to be expected for construction of a production shaft.

The O-cell provides the only practical method for load testing drilled shafts with such large axial resistance; however, the verification of axial resistance was complicated because the base resistance of a production shaft was anticipated to exceed the side resistance available as a reaction. Therefore, a scaled prototype test shaft 6 feet in diameter was selected in order to more closely balance the side and base resistance at the target tip elevation.

The load test shaft was constructed and tested in a non-production location in the center of the main pylon foundation. This was considered to be the optimal location and very representative of the rock at the production locations. Additionally, no additional borings to evaluate conditions at the test location were required.

Excavation of the shafts would be completed using digging buckets and augers, as illustrated in Figure 8. Similar tools were used for the load test shaft and the production shafts. The “back-scratcher” used to scarify the sidewall of the rock socket is illustrated in Figure 9.

Clean-out of the shaft base was completed using cleanout buckets and a simple hydraulic pump, as illustrated in Figure 10. Video inspection of the shaft base was conducted with a mini Shaft Inspection Device (mini-SID) for the load test shaft and the first two production shafts at the pylon. The mini-SID was also used on the first two shafts at the approach piers to verify that the construction procedures used were achieving the desired level of base cleaning.

Test Shaft Installation. Excavation of the test shaft began on April 25, 2008 following the installation plan outlined previously. The top of bedrock was encountered at approximately Elevation 646.5 ft. The 76-inch I.D. permanent casing was placed the following day and seated into the shale approximately 4 feet. After the casing was seated, the bentonite slurry was exchanged for polymer slurry.

The rock-socket excavation was started on April 29 and completed the morning of April 30. The “back-scratching” of the socket with a wire brush tool to remove loosened material, final clean-out (cleanout bucket and dredge pump), and video inspection of the base were performed on May 1. Inspection with the mini-SID determined that the shaft base was reasonably clean after the use of the hydraulic pump. Sonic caliper testing was performed the following day, and determined that the actual as-built dimensions were very close to the planned dimensions. Concrete was successfully placed by tremie pipe on May 5, approximately 96 hours after completion of the “back-scratching” operation. The O-cell test was performed the morning of May 9.



Figure 8: 6-ft diameter drilling bucket and auger used for excavation of test shaft.



Figure 9: "Back-scratcher" for Preparation of Rock-Socket Sidewall



Figure 10: Test shaft clean-out tools: one-eyed bucket and hydraulic pump

The O-cell test was performed with the test shaft tip at Elevation 611.7 feet. The bottom of the permanent casing was at Elevation 641.8, resulting in a 30.1-ft rock socket. Three 26-inch diameter O-cells (approximately 3600 kip per O-cell) were used to provide the required bi-directional loading. Figure 11 shows the O-cells being prepared for installation. The O-cells were set 20 inches above the tip with four levels of strain gages above the O-cells to evaluate the distribution of side shear along the shaft. Two diametrically opposed sister bar vibrating wire strain gages were installed at each level, and the mean strain was used. Very little variation was observed between the two gauges at each level. During concrete placement, concrete was pumped between the base of the O-cells and the bottom of the excavation to provide a bearing surface for the O-cells. The top of concrete was at Elevation 694.3 feet.



Figure 11: Three O-cells attached top and bottom to 2-in thick steel plates.

Load Test Results. The O-cell test indicated that at the maximum upward displacement of 0.2 to 0.3 inches, the shaft mobilized a unit side resistance of 12 ksf in the four feet of the socket immediately below the tip of the casing, and 16 ksf in the remainder of the socket. A unit base resistance of 275 ksf was mobilized at a downward displacement of 1.5 inches. The test was successful in that the shaft was installed in a manner similar to that anticipated for the production shafts without any complications, the measured data appeared to be reliable, and the test mobilized values of side shear and end bearing that approached the geotechnical strength limit condition. It is worth noting that

the unit side friction values were in a similar range to those developed by drilled shafts in other western Missouri shale formations as referenced in Miller (2003), even though those shafts were not in the Pleasanton Formation.

Evaluation of Base Resistance for Design of Production Shafts. Displacements required to mobilize the unit base resistance values are related to shaft diameter. The measured unit base resistance was obtained at a displacement of 1.5 inches, or approximately 2% of the diameter of the test shaft. For a 10.5ft diameter shaft, similar values would be anticipated at a displacement approximately 2.5 inches (2 percent of the 10.5-ft diameter) in the production shaft. Although typical design guidelines for geotechnical strength are based on a larger displacement value of 5% of diameter, the measured unit base resistance was considered an upper bound for design purposes because of the large creep movements which were observed at this pressure.

The maximum unit end bearing at the test location was correlated with a location within the shale bearing formation which was determined to have unconfined compressive strengths in the rock of approximately 2000 psi (288 ksf). Thus the nominal unit end bearing was approximately equal to the unconfined compressive strength, and therefore $q_b = N_b q_u$ with $N_b = 1$.

In order to accommodate potential variation of unconfined compressive strength across the footprint of the main pylon foundation, a lower value of 165 ksf (0.6 times the maximum tested value) for base resistance was used for design of the production shafts. Values of maximum base resistance at Bents 1 through 4 were correlated with typical values of unconfined compressive strength at those respective locations.

Evaluation of Side Resistance for Design of Production Shafts. Although the maximum side shear occurred at a displacement smaller than the displacement at which the maximum base resistance was mobilized, the test data showed no evidence of strain softening and therefore strain compatibility was not a factor in combining side and base resistance. This tendency is likely related to the dilatancy at the shaft/rock interface. It is also noted that load test measurements in similar (even softer) shale

materials from nearby projects referenced previously as reported by Miller (2003) showed ductile behavior at significantly larger displacements. Therefore, the maximum unit side resistance mobilized in the load test was considered as the maximum available side resistance for design in rock of similar strength characteristics.

The load test results indicate that an maximum unit side resistance of 12 ksf was appropriate for the upper four feet of shale beneath the tip of the permanent casing, and 16 ksf was appropriate for the remainder of the Stratum II shale within 30 feet below the tip of the permanent casing. This assessment is consistent with the slightly lower rock core compressive strengths recorded in the upper part of Stratum II as compared to the remainder of the stratum. The side resistance values mobilized at the test shaft location were appropriate for the Main Pylon production shaft locations.

It is useful to compare the maximum unit side resistance with the values used for preliminary design, based on the relationship between side resistance, f_s and $\sqrt{q_u}$. The method contained in Turner (2006) originally dating back to Horvath & Kenney (1979), and normalized to dimensionless units is:

$$f_s = Cp_a \sqrt{q_u / p_a}$$

where p_a is equal to atmospheric pressure and C is a constant.

The original expression of Horvath & Kenney (1979), as contained in the current edition of the FHWA Drilled Shaft Manual, included a constant, C , equal to 0.65. The most recent regression analysis of available load test data is reported by Kulhawy et al. (2005) and demonstrates that the mean value of the coefficient C is approximately equal to 1.0. A lower bound value of $C = 0.63$ was shown to encompass 90 percent of the load test results.

The average unconfined compressive strength of the rock along the length of the test shaft socket was around 1200 psi (170 ksf), and thus the measured unit side shear of 16 ksf would correlate to a value of $C = 0.86$. This value compares well with the mean and lower bound values cited above.

A final point is worth noting relating to the measured unconfined compressive strengths, q_u , of the rock at the main pylon. Pre-bid borings indicated significantly lower values, on the order of ½ or less, than were obtained during the final geotechnical investigation. Since the writers had no record or direct knowledge of the care and handling of the pre-bid cores, an explanation of the variation between pre-bid and post-award compressive strength is not available. However, this issue highlights the difficulties in predicting performance of deep foundations in shale formations and underscores the need for high-quality geotechnical data and site-specific drilled shaft load test information.

Axial Design of Production Shafts

The anticipated service load capacity for each of the eight drilled shafts supporting the Pylon is approximately 9,700 kips for corner shafts and 9,500 kips for non-corner shafts. Design of the drilled shafts is based on the use of a socket into the shale of Stratum II above Elevation 600, with a factor of safety of 2.0 on side and 3.0 on base resistance. The higher factor of safety on base resistance is included because of the greater influence of potential variability in rock strength and the presence of softer shale strata at greater depth.

Based on the design values outlined in the previous section, a 20-foot rock-socket below the casing provided the required resistance to support the design loads with the target factors of safety. The side resistance above the casing tip was ignored due to possible scour, weathering within the shallow zone and the effect of casing installation. The preliminary design included a 30-foot rock socket below the casing, so the results of the load test allowed the length of the shaft sockets to be reduced by one-third.

Construction of Production Shafts

Construction of the production shafts at the main pylon was completed without significant incident. The use of the permanent casing is considered helpful in avoiding any significant issues with bottom cleanout or entrapped debris. The crosshole sonic logs indicated that all eight shafts were free of detectable defects.

The contractor was able to complete the shafts within the specified time limit established by the

load test program. In general, shafts were completed within 2 to 3 days after “back-scratching”. However, a broken tool resulted in a delay of about 1-1/2 days on one particular shaft. Because of the 4 day window allowed by the proven performance of the load test shaft, this shaft was completed without issues of noncompliance.

APPROACH BENTS

The River Bridge includes five approach bents; Bent 1 is south of the Missouri River and Bents 2 through 5 are north of the Missouri River. Each bent includes five columns supported on individual drilled shafts. The preliminary foundation concept at the time of the bid included drilled shafts permanently cased through the overburden and socketed into shale bedrock, similar to the Main Pylon foundations. The permanent casing was to be installed in an over-sized hole excavated under bentonite slurry, thereby significantly reducing or eliminating any side resistance that may develop in the approximately 100 feet of soil overburden.

The pre-bid scheme was modified to achieve improved economy, avoid installation risks related to boulders, and avoid potential negative impacts on nearby federal flood control levees related to deep permanent casing. The selected foundation scheme at Bents 1 through 4 includes a permanent casing at the surface (10 to 20 feet) and uncased drilled shafts extending 4 ft into shale bedrock (considered only as a “seating socket”).

Although the load test at the main pylon provided information on the performance of shafts in the shale at this site, the shafts for the approach foundations were considered sufficiently far removed that the design was completed using a higher factor of safety of at least 2.5 in side resistance and 3.0 in base resistance. All shafts for Bents 1 through 4 were base-grouted for quality assurance purposes. Because the foundation construction did not include an over-sized hole, the contribution of side resistance below the scour elevation was included in the design of the shafts. The potential for soil piping vertically adjacent to the shafts during floods was also overcome by eliminating the oversized hole associated with permanent casing. The cost of a full-length steel casing is also removed.

The foundations at Bent 5 are similar to Bents 1 through 4, but the shafts bear in sand above the shale bedrock. During the subsurface exploration at Bent 5 (after award of the contract) a large cobble and boulder field was encountered on the order of 20-feet thick, directly above bedrock and approximately 110 feet beneath the surface. Due to risks relating to installation of drilled shafts through these boulders, an alternative approach was used.

The five shafts at Bent 5 were designed using base-grouting to enhance the base resistance of the shaft such that the required axial resistance the boulder field. Two of the five shafts that were base-grouted were installed with sister-bar strain gauges so an indication of axial capacity could be obtained during the grouting operation.

Three independent grouting circuits were installed via the six available crosshole sonic logging tubes and used for all approach bent shafts. Base grouting was completed without incident, although two of the shafts which were constructed within areas where boulders were encountered required larger than normal grout volume.

CONCLUSIONS

The Main Pylon foundations exhibit several instances in which constructability issues played a critical role in the design. Slake durability testing was conducted after award of the contract to evaluate the effect of drilling fluid on the decomposition of the shale bedrock. Installation methods, namely the “back-scratch” tool, were instituted to ensure adequate bond between the concrete and the rock socket. A load test was successfully designed and executed to replicate the maximum allowable exposure time for the rock socket. Results from the load test were used to significantly reduce the size of the drilled shafts at the Main Pylon.

The drilled shaft foundations at approach Bents 1 through 4 were significantly reduced in length, taking advantage of side resistance developed in the soil above bedrock. The installation technique was also modified to alleviate concerns of a potential negative impact on the adjacent flood control levees. The foundations at Bent 5 were modified to include enhancement of the base resistance by base-grouting, thereby allowing for shorter shafts and avoiding difficult drilling through a boulder field.

Design/build contract delivery proved to be a successful approach for the design and construction of the kclCON River Bridge foundations. Because the design and construction team worked collaboratively, a constructible preliminary design was proposed which provided flexibility and efficiency. The consideration of construction issues was an integral component of the drilled shaft load test program. The load test shaft was used to test the installation plan and to measure the performance of foundations constructed with a plan that addressed critical issues related to the effect of construction on performance. At the same time, specifications were implemented through the test program which allowed flexibility in the construction approach. As the final stratigraphy was identified and the risks related to boulders and deeper, poor quality rock became evident, the design-build team was able to modify the design and achieve economy and efficiency.

Besides the designers and constructors, the owner represents the critical third member of the design-build team. In this case MoDOT engineers played a critical role in working with the design-builder in a timely manner to allow innovation and flexibility while ensuring that standards of accountability and performance requirements are achieved.

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