Design and Construction Challenges at the kcICON Bridge

The Christopher Bond Bridge is a cable-stayed structure that will span the Missouri River and create a new gateway into Kansas City, replacing the existing Paseo Bridge. Also known as kcICON, the name of a larger area project, the bridge will open in 2011. The bridge has 7 spans totaling 1,715 ft, with a 550 ft main span and a 452 ft back span. The main pylon foundation is in the river, near the navigation channel, and subjected to significant vessel impact forces. This layout was efficient because the robust main pylon support could be readily proportioned to resist lateral forces from barge impact, and the more lightly-loaded approach pier foundations could be located away from potential vessel collision. The trade-off was the fact that marine construction was required for the largest foundation unit. The approach piers utilized individual drilled shafts under each column.

Support for the main pylon of the bridge consists of large-diameter, cast-in-place drilled shaft foundations 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 used on drilled shafts at the approach piers.

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

The majority of the bedrock was shale with lesser amounts of limestone at depth. The bedrock is from the Pleasanton Group of Pennsylvanian Age and weathered in the upper 3 to 5 ft. The shale included some limestone laminations and occasional 1 to 2 in coal seams. Most core runs had full recovery and, excluding the weathered portion near the surface of the bedrock, the majority of the rock quality designations (RQD) measured in the bearing stratum exceeded 70%, with only two exceptions that measured 60 and 65%. Unconfined compression results in the bearing stratum ranged from 800 to 3750 psi.

A highly-weathered, relatively soft shale layer appeared in all 15 borings with coring. The top of this 6-ft-thick soft shale layer lies about 30 ft beneath the shaft. While the recovery in this zone was high, the RQDs were very low, as were the unconfined compression strength test results. 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 present atop the rock and more prevalent at locations where top-of-rock elevation was lower. Some boulders were hard granitic rock, likely as a result of glacial deposition.

All of the soil overburden was neglected during design of the main pylon foundations because of scour. For the approach structures, the scour generally extended to the shale bedrock at Bents 1 and 2 in the river, and shallower to the north at Bents 3 and 4. Bent 5 is on the opposite side of a Federal flood control levee, so scour at that location is not anticipated.

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Main Pylon Foundation

The main pylon foundation consists of a single footing, approximately 116 ft by 48 ft in plan, supported by a group of 8 drilled shafts (Figure 1). The drilled shafts are constructed with a permanent steel casing extending into the top of the shale bedrock, with a 10.5-ft-diameter socket extending into the shale formation. Each shaft design provides a required axial resistance of approximately 10,000 kips.

This single, large pile cap with multiple shafts provides a robust and reliable foundation that is not sensitive to scour,

and that 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, designers selected the large-diameter shaft to the necessary axial resistance within the rock of Stratum II and thereby avoided the softer deeper strata. Using fewer larger shafts also provided a minimum footprint dimension so the required navigation clearance could be maintained with the minimum span length.
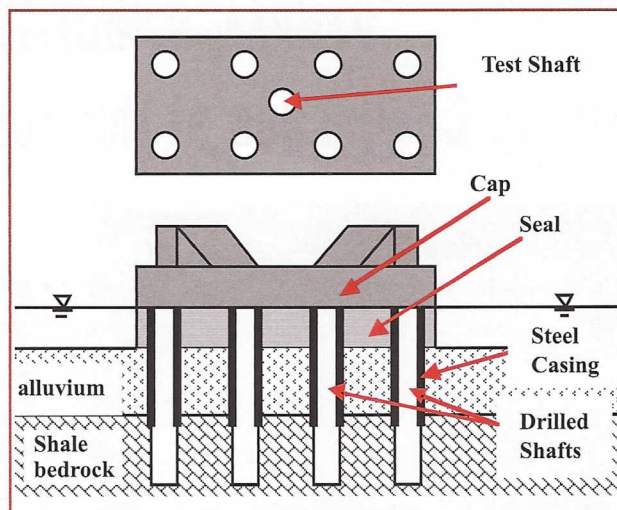

Figure 1. Schematic diagram of main pylon foundation

bearing formation were tested for slake durability (ASTM D 4644). These tests provide a measure of the relative susceptibility of the shale to deterioration under agitated conditions similar to drilled shaft construction in the presence of drilling fluids. The tests were performed on samples

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

Figure 2. Slake durability test result

A 6-ft-diameter test shaft was tested in the center of the main pylon to evaluate the design values of side shear and end bearing in the rock socket using the O-cell load test method. Although 10.5-ft-diameter production shafts were planned, the somewhat smaller-diameter test shaft provided a balance between the anticipated base and side resistance in an O-cell test with load cells placed at the base of the shaft. The test shaft rock socket was drilled using similar tools, installation and inspection techniques as was used on the production shafts.

The design team was concerned about deterioration of the shale in the presence of various drilling fluids. To evaluate potential deterioration, core samples of rock from the

of shale exposed to both Missouri River water and polymer drilling fluid. The polymer was POLY-BORE™ Borehole Stabilizing Agent mixed per the manufacturer's instructions at the target density and viscosity. Soda ash was used to achieve the proper pH in the mixing water.

The test results suggest that the polymer slurry appears to preserve the integrity of the shale better than river water alone. The shale was not expected to experience significant decomposition if polymer slurry was used to drill the rock socket.

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 plan details. The exposure time of the excavation was intentionally extended to four days to simulate the worst possible conditions for construction of a production shaft.

Axial Performance

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

The excavation tools used for the load test shaft replicated the methods used for the production shafts. These tools included digging buckets and augers to excavate the rock socket, followed by the use of a "back-scratcher" to scarify the sidewall of the rock socket prior to final clean-out with a hydraulic pump. 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 contractor also used the mini-SID on the first two shafts at the approach piers to verify that the procedures achieved the desired level of base cleaning.

The O-cell test was conducted on a 30-ft-deep rock socket, with sonic caliper testing to indicate the actual as-build dimensions. Three 26-in-diameter O-cells (approximately 3,600 kip per O-cell) provided the required bi-directional loading. The O-cells were set 20 in above the tip, with 4 levels of strain gages above to evaluate the distribution of side shear along the shaft.

The O-cell test indicated that at the maximum upward displacement of 0.2 to 0.3 in, the shaft mobilized a unit side resistance of 12 ksf in the 4 ft 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 in.
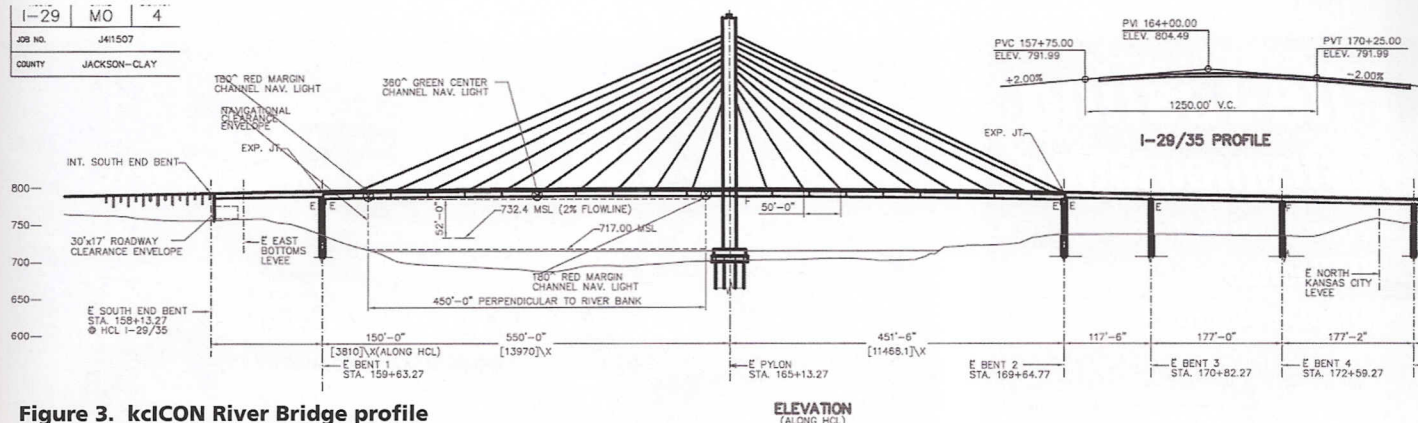
Figure 3. kcICON River Bridge profile

The test was successful in that the shaft was installed in a manner similar to the method planned 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.

The displacement required to mobilize the base resistance is typically proportional to shaft diameter, and so the variation in diameter between the test and production shafts must be considered. The measured unit base resistance was obtained at a displacement of 1.5 in, or approximately 2% of the diameter of the test shaft. For a 10.5-ft-diameter shaft, similar values would be anticipated at a displacement approximately 2.5 in (2% 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 taken at a more conservative displacement for design purposes because of the creep movements observed at this pressure and because of the large-shaft diameter. The shale bearing formation at the test location had unconfined compressive strengths in the rock near the base of the test shaft of approximately 2,000 psi (288 ksf). Therefore the nominal base resistance (at a displacement of 2% of the shaft diameter) was approximately equal to the unconfined compressive strength. The shape of the load versus displacement relationship suggests that greater base resistance was likely available at larger displacement.

To allow for the 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. The values of maximum base resistance at Bents 1 through 4 were correlated with typical values of unconfined compressive strength at those locations.

Although the maximum side resistance 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. Therefore strain compatibility was not a factor in combining side and base resistance. This tendency is likely related to the dilation at the shaft/rock interface because of the rough interface surface. Load test measurements in similar (even softer) shale materials from nearby projects referenced by Miller (2003) showed ductile behavior at significantly larger displacements. Thus, the designers considered the maximum unit side resistance mobilized in the load test as the maximum available side resistance for design in rock of similar strength characteristics.

The load test results showed that a maximum unit side resistance of 12 ksf was appropriate for the upper 4 ft of shale beneath the tip of the permanent casing, and 16 ksf was appropriate for the remainder of the Stratum II shale within 30 ft 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.

The average unconfined compressive strength, q_u, of the rock along the length of the test shaft socket was around 1200 psi (170 ksf), and thus the measured unit side resistance, f_s, of 16 ksf correlates to $f_s = 0.86\sqrt{q_u}$, where f_s and q_u are in units of atmospheres of pressure.

The service load capacity for the 8 drilled shafts supporting the pylon is approximately 9,700 kips for corner shafts and 9,500 kips for non-corner shafts. The drilled shafts design is based on the use of a socket into the shale of Stratum II, 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 above, a 20 ft 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.

Each of the five approach bents includes five columns supported on individual drilled shafts. The foundation scheme at Bents 1 through 4 includes a permanent casing at the surface (10 to 20 ft) and uncased drilled shafts extending 4 ft into shale bedrock (considered as a "seating socket"). 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 the contract award), the rock was found to be deeper and overlain by a large cobble and boulder field on the order of 20 ft thick, directly above bedrock and approximately 110 ft beneath the surface. These conditions presented a significant risk of difficulties during construction, and so the plan to bear on rock was modified to accommodate the conditions. To provide the necessary axial resistance in the soils above the bedrock, the design for the 5 shafts at Bent 5 utilized base-grouting to
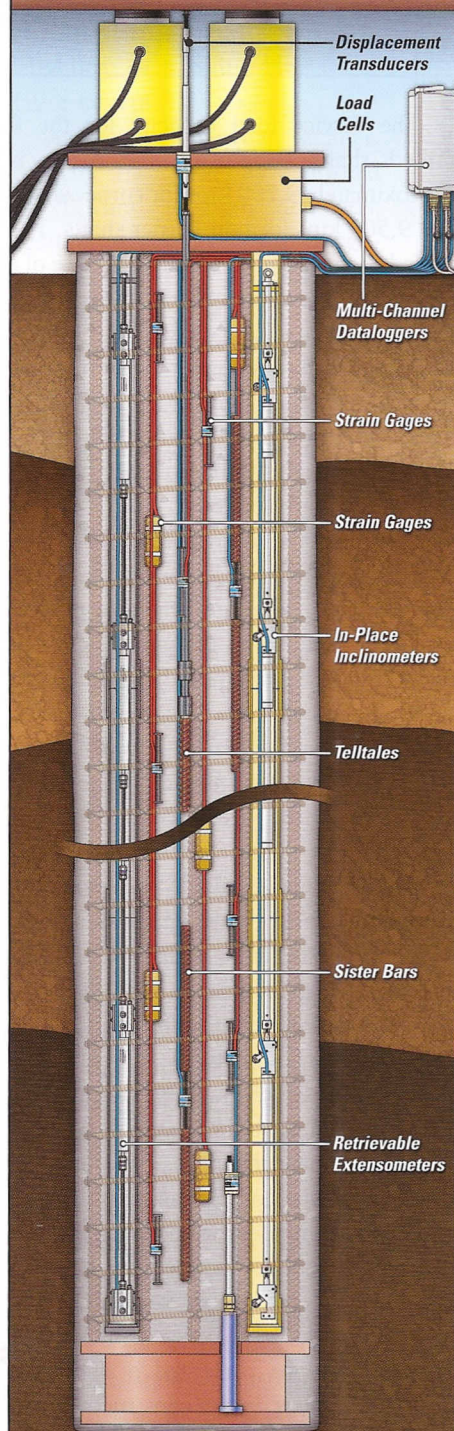
Drill rig and "back-scratcher" tool

enhance the base resistance of the shafts bearing in granular soils. Two of the five base-grouted shafts were installed with sister-bar strain gauges so that an indication of axial side resistance could be observed during the grouting operation as grout pressure applied force to the base of the shaft.

The base grouting was accomplished via the crosshole sonic logging tubes, connected across the base of the shaft with a sleeve-port tube to form three independent grouting circuits. Several of the drilled shafts at Bent 5 encountered boulders near the base of the excavation, and two of the shafts constructed within areas with boulders required significantly more grout than the others.

In summary, the design-build system worked well, encouraging collaboration between construction and design resulting in foundations for the kcICON bridge that provided reliability and met the goals of cost-effectiveness and scheduling. The main pylon foundations incorporated a reasonable exposure limitation on the shale bedrock thanks to a load testing program, which addressed construction and design objectives. Using polymer slurry and the

"back-scratch" tool ensured an adequate bond between the concrete and the rock socket. The base cleaning methods were developed using downhole inspection tools and verified with the load tests. The design of the drilled shafts for the approach structure incorporated base grouting to minimize the construction risks associated with deep bedrock overlain by boulders at some locations.

Acknowledgments