Evaluation of Base Grouted Drilled Shafts at the Audubon Bridge

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Abstract: The load test program for the Audubon Bridge project provides a comparison of the performance of base grouted and ungrouted shafts on large diameter foundations in sand. Comparison of several field load tests at the two pylon foundations indicate that the base grouting operations more than doubled the available base resistance, and provided for consistent load-deflection behavior in sand with varying properties. Base grouting of all the production shafts also provided a verification of axial resistance. The Audubon Bridge will be a new Mississippi River crossing near St. Francisville, Louisiana, and the two main pylons of the cable-stayed span are each supported by a 3 x 7 group of 2.29m (7.5ft) diameter drilled shafts. The drilled shafts bear in an alluvial sand and gravel formation at an elevation about 61 m (200ft) below the surface. Underlying this formation was a clay with less potential base resistance for deep foundations.

Introduction

The John James Audubon Bridge design-build project will connect US Highway 61 in St. Francisville, West Feliciana Parish with LA Highway 10 in New Roads, Pointe Coupee Parish. This crossing will be accomplished via a 19.3km (12mile) stretch of roadway that includes a 4 lane cabled stayed bridge across the Mississippi River as well as several approach bridges. The main bridge structure crossing the Mississippi River is a five span bridge with the main center span length of 482.5m (1583feet).

A load testing program performed at the start of construction included tests of base grouted shafts on both sides of the river for both the main piers, the high approach structures, and a single test of an ungrouted shaft. Subsequent testing was performed on seven additional test and production shafts at or near the main piers for verification testing. All tests were performed using the embedded bi-directional O-cell load testing method. Initial testing indicated that the base resistance exceeded the upward reaction force (side shear resistance + buoyant weight); subsequent tests had a reaction frame installed at the top of shaft, which was engaged when needed to supplement the upward reaction force. Comparisons of the magnitude of end bearing resistance from the grouted and ungrouted shafts indicated that the base grouting operations more than doubled the available base resistance and proved very effective in enhancing the axial performance of the drilled shafts.

Project Background

The main bridge structure crossing the Mississippi River will be a five span bridge with the main span length of 482.5m (1583 feet), side spans of 195.5m (641.5ft), and transition spans (to the high approach structures at either side) of 48.8m (160 feet). The main span will be slightly longer than the Arthur Ravenel Jr. Bridge in South Carolina, making it the longest-span cable-stayed bridge in America. Two planes of 68 stays (126 stays total) will radiate from the two reinforced concrete H-type pylons approximately 157m (515ft) above low water level in a semi-fan arrangement to the girders.

Each of these main piers consists of 21 shafts (3 x 7 grouping) with center to center spacing 3 times the diameter of the permanently cased portion, i.e. 7.32m (24ft). A drilled shaft foundation with partial-height permanent casings and tip grouting was selected as the foundation elements for both the main piers (1W and 1E), and were chosen for reasons of performance, constructability, and economy. The partial depth casing provides improved performance over the uncased drilled shafts when subjected to vessel impact loads. The shafts were approximately 61m (200ft) deep, and were subjected to approximately 21.3m (70ft) of scour. The permanently cased sections of the main piers had a nominal diameter of 2.44m (8 ft), while the uncased section below had a nominal diameter of 2.29m (7.5ft). Ultimate resistances were in excess of 53 MN (6,000tons) per shaft.

The anchor piers (2W and 2E), the transition piers (3W and 3E), and a portion of the West high approach structure (inside the levee) were similar, but without permanent casing as the lateral loads and bending stresses were less severe. These piers had two shafts each, approximately 36.6m (120ft) deep, and nominal diameters of 2.50m (8.2 ft), except shafts at Pier 3W were 2.29m (7.5ft) diameter, and all had large center to center spacing between shafts. Ultimate resistances of these shafts were in excess of 36MN (4000tons) per shaft. Similar improvements in base resistance to the main piers were measured on these piers as well.

Group effects were considered for the sustained average loads on the entire group of shafts for the main pier foundations. The primary concern for group effects was related to the stresses transferred to the underlying clay stratum; however, group effects were found to not control the shaft design due to the relatively high strength and incompressibility of the older Pleistocene-age clay soils.

Soils

The site is located along the eastern boundary of the Mississippi River alluvial plain located to the west of the Mississippi River. The shafts were embedded in the alluvial deposits primarily consisting of sands and gravels, and were tipped above the underlying Pleistocene-age clay soils. A total of 8 test borings (2 pre-bid and 6 post) were made in and around the main piers. Figure 1 summarizes the soil stratigraphy and pertinent geotechnical properties.

The alluvial sediments were predominantly granular soils associated with the meandering Mississippi River. These Holocene-age fluvial deposits include point bar and deeper channel lag deposits. The deposits of point bar sands result from meandering of the major rivers and streams in the area, which form cross-bedded layers of granular soil that tend to be coarser with increasing depth. During movements of the streambed, "cut-offs" were formed resulting in oxbow lakes and a natural levee system that left remnant fine-grained infill sediments at shallow depths from flooding. The channel lag deposits consist of layers of sand and gravel from the trough of the ancient channel, and these tend to be more frequent with increased depth in the alluvial sequence. The result of this depositional environment is a highly variable soil profile both in plan location and vertical stratigraphy.

Underlying the alluvial sediment, below elevations of approximately -61m to -67m (-200ft to -220ft), are older Pleistocene-age soils of the Prairie Formation which were primarily composed of marine deposited clay. These older soils are overconsolidated due to desiccation during the most recent period of glaciation, when the sea level was perhaps 400 feet lower than at present, resulting in high shear strengths and preconsolidation pressures, as well as producing a secondary structure of fissures and/or slickensides. Silt pockets and lenses within the clay are also common and may be associated with small scale infilling of fissures within the clay. Shear strengths from UU and UC tests ranged from approximately 200kPa to 300kPa (2tsf to 3tsf), and exhibited OCR's ranged from approximately 2 to 3.

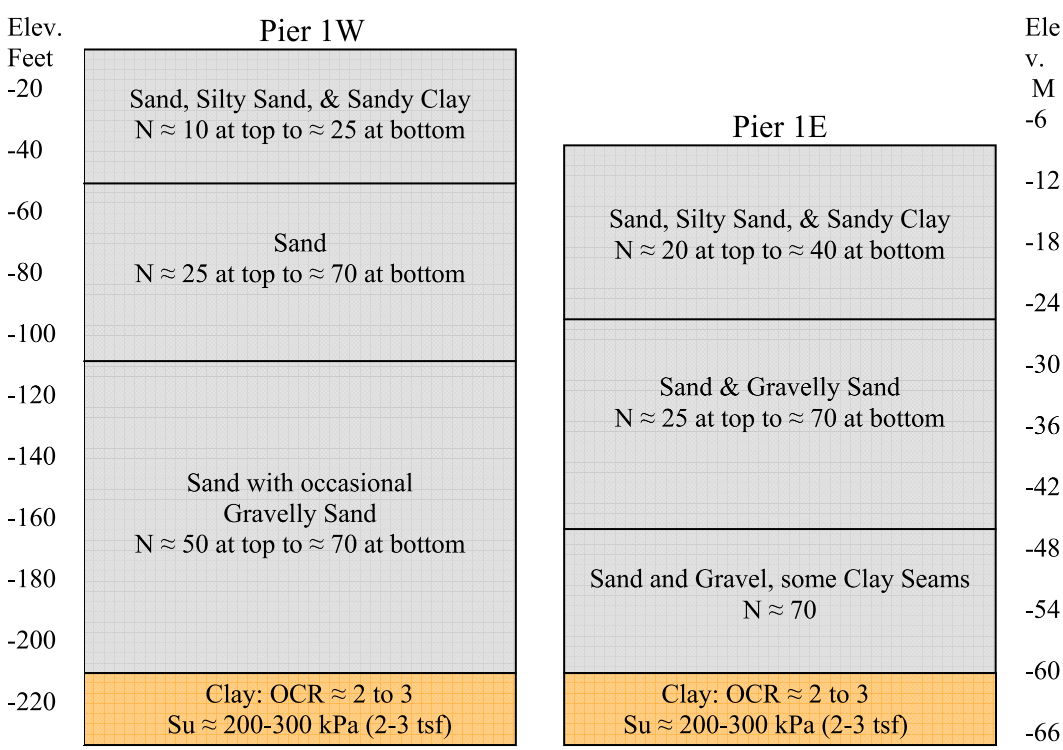

Figure 1. Soil Stratigraphy and Pertinent Geotechnical Properties

Drilled Shaft Construction

The shafts were constructed using a casing oscillator and full depth temporary casing. A permanent casing was first installed with the use of a vibro-hammer, then the temporary casing was inserted through the permanent casing. The temporary casing was advanced a minimum of approximately 0.9m to 1.2m (3ft to 4 ft) ahead of the shaft excavation at all times during excavation. Excavation was performed predominantly with down-hole grabs at the higher elevations where cohesive soils were frequent, and by use of a water jet-ring and airlift for the majority of the shaft. Figure 2 is a picture of the oscillator and excavation process.

This method of construction has advantages in terms of foundation performance because only water is needed as a drilling fluid. Since bentonite slurry is not necessary, contamination of the side wall and loss of bond strength in side shear is not an issue. By controlled advancement of the casing simultaneously with the shaft excavation, reduction of in situ soil stress and loosening of sands is also reduced. The full depth temporary casing also provides the time necessary to consider and implement options when occasional excavation difficulties are encountered.

Figure 2. Shaft Excavation Using Oscillator, Grabs, and Air-lift.

Base Grouting

Base grouting involves injecting grout at high pressures at the base of a cured shaft through a system of tubes that are incorporated into the rebar cage. Other than the grout plumbing and a grout distribution mechanism tied into the rebar cage, the construction of a grouted shaft is identical to that of an ungrouted shaft. Similar to bi-directional load testing, the application of grout pressure across the base of the shaft mobilizes base resistance acting against the side shear. Base grouting more reliably accounted for the contribution of the end bearing component of resistance, resulting in a lower allowable safety factor of 2.

Increased confining pressure associated with base grouting increased the resistance of the granular bearing materials. The grout pressure also compensated for any stress relief resulting from construction operations as well as any loosened residual material at the base of the shaft which may have not been completely removed. The base grouting process pre-compressed the shaft and allowed the base resistance component to be developed at smaller displacements. The grout pressure was released immediately after grouting; however, the densification of the soils surrounding the tip had been accomplished, and the base resistance then operated along a much stiffer re-load response.

The sleeve-port system (also known as tube-a-machetes) shown in Figure 3 was used for grouting. Four individual grouting circuits were formed by joining pairs of CSL (cross-hole sonic logging) tubes below the shaft (forming a “U-tube” shaped arrangement), and the grout was injected through holes in the tube portion at the bottom of the shaft which was covered by sections of tightly fitting rubber tubes. A steel base plate was used above the sleeve-ports to separate the grout distribution system from the drilled shaft concrete and to aid in distribution of the grout across the base of the shaft. This type of system has become common practice in the United States, Dapp et al. (2006). A gravel pack (several inches thick) was placed at the bottom of several shaft excavations where isolated clay stringers were encountered in the bearing formation. This proved to be an effective mechanism to encapsulate the remaining small clay balls and aide in the grout distribution across the shaft tip during grouting.

Figure 3. Sleeve-Port with Top Plate Grouting Mechanism.

The base grouting criteria used on this project include the following:

(A) *Grout Pressure*: Achieving the specified grout pressures was the most definitive way to verify the base resistance on a shaft by shaft basis. The sequential pressure grouting of redundant grout circuits (four U-tube arrangements total) helped to ensure grout distribution across the base of the shaft. All the load tested shafts were grouted to pressures of 5200kPa (750 psi) except shaft 11 at Pier 1W was grouted to a pressure of 3800kPA (550psi) after staged grouting had been conducted. Many production shafts of Pier 1W were grouted to pressures in excess of 6900kPa (1000psi). The grout pressures at Pier 1W were predominantly only limited by the maximum capacity of the grout pump being used, the initial pump used was capable of 5200kPa (750 psi) and was replaced with one capable of 6900kPa (1000psi). The grout pressure at Pier 1E was predominantly dependent upon the pressure at which the grout hydro-fractured the soil (presumably along the gravel lenses/layers).

(B) *Grout Volume*: Net grout volumes (grout delivered to the shaft tip) at the Pier 1W shafts were relatively small, approximately 230L to 770L (0.3 to 1 cubic yards), as the Pier 1W bearing soils consisted of predominantly fine sands, with occasional gravel or course sand pieces. Net grout volumes at the Pier 1E shafts were relatively large, approximately 2300L (3 cubic yards), as the bearing soils at Pier 1E consisted predominantly of gravelly sands with isolated clay stringers. A neat cement grout with water-to-cement ratios ranging from 0.60 to 0.45 was used. When a shaft continued to take grout volume without building to the desired grout pressure, typical at Pier 1E, the water to cement ratio was reduced. If grout take continued without building to the desired pressure, then stage grouting was employed. Staged grouting consists of wash out the grout circuit(s) that was utilized for the first stage with water, and then repeating the grouting procedure at some later time (second stage of grouting). The first test shafts at the main piers were stage grouted as both took appreciable grout volume during the initial grouting session without obtaining the design grout pressure, Shaft 11 at Pier 1W took 2500L (90cf) net, and Shaft 11 at Pier 1E took 1100L (40cf) net. This may have been due to avenues for the grout to escape (such as gravel lenses/layers), or due to the soil at the base of the shaft being disturbed by the installation process. Shaft 11 at Pier 1W was grouted in a total of 3 stages, and Shaft 11 at Pier 1E was grouted in a total of 2 stages.

(C) *Shaft Uplift*: Shaft uplift movement was limited to 6.35mm (¼ inch) uplift during grouting, as measured at the top of shaft. While some shafts had appreciable uplift, particularly the first test shafts, the majority of the shafts had essentially no measurable uplift at the top of shaft.

Load Test Program

A load testing program performed at the start of construction included tests of base grouted shafts on both sides of the river at the center shaft location of the main piers (Pier 1W Shaft 11, and Pier 1E Shaft 11); a single test of an ungrouted shaft (T3 near Pier 1W) was made to asses the improved effects of base grouting by direct comparison. Subsequent testing was performed on seven additional production shafts and nearby non-production locations at the main piers for verification testing, as well as two shafts at the high approach structures.

All tests were performed using the embedded bi-directional O-cell load testing method, single level (near the shaft tip) using four O-cells per level. The initial tests conducted at Pier 1W Shaft 11, and Pier 1E Shaft 11 indicated that the base resistance exceeded the upward reaction force available (side shear resistance + buoyant weight of the shaft), and the base resistance could not be fully developed at these two test shafts. Subsequent tests had a reaction frame installed at the top of shaft with a hydraulic jack which could be engaged near the end of a test, when needed, to supplement the upward reaction force, shown in Figure 4. Test shaft T2 (near Pier 1W) was the next shaft tested, and was the only shaft to utilize the installed reaction frame, as shafts produced subsequent to these test shafts had markedly improved side shear resistance.

Figure 4. Top of Shaft Reaction Frame to Supplement Upward Resistance against Base Resistance during O-cell Testing.

Comparison of Base Grouting

The test data from the main pier locations shown in Figure 5 illustrate the base resistance vs. displacement from O-cell tests near the base of a single ungrouted shaft (Shaft T3 near Pier 1W; the solid heavy line) and of nine grouted shafts. The base grouted shafts were all grouted to similar pressures of around 5200kPa (750psi) and show consistent load-deflection behavior even though the relative density and composition of the soil near the base of these shafts was known to vary. Comparisons of the magnitude of end bearing resistance from the grouted and ungrouted shafts indicated that the base grouting operations more than doubled the available base resistance and proved very effective in enhancing the axial performance of the drilled shafts.

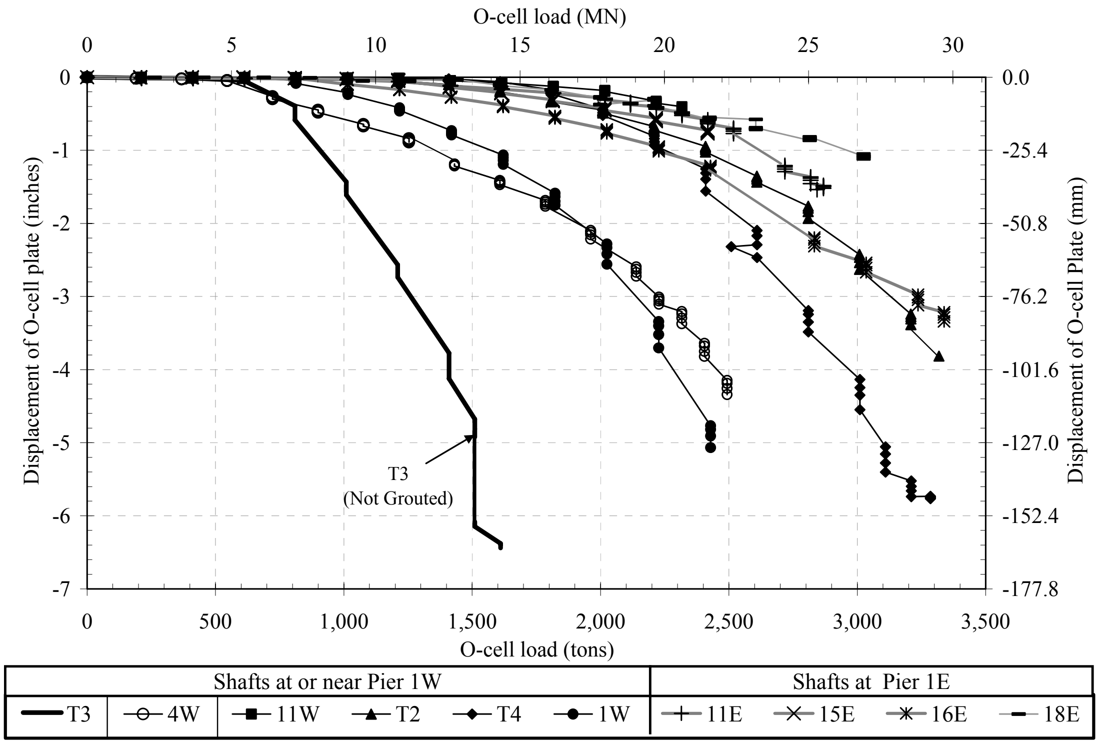

Figure 5. Base Resistance vs. Displacement for the Main Pier Load Test Shafts.

The soil at the base of one of the grouted shafts (11W) was known to have been disturbed by the installation process; this shaft required a substantially larger volume of grout to complete the grouting, which had to be accomplished in 3 stages. Although the end bearing could only be mobilized to a displacement of approximately 0.5% of the diameter, the data indicates that the grouting operations successfully mitigated the installation problems at the base of this shaft, as the measured base resistance trended similar to or in excess of the other tested shafts at Pier 1W which required a much smaller volume of grout to meet the target pressure.

The 2.29m (7.5ft) diameter temporary casing at Shaft 4 of Pier 1 had become stuck during construction. This casing was left in place, and a 2.13m (7ft) diameter pipe pile was inserted and driven open ended to a tip elevation that was 1.52m (5ft) deeper than the other shafts at Pier 1W, excavated to near tip, and then concreted. The resulting Cast-In Steel Shell (CISS) was then tip grouted to approximately 5200kPa (750psi). Shaft 1 of Pier 1W had some construction difficulty, and was extended 5 feet deeper than the other shafts of Pier 1W in an effort to gain total capacity.

The measured responses were stiffer than the original design estimates made using the procedure recommended in Mullins, et al. (2006) at small displacements (approximately 1.2% of the shaft diameter or less), but exhibited less strain hardening at larger displacements. The parameters are modified, below, to better correlate the grouted end bearing response to that observed. The equation shows the Tip Capacity Multiplier (TCM) as a function of the Grout Pressure Index (GPI) and the displacement which is expressed as a percentage of the shaft diameter (%D).

Mullins, et al. (2006): $TCM = 0.713\cdot(GPI)\cdot(\%D)^{0.364} + (\%D)/[0.4\cdot(\%D)+3]$

Modified to fit Data: $TCM = 0.713\cdot(GPI)\cdot(\%D)^{0.200} + (\%D)/[4.0\cdot(\%D)+6]$

The GPI is the sustained grout pressure divided by the ungrouted unit base resistance at 5% Diameter displacement. The ungrouted base resistance is calculated with the correlation by Reese and O'Neill (1988), as outlined in the FHWA drilled shaft manual (FHWA-IF-99-025). The grouted unit base resistance at any given displacement (expressed as % Diameter) is then the TCM (for that displacement) multiplied by the ungrouted unit base resistance at 5% Diameter displacement.

The measured unit base resistances are shown in Figure 6; the measured ungrouted resistance (T3 near Pier 1W) is also shown. Included in this figure are the original calculations for both the ungrouted tip (Reese and O'Neill, 1988) and grouted tip (Mullins et al., 2006), as well as calculations made using the modified parameters presented. The measured ungrouted response (T3 of Pier 1W) exhibited the same trend of a stiffer response than estimates made using the procedure recommended in Reese and O'Neill (1988) at small displacements (less than approximately 1% Diameter), but exhibited less strain hardening at larger displacements. This is the source for the same observed trend in grouted tip comparisons, as the grouted capacity estimates are based upon the ungrouted capacity.

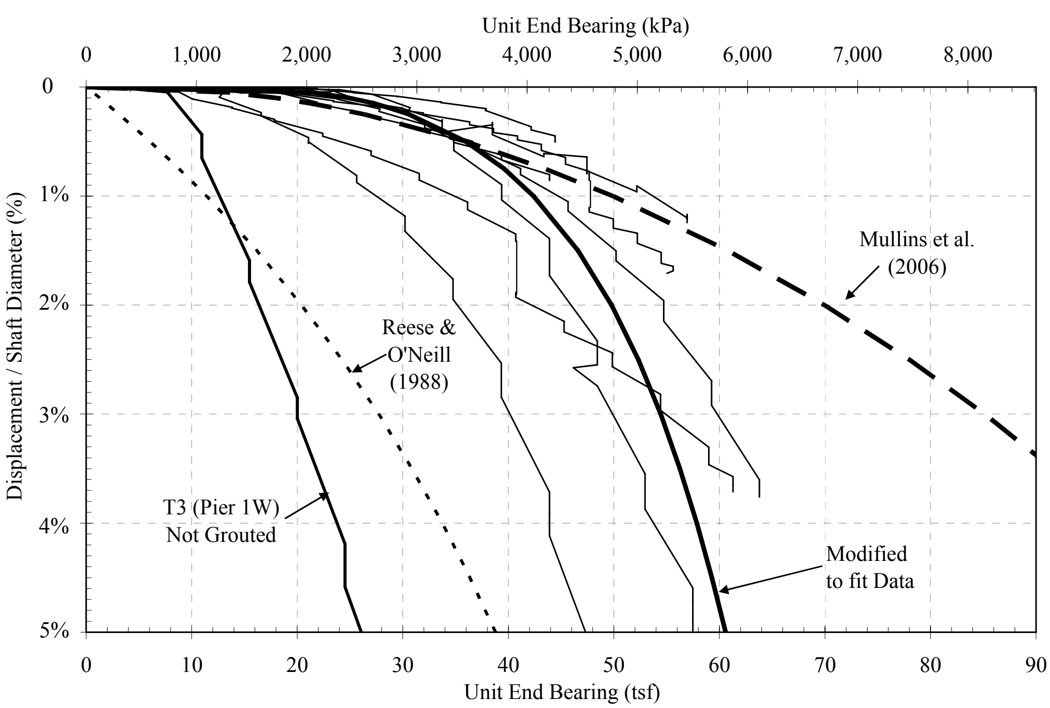

Figure 6. Unit Base Resistance.

Conclusions

A load testing program was conducted on nine grouted shafts, all grouted to similar pressures of around 5200kPa (750 psi), and a single ungrouted near Pier 1W. The shafts show consistent load-deflection behavior even though the relative density and composition of the sand near the base of these shafts was known to vary. Comparisons of the magnitude of end bearing resistance from the grouted and ungrouted shafts indicated that the base grouting operations more than doubled the available base resistance and proved very effective in enhancing the axial performance of the drilled shafts.

Both the measured ungrouted response and grouted response exhibited the same trend of a stiffer response than estimates at small displacements (less than approximately 1.2% Diameter), but exhibited significantly less strain hardening at larger displacements. The estimate of ungrouted base capacity is based upon the estimate of the ungrouted capacity (along with grout pressure), and therefore reflected the same trends when compared to the measured response. Modified parameters are presented to better correlate the procedure by Mullins et al. (2006) to the response measured at this project.

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