ST. CROIX BRIDGE, MINNESOTA: THE INFLUENCE OF CONSTRUCTION ON THE AXIAL RESISTANCE OF DRILLED SHAFT FOUNDATIONS IN WEAK SANDSTONE

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The St. Croix River Crossing project on the Minnesota-Wisconsin border has been in the process of planning and public debate for two decades, with several changes in alignment. During the course of the earlier planning process in 1995, a drilled shaft load test was performed to obtain preliminary design information. There were some difficulties in maintaining stability within the weak sandstone bearing formation, and the measured axial resistance was relatively low. Subsequent experience by the Minnesota Department of Transportation with similar weak sandstones on the I-35W project suggests that improved construction techniques using modern drilling fluids may offer opportunities for improved performance. During final design in 2012 a second load test program was performed of a full scale test shaft constructed using polymer drilling fluids. The stability of the socket excavation in the weak sandstone appeared to be greatly improved and the measured axial resistance significantly higher.

This case history illustrates the excellent performance that drilled shafts can provide in even relatively weak sandstone bedrock as well as the profound influence on axial resistance associated with the construction techniques used. The improved resistance values obtained by the new load test with more recent construction technology allowed the foundation scheme to be substantially optimized with respect to the number and size of required drilled shafts, reducing the size, cost, and risk of the coffercells and footings.

Introduction

The Minnesota Department of Transportation (MnDOT) has awarded the first phase of a bridge construction project that will cross the St. Croix River between Oak Park Heights, MN and St Joseph, WI. The first phase construction contract involves building the river pier substructures including groups of drilled shafts beneath each tower leg. A general location of the bridge project is provided in Figure 1.

Prior to award of the construction contract, two separate design-phase axial load tests were performed on non-production drilled shafts with rock sockets. Both tests were static, bottom-up tests using the Osterberg Cell (O-cell). The first test was performed in 1995 when final design and construction was thought to be imminent. After years of public debate and literally, an act of Congress, design of the project was finally restarted and a second load test performed in 2012. The 1995 test was conducted on the Minnesota bank approximately 900 ft southwest of the St. Croix River bank. The 2012 test was conducted in the river about 2,100 ft northeast of the1995 test location. A plan-view of the project indicating the test program locations relative to the bridge alignment is provided in Figure 2.

The axial design recommendations submitted to MnDOT following the two load test programs are significantly different although the tested rock sockets are in similar sandstone bedrock. A significant portion of the differences in the test results and subsequent recommendations are thought to be related to improvements in construction techniques.

Figure 1: General Location of Project.

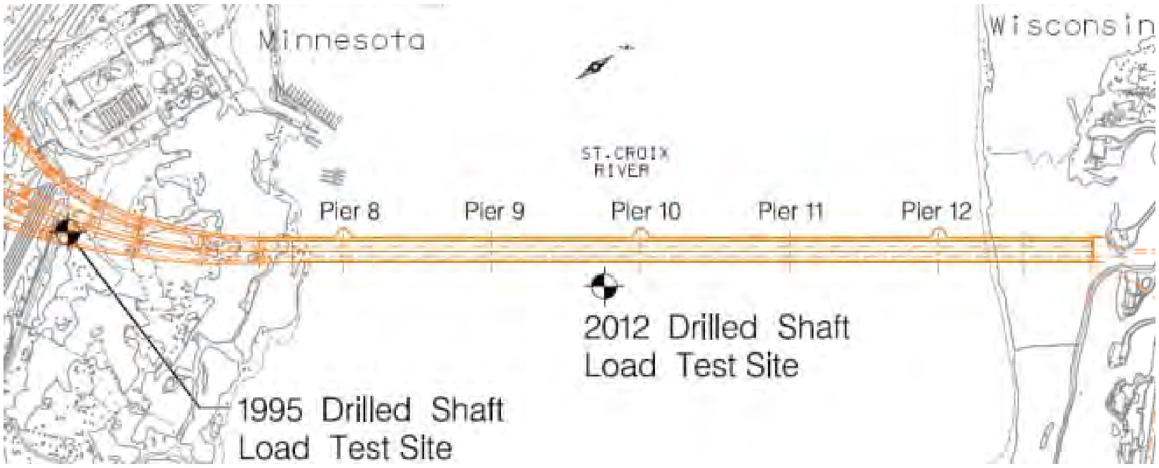

Figure 2: Test Shaft Locations.

Subsurface Conditions

The ground surface elevation and the subsurface conditions above the tested rock sockets at the 1995 test location are different than those at the location of the 2012 test. However, the geologic origin and properties of both sandstone bedrock sockets tested are very similar, if not the same. Both sockets were constructed in sandstone of the Franconia Formation including both the Reno Member and the Mazomanie Member. The bottom 8 ft or 17 percent of the 2012 test also encountered the Tomah Member. The primary difference between the two test sites is the depth to the top of the tested socket. In the 1995 test, the depth from the ground surface to the top of the socket was approximately 149 ft because the test shaft was constructed on land. In the 2012 test, the depth from the ground surface to the top of the socket was approximately 91 ft because the test was constructed in the river.

The observed core drilling results are similar with respect to recovery (REC) and Rock Quality Designation (RQD) and similar equipment was used to perform the coring. Boring T-78 was performed by MnDOT drill crews near the location of the 1995 test. Borings T-504 and T-512 were drilled near the location of the 2012 test and were also performed by MnDOT drill crews. The REC and RQD from these three borings are shown in Figures 3 and 4 for the 1995 site and the 2012 site, respectively.

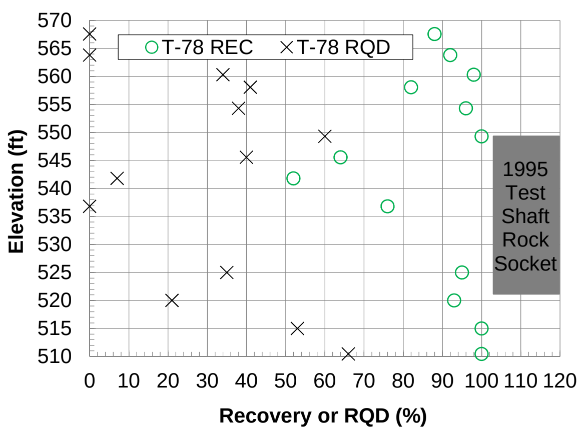

Figure 3: 1995 Test Site Bedrock Core Information.

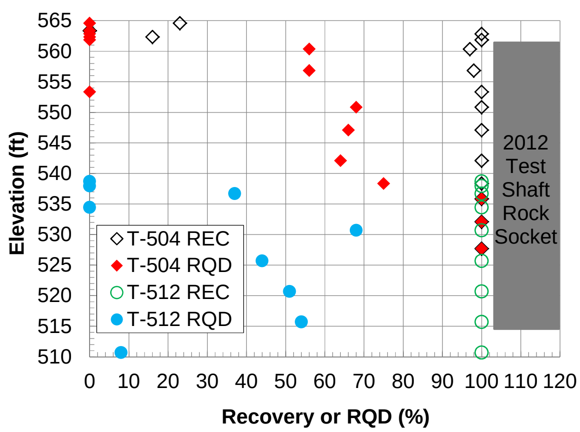

Figure 4: 2012 Test Site Bedrock Core Information.

At the 1995 test site, the REC in the vicinity of the rock socket was between 50 and 100 percent; the RQD was between 0 and 60 percent. At the 2012 test site, the REC in the vicinity of the rock socket was generally around 100 percent; the RQD was between 0 and 100 percent, predominantly in the 40 to 70 range.

The range and variation of unconfined compression tests performed in the laboratory on recovered core samples from both test locations are similar although highly variable. The unconfined compression tests from Borings T-78, T-504, and T-512 are shown in Figure 5 along with the two super-imposed test sockets.

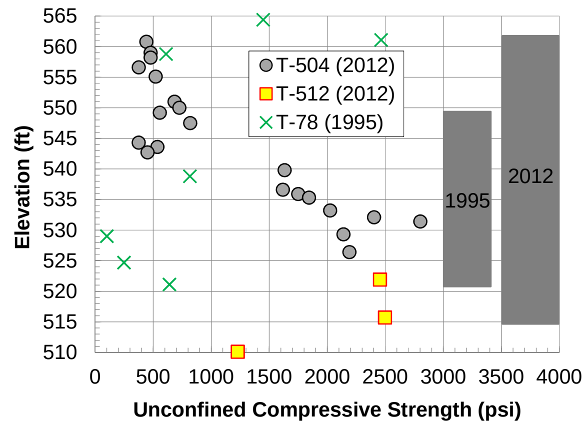

Figure 5: Bedrock Core Strength.

As is typical with bedrock that exhibits low RQD's, the unconfined compression test results may not be the best indicator of load resistance since only a small portion of intact samples are suitable for laboratory testing. In addition, RQD may also be of minimal benefit because, on the basis of visual observations, it does not fully capture how friable a substantial portion of the recovered sandstone core was.

Test Shaft Construction

Case Foundation out of Chicago, Illinois, constructed the shafts for both test programs. In each program, a permanent steel casing was installed to the top of bedrock and a smaller diameter rock socket was constructed beneath the casing. The rock socket in both programs was constructed by initially drilling a pilot hole and subsequently drilling the full diameter socket with a rock auger.

The 1995 test program consisted of a 5-ft diameter shaft in the soil overburden and a 4-ft diameter rock socket in the bedrock. The 2012 test program consisted of an 8-ft diameter shaft in the soil overburden and a 7.5-ft diameter socket in the sandstone bedrock. Generalized profiles of the two test shafts are provided in Figures 6 and 7 for the 1995 and 2012 tests, respectively. A construction timeline noting key activities is provided in Tables 1 and 2 for the 1995 and 2012 tests, respectively.

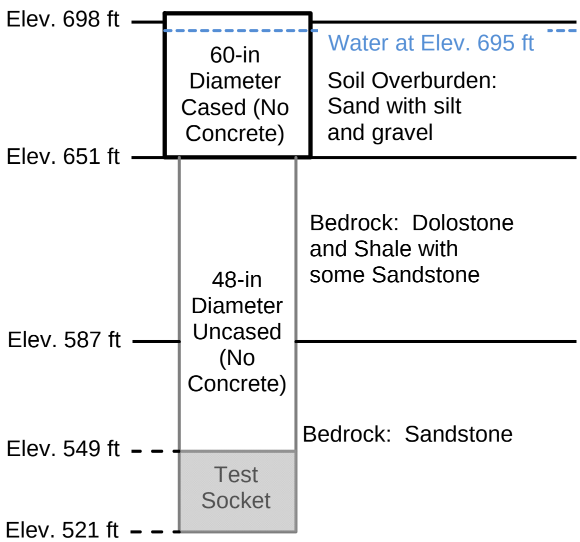

Figure 6: 1995 Test Shaft Schematic.

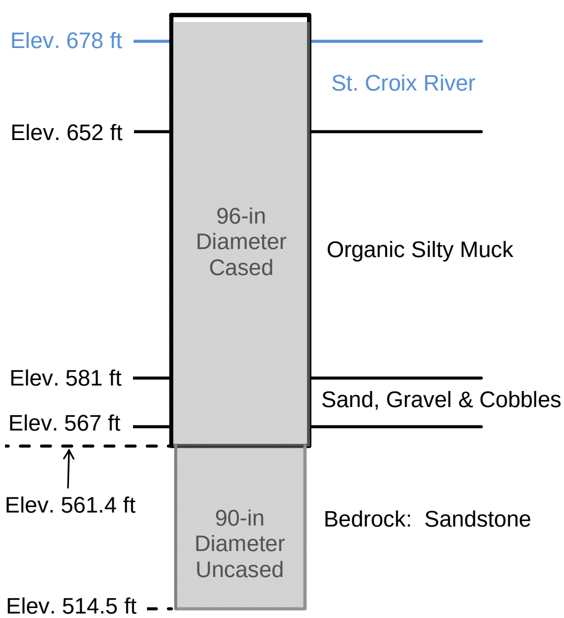

Figure 7: 2012 Test Shaft Schematic.

Concrete was only placed in the bottom portion of the test shaft in 1995. The area containing concrete is shaded in Figure 6. In 2012, concrete was placed throughout the entire shaft.

In 1995, the casing was installed after the soil overburden was excavated. The casing and the excavation were equal in diameter. In 2012, the casing was installed with a vibratory hammer. Initially, the casing was advance ahead of the excavation, although relief drilling inside the casing was necessary to achieve the desired casing tip elevation.

In 1995, no drilling fluid other than water was utilized in the construction. In 2012, the rock socket pilot hole was initially attempted using only water as the drilling fluid, but it was quickly determined that polymer slurry was desirable. Polymer slurry was introduced at that time and maintained throughout the remainder of construction.

Table 1: 1995 Test Shaft Construction.

Begin Date	Day	Construction Activity
Oct. 16	-9	Drill 60-in diameter hole to 47 ft. Install 60-in diameter permanent casing. Drill 48-in diameter hole beneath casing to top of socket.
Oct. 25	0	Begin and complete rock socket excavation.
Oct. 26[A]	+1	Clean-out with 1-eyed bucket. Perform mechanical caliper test.
Oct. 27[A]	+2	Clean-out with 1-eyed bucket. Perform mechanical caliper test.
Nov. 3[A]	+9	Clean-out with 1-eyed bucket. Place the testing frame Tremie-place concrete.
Nov. 7	+13	Perform O-cell load test.

<u>*Note A:*</u> Several feet of sediment accumulated at the base of the excavation after drilling.

The method employed for final clean-out in 1995 was a one-eyed clean-out bucket. The clean-out bucket was used on four different days in

1995 because several feet of sediment kept accumulating at the base of the shaft.

During the 2012 program, an airlift was used. The airlift was used on two separate occasions to remove sediment that accumulated after the excavation was complete and to exchange drilling fluid.

Inspection of the socket and base cleanliness prior to concrete placement in 1995 consisted of mechanical calipers and traditional weighted tape measurements. An attempt was made to observe the socket excavation with an underwater camera but it proved unsuccessful. The specifications for base cleanliness in 1995 required 0.5 inches or less of sediment at the base. Reportedly, this requirement was not achieved but construction proceeded regardless.

Table 2: 2012 Test Shaft Construction.

Begin Date	Day	Construction Activity
July 23	-7	Vibro-install 96-in diameter permanent casing
July 26	-4	Alternate relief drilling inside casing and advancing casing with vibro-hammer.
July 30	0	Begin drilling rock socket. Alternate between relief drilling and advancing casing with vibro-hammer.
July 31	+1	Introduce polymer slurry into excavation.
Aug. 1	+2	Advance casing to final location.
Aug. 3	+4	Complete rock socket excavation.
Aug. 6[A]	+7	Airlift.
Aug. 7	+8	Prepare cage including load testing apparatus.
Aug. 8[B]	+9	Airlift for final time. Perform Mini-SID and SoniCaliper inspections. Install cage.
Aug. 9	+10	Tremie-place concrete.
Aug. 15	+16	Perform non-destructive CSL testing.
Aug. 27	+28	Perform O-cell load test.

Note A: Approximately 4 ft of sediment accumulated and was removed August 6.
Note B: Approximately 3 inches of sediment accumulated and was removed August 8.

The inspection conducted in 2012 included the SoniCaliper performed by Loadtest, the Mini-SID performed by Case Foundation under the direction of DBA, and traditional weighted tape measurements performed by DBA. The specifications for base cleanliness in 2012 required 0.5 inches or less of sediment over 50 percent or less of the base; the maximum amount of sediment allowable at any one location was 1.5 inches. On the basis of the Mini-SID inspection and the weighted tape soundings, these requirements were achieved.

The Mini-SID was used to view five locations; four around the circumference and one at the center. In three of the four viewings around the circumference, no sediment at all was observed. In the remaining view, approximately 10 percent of the viewing area contained 0.5 inches of sediment. In the center view, approximately 25 percent of the viewing area contained 0.5 inches of sediment.

The concrete for both test shafts was tremie-placed under fluid. In 1995, a 4-in diameter steel tremie was used. In 2012, a 10-in diameter open-top tremie was used and the hopper attached to the top of the tremie was supplied by two separate pump trucks. A plug or pig was used to separate the concrete from the drilling fluid in both instances. Non-destructive integrity testing was performed in 2012 but not in 1995; on the basis of cross-hole sonic logging (CSL), no anomalies were detected in the 2012 shaft.

The concrete utilized in 1995 contained a 0.75 inch maximum coarse aggregate and had a 9 in slump. The concrete utilized in 2012 was similar to a traditional self-consolidating concrete (SCC) mix although all tremie-placed concrete in drilled shafts is SCC by definition. The 2012 concrete had a slump flow of 27 to 28 inches and displayed no apparent bleed or segregation. At the time of the load tests, the unconfined compressive strength of a concrete cylinder made during the placement was 4,610 psi and 6,644 psi for the 1995 and 2012 shafts, respectively. Both exceeded the design strength of 4,000 psi.

Test Results

The results of the 1995 and 2012 tests are shown graphically in Figures 8 and 9 for measured unit side resistance and measured unit base resistance, respectively. In Figure 8, a large discrepancy in measured unit side resistance is obvious with the 2012 values far exceeding the 1995 values, even at small deflections. Unfortunately, the 2012 test was not able to demonstrate additional side resistance as the capacity of the O-cells was exceeded prior to mobilization of the full nominal resistance.

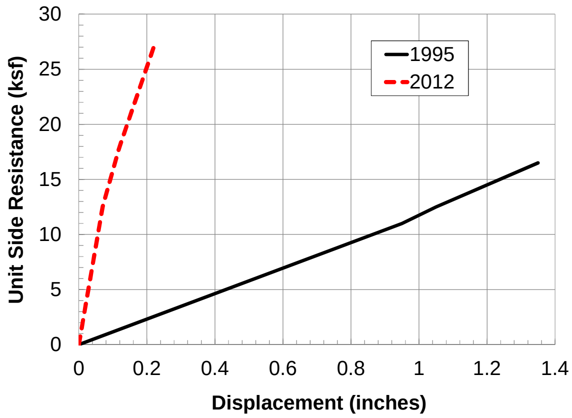

Figure 8: Average Measured Unit Side Resistance in Rock Sockets.

In addition, the strain gage measurements in the 1995 test indicated that bending moments were present in the shaft during the test. Half the cage was in compression and the other half was in tension. These measurements agree with the fact that the bottom of the shaft could not be adequately cleaned to satisfy the construction specifications and the tremie pipe was offset to one side of the shaft during concrete placement. To estimate the nominal unit side resistance measured in the 1995 test, the average strain gage values were used despite the fact that one side of the shaft was in compression whereas the other was in tension. The impact of this averaging is unknown but is suspected to have substantial consequences on the reported results and associated recommendations.

Another O-cell load test on a drilled shaft that was conducted in sandstone bedrock of similar quality was performed by MnDOT for the I-35W replacement bridge in 2007. In that test, the average unit side resistance measured in the sandstone bedrock was approximately 30 ksf at a maximum displacement of 0.64 in. The maximum unit base resistance was measured to be about 250 ksf at a displacement of 2.35 inches. That rock socket was also constructed under polymer slurry and utilized a SCC mix. The I-35W results are more similar to the St. Croix 2012 test than the 1995 test.

In Figure 9, the measured unit base resistances during the 1995 and 2012 tests are similar at small deflections and the stiffness response is similar as well. Unfortunately, the 1995 test was not able to demonstrate additional resistance as a result of testing limitations.

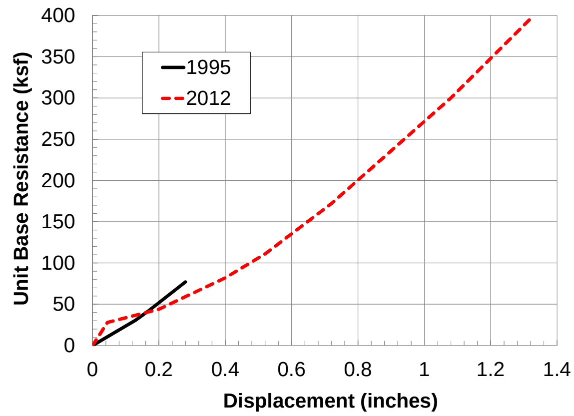

Figure 9: Measured Unit Base Resistance.

It is important to recognize that neither the 1995 nor the 2012 test achieved nominal resistance values, and both the unit base and unit side resistances were increasing rather dramatically at the conclusion of each test.

Design Recommendations, Differences, and Impacts

The recommended axial design values following both test programs are provided in Table 3. The nominal unit side resistance, f_{sN}, recommended for design following the 2012 test program is more than three times that following the 1995 program. The nominal unit base resistance, q_{bN}, recommended for design following the 2012 test program is more than six times that following the 1995 program. Note that these values do not necessarily represent the maximum resistances measured during the test but are those recommended for use in design. The difference between measured and recommended values is necessary to account for variability across the site, scaling factors since the 1995 test was on a smaller prototype foundation, and the compatibility of observed deflections between the two test programs.

Table 3: Recommended Axial Design Values.

Test Program	f_{sN} (ksf)	q_{bN} (ksf)
1995	7.5	40
2012	25	250

The current bridge design includes five river piers with each pier containing two legs. Each pier leg is supported on a 2x2 group of 9-ft diameter drilled shafts with 8.5-ft diameter rock sockets. The rock sockets are planned to be 25-ft long and four of the five pier locations and 35-ft long at the remaining pier. To provide the same axial resistance using the same individual drilled shaft dimensions, approximately 16 shafts would be required if the 1995 test program design values were utilized. It is recognized that reconfiguring the group would change the demand on individual shafts through group action. Regardless, this substantial increase in the quantity of drilled shafts would necessarily increase the size of the coffercell and pile cap. Furthermore, the additional number and size of the foundations would negatively impact the hydraulic performance and greatly increase the scour potential. Alternatively, if the current 2x2 group configuration and shaft diameters were to be maintained, the socket length would have to increase from 25 ft to 143 ft if the 1995 design recommendations were utilized. Obviously, 143-ft long rock sockets are not practical.

Considering the differences in construction and measured behavior during the load tests, the following items are identified that may contribute to the observed results:

1. The use of a very workable SCC concrete mix in 2012 versus a traditional mix in 1995;
2. The increased fluid head pressure in the freshly placed concrete in 2012 versus 1995;
3. The use of polymer slurry in 2012 versus water in 1995;
4. The practice of airlifting in 2012 versus a clean-out bucket in 1995; and
5. The numerical scaling approach utilized in 1995 that was not necessary in 2012.

It is possible, perhaps probable, that potential factors 1 and 2 can be combined. The use of a relatively fluid concrete mix with substantially more fluid pressure almost certainly creates a better bond at the interface between the socket wall and the concrete. The enhanced bond would serve to increase the unit side resistance measured during the 2012 test relative to the 1995 test. It is not known if the properties of the mix or the increased fluid pressure in the concrete at the location of the rock socket provided the most benefit although it is probable that the increased fluid pressure would have helped more because the difference in workability between an SCC mix (2012) and a traditional mix with a 9 in slump (1995) is probably minor or perhaps non-existent.

The use of polymer slurry may have helped preserve the integrity of the socket walls. The duration in which the rock sockets were open is similar between the 1995 and 2012 tests. In 1995, the concrete was placed nine days after the socket was started and completed. In 2012, the concrete was placed 10 days after the socket was started and 6 days after the socket was completed. Therefore, the substantial difference in the measured unit side resistance values could be attributed to the use of the polymer slurry. It is generally recognized that sandstone is typically not considered to degrade in the presence of water like a shale bedrock would. However, both sandstone and shale are of sedimentary origin and of relatively low-strength compared to limestone which is also considered to be resistant to water during drilling.

By using polymer slurry in weak sandstone, it may be that the weak, uncemented layers are less likely to slough from the excavation through the socket wall into the bedrock formation. The presence of fine-grained particles in suspension could potentially contaminate the sidewall of the rock socket and reduce the measured side resistance. It is also possible that incomplete concrete filling of voids created by sidewall sloughing (or filling by debris rather than concrete) could contribute to reduced side resistance by weakening the rock along the sidewall.

The use of an airlift or other similar means to perform the final clean-out of drilled shafts is beneficial for two reasons. First, for relatively large shafts, airlifting provides an excellent means for cleaning the drilling fluid. Regardless of sand content, density, viscosity, or results of other slurry tests, exchanging the drilling fluid during airlifting is good and necessary practice. Preferably, the entire volume of drilling fluid would be exchanged during airlifting so that cuttings held in suspension in the relatively large drilled shaft don't precipitate out of suspension during concrete placement or after final inspection. This exercise is necessary and important to reduce the risk of construction defects. In the 2012 test shaft, it is estimated that approximately two-thirds of the polymer slurry was exchanged during airlifting. Prior to airlifting for the first time in 2012, approximately 4 ft of sediment accumulated at the bottom of the completed excavation over a weekend. After airlifting, a couple of inches of sediment accumulated over the next two days. In 1995, several feet of sediment accumulated overnight on several occasions likely because of cuttings temporarily held in suspension in the drilling fluid. To exacerbate the problem, every time a clean-out bucket is used, the particles are re-introduced into suspension and more harm than good usually results. Note that calipers were used on several occasions in 1995 following the repeated accumulation of material at the base and no evidence of sloughing was discovered.

Furthermore, by sufficiently removing sediment or loose material from the base of the shaft, the potential for construction defects is reduced because the initial charge of concrete at the on-set of the tremie-placement will not cause displaced loose material to become trapped around the cage or CSL tubes. Ideally, the low-density sediment would be displaced by the concrete and ride to the top of the pour on top of the fluid concrete. However, in reality, research indicates the low-density sediment is pushed to the side and often becomes trapped around the cage, particularly in relatively large diameter shafts. The CSL tests following the 2012 construction indicated no anomalies were present. Similar testing was not conducted in 1995 so no correlations can be made.

In addition to the reduction in construction risk, airlifting can be an excellent way of cleaning the base of shafts that rely on the contribution of base resistance. However, considering the measured unit base resistance in 1995 and 2012, the potential benefit of airlifting on base resistance is inconclusive. The low value of base resistance recommended for design in 1995 was an artifact of the low resistance mobilized during the test. It is quite likely that additional base resistance would have been mobilized but the test was not capable of demonstrating the additional resistance.

The 1995 test was performed on a prototype foundation that was smaller diameter than that being considered for the project. Scaling factors were computed using sophisticated analytical methods to estimate the scale effects following the 1995 test. The scale effects were included in the design recommendations and ultimately reduced the resistance values measured during the test. While scaling factors may be important, they do not bridge the gap in recommended design values between the 1995 and 2012 tests.

Conclusions

Two static O-cell load tests were performed by MnDOT for the St. Croix River bridge project near Oak Park Heights, Minnesota. The first test was performed in 1995 and the second in 2012. Both test shafts were constructed by the same drilled shaft contractor. Following the tests, very different design recommendations were provided. Under the current 2x2 group configuration with 8.5-ft diameter rock sockets, the 1995 recommendations would require 143-ft long rock sockets compared to the 2012 requirements for 25-ft long rock sockets. Because 143-ft long rock sockets are not expected to be practical, as an alternative, 16 8.5-ft diameter by 25-ft long rock sockets would also suffice.

Major differences in the measured axial resistance values between the 1995 and 2012 drilled shaft load test program were observed, and much of the lower resistance measured in the 1995 tests are attributed to instability and inadequate cleaning in the rock socket during construction.

Three factors are identified that could have all contributed to the increased side resistance measured in 2012 relative to 1995; the type of concrete mix, the increased fluid pressure at the time of concrete placement, and the use of polymer slurry. It is not known if all the potential factors contributed or, if they did, to what degree. Regardless, when comparing the measured side resistance values in 1995 versus 2012, it can be concluded that a mix with a relatively high workability should be used in conjunction with polymer slurry when constructing rock sockets in sandstone bedrock.

Airlifting is beneficial and when performed sufficiently, reduces the risk of construction defects. It is not known if this factor contributed to the difference in load test results. However, the bending measured in the shaft during the 1995 test and the necessary averaging of strain gage data likely affected the results. This bending could possibly have been avoided if airlifting were conducted. Airlifting is also beneficial for helping design engineers rely on the contribution of base resistance. However, since the measured base resistance response was similar for the two tests, it cannot be concluded that airlifting alone produced better base resistance.

Numerical analyses were performed in 1995 to account for scaling factors between prototype and full-size drilled shafts and for the higher effective stress in the bearing stratum at the test location relative to the bridge site; these factors suggested that design values of axial resistance should be even lower than the measured values. The 2012 test shaft had a diameter within one foot of the estimated production shafts and at a location of similar overburden conditions, and so the test results were directly applicable for production locations in rock with similar characteristics. Full-scale load test shafts near the actual production locations are preferred despite their increased cost. The degree of influence contributed by *possible* errors in the scaling factors is not known.

Finally, the comparison of the two load tests suggest that today's drilled shaft professionals have better tools and understanding of the techniques to obtain higher quality foundations. Improvements in the use of polymer slurry, downhole cleaning procedures, and inspection techniques will all contribute to improved foundation design and reliability. These tools should be specified, utilized, and embraced to enhance foundation performance and reduce the risk of construction defects.

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