LOAD TESTING FOR ACCEPTANCE OF SHAFTS WITH QUESTIONABLE INTEGRITY – CASE STUDIES

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ABSTRACT

Integrity testing of drilled shafts through non-destructive methods has become commonplace in the construction industry. Many test methods are available and established, each having their own individual strengths and limitations. However, shaft acceptance should not be based solely on these test results. Even when integrity testing raises a concern or indicates a potential anomaly, the shaft may still be capable of performing as designed. All available installation information and test results should be considered. When further investigation is warranted, there are several options to provide additional information for consideration of shaft acceptance. Common methods include additional testing utilizing alternate methods, concrete coring, or an engineering analysis with stated assumptions. A less common option is to perform a full-scale load test on the shaft in question to evaluate the ability of the shaft, as installed, to carry the structural loads. This paper will discuss two case histories where load testing was performed to support shaft acceptance where anomalies were indicated in multiple integrity testing methods.

Two county road bridges in close proximity and being constructed in the same timeframe were designed to be supported by drilled shaft foundations. The specifications required one shaft at each abutment be tested using Thermal Integrity Profiling. The Thermal Integrity Profiling results indicated significant anomalies at the base of each shaft. The contractors and owners agreed to perform dynamic load tests to evaluate the load carrying ability of the shafts as installed. Prior to dynamic testing, Pile Integrity Testing was also performed on each shaft. This paper will discuss and compare the multiple test type results for the shafts on these projects. The advantages and limitations of using this approach will be presented so others can evaluate the suitability of this testing option for foundation acceptance on future projects.

Keywords: shaft, integrity, testing, acceptance

INTRODUCTION - INTEGRITY ASSESSMENT OF DRILLED SHAFTS

Integrity assessment of drilled shafts can be determined by considering information from many points in the construction process. Observation of the drilling and shaft installation can provide additional confidence in the foundation element. Material testing is commonplace and provides another quality assurance. All of this information should be considered and is critical to the final evaluation of shaft integrity.

Integrity testing is an in-situ method that evaluates the shaft as constructed. Several testing methods, each with their own strengths and limitations, are available to evaluate the integrity of a shaft. This paper will discuss three types of tests, Thermal Integrity Profiling, Pile Integrity Testing, and High-Strain Dynamic Testing, and their results from drilled shafts installed to support two county road projects.

Thermal Integrity Profiling (TIP)

Thermal Integrity profiling uses the heat generated from cement hydration to evaluate the shaft integrity and is standardized through ASTM D7949. The temperature measurements are typically collected using instrumented wires that are attached to the reinforcing cage or some element that is inserted into the excavation (ASTM D7949 method B). Temperature measurements are typically collected every 15 minutes along the full length of the shaft. Temperature measurements should begin prior to, or soon after, concrete placement and continue until the average temperature of the shaft has passed the highest temperature, or the peak temperature. Alternatively, temperatures can be measured using a temperature probe placed in access tubes that were cast into the shaft at a selected time or multiple times after concrete placement (ASTM D7949 method A).

The collected temperature data is analyzed for unexpected temperature reductions, which may indicate an anomaly in the shaft. Additionally, differences in temperature measurements from opposing wires can indicate shifting or misalignment of the reinforcing cage. Using dedicated software, temperature measurements can be correlated to an effective radius and effective concrete coverage if an accurate measurement of placed concrete volume is available. Effective radius and coverage are terms which describe these measurements with the assumption of uniform material properties. Therefore, the effective radius is affected by both geometric changes in the shaft (diameter change) or material quality changes (Belardo et al. 2021). This is important to understand when evaluating a shaft effective radius model generated from TIP.

Pile Integrity Testing (PIT)

Pile integrity testing, also referred to as low strain testing or pulse echo testing, is standardized through ASTM D5882. This test method uses measured pile top acceleration versus time under an impact to evaluate shaft integrity. Acceleration measurements are collected with an accelerometer attached to the shaft, typically at the top (e.g., Rausche et al. 1998). The impact is applied with a relatively small, generally hand-held, hammer and induces a stress wave travelling down the shaft. Acceleration measurements are integrated to obtain velocity. The velocity versus time curve is amplified and filtered through dedicated software and investigated for pile top velocity changes due to reflected stress waves. The stress wave is reflected only by a change in impedance, denoted by Z (a term referring to the product of the shaft elastic modulus and area divided by the wave propagation speed, i.e., EA/c) or by the end of the shaft. Therefore, reflections prior to the expected time of wave arrival from the toe of the shaft are caused by changes in impedance along the pile length, which may be caused by changes in material or cross section.

High Strain Dynamic Testing (HSDT)

High strain dynamic testing is standardized through ASTM D4945 and was originally developed to monitor driven piles; however, this method has been used on cast in place foundations for decades. For this paper we will be referring to HSDT as applied to drilled foundation testing.

Measurements of strain and acceleration are collected using an instrumented top transducer and accelerometers mounted on the side of the shaft, respectively. For driven piles, the installation hammer provides the impact that generates the stress wave down the pile. For drilled shafts, typically a drop hammer is utilized. A sufficient drop weight should be utilized, generally 1% to 2% of the anticipated maximum test load (Hussein, M. et al. 1996).

The stress wave induced by an impact travels down the shaft and is reflected only by changes in impedance or the end of the shaft. The measured strain is converted to force using input shaft properties and the acceleration is integrated to obtain velocity. The measured force and velocity data is evaluated using dedicated software. The results of HSDT typically include evaluation of the stresses near the pile top, energy transferred to the shaft, major integrity issues, and static soil resistance. In addition, due to the non-uniform nature of drilled shafts, signal matching software is utilized to further refine the results for stresses in the pile, integrity evaluation, and static soil resistance.

CASE STUDIES

Case Study #1 – Single Span County Road Bridge Replacement

This project was the replacement of a 91-foot single span, pony truss bridge with a prefabricated, 101-foot single span, truss bridge. Each abutment is founded on five drilled shafts with maximum factored load of 254 kips. The rear abutment shafts are 36 inches in diameter, 35 feet long, and bear in very dense sands. The designed factored side resistance was 104 kips and was assumed to act along the bottom 25 feet of the shaft. The factored tip resistance was 170 kips. The forward abutment shafts are 36 inches in diameter, 26 feet long, with the bottom 4.5 feet being 30-inch diameter rock sockets bearing in moderately strong sandstone. Base resistance of the rock socket provided all of the factored resistance for the shaft, 1,350 kips. Therefore, any shaft resistance was neglected.

Following typical practices, the shafts were drilled to depth with augers, and temporary casing was used to maintain sidewall stability of the hole. The bottom of the hole was cleaned with a cleanout bucket prior to installing the reinforcing cage. For holes with limited groundwater inflow, a submersible pump dewatered the hole immediately prior to concrete placement, while holes with substantial inflow were not initially dewatered, they were dewatered during concrete placement. A pump truck delivered the concrete to the bottom of the hole through a tremie pipe. Casing removal was coordinated with concrete placement. No problems were noted to have occurred during drilling, concrete placement, or casing removal.

Three shafts, two at the rear abutment and one at the forward abutment, were instrumented with TIP wires on the reinforcing cages to collect the TIP data for the evaluation of shaft integrity. Evaluation of the three TIP testing results indicated similar anomalies in all three shafts over the bottom 3 to 5 feet of the shafts. Based on the elevations and the TIP results, it also appeared the reinforcing cage had sunk into the soil below the base of the shaft. In addition, one shaft also had an anomaly at approximately the midpoint of the shaft. Figure 1 presents the TIP results of effective radius vs depth and a 2-dimensional model of the shaft with the shaft exterior surface overlayed on the reinforcing cage and generalized soil profile for both shafts.

The anomalies could be caused by changes in geometry of the shaft, but this was unexpected. Concrete quality issues could arise from soil inclusions or contamination of the concrete by water and cuttings, but no observations during installation indicated these issues. The contract specifications required that anomalies were to be evaluated by coring the concrete and testing its strength. If the strength was found to be deficient, then the contractor had to prepare a remediation plan. However, since the anomalies appeared to be across the full section, including outside the reinforcing cage, coring was deemed to not be able to provide sufficient information for repairs or to evaluate reduced capacities for the shafts. In addition, all ten shafts were suspected of having similar anomalies, because construction methods and observations did not vary among the shafts. Therefore, all the shafts were considered to be deficient and required a remediation plan. Due to the spacing of the shafts, adding more drilled shafts to the abutment was impractical.

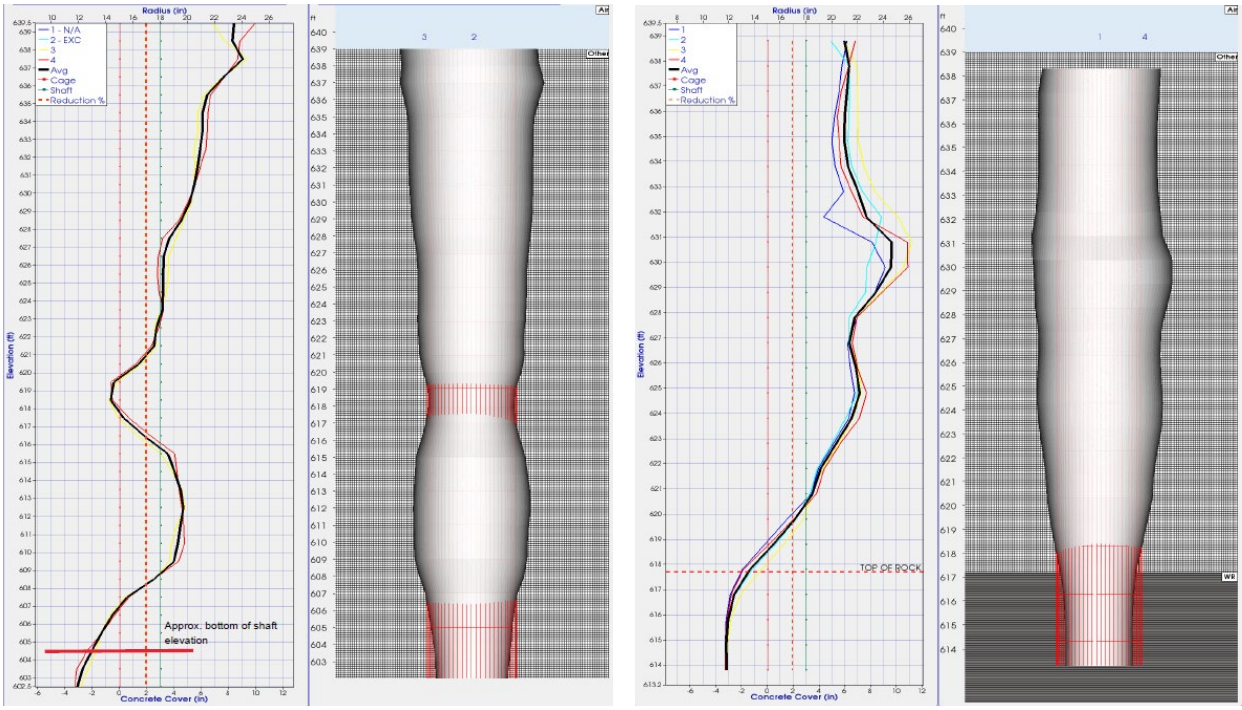

Fig. 1: TIP Results Indicating Anomalies at the Base and Near the Midpoint of Shaft 1 (Left) at the Rear Abutment and Near the Base for Shaft 8 (Right) at the Forward Abutment

The contractor and their testing consultant proposed performance testing, HSDT, on one shaft in each abutment rather than attempting to core and remediate the shafts. The owner was familiar with HSDT having used it over the past 10 years in unique circumstances: verifying pile refusal on rock of 200-foot long HP18x204 that was overlain by thick, hard till; verifying capacity of 4-foot diameter friction shafts drilled in coarse granular soils in artesian condition; and proof-of-concept that ACIP piles can be used for bridge foundations. The designer and the owner accepted the proposed testing to evaluate the ultimate resistance of a shaft at each abutment which would determine the magnitude of any necessary remediations.

The testing consultant also performed PIT prior to HSDT. The resulting velocity curves are presented in Fig. 2 below for the same shafts.

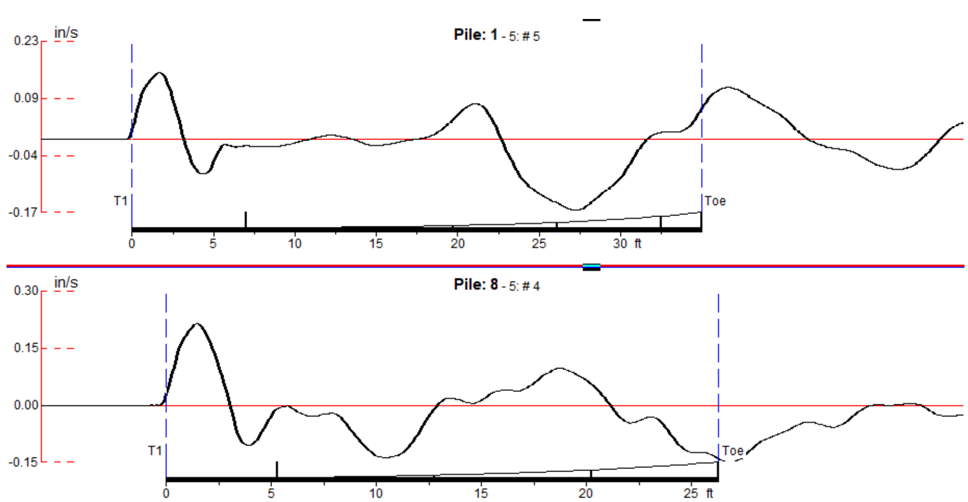

Fig. 2: PIT Results for Shafts 1 and 8

The results from PIT agreed fairly well with the results of TIP. For Shaft 1, a clear anomaly is indicated at approximately the midpoint of the shaft with a positive velocity response. After the first anomaly was confirmed, the subsequent anomaly at the base is difficult to evaluate, which is a limitation of the test method. The results from Shaft 8 indicate a bulge at approximately 8 feet with a negative velocity response, and are inconclusive below the bulge, again, a limitation of the test method.

HSDT was performed on both shafts. A drop hammer system with a 9-kip drop weight was used to provide the impacts to the shafts. After preparing and instrumenting the shafts, three to four impacts were applied to each shaft with increasing drop heights. The drop heights ranged from 8 to 24 inches. After each impact, the shaft's set was measured using an optical level and a scale attached to the shaft.

Fig. 3: Shaft instrumented for HSDT

Fig. 4: Drop hammer system positioned on shaft

With the understanding that integrity testing results indicated significant anomalies over the bottom 3 to 5 feet in each of the shafts, the shafts were modeled as approximately 4 feet shorter in the signal matching analysis, which created a reasonable match. In addition, an impedance reduction was evident in the Shaft 1 results which corresponded to the integrity testing results and was modeled in the signal matching software.

As previously mentioned, the maximum factored load for each shaft was indicated to be 254 kips. At the forward abutment where the shafts bear in rock, Shaft 8 demonstrated a mobilized soil resistance of 372 kips with nearly zero movement under the impacts. This mobilized capacity exceeded the maximum factored load and the forward abutment shafts were determined to be acceptable.

At the rear abutment where the shafts bear in dense sand, Shaft 1 experienced significant movement during testing, a total of more than 0.5 inches and signal matching indicated a nominal soil resistance of 181 kips. The fully mobilized soil resistance did not exceed the maximum factored load and these shafts were determined to be deficient and required a remediation plan. The contactor's designer proposed to increase the total capacity of the rear abutment foundation by adding four vertical H-piles located midway between the shafts to increase the total resistance of the foundation elements to 1,270 kips. The plan was accepted, and the contractor drove four HP10x42 piles to refusal on bedrock, which provided more than the required additional factored resistance, but did not require load testing.

An important aspect from performing these drilled shaft tests was that schedule delays were mitigated. The TIP results quickly alerted the contractor that there were potential shaft deficiencies, so that planning for corrective actions could be considered before formal findings were issued. The contractor and testing consultant had the proposed HSDT plan prepared, and presented it along with the TIP results. The HSDT provided the actual soil resistances that determined which foundations needed remediation, and by how much. The added piles were driven within six weeks from the completion of the drilled shafts, and the bridge was completed and opened on time.

Case Study #2 – Three Span County Road Bridge Replacement

This project was the replacement of a 343-foot three span structure. Each abutment is founded on 8 or 9 drilled shafts, 42 inches in diameter with 42-inch diameter rock sockets. The rock sockets were designed to be 23 feet and 6 feet in length for the rear and forward abutments, respectively. The pier foundations consisted of 3 drilled shafts each, 48 inches in diameter with 42-inch diameter rock sockets. The rock sockets were designed to be 20 feet and 10 feet in length for Piers 1 and 2, respectively. In all cases, permanent casing was utilized and installed to the top of bedrock.

The factored loads to be supported by each shaft were 417 kips and 881 kips, for the abutments and piers, respectively. Project documents present the factored side resistance (and the assumed location of the resistance) and factored tip resistances. Table 1 presents the factored resistances provided in the project plans.

Table 1: Factored Resistances Provided from the Drilled Shaft Design

Substructure	Estimated Total Shaft Length (feet)	Factored Side Resistance (kips)	Assumed Location of Resistance	Factored Tip Resistance (kips)	Total Factored Resistance (kips)
Rear Abutment	27.4	324	Bottom 21 feet of rock socket	1732	2056
Pier 1	37.4	1728	Bottom 9 feet of rock socket	5715	7443
Pier 2	54.8	37	Bottom 8 feet of rock socket	866	903
Forward Abutment	75.3	187	Bottom 4 feet of rock socket	866	1053

The project notes only directed to perform TIP on two shafts at the forward abutment of the structure. The project schedule dictated that the forward abutment shafts were the last shafts installed on the project. Furthermore, due to a miscommunication, TIP was performed on the last 2 shafts installed at the forward abutment, and therefore, the last 2 shafts installed on the project. The TIP tested shafts were designated as shafts 19 and 20.

Documentation and details of the shaft installation provided to the testing consultant were minimal. A tremie was used to place the concrete, however, the tremie pipe onsite was measured to be 60 feet in length, while the forward abutment shafts were designed for a length of approximately 75 feet. Shaft bottom cleanout procedures were not provided. TIP results indicated very little temperature increase in the bottom 10 feet and 20 feet of shafts 19 and 20 respectively. Both results indicated anomalies in the bottom of the shafts, in both cases, the anomalies extended above the reported top of rock. Figure 5 presents the TIP results of effective radius vs depth and a 2-dimensional model of the shaft with the shaft exterior surface overlayed on the reinforcing cage for both shafts.

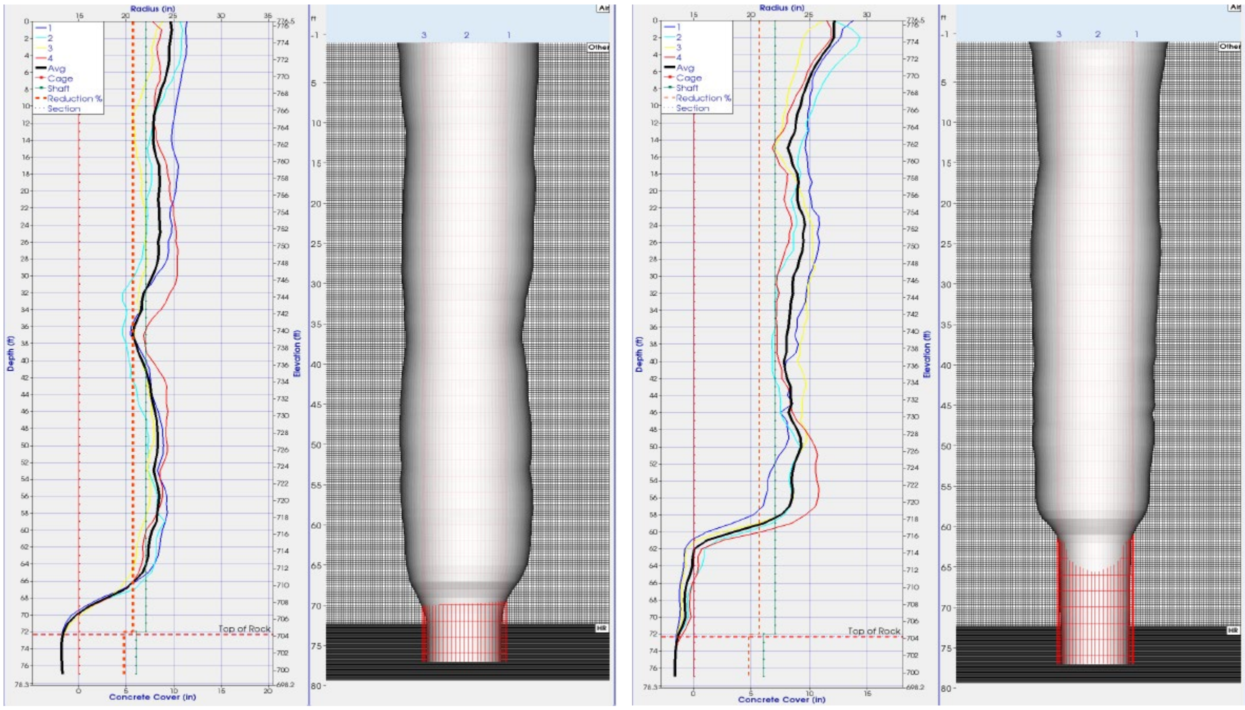

Fig. 5: TIP Results Indicating Anomalies at the Base of Shaft 19 (Left) and Shaft 20 (Right)

Similar to Case Study #1, the anomalous area was to be evaluated through coring per the project documents. Again, however, the anomalies appeared to be across the entire shaft section, and remediation may not be effective. The contractor and testing consultant elected to propose performance testing through HSDT to confirm that the shafts, as installed, could provide the required factored loads shown in the plans. Again, the designer and the owner accepted the proposed testing to evaluate the ultimate resistance of a shaft at each abutment which would determine the magnitude of any necessary remediations.

The testing consultant again performed PIT prior to the HSDT. The resulting velocity curves are presented in Fig. 6 below.

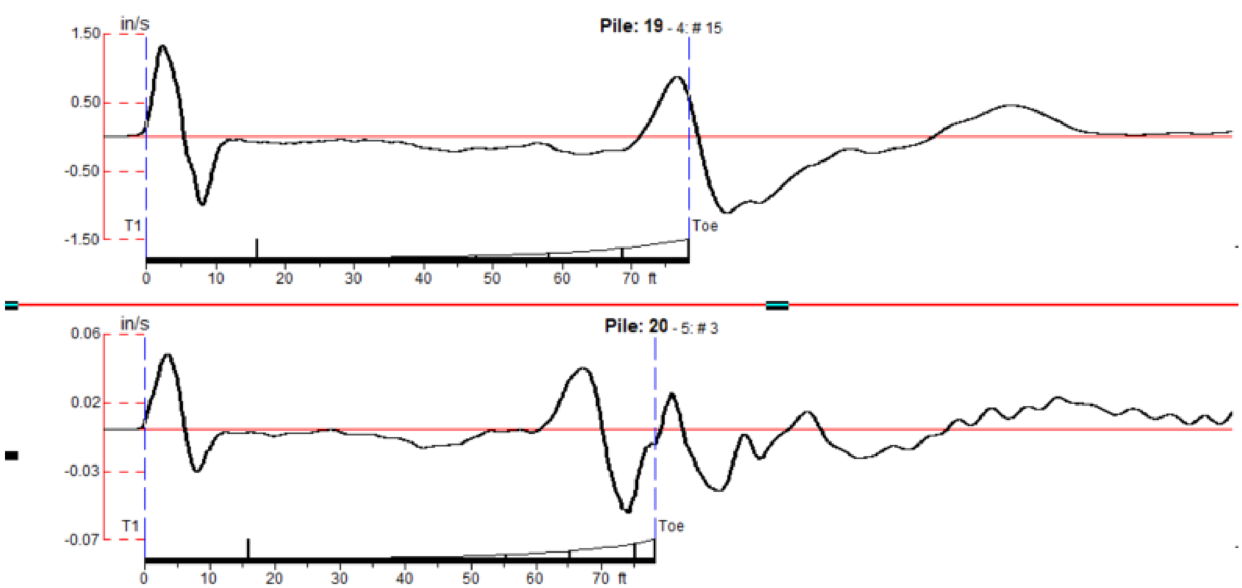

Fig. 6: PIT Results for Shafts 19 and 20

The PIT results agreed well with the TIP results. Shafts 19 and 20 indicated early reflections from the bottom of the shafts at depths of approximately 67 feet and 60 feet, respectively.

HSDT was performed on both shafts. The same drop hammer system from Case Study #1 was used to provide the impacts to the shafts. After preparing and instrumenting the shafts, four impacts were applied to each shaft with increasing drop heights. The drop heights ranged from 10 to 30 inches. After each impact, the shaft set was measured with an optical level and a scale attached to the shaft.

Fig. 7: Shaft instrumented for HSDT

Fig. 8: Drop hammer system positioned on shaft

Both shafts performed well during the HSDT. The total movement under 4 impacts was less than 0.1 inches. As these were production shafts, impacts with enough force to mobilize the required factored loads, but not significantly higher, were applied. The intent was to limit the risk of unnecessarily damaging the shaft during testing.

With the understanding that integrity testing results indicated significant anomalies over the bottom 10 to 20 feet, the shafts were modeled as approximately 10 and 20 feet shorter in the signal matching analysis, which created a reasonable match. The mobilized soil resistances under the highest energy impacts were 615 kips and 590 kips for shafts 19 and 20, respectively. The shaft resistance mobilized for both shafts was approximately 540 kips, which exceeded the required factored load, but was completely neglected during design. Note that the factored side and tip resistances were assumed to only be provided from the bottom 4 feet of the rock socket and in end bearing for the forward abutment shafts. It appears that the soil resistance above the locations assumed to provide soil resistance was enough to support the structure. As the mobilized soil resistance was greater than the required factored load, the shafts were determined to be acceptable.

ADDITIONAL CONSIDERATIONS

It is important to note that in the two cases presented here, most shafts were accepted based on the HSDT results indicating a higher soil resistance than the required factored load, however, there are many other considerations for acceptance of shafts with questionable integrity.

Shear and Moment and Uplift Loads

Depending on the lateral and uplift load requirements of the shafts, the location of the anomaly could be of concern, even if the shaft demonstrates adequate axial compressive soil resistance. Shaft 1 had a potential anomaly at approximately 20 feet depth that could have been a concern for the lateral resistance of the shaft, however, the shafts in Case#1 did not have any shear or moment load requirements. The shafts in Case#2 did have shear and moment load requirements, but the anomalies were sufficiently deep enough that they were likely below the depth to fixity of the shafts. Neither case had uplift load requirements.

Shaft Durability

As foundation design lives increase, so does the concern of the durability of the shaft, or the ability of the shaft to perform throughout its design life. Clearly this could be a concern for shafts with questionable concrete quality or concrete cover and should be evaluated for each case.

Certainly, this is not an exhaustive list of considerations for accepting shafts with questionable integrity based on HSDT results, any specific shaft performance requirements should be addressed and considered prior to acceptance of HSDT as a method of providing shaft approval.

CONCLUSIONS

Two cases were presented where multiple integrity test methods indicated potential anomalies in drilled shafts. Due to the nature of these anomalies, the contractors elected to propose High Strain Dynamic Testing (HSDT) rather than coring and remediating the shafts. In both cases, the shafts were approved as installed, however, for one abutment the shafts were supplemented with driven piles to offset the insufficient soil resistance indicated by the HSDT results.

HSDT is a viable option for evaluating the axial soil resistance of a shaft as installed, including those with potential anomalies. This information, along with careful consideration of many other factors, can be used to justify approval of drilled shafts with questionable integrity. Prior to an owner or owner's representative specifying HSDT or accepting a contractor proposal to perform HSDT, the test method and its limitations should be understood. The goal of both the contractor and the owner should be to use the HSDT results, in conjunction with all records and integrity test results, to facilitate shaft acceptance.

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