Case History: Foundation Evaluation for the Virginia Highway 288 Project

Emad Farouz
CH2M HILL, Herndon, Virginia

Paul Landers
CH2M HILL, Herndon, Virginia

Scott Webster
GRL Engineers, Inc., Charlotte, North Carolina

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Emad Farouz
CH2M HILL
Herndon, VA (USA)

Paul Landers
CH2M HILL
Herndon, VA (USA)

Scott Webster
GRL Engineers, Inc.
Charlotte, NC (USA)

ABSTRACT

For the first time in Virginia highway construction history, a consortium of contractors, engineers, and designers proposed an expansion of the VA-288 highway around the fast-growing western half of Richmond. The design-build project was approved in December 2000 and construction began April 2001. The project includes constructing approximately 17 miles of new highway with 23 bridges and overpasses. The fast-track, design-build process requires that bridge design and construction be carefully evaluated to determine the most cost-effective approach. Figure 1 presents the Site Location Map.

In keeping with the design-build process, the bridge foundations were varied depending upon the crossing's length and the bridge's height. Many smaller bridges were supported on pile foundations, while larger structures were designed with a combination of piles and drilled shafts. To ensure cost-effective foundations, it was desirable to use the highest loading possible without compromising safety. This required extensive foundation testing using non-destructive techniques.

Testing with the Pile Driving Analyzer (PDA) was proposed for driven-pile foundations to confirm the ultimate pile capacity, evaluate driving stresses and hammer performance, and establish the driving criteria. PDA testing was performed at all bridge locations where piles were used. CAPWAP and GRLWEAP analysis were used to establish the pile-driving criteria, which allowed the most efficient means available for pile installations. Finally, PDA testing and evaluations were used to further evaluate the pile performance and suitability whenever unusual situations were encountered.

Crosshole Sonic Logging (CSL) and Pile Integrity Tests (PIT) were used to evaluate the overall quality of the constructed shafts for drilled-shaft foundations. CSL using the Crosshole Analyzer (CHA) was performed on each of the project's 120 shafts. The shaft diameter varied from 4.0 to 6.5 feet, with design loads between 600 and more than 2,500 kips. Remedial actions were developed and implemented to repair the defect as necessary where CSL results indicated poor quality concrete or defects in the shaft.

This paper presents a case history detailing the benefits of the latest techniques for deep-foundation evaluation, various construction anomalies, and defects encountered while testing the drilled shafts. The paper also discusses the remedial measures developed and implemented to repair the defects.

DESIGN SUMMARY

The bridge foundations for the VA-288 project were designed using the same design methodology to maintain continuity in design, expedite construction, and provide consistency. Most project bridges were typical road crossings, wherein the abutments were typically supported on driven H-piles while the piers were supported on drilled shafts. This system was implemented in 20 of 23 bridges. Figure 2 presents the plan and profile for a typical bridge foundation.

Driven Steel H-Piles

The entire project utilized driven 12x53, 50 ksi steel H-piles for several reasons. Use of the same piles allowed contractors to utilize one pile-driving hammer for all piles. Scheduling contractors was more versatile, as they could move their hammer and piles to any bridge without special orders. For design, using one pile type served as a quality-control measure since the designers became familiar with achievable axial and lateral pile capacities. Finally, the field engineers were able to become very familiar with the performance of the pile-driving hammer, which added to quality control during construction.

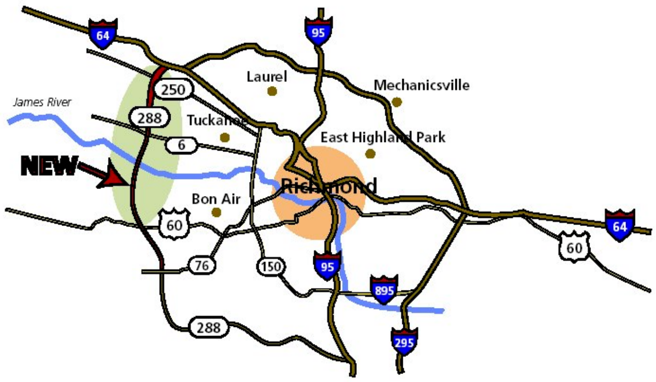

Fig. 1. Site location map

dimensional modeling of the abutment with the piles in their actual design location and orientation. Based on these analyses, it was possible to ensure that the pile-supported abutment could resist the axial and lateral loads.

Some bridge abutments were constructed using Mechanically Stabilized Earth (MSE) walls; H-piles were driven prior to constructing the MSE walls around the piles. Therefore, downdrag forces were induced as a result of the settlement of the MSE-wall backfill material. These downdrag forces were incorporated into the design by including them in the ultimate capacity required during driving.

PDA testing was performed at all bridges on a select number of piles during construction to verify the design; production pile lengths and the final pile-driving criteria were developed based on the PDA testing results.

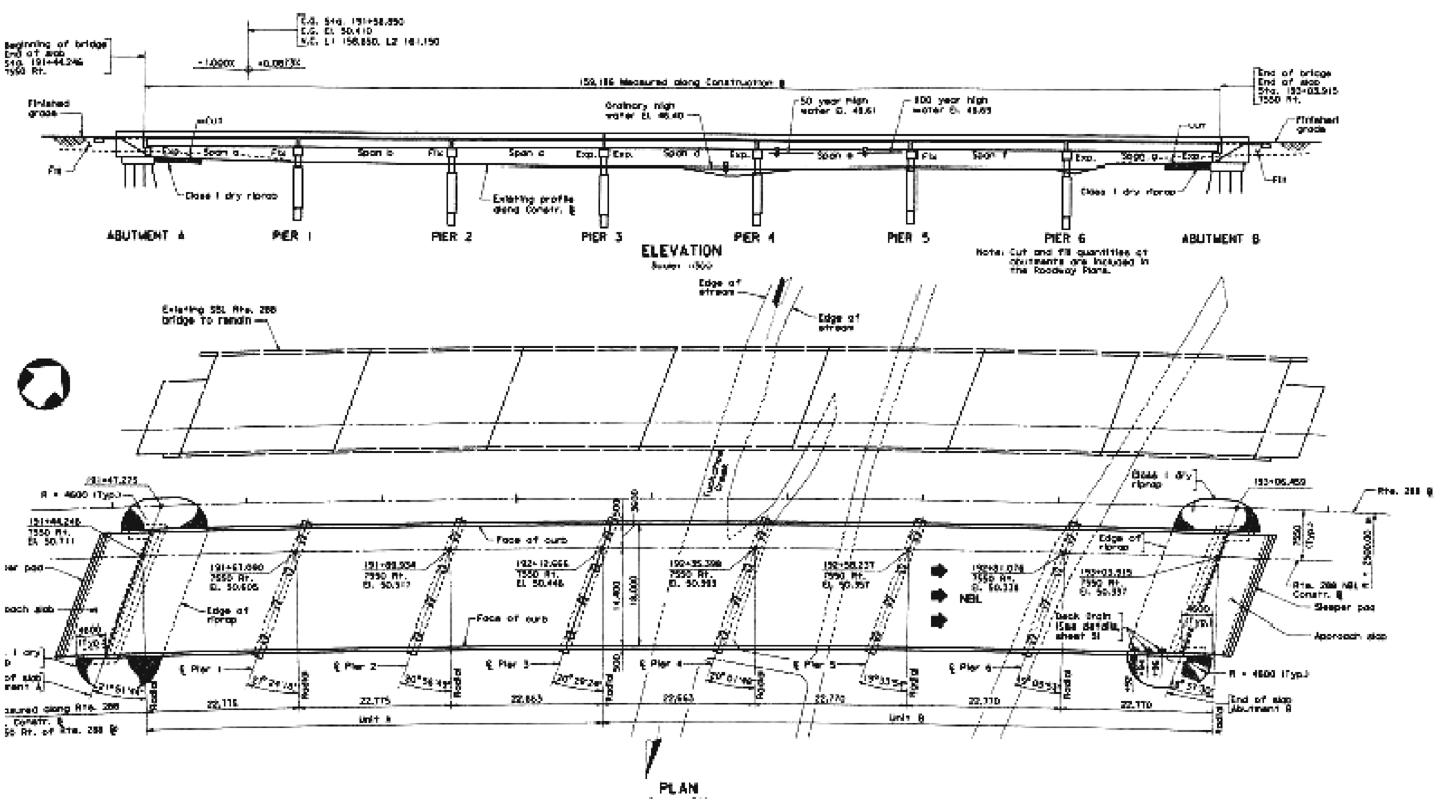

Fig. 2. Plan and profile for a typical VA288 bridge

The FHWA computer program DRIVEN was used to estimate the design lengths of the H-piles, based on the bridge loads provided by the structural engineer. The GRLWEAP computer program was then used to verify that the piles could be driven to design depths with the proposed hammer without damage from high compressive stresses or reaching refusal. The entire abutment's overall stability was assessed using the FB-Pier computer program, which permitted three-

Drilled Shafts

The SHAFT computer program developed by Ensoft was used to design drilled shafts for axial loading based on the pier loads provided by the structural engineer. The FB-Pier computer program was used to model drilled-shaft-supported piers in three dimensions to assess the piers' overall stability. Drilled shafts were either socketed into rock or designed as a combination of skin-friction and end-bearing shafts into soils and weathered rock. The rock sockets were inspected to verify that cutting spoils were thoroughly cleaned from the sides and bottom either by hand in dry excavations, or by vacuum in wet ones.

Groundwater was typically encountered in the overburden material above the top of rock. This condition created the likelihood that drilled shaft excavations in the overburden material would not be capable of free standing. Therefore, permanent or temporary steel casings were used to maintain an open hole and facilitate installation of the drilled shafts. Permanent casings used were typically seated into the top of rock and included into the shaft design.

Since good construction methods are required to install a quality drilled shaft below the groundwater table, demonstration shafts were required to be constructed at each bridge location. The construction methods used in the demonstration shafts were assessed and then Crosshole Sonic Logging (CSL) non-destructive testing was used to verify the shaft's final integrity. Once the demonstration shaft was approved, the constructor proceeded with the installation of the production shafts.

All drilled shafts for the VA-288 project were inspected to verify that the same proven techniques used in the demonstration shafts were maintained for the remainder of the shafts. CSL testing was also used in all drilled shafts as a final quality-control measure.

Bridge piers for the VA-288 bridge in question were supported by 48-inch diameter drilled shafts constructed with permanent casings seated into rock with rock sockets ranging from 7 to 18 feet in length. Drilled shafts were designed for a 562-kip capacity and were constructed with 4,350-psi compressive-strength concrete. Due to limited site access and shaft excavations filling with water, concrete was placed by a pump truck with a tremie extension to deliver concrete to the bottom of the shaft.

Figure 3 presents a typical shaft design at this bridge.

QUALITY CONTROL INSPECTION

As discussed above, drilled shaft foundations were determined most suitable for the larger bridge structures. In keeping with good general foundation design procedures as well as VDOT practices, post-construction evaluation of the drilled shafts was considered desirable. Such inspections allowed for the design loads and therefore the number of drilled shafts per bridge bent to be optimized for the soil conditions at each bridge location.

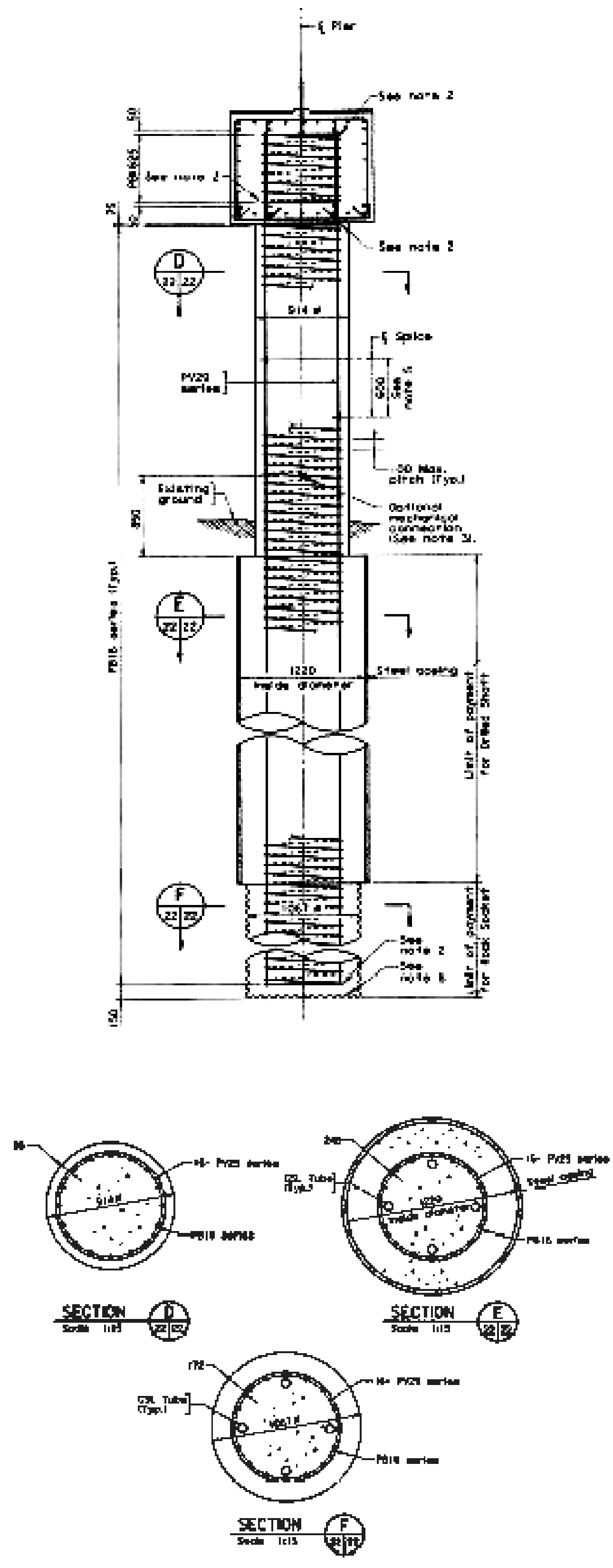

Fig. 3. Typical Drilled Shaft Design Details

Pile Integrity Testing (PIT) and Crosshole Sonic Logging (CSL) were considered for the post-construction evaluations of the drilled shafts. PIT testing is performed by striking the shaft top with a small hand-held hammer and measuring the reflected stress waves using a small accelerometer mounted at the shaft top. The reflected stress wave will be influenced by changes in the cross-sectional area or concrete modulus of the shaft. Where the shaft diameter is decreased or the concrete modulus reduced, the reflected wave will increase in velocity. Since the design and construction of the drilled shafts for this project contained planned changes in cross-section and the length of shafts was expected to be relatively short, it was determined that PIT testing of these shafts would yield results that were difficult to interpret. In addition, all of the designed shafts would include a significant amount of end-bearing resistance and the toe condition was considered critical for evaluation. Using PIT testing to determine the toe conditions can also be very difficult. For these reasons, CSL testing was selected to provide the post-construction evaluation for all shafts and PIT testing would be performed only where unusual conditions or results were obtained from the CSL testing or from visual shaft inspection.

Crosshole Sonic Logging (CSL) is performed in drilled shafts once the shaft has been drilled and concrete poured. CSL testing requires that access tubes be tied to the interior of the shaft reinforcing steel at selected intervals. Usually one tube per foot diameter is used to provide sufficient coverage of the shaft cross-section. For the VA-288 project, shafts were provided with 4 to 6 CSL access tubes with shaft diameters ranging from 4 to 6.5 feet. The access tubes are filled with water either just before or after concrete placement to prevent debonding between the access tube and shaft concrete, as well as to provide a transmission medium for the ultrasonic signal. CSL testing may be performed after an appropriate curing time, usually 3 to 7 days.

CSL testing is conducted by lowering transmitter and receiver probes down separate tubes and raising them from the shaft bottom to the top of the access tubes. The transmitter probe emits an ultrasonic signal across the shaft concrete to the receiver probe signal at 2-inch increments along the tubes. The probes are maintained at the same elevation to maintain a constant distance between the sensors throughout the test. A log of the shaft is then produced for each pair of access tubes. Typically, testing is performed for all perimeter tube pairs and the major diagonals to develop shaft profiles. Therefore, six profiles are performed for a shaft with four access tubes to fully assess the shaft's integrity. Figure 4 shows the typical CSL testing setup using the Crosshole Analyzer (CHA) manufactured by Pile Dynamics, Inc.

CSL results are plotted for each profile performed. The results include the historical "waterfall diagram," which presents results in a binary fashion where positive signal components are displayed and negative or unreceived records are not. The waterfall diagram is an intuitively clear representation of concrete quality over depth, but does not provide sufficient detailed information where marginal results are obtained. For these situations, the first arrival time (FAT) and relative signal-energy plots are more informative. The FAT plot is a single line plot showing the arrival time of the CSL signal over the length of the shaft tested. For the CHA system the FAT may be selected either manually or by setting absolute and relative thresholds. The relative threshold is relative to the maximum signal received for the individual profile.

The signal strength can also be used to analyze CSL test data to evaluate shaft integrity. The signal strength is evaluated by digital integration over time of the absolute value of the signal.The duration of the signal integration is typically around 10 to 20 samples. The result of this integration is called the signal energy. There are no absolute values of energy that can be used for concrete quality assessment; however, a local relative reduction of energy by more than a factor of 10 usually indicates a serious reduction in concrete quality.

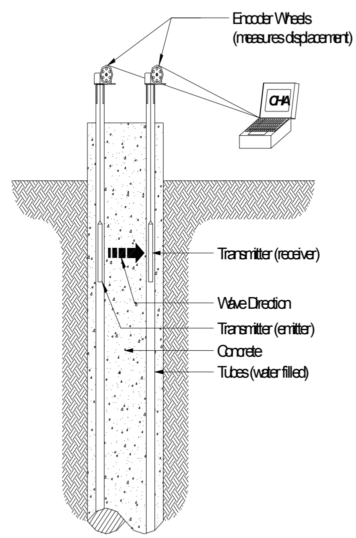

Fig. 4. Typical crosshole setup

CSL TESTING RESULTS

As stated above, all the drilled shafts for the VA-288 project were subjected to post-construction quality control testing using the Crosshole Analyzer (CHA) manufactured by Pile Dynamics, Inc. Considering that the project consisted of approximately 150 drilled shafts, the amount of testing was considerable. In order to make CSL testing more cost-efficient, the testing was usually only performed when a large number of shafts were ready to be tested. As such, the CSL testing took place anywhere between approximately 5 and 30 days after completion of the drilled shafts. Since only steel tubing was used for the access tubes, the duration between concrete placement and CSL testing could be extended beyond the 10- to 14-day limit that is often specified due to debonding concerns associated with the use of PVC tubing.

CSL results indicated that the vast majority of the drilled shafts were of high quality and integrity. However, approximately 10 percent of the drilled shafts indicated some sort of problem with the shaft concrete. Of these, two drilled shafts indicated a significant defect in the middle. Figure 5 presents the CSL results for one profile selected from one of these two shafts. As the figure indicates, a complete loss of the CSL signal was indicated at a depth between 6.5 and 8 feet. Although only one profile is presented here, the results for the other five profiles performed for this shaft were nearly identical. Such results clearly demonstrate a significant deficiency in the drill shaft concrete between these depths.

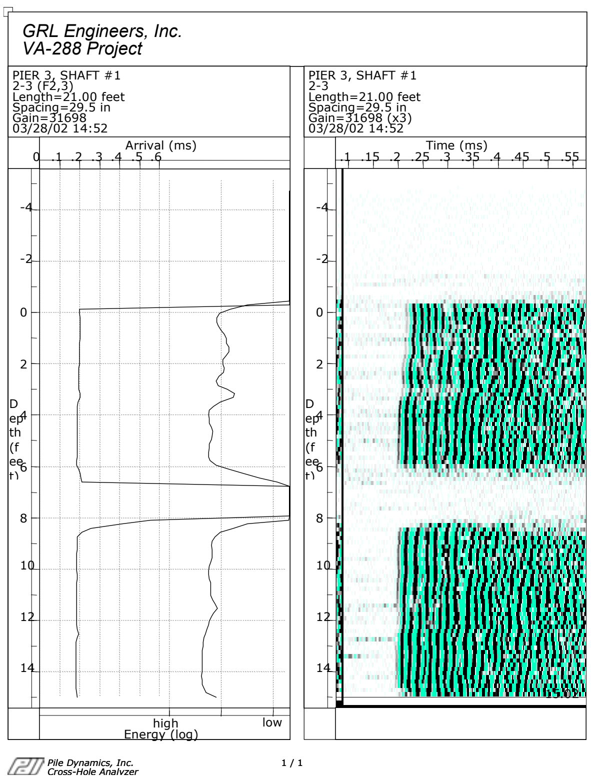

Fig. 5. CSL results for shaft defect in middle of shaft

In addition to the above results, a defect was indicated near the shaft bottom. Approximately four shafts at one bridge location were identified to have CSL results indicating a so-called “soft toe.” Such results are indicated by delayed signal-arrival time or loss of signal at the shaft bottom. Figure 6 illustrates the CSL results from one profile of one of these shafts. As indicated in the figure the CSL signal arrival time is first delayed and then completely lost beginning approximately 2 feet above the shaft bottom. Variations in the results for the four shafts indicated that this “soft toe” condition only existed in two or three of the six profiles. Finally, approximately nine shafts were identified as having a significantly lower CSL signal energy and/or a minor delay in the CSL signal arrival time near the shaft top.

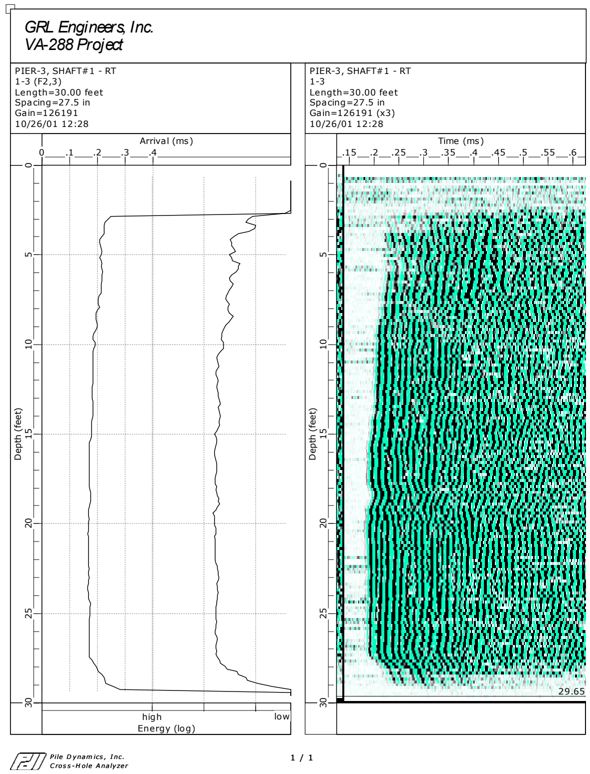

Fig. 6. CSL results for shaft defect at shaft toe

Figure 7 presents the results for one of these shafts. As indicated the CSL energy is clearly reduced over the top 3 feet of the shaft. In addition, the CSL signal arrival time is slightly delayed over portions of this area. Such results appear to indicate a variation in concrete quality between the upper shaft concrete and that present over the lower portion of the shaft. In general, results such as these were encountered only for the major diagonal profiles and where shaft diameters of 6 feet or greater were used. As such, it appears that the results may have resulted from the partial debonding between the steel access tubes and the shaft concrete. Debonding is described as a separation between the shaft concrete and the access tube resulting in a small air gap. Such a gap will prevent or degrade the transmission of the ultrasonic signal from one access tube to the next. Where CSL results indicated this condition, either PIT testing or core samples were collected from the shaft top to further evaluate the shaft integrity. Unconfined compressive tests were performed on core samples obtained from the shaft top to assess the concrete strength.

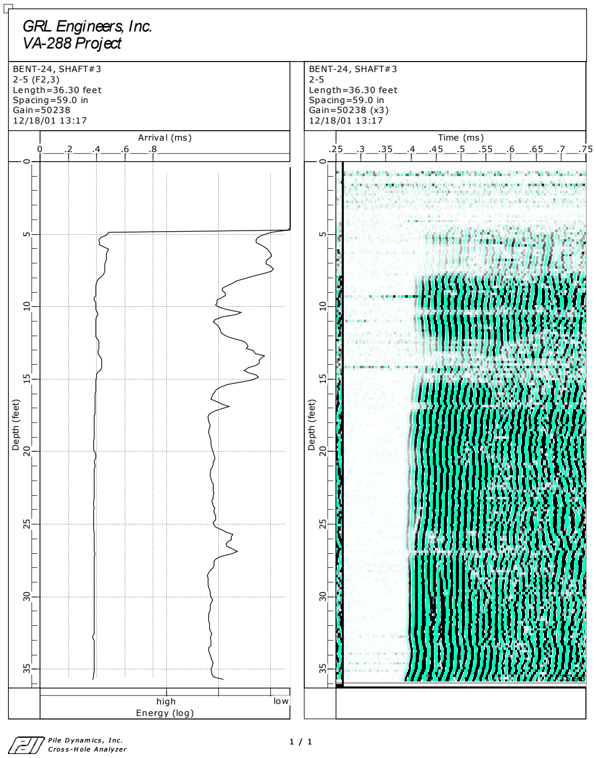

Fig. 7. CSL results for shaft with poor quality concrete at top

CONFIRMATION CORING AND POTENTIAL CAUSES OF SHAFT DEFECTS

Several shafts were selected to be cored to confirm the results of the CSL testing. Coring was performed using an NX-size diamond core bit to retrieve samples for testing in the lab. Typically, three cores were drilled to 3 feet past the defective zone as identified from the CSL testing. Unconfined compression tests were performed on the core samples and the results are summarized in Table 1. The results indicate that the concrete strength in the defective zones varied between 661 and 1,337 psi. The design concrete strength was 4,350 psi. These test results confirmed the CSL test finding. The shaft's vertical and lateral capacity were reanalyzed to assess the shaft capacity using the lowered concrete strength. The results indicated that the shaft capacity would not be adequate to support the structural loads imposed, with an adequate safety factor. Therefore, the shaft defects had to be repaired.

Table 1. Unconfined Compression Test Results

Depth (feet)	Unconfined Compressive Strength (psi)
<15	5828
15 to 16	1178
15 to 16	661
15 to 16	1337

The causes of the defects were investigated to avoid the installation of additional defective shafts during the completion of the bridge foundations. The potential causes of the shaft defects were categorized by the location of the defects in the shafts:

1. At the top of the shafts
2. At the middle of the shafts
3. At the bottom of the shafts

Defects at the top of the shaft were caused either by defective concrete or by soil/spoil contamination that may not have been thoroughly removed during construction. Inadequate over-pouring may have been the cause or inadequate soil/spoil removal. Although, the testing performed on concrete samples indicated the concrete strength exceeded the design strength (4,350 psi), the concrete supplier was changed since large gravel were identified during the concrete placement which caused the pump lines to clog and interrupt the concrete placement.

A single shaft had a most puzzling defect at the middle, since the shaft was installed using steel casing to the top of rock. The defect was encountered approximately 3 feet above the top of rock and within the steel casing. Installation records indicate that the defect was encountered at a depth that coincided with the end of pumping from one concrete truck and the start of pumping from another. Two possible causes were debated; the first was defective concrete and the second was from pulling the pump line too close to the top of concrete. Typically 5 to 7 feet of concrete head are required to maintain the concrete flowing without being contaminated with the spoil floating on top of the concrete.

Inadequate cleaning of the bottom most likely caused the shaft defects encountered at the bottom of the shafts. The rock and the weathered rock are clay based and when they were mixed with the water the fines were suspended in the water. If enough time elapsed from completion of the drilling to the concrete placement, the suspended sediments will settle to the bottom of the shaft causing the bottom to be soft as shown in Figure 7.

REMEDIAL ACTIONS TO FIX THE DEFECTIVE SHAFTS

The remediation program is summarized as follows:

1. Drill six 4-inch diameter holes in the defective shafts using air-track drilling equipment. No samples were recovered, however the drillers were able to indicate that the bottom of the shafts contained softer material.
2. Drilling depths extended approximately 1-foot beyond the bottom of the defective zone as identified from the CSL testing.

3. The drill holes served as access points for cleaning and removing material from the deficient zones.
4. Upon the completion of the drilling, each core hole was water blasted at 10,000 psi to break up inferior concrete.
5. Cuttings and concrete debris were then vacuumed from the deficient zone using a vacuum truck.
6. The hole was inspected using a microcamera to ensure that the defective concrete was removed. Figure 8 shows a picture of intact concrete while Figure 9 shows a picture of the defective zones.
7. A 1.5-inch diameter steel pipe was installed through one of the holes to serve as a grout port.
8. The voids were pressure grouted through the grout port. The pressure was limited to 200 psi.
9. Seven days after the completion of the grout operation, the defective shafts were retested using CSL testing, the results of which appear in Figure 10.

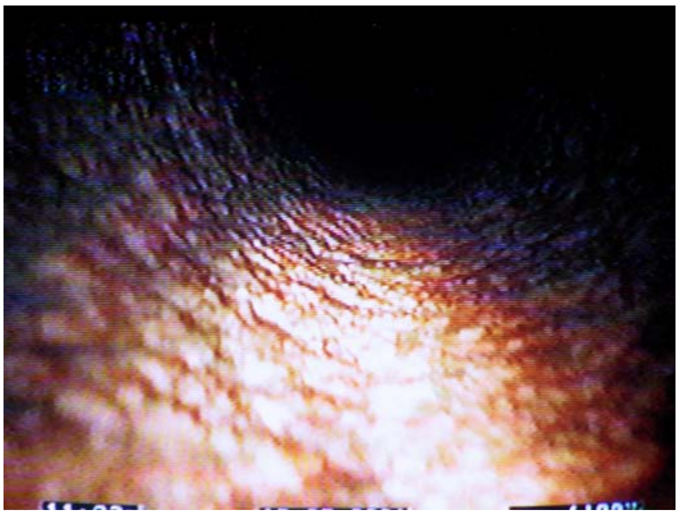

Fig. 8. Picture of intact concrete with microcamera

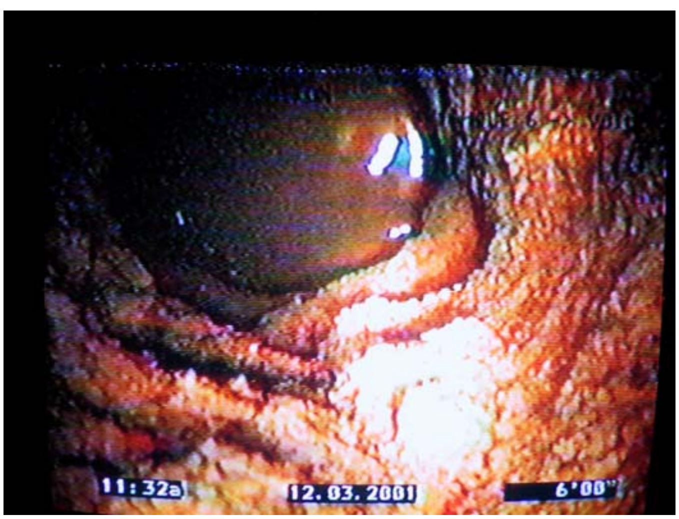

Fig. 9. Picture of defective concrete with microcamera

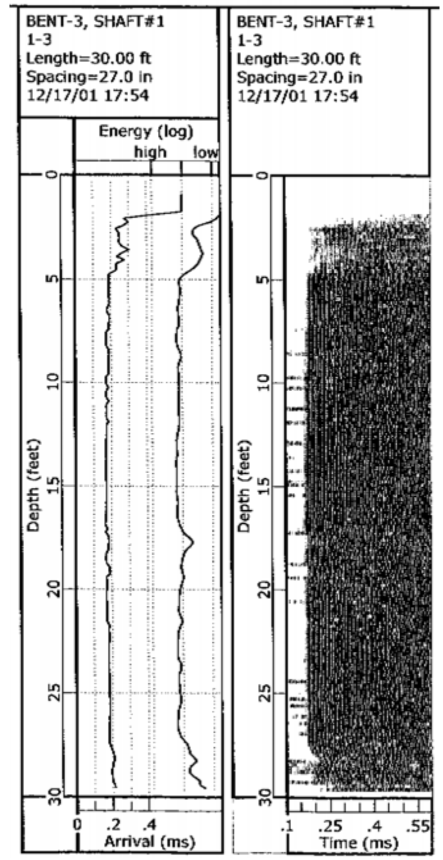

Fig. 10. CSL results for repaired shaft

CONCLUSIONS

- Using CSL testing to provide quality control is a valuable tool and should especially be implemented if there is no redundancy in the foundation system.
- It is crucial to have a qualified testing agency interpret the test results. The results will have to be evaluated in light of the applied loads, foundation system, redundancy, and many other factors.
- During the installation of the drilled shafts, special attention should be given to the cleaning of the shaft bottom and concrete placement.
- During the placement of the concrete, if the pumplines are pulled too close to the top of the concrete, it may be more cost effective to remove the placed concrete, clean the shaft and restart the concrete placement, than having to fix defective drilled shafts.
- Fixing drilled shafts is possible but costly.