

Case History: Foundation Evaluation for the Virginia Highway 288 Project

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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 27.4 km (17 miles) of new highway with 23 bridges and overpasses. The fast-track, design-build process required careful evaluation of bridge design and construction to determine the most cost-effective approach. Figure 1 presents the Site Location Map.

In keeping with the design-build process, bridge foundations were varied depending upon the bridge length and 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. Extensive foundation testing was required.

Pile Driving Analyzer (PDA) testing was proposed for driven-pile foundations to confirm the ultimate pile capacity, evaluate driving stresses and hammer performance, and establish driving criteria. PDA testing was performed at all bridge locations where piles were used. Pile-driving criteria were established using CAPWAP and GRLWEAP analyses, allowing for the most efficient pile installations. Finally, PDA testing provided further evaluations of pile performance and suitability when unusual situations were encountered.



Fig. 1. Site location map

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The overall quality of constructed drilled shaft foundations was evaluated using Crosshole Sonic Logging (CSL) and Pile Integrity Tests (PIT). CSL using the Crosshole Analyzer (CHA) was performed on each of the project's 120 shafts. Shaft diameters varied from 1.22 to 1.98 m (4.0 to 6.5 feet), with design loads between 2,670 kN (600 kips) and more than 11,120 kN (2,500 kips). Remedial actions were developed and implemented to repair the shafts where CSL results indicated poor quality concrete or defects in the shaft.

This paper presents a case history detailing the benefits of the latest deep-foundation evaluation techniques, and their ability to identify foundation performance or defects in completed drilled shafts. The paper also discusses remedial measures developed and implemented to repair defects.

Design Summary

A consistent design methodology was used for all VA-288 project bridge foundations, which maintained design continuity, expedited construction, and provided consistent quality. Most project bridges were typical road crossings, wherein abutments were 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.

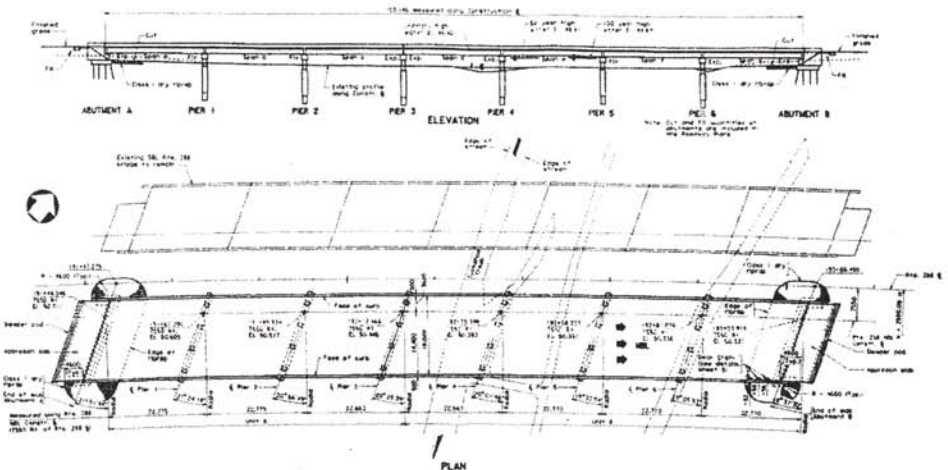


Fig. 2. Plan and profile for a typical VA288 bridge

Driven Steel H-Piles

The entire project used driven HP 310x79 (HP 12x53), 344 MPa (50 ksi) steel H-piles for several reasons. Use of the same piles allowed contractors to use one pile-driving hammer for all piles. Scheduling contractors was more versatile, as the hammer and piles could be moved to any bridge without special orders. For design, using one pile type served as a quality-control measure because the designers became familiar with achievable axial and lateral pile capacities. Finally, the field engineers became very familiar with the pile-driving hammer performance, providing greater quality control during construction.

Using the FHWA computer program DRIVEN, the design lengths of the H-piles were estimated based on the bridge loads provided by the structural engineer. The GRLWEAP computer program provided verification that piles could be driven to design depths with the proposed hammer without damage from high compressive stresses or reaching refusal. The overall stability of the entire abutment was assessed using the FB-Pier computer program, which permitted three-dimensional modeling of the abutment with the piles in actual design locations and orientation. Based on these analyses, it was possible to ensure that the pile-supported abutment could resist 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 MSE-wall backfill material settlement. These downdrag forces were incorporated into the design by including them in the ultimate capacity required during driving.

During construction, PDA testing was performed at all bridges on a select number of piles to verify the design, provide production pile lengths, and develop final pile-driving criteria based on the PDA testing results.

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 modeled 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 either visually or with a sounding rod 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.

Because of groundwater in the overburden material above the top of rock, it was unlikely that drilled shaft excavations in the overburden material would be capable of free standing. Therefore, permanent or temporary steel casings were used to maintain an open hole and facilitate drilled shaft installation. Permanent casings were typically seated into the top of rock and were included in the shaft design.

Since good construction methods are required to install a quality drilled shaft below the groundwater table, construction of demonstration shafts was required at each bridge location. The demonstration shaft construction methods were assessed, and final integrity of the shaft was verified by CSL testing. Once the demonstration shaft was approved, the constructor proceeded with production shaft installation.

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

Bridge piers for the VA-288 bridge in question were supported by 1.22 m (4-foot) diameter drilled shafts constructed with permanent casings seated into rock with rock sockets ranging from 2.13 to 5.49 m (7 to 18 feet) in length, as shown in Figure 3. Drilled shafts were designed for a 2,500 kN (562-kip) capacity and were constructed with 30 MPa (4,350-psi) compressive-strength concrete. Because of 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.

Quality Control Inspection

As discussed previously, 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 required. Such inspections allowed for higher design loads because of a reduced factor of safety and therefore the number of drilled shafts per bridge bent was optimized for the soil conditions at each bridge location.

PIT and CSL were considered for the drilled shaft post-construction evaluations. 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 is influenced by changes in the shaft's cross-sectional area or concrete modulus. Where shaft diameter is decreased or the concrete modulus is reduced, the reflected wave will increase in velocity. Since the drilled shaft design and construction 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 CSL testing or visual shaft inspection indicated unusual conditions.

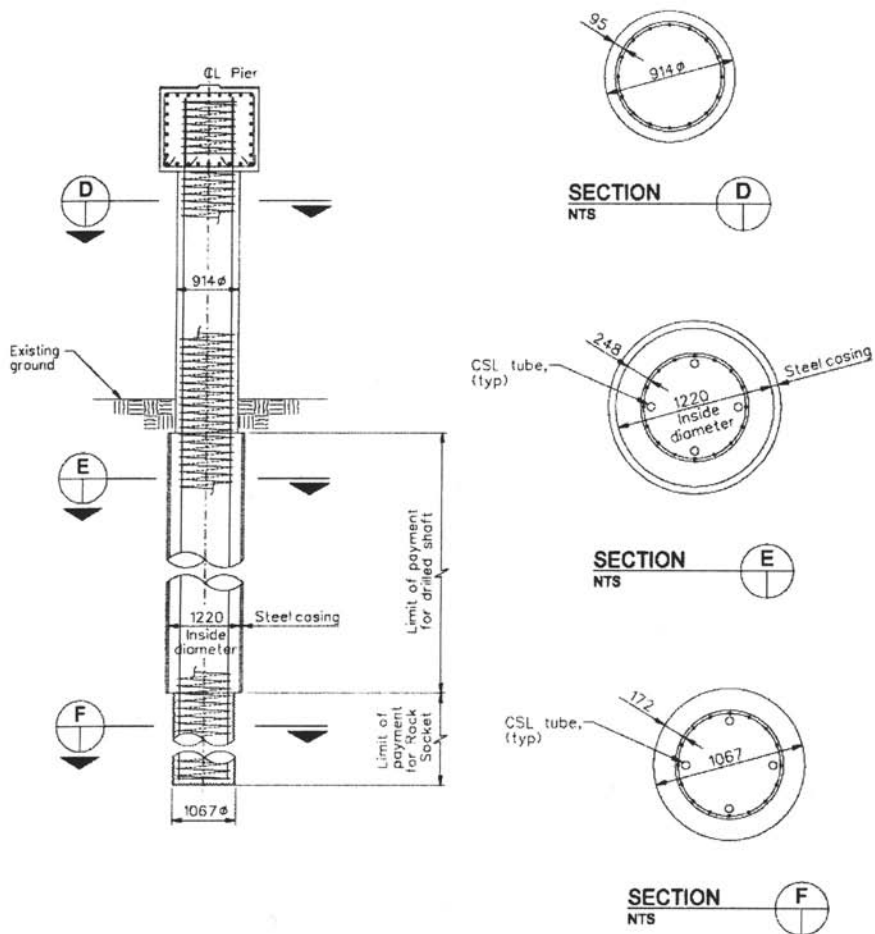


Fig. 3. Typical drilled shaft design details

In drilled shafts, CSL is performed after the shaft has been drilled and the concrete poured. CSL testing requires tying access tubes 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 during testing. For the VA-288 project, shafts were provided with 4 to 6 CSL access tubes with shaft diameters ranging from 1.22 to 1.98 m (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 is typically 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 50 mm (2-inch) increments along the tubes. Probes are maintained at the same elevation to maintain constant distance between sensors throughout the test. A log of the shaft is produced for each pair of access tubes. To develop shaft profiles, testing is typically performed for all perimeter tube pairs and the major diagonals. 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 CHA.

CSL results are plotted for each profile performed. The results include a “waterfall diagram,” as shown in Figure 5, 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, also shown in Figure 5, is a single line plot showing the arrival time of the CSL signal over the length of the tested shaft. 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.

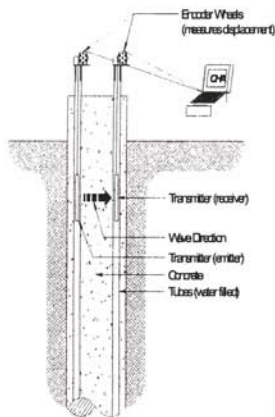


Fig. 4. Typical crosshole setup

CSL Testing Results

As stated previously, all VA-288 project drilled shafts were subjected to post-construction quality control testing using the CHA. 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 because of 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 1.98 and 2.44 m (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 drilled shaft concrete between these depths.

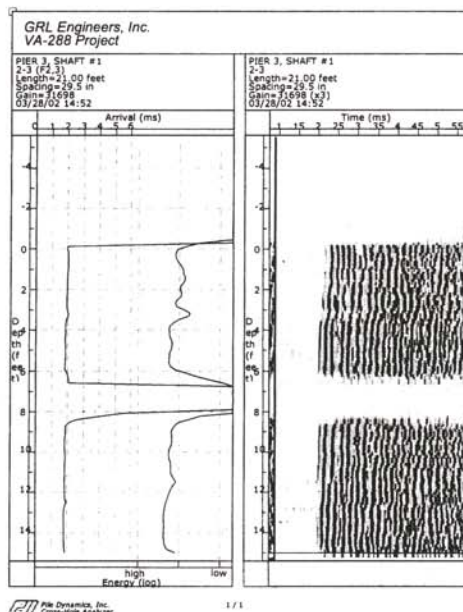


Fig. 5. CSL results for shaft defect in middle of shaft

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 0.6 m (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. Figure 7 presents the results for one of these shafts. As indicated, the CSL energy is clearly reduced over the top 0.9 m (3 feet) of the shaft. In addition, the CSL signal arrival time is slightly delayed over portions of this depth. 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 1.83 m (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 shaft integrity. Unconfined compressive tests were performed on core samples obtained from the shaft top to assess the concrete strength.

Confirmation Coring and Potential Causes of Shaft Defects

Several shafts were selected to be cored to confirm the CSL testing results. Coring was performed using an NX-size diamond core bit to retrieve samples for laboratory testing. Typically, three cores were drilled to 0.9 m (3 feet) past the defective zone identified by 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 4.56 and 9.22 MPa (661 and 1,337 psi). The design concrete strength was 30 MPa (4,350 psi). These test results confirmed the CSL test finding. The shaft’s vertical and lateral capacities 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 imposed structural loads, with an adequate safety factor. Therefore, the shaft defects required repair.

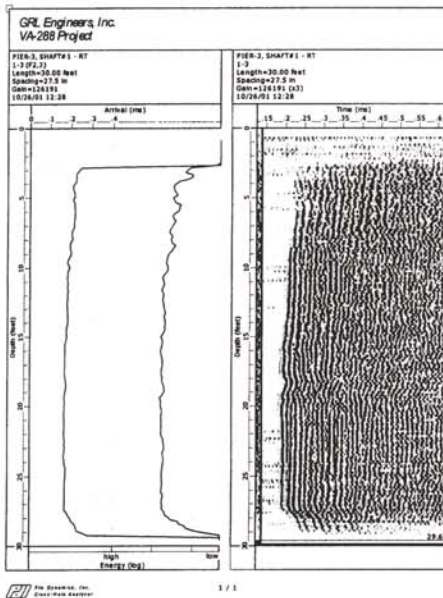


Fig. 6. CSL results for shaft defect at shaft toe

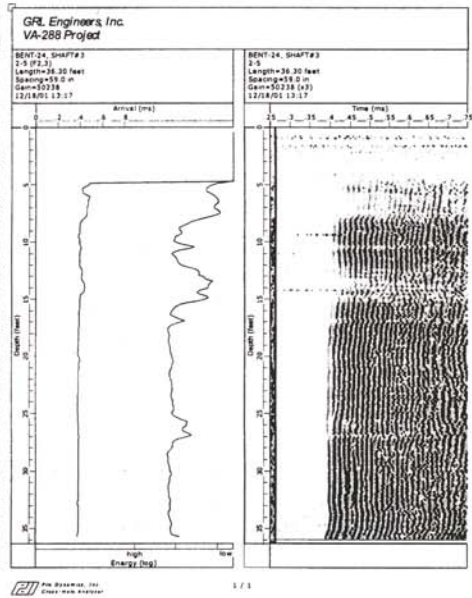


Fig. 7. CSL results for shaft with poor quality concrete at top

Table 1. Unconfined Compression Test Results

Depth m (feet)	Unconfined Compressive Strength MPa (psi)
<4.57 (<15)	40.18 (5,828)
4.57 to 4.88 (15 to 16)	8.12 (1,178)
4.57 to 4.88 (15 to 16)	4.56 (661)
4.57 to 4.88 (15 to 16)	9.22 (1,337)

The causes of the defects were investigated to avoid 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 30 MPa (4,350 psi), the concrete supplier was changed since large gravel, identified during concrete placement, caused the pump lines to clog and interrupt concrete placement.

A single shaft had a defect at the middle, which was puzzling since the shaft was installed using steel casing to the top of rock. The defect was encountered approximately 0.9 m (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 1.52 to 2.13 m (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 shafts. The rock and weathered rock are clay based, and when they were mixed with water the fines were suspended in the water. If enough time elapses from completion of the drilling to concrete placement, 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 100 mm (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 0.3 m (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 completion of drilling, each core hole was water blasted at 69 MPa (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 defective zones.
7. A 38 mm (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 1.38 MPa (200 psi).
9. Seven days after completion of the grout operation, the defective shafts were retested using CSL testing, the results of which appear in Figure 10.

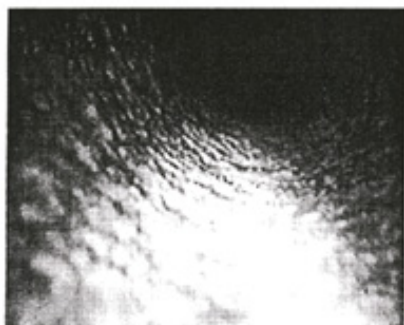


Fig. 8. Picture of intact concrete with microcamera

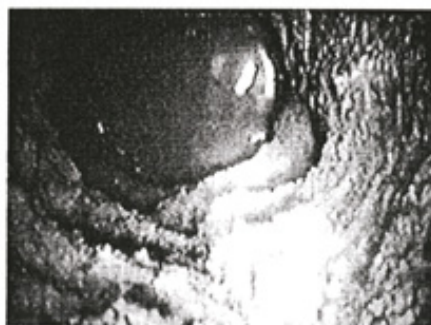


Fig. 9. Picture of defective concrete with microcamera

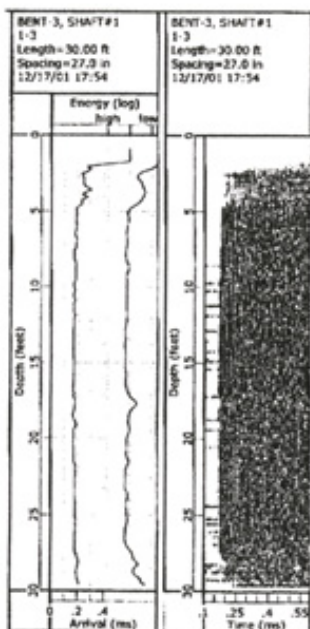


Fig. 10. CSL results for repaired shaft

Conclusions

- PDA testing and CSL testing are both very good tools for assuring installation of deep foundation systems with the expected design quality.
- 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 require evaluation with respect to applied loads, foundation system, redundancy, and many other factors.
- During installation of the drilled shafts, special attention should be given to the cleaning of the shaft bottom and concrete placement.
- During concrete placement, if the pump lines 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 concrete placement, rather than fixing defective drilled shafts.
- Fixing drilled shafts is possible but costly.
- As was shown on this project, shaft defects are still possible, but greatly reduced, with a thorough inspection process.