Cast-in-place Drilled Shafts:
A State of the Art Report

Neil T. Greenlee, P.E.
HNTB Corporation
343 E. Six Forks Rd.
Suite 200
Raleigh, NC  27609

Ph. (919) 546-8997
Fax (919) 546-9421
ABSTRACT

Cast-in-place drilled shafts have lately emerged as a viable railroad bridge foundation alternative that, under certain conditions, offers cost and/or constructability advantages over driven piles. This is due in part to the increasing number of new and replacement railroad underpass grade separations being coordinated through state and municipal DOT’s and their consultants, many of whom routinely utilize drilled shafts on highway bridge projects. Consequently, many railroads are encountering an uncommon foundation type with greater frequency on their bridge projects.

In consideration of the growing number of cast-in-place drilled shafts utilized on railroad bridge projects as well as current technologies related thereto, a comprehensive report on this subject will benefit railroad personnel who oversee public improvement and outsourced engineering design and construction projects. Accordingly, this paper presents a “state-of-the-art” look at this specialized foundation type with emphasis on current industry standards for design, construction, and field testing. Topics of discussion include railroad applicability and cost factors; analytical methods for geotechnical and structural design; construction details and specifications; common construction methodologies; and modern field testing methods for quality control and assurance. The information presented will provide practical, up-to-date guidelines on drilled shaft design and construction for railroad bridge engineers and field inspectors.

Key Words

Drilled shafts, design, specifications, construction, field testing.
INTRODUCTION

Cast-in-place drilled shaft foundations are being used increasingly on railroad bridge construction projects throughout the United States. This is due in part to the growing trend of underpass grade separations being replaced in conjunction with state and municipal DOT highway improvement projects, though several Class 1 railroads use drilled shafts extensively on bridge projects of all types. Another contributing factor is the outsourcing of bridge design contracts by Class 1 railroads to consulting engineering firms that utilize drilled shafts on highway bridge projects and are likely to consider their use on railroad bridge projects. Increased longitudinal design forces for railroad bridges also favor drilled shafts because of their inherent ability to withstand corresponding increases in design lateral loads and overturning moments at the foundation level.

The intent of this paper, therefore, is to provide a brief survey of modern drilled shaft technology along with practical guidelines for drilled shaft design and construction inspection. Accordingly, this report addresses the applicability of drilled shaft foundations in the railroad industry along with general cost factors, and describes modern design, construction and field testing procedures commonly associated with this foundation type.

GENERAL APPLICABILITY AND COST CONSIDERATIONS

Under certain conditions, drilled shafts can compete with driven piles in a railroad construction environment from both a cost and constructability standpoint. For example, foundation excavations for a new underpass structure constructed on permanently realigned track immediately adjacent to an existing underpass structure often requires extensive use of temporary railroad shoring. Drilled shafts can be constructed in this case with little or no temporary shoring (Figure 1), and the cost of such shoring often exceeds the cost differential between the shafts and driven piles. Drilled shafts are also appropriate for on-line bridge replacements, since they can be constructed immediately adjacent to live tracks with minimal or no disruption to scheduled train traffic depending on the operational characteristics of the railroad in question. Furthermore, drilled shafts can withstand very high levels of combined vertical and lateral forces, thus making them particularly useful in cases where longer bridge spans result in substantial amounts of design longitudinal force.

In addition to site and loading conditions, certain subsurface conditions may favor the use of drilled shafts as opposed to driven piles. For example, in softer soils or in cases where thick layers of “muck” overlie stiffer soils, permanently uncased drilled shafts are advantageous because they can mobilize large amounts of skin friction in
addition to end bearing resistance. In the case of stiffer soils, large diameter drilled shafts may be less costly to install than multiple rows of driven piles, especially if the latter must be battered and pre-drilled. In cases where artesian waters are trapped in confining soil layers through which piles will be driven, the resultant artesian releases will aggravate the ability to confirm specified pile capacities and will be of great concern to environmental agencies with jurisdiction over the project. Drilled shafts are particularly advantageous in these conditions because, in contrast to the special (i.e. costly) environmental and soil stability measures that driven piles would require, they can be constructed with methods that are germane to this foundation type. Further discussion of subsurface related design and construction issues will occur later in this paper, so it is sufficient here to say that drilled shafts are applicable in a variety of subsurface conditions, and their use should be considered accordingly.

Foundation construction costs will, of course, depend on project specific design criteria and physical parameters. Accordingly, drilled shaft plans and specifications must be practicable from a construction standpoint while thoroughly addressing project design criteria. Some common factors influencing drilled shaft costs include the aforementioned subsurface conditions, environmental protection requirements, drilling and concrete placement methods and time limits, and field testing requirements. In addition, individual railroad operating conditions and temporary track closure requirements are cost affecting factors, and will vary from railroad to railroad. Well crafted plans and specifications that appropriately address all of these issues will enable contractors and inspectors to achieve a high quality end product for the most reasonable cost.

**DESIGN CONSIDERATIONS**

Proper design of drilled shaft foundations requires a well coordinated effort between the geotechnical and structural engineering disciplines. This section describes the salient aspects of each discipline as they relate to analysis and design of drilled shafts.

**Geotechnical Analysis & Design**

Drilled shaft sizes and depths are governed primarily by subsurface conditions and design loads, the latter of which is primarily a function of span length. Economy of construction is achieved by minimizing the diameter, length and number of drilled shafts required for the horizontal and vertical design loads to which they are subjected. As previously discussed, the dominant horizontal load is longitudinal force which, when combined in particular with
vertical live load, has the potential to produce very large out-of-plane design moments and displacements in the supporting piers. Achieving the aforementioned economy under such conditions requires a practical understanding of relevant geotechnical design issues as well as judicious application of the inherent load resisting characteristics of drilled shafts.

In stiffer soils, the primary vertical load resistance mechanism of a drilled shaft will be end bearing at its design tip elevation. If the soil is “hard” enough, the shaft diameter will be solely dependent on the soil bearing capacity at the shaft tip. In softer soils, however, end bearing may need to be supplemented by skin friction in order to achieve the required vertical resistance of the shaft without making the shaft diameter excessively large. Skin friction could, in fact, be the majority component of overall vertical shaft resistance in soils that are incapable of providing enough end bearing capacity for vertical loads. Thus, shaft capacity is directly proportional to shaft diameter in both stiff and soft soils, but in the latter case greater depths will normally be required in order to mobilize enough skin friction to keep shaft diameters to a reasonable minimum. In this regard, the impact of temporary and/or permanent steel casing on the load carrying capacity of drilled shafts is of critical importance in the design of drilled shafts.

The most important point to make within the context of this discussion is that, in an uncased drilled shaft, the embedded vertical surface area of the completed shaft will be in direct, permanent contact with residual soils. This results in a condition that generates high amounts of skin friction depending on the properties of the surrounding soils. The combination of diameter and embedment in uncased drilled shafts, therefore, can be manipulated to develop the required amount of skin friction and end bearing resistance in softer soils, as long as they are fairly stable and/or impermeable. On the other hand, if smooth wall, permanent steel casing is used, skin friction resistance in the completed shaft can be diminished to a great degree. Similarly, temporary steel casing used to stabilize an excavation prior to concreting could effectively “smooth” the walls of the excavation when it is extracted during concrete placement. Therefore, since skin friction resistance must be maximized to produce smaller diameter and shorter shafts for a given axial load, it is generally prudent not to use casing if at all possible in order to achieve the most cost efficient design. Conversely, if temporary or permanent steel casing must be used, the soil parameters upon which skin friction resistance is based should be adjusted as appropriate.

Economy of design has so far been discussed only in light of required vertical capacity of drilled shafts. However, vertical and lateral shaft deformations must be considered in tandem with soil resistance parameters in order to arrive at a design solution that completely addresses the combined effects of vertical and lateral loads. Under these
combined load conditions, the shaft diameters and depths that would be required solely for resistance to vertical loads are nearly always increased, often significantly. Diameter and depth are particularly sensitive to the combined effect of vertical and lateral loads, and the recent adaptation of increased longitudinal forces in the AREMA bridge design guidelines has elevated this sensitivity to a much higher degree. As with any foundation type, geotechnical engineers should be cognizant of the impact of vertical and lateral deflections on the structure as a whole when making foundation recommendation for drilled shafts. This will allow the most appropriate combination of physical design parameters to be selected by the structural engineer for final design of foundations and substructures involving drilled shafts.

**Structural Analysis & Design**

Generally speaking, drilled shafts are most efficiently utilized by transitioning them directly into a single row of above grade columns that frame into the bottom of a substructure pier cap. This approach is especially applicable in the aforementioned situations where footing cap excavations would require extensive amounts of temporary railroad shoring (Figure 1), but it also holds true for on-line bridge replacements where the shafts can be installed just outside the footprint of the existing structure (Figure 2). In these direct transitioning situations, drilled shafts are generally spaced at least three diameters (i.e. 3*D) on center and poured either continuously to the bottom of the pier cap or in segments with a construction joint at the shaft-to-column transition point just below grade. Specific project conditions and requirements will dictate the most appropriate approach.

From a structural engineering standpoint, drilled shafts so utilized must be analyzed and designed as “compression members with flexure” per the AREMA Manual for Railway Engineering. As previously discussed, flexural loads and displacements resulting from current AREMA longitudinal design forces can easily dominate the design of drilled shaft foundations, depending on site conditions and the overall structural modeling approach. Accordingly, it is appropriate to briefly describe two of the most common structural modeling approaches used for analysis and design of drilled shafts.

The first modeling approach is the “point-of-fixity” method. In this approach, an isolated drilled shaft is modeled in a program such as L-PILE along with its approximated structure loads and properties of the surrounding soil. The behavior of the drilled shaft under these conditions is examined, and a theoretical “fixity” elevation is
determined based on the magnitudes of the lateral deflections. The drilled shaft is then presumed to be fully fixed at this elevation in the final structural model. Some key characteristics of this approach are as follows:

- **Implicit soil-structure interaction.** Soil properties are implicitly represented in the structural model via the elevation of the theoretical fixity point.

- **Iterative methodology.** The initial fixity point is assumed based on preliminary loading conditions, then verified and adjusted by the geotechnical engineer based on the outcome of initial and subsequent system analyses performed by the structural engineer.

- **Analytical simplicity.** Design axial loads and moments can be computed manually or by using commercially available software familiar to most structural engineers, such as GTSTRUDL and STAAD.

- **Conservative support reactions.** Frame model support reactions are generally exaggerated with theoretical fixity points, resulting in somewhat decreased internal forces elsewhere in the model.

The second modeling approach is the “soil-spring” method. In this approach, incremental springs are placed along the full length of the drilled shaft to simulate the surrounding soil resistance. This allows the drilled shaft to deflect and rotate along its entire length as opposed to having its movement absolutely restrained at some theoretical, predetermined depth of fixity. Key characteristics of this approach are as follows:

- **Explicit soil-structure interaction.** Soil properties are explicitly modeled via incremental soil springs.

- **Analytical complexity.** Structural models are more complex and difficult to develop and adjust if necessary, especially if shaft depths increase and/or refinement of soil spring increments is required.

- **Non-conservative support reactions.** Results more closely approximate actual behavior, provided subsurface conditions and global structural boundary conditions are properly modeled.

Despite its iterative nature, the author favors the point-of-fixity approach because of its relative analytical simplicity. Both approaches, however, are valid, rational techniques that yield reasonable results for design when properly employed by geotechnical and structural engineers. A schematic representation of both methods is shown in Figure 3.

An important aspect of this discussion is the distinction between a “free-head” and a “fixed-head” shaft within the context of geotechnical and structural modeling. A “free-head” shaft (i.e. a “flagpole”) is one in which the top of the shaft is fully free to rotate and deflect in response to external loads. This condition is generally applicable when considering out-of-plane behavior of a drilled shaft relative to the entire pier (i.e. global FZ and MX per the coordinate
system shown in Figure 3). In contrast, the top of a “fixed-head” shaft is rotationally restrained by virtue of its framed attachment to another structural member such as a cap beam or footing. This condition is generally applicable when considering in-plane behavior of a drilled shaft relative to the entire pier (i.e. global FX and MZ per the coordinate system shown in Figure 3). Since both of these boundary conditions are idealized and seldom exist in the real world, analytical results obtained with either approach will always be approximate to some degree, regardless of whether a “point-of-fixity” or “soil-spring” approach is used. Consequently, overall project complexity, the nature of subsurface conditions, and the magnitude of longitudinal loads relative to vertical loads may warrant a more refined modeling approach that accounts for non-linear behavior of the soil as well as the drilled shaft. Furthermore, the out-of-plane response at each pier might possibly be minimized by distributing the total longitudinal design load across the structure, taking into account the relative pier stiffnesses and the passive resistance of the embankment behind the abutment backwalls. This approach may offer some relief to designers who are wrestling with the combined effects of vertical and longitudinal loads on their bridge projects.

After the most appropriate analytical approach for the drilled shafts has been identified and utilized, the resultant member forces are used to design the drilled shaft reinforcing per the applicable Load Factor Design requirements of AREMA Chapter 8. In order to use the larger compression strength reduction factor of $\phi = 0.75$ in design, containment reinforcing must be continuous spirals conforming to the reinforcing ratio limits set forth in AREMA Chapter 8, Article 2.11.2a. In larger diameter (i.e. roughly 4′-6″ and greater) drilled shafts, this criteria cannot be met without reducing the spiral pitch to a level that restricts the free flow of fresh concrete through the rebar cage. Therefore, unless seismic requirements dictate otherwise, designers should consider using individual ties for drilled shaft containment reinforcing along with the appropriate strength reduction factor of $\phi = 0.70$. Other options would be to use continuous spirals with an oversized pitch and design the shaft as a tied column (i.e. $\phi = 0.70$), or design and detail the shaft with individual ties and allow spiral substitution. A primary consideration here is that spirals cost more than lateral ties, but a spirally reinforced rebar cage is generally more stable and easier to handle than a tied cage. In either case, the pitch of a spiral so used should be large enough to permit unrestricted flow of fresh concrete as previously mentioned.

A final point worth making in regard to drilled shaft design is that some designers advocate reducing the design diameter of uncased drilled shafts, effectively ignoring an outer portion of the hardened concrete in contact with residual soils. This approach assumes that this concrete is compromised by soil intrusion or uneven excavation walls
and should therefore be neglected in design. This assertion has some validity, but neglecting more than 1.5 to 2 inches of concrete in the uncased condition (assuming the normal case of 5 to 6 inches of total concrete cover in drilled shafts) may be unnecessarily conservative.

CONSTRUCTION AND TESTING

Construction of high quality drilled shafts requires efficient implementation of a number of coordinated activities. Since visual inspection of a completed drilled shaft is not possible, it is important to have a well-organized plan for these construction activities as well as an economical, reliable means of ascertaining the overall quality of the completed drilled shafts. Accordingly, this section will briefly discuss specifications, construction techniques and field testing methodologies commonly employed in today’s drilled shaft industry.

Specifications

A comprehensive, practicable drilled shaft specification is essential for promoting a well executed program of drilled shaft construction and testing, especially in the uniquely challenging construction environment of an active railroad corridor. Local state DOT specifications are usually good sources for specifications that can be readily modified for use on railroad bridge projects. In any event, whether a DOT specification or one from another source is used, there are a number of issues—some of a general nature and some relating specifically to railroads—to consider when setting up drilled shaft specifications for a given project, as follows:

- **Effect of train vibrations.** Even if soil parameters indicate that casing is unnecessary, temporary casing should be seriously considered when drilled shafts are in close proximity to an active track.

- **Track time.** Mobile equipment can be temporarily moved so as not to foul an active track, but splicing of permanent casing during its installation or continuous extraction of temporary casing during a concrete pour may require more careful coordination with local train operations depending on the amount of traffic on the line. For this reason, project specifications should explicitly address the number of trains per day in the project area as well as provisions for obtaining extended track time if necessary.

- **Foundation undermining.** Existing bridge piers immediately adjacent to wholly or partially uncased excavations could potentially be undermined by collapsing of nearby shaft excavations. Careful
monitoring and sound construction practices will generally mitigate this concern, but if special measures are deemed appropriate they should be so specified.

- **Concrete criteria.** Tremie-placed or pumped concrete should be specified with a slump on the order of 8 to 9 inches as opposed to 5 or 6 inches. Water reducers and/or super-plasticizers utilized for this purpose should be batched along with the mixing water to ensure proper control of the mix. In addition, #67 or #78 coarse aggregate will facilitate better flow of concrete through and around the rebar cage. Concrete placement should progress smoothly and continuously from start to finish so that cold joints, wall collapses or other defects are not introduced in the process, thereby resulting in a sound, monolithic drilled shaft.

- **Standard Penetration Tests (SPT’s).** SPT sampling is sometimes required after excavation and prior to rebar cage placement in order to verify the condition of the bearing material at the shaft tip. This is generally the case when original soil sampling was performed at random locations and/or design tips extend deeper than the depth of the soil sampling. Depending on applicable project factors, SPT sampling could be performed prior to drilling in order to negate its potential impact on construction time limits. This should be clearly stated in the specifications if allowed.

- **“Rock” Excavation.** “Rock” is sometimes contractually defined on the basis of rock auger penetration rate at full crowd force, and in this case a separate pay item for drilling in material so classified is provided. “Crowd force” depends on the type of drill rig being used, so minimum drill rig size should be specified since an undersized drill rig could conceivably encounter greater amounts of “rock” than a larger, more powerful drill rig. The cost implications of this are readily apparent, so specifications should address this issue with clarity when appropriate.

- **Field Testing.** Non-destructive testing (NDT) requirements should be explicitly identified, along with steps to be taken if NDT results are inconclusive or reveal serious defects in a completed drilled shaft. Load testing is generally quite expensive, and should be kept to a practicable minimum if required.

**Construction Techniques**

In theory, construction of drilled shafts is a fairly straightforward process involving drilling a hole in soil, placing a prefabricated rebar cage in the hole, and filling the hole with concrete. In practice, however, this process involves a
host of interrelated tasks that must be properly coordinated in order to ensure production of a high quality drilled shaft. It is most convenient to discuss the basic process within the context of “dry” and “wet” concrete placement conditions, and to subsequently discuss situations in which steel casing is appropriate and its effect on the process.

“Dry” Placement Conditions

“Dry” placement conditions occur when the excavated hole is free of significant amounts of ground water (i.e. less than some marginal rate of inflow in inches of water per hour) or when slurry is not required to stabilize the excavation. “Dry” conditions are optimal because it is easier to inspect and place concrete in a “dry” hole than one that is partly or wholly filled with water or slurry. In addition, “dry” placement conditions require less equipment and less complicated environmental protection measures during construction (Figure 4).

A historically contentious issue related to “dry” conditions is the validity of “free-fall” concrete placement methods whereby concrete is dropped from a concrete truck directly into the hole without the use of a tremie tube or pump. Some states allow “free-fall” concrete placement only for drilled shafts less than 25 feet deep, while some states allow it for any depth and diameter of drilled shaft. Still other states disallow this method altogether, primarily due to concerns that it results in aggregate segregation problems. Good results can be obtained with the “free-fall” method regardless of shaft depth as long as the concrete is directed in a continuous stream down the center of a reasonably sized shaft such that it won’t “splatter” off of the rebar cage. This can be accomplished without special equipment via creative arrangement of the chutes with which concrete trucks are normally outfitted.

“Wet” Placement Conditions

“Wet” placement conditions occur when concrete must be deposited through significant levels of water or stabilizing slurry by means of a tremie or pump. The discharge end of the tremie or pump tube is plugged with a sacrificial seal (usually a piece of Styrofoam called a “pig” or a “rabbit”) and placed within inches of the bottom of the excavation at the beginning of concrete placement. The tube is extracted as the level of concrete rises within the shaft, but its discharge end is always kept submerged in at least 5 to 10 feet of fresh concrete. Rising concrete scours loose soil and slurry mud from the excavation walls, and ultimately displaces water, slurry, loose soil and other deleterious material out of the top of the hole. The concrete is thus protected from being infiltrated or fouled by excess water, slurry mud or other materials as it is deposited.
Two significant logistical aspects of “wet” placement conditions are environmental protection measures and the additional amount of equipment required, especially when slurry mud is used and/or when the shaft is constructed in a river or stream bed. Containment systems are generally required in such situations in order to prevent spoil materials, displaced slurry mud, or wasted concrete from contaminating the project area. Furthermore, when slurry is used to stabilize an uncased excavation and/or suppress artesian waters, slurry tanks and “desanding” units (Figure 5) must be placed on site as close to the constructed drilled shaft as practicable in order to reclaim and recycle the slurry mixture for use on other shafts. The space required for this additional equipment must be considered when evaluating potential staging and work access areas.

**Steel Casing**

In general, steel casing is used to prevent an excavated hole from collapsing due to non-cohesive or saturated soils or to exclude ground water from the excavation during concreting. If the surrounding soil is subject to collapse but otherwise stable enough to provide lateral support for the fresh concrete, the excavation can be supported with temporary steel casing that is extracted during concrete placement. If surrounding soils are unstable or “mucky”, or if the drilled shaft is constructed in a lake or stream bed, permanent steel casing will be needed to stabilize the excavation and act as a form for the fresh concrete. Thus, specific project site and subsurface conditions will determine whether or not steel casing is required, and whether it should be temporary or permanent.

It is important to emphasize here that there is no absolute correlation between the presence or absence of steel casing and “dry” or “wet” placement conditions. For example, it is generally true that steel casing is not required for an excavation in cohesive soil that is completely above the water table, and this situation is typically associated with “dry” concrete placement methods. However, if otherwise cohesive soils are rendered unstable due to a high water table or artesian flows, the excavation could be stabilized with slurry mud and “wet” concrete placement methods can be utilized accordingly. Steel casing is not required in either of these hypothetical scenarios, but the concrete placement methods are different. Conversely, temporary casing can be utilized with “dry” placement methods when the casing is socketed into rock that is overlain with saturated soils. In this scenario, the socketed casing seals out potential water intrusion, thereby allowing “dry” placement of shaft concrete. When the level of the concrete reaches 5 to 10 feet above the bottom of the excavation, the casing can be extracted while consistently maintaining that same level of static concrete head above its bottom edge. In contrast to the earlier hypothetical scenario, the entire drilled
shaft is poured using the “dry” placement method while temporary steel casing is used to stabilize the excavation. Again, actual project conditions and design requirements will dictate the need for temporary or permanent casing, as well as the applicability of “dry” or “wet” concrete placement methods.

Finally, railroad environments can promote unique uses of temporary casing, primarily in the aforementioned situation where drilled shafts are constructed for a new underpass immediately adjacent to an active railroad track. In this case, an outer temporary casing with a larger diameter than the drilled shaft is installed in the active railroad embankment such that its tip is just below the top of the proposed drilled shaft. Subsequently, an inner temporary casing with a diameter equal to the design diameter of the drilled shaft is installed to the design shaft tip elevation. The inner casing is extracted during the concrete placement operation as described above while the outer casing remains in place. After the completed drilled shaft has been tested and approved, a column form is placed inside the outer casing and the column is poured from the top of the drilled shaft to the bottom elevation of the pier cap. In this manner, the new underpass can be completely constructed with minimal or no disturbance to the existing railroad embankment, which is generally maintained at a minimum 2:1 (H:V) slope during construction. After train traffic is permanently diverted onto the new structure, the existing structure and embankment are removed as dictated by project requirements and final site grading conditions. The new underpass shown in Figure 1 was constructed in just this manner.

Field Testing Procedures
This section will focus on testing procedures utilized by inspectors and engineers to reasonably assess the overall quality of drilled shaft excavations and completed drilled shafts. Only those testing procedures most frequently specified for drilled shaft construction are subsequently discussed, since they are generally the simplest and most cost effective procedures to employ.

Excavation & Concrete Testing
Immediately prior to placing concrete in drilled shaft excavations, they must be inspected and certified as to bottom integrity and cleanliness. Bottom integrity is generally verified by means of “Standard Penetration Tests” (SPT) and subsequent analysis to ensure that actual soil properties at the bottom of the excavation are consistent with properties used in design. As previously stated in the Specifications section, performing SPT at the exact location of a drilled shaft prior to drilling can be advantageous from the standpoint of reducing the total number of tasks that must be
performed during the critical time interval between completion of drilling and commencement of concrete placing operations.

Bottom cleanliness can be verified with “down hole” inspection for “dry” shafts that are large enough in diameter to accommodate the inspector, or with a portable “shaft inspection device” (SID or Mini-SID) in the case of smaller diameter shafts or “wet” shafts. The Mini-SID is basically a remotely operated video camera mounted inside a metal canister that is lowered to the bottom of the excavation at the completion of sediment removal operations (i.e. desanding) and prior to rebar cage placement. The camera transmits images of the bottom of the excavation to a monitor that allows a qualified inspector to assess the thickness of bottom sediments that have settled during the initial desanding operations. If excessive amounts of sediment remain on the bottom of the excavation, it is “vacuumed” and reinspected until specified limits are met. This ensures that shaft concrete will bear on sound, undisturbed material with properties equivalent to those used in design.

Figure 6 contains a photograph of a Mini-SID and sample test results. In this case, the specifications considered the bottom excavation to be “clean” if a minimum of 50% of the bottom area had less than 15 mm of sediment and no portion of the bottom area had more than 40 mm of sediment. The test results shown indicate a “clean” bottom based on these criteria, with a majority sediment accumulation of approximately 10 mm and maximum sediment accumulation of 40 mm (bottom of test area).

In addition to the standard procedures of recording slump, air and temperature measurements and making concrete cylinders during concreting operations, a record of volume of concrete placed vs. depth of shaft is made for each drilled shaft. After each truckload of concrete has been placed, a weighted tape is used to measure the distance to the top of the concrete from a preset datum elevation. Field measured data and theoretical data are subsequently plotted on a shaft volume vs. depth chart. The relationship between actual and theoretical volume lines can be used, along with other integrity testing data, to ascertain the general condition of the completed drilled shaft. A sample volume vs. depth chart is shown in Figure 7. Actual volumes that are consistently equal to or greater than (i.e. to the right of) theoretical volumes are typically indicative of the absence of collapsing excavation walls or other major problems during concreting operations.
Shaft Integrity Testing

Integrity testing procedures are generally non-destructive tests (NDT) based on wave propagation techniques. Stress or sound waves are introduced into a drilled shaft from external or internal sources, and response measurements are used to assess the integrity of the drilled shaft concrete. These tests are advantageous from a time standpoint because they can usually be performed within a week of constructing a drilled shaft and test results can be viewed and interpreted on site immediately upon test completion.

Stress wave testing procedures generally fall under the broad categories of “high strain dynamic pile load testing” (HST) and “low strain dynamic pile load testing” (LST). HST procedures require a pile driving hammer or heavy drop weight, so they are generally utilized on driven piles since pile installation equipment can also be used to test the piles. For drilled shafts, however, LST procedures—commonly referred to as “pile integrity testing” (PIT)—are preferred because they can be performed by a single person using a small, hand-held hammer that normally weighs between 1 and 10 lbs. The stress wave initiated by striking the top of the shaft with the hammer travels the full length of the shaft while a motion sensing device (normally an accelerometer) mounted to the top of the shaft measures the resultant head motion as a function of time. The hammer can also be instrumented to measure shaft head force as a function of time. When motion only is measured, the procedure is referred to as pulse or sonic echo (SE) testing. When force and time are measured, the procedure is referred to as transient or impulse response (IR) testing.

PIT procedures fall under the jurisdiction of ASTM Standard D 5882, Standard Test Method for Low Strain Integrity Testing of Piles. The basic theory behind PIT testing is that stress wave impedance (i.e. resistance) is a direct function of the cross sectional area of the shaft (among other parameters). Consequently, actual wave response echo times will differ from theoretical response values when an anomaly is present in the design cross section of the drilled shaft. Generally, a decreased time response indicates a decrease in shaft section (i.e. “necking”) and an increased time response indicates an increase in shaft section (i.e. “bulging”). Other anomalies that can affect response time are changes in concrete density, the presence of voids in the drilled shaft, and hard surrounding soils with very high skin friction capacity. An impulse response test schematic and sample stress wave output is shown in Figure 8.

Sound wave integrity testing is performed by means of the “crosshole sonic logging” (CSL) procedure. The CSL procedure is governed by ASTM Standard D 6760, Standard Test Method for Integrity Testing of Concrete Deep Foundations by Ultrasonic Crosshole Testing, and is based on the theory that the velocity of sound wave propagation in a drilled shaft is a function of its concrete material properties. CSL involves initiation of ultrasonic waves via...
transmitter probes placed inside the drilled shaft and recording their travel time through the concrete by means of receiver probes placed simultaneously at other locations inside the shaft. The probes are inserted and pulled vertically through water-filled PVC or steel tubes that are tied to the rebar cage and cast into the drilled shaft when it is constructed, thus allowing the entire length of the drilled shaft to be ultrasonically tested at increments as small as several inches. A CSL test schematic and sample test output is shown in Figure 9.

PIT and CSL procedures are effective and relatively inexpensive to perform, but they offer no capacity information in addition to having other limitations. For example, a “rule of thumb” is that PIT is reliable only for piles or shafts with a length to diameter (L/D) ratio less than 30. Furthermore, PIT generally detects only major sectional defects that often render evaluation of piles and shafts below such defects impossible. CSL testing detects major or minor internal defects between transmitter and receiver tubes, but cannot detect defects outside the rebar cage. These limitations notwithstanding, PIT and CSL procedures are the preferred integrity test methods for drilled shafts and, in combination with volume vs. depth curves, provide useful results when performed and interpreted by qualified testing personnel.

**Shaft Capacity Testing**

Currently, there are three specialized approaches to load capacity verification testing for drilled shafts. These are static, dynamic, and rapid load testing, the latter of which is sometimes referred to as “Statnamic” testing. Exhaustive discussion of each approach is well beyond the scope of this paper, so only a general discussion of their relative merits within the context of axial compressive capacity verification will be undertaken herein.

Static load testing is performed by straddling the drilled shaft with a stationary reaction frame and applying axial load by means of a frame mounted hydraulic jack. Depending on the size and design capacity of the drilled shaft, the frame and jack capacities can become quite large, and frame supports must be capable of handling substantial amounts of uplift. This test method can be quite costly, requires a large amount of space, and is generally not as convenient to implement as dynamic and rapid load testing methods.

Dynamic load testing has been utilized frequently since its development nearly 40 years ago. For drilled shafts, this approach requires a crane to drop a heavy, frame guided ram onto the exposed head of the drilled shaft. Accelerometers and strain sensors mounted on or near the shaft head measure its response, from which a Pile Driving Analyzer (PDA) calculates the bearing capacity of the shaft. PDA field calculations are further refined using signal
matching software such as the CAse Pile Wave Analysis Program (CAPWAP). This common load test procedure is familiar to most geotechnical engineers because of its use with driven piles, and is particularly advantageous in this regard as compared to static or rapid load testing.

Rapid load, or “Statnamic”, testing is a fairly new technology that involves detonating an explosive charge underneath a heavy weight placed on top of the drilled shaft. The charge and the amount of weight are dually calibrated to impart a predetermined test load into the shaft, and the total load application time is on the order of 0.5 seconds. The force of the explosion lifts the weight into a frame mounted catching mechanism that keeps it from falling back down onto the shaft (Figure 10). During the test, applied load as well as total and permanent axial deformations and response behavior of the shaft are measured by means of a load cell, accelerometers and embedded strain gages, all of which are simultaneously monitored with a field data acquisition system. Rapid load tests are generally more costly to employ than dynamic load tests, but they are capable of generating enormous test loads on larger drilled shafts. This could be advantageous in certain situations, depending on the size and design capacity of the drilled shafts to be tested.

In conclusion, it should be noted that load testing is costly compared to integrity testing, so it is often reserved for situations where there is concern about a particular drilled shaft based on unfavorable or inconclusive integrity test results and/or problems that may have occurred during the construction process. Thus it is important to clearly specify the nature and requirements of “proof” load testing since the contractor will generally be liable for its associated costs. Conversely, an owner can take the proactive approach of conducting a load test program prior to or during construction of the actual production drilled shafts for the purpose of verifying the geotechnical engineering assumptions used in their design. Again, due to the potential costs involved—in this case on the part of the owner—the need for a testing program and the type and number of load tests performed should be carefully weighed against the relative “scale” of the overall project in order to avoid excessive total construction costs.
LIST OF FIGURES

Figure 1    New Railroad Underpass on Drilled Shaft Foundations
Figure 2    New “On-line” Railroad Bridge on Drilled Shaft Foundations
Figure 3    Structural Modeling Schematics for Drilled Shafts
Figure 4    “Dry” Concrete Placement Method for Drilled Shaft
Figure 5    Portable Equipment Required for Slurry (i.e. “Wet”) Placement Method
Figure 6    Shaft Inspection Device & Sample Test Output
Figure 7    Sample Drilled Shaft Volume vs. Depth Chart
Figure 8    Impulse Response (IR) Testing Schematic & Sample Test Output
Figure 9    Crosshole Sonic Logging (CSL) Schematic & Sample Test Output
Figure 10   16 MN “Statnamic” Rapid Axial Load Test – Before & After
New Railroad Underpass on Drilled Shaft Foundations

(Drilled shafts constructed w/o temporary shoring. Existing underpass will be removed and highway widened after new underpass is opened to railroad traffic.)

Figure 1
New “On-line” Railroad Bridge on Drilled Shaft Foundations

(9'-0” dia. drilled shafts constructed w/permanent steel casing to bottom of concrete pier cap. Superstructure will consist of two 113’ through plate girder spans.)

Figure 2
Figure 3

Structural Modeling Schematics for Drilled Shafts
“Dry” Concrete Placement Method for Drilled Shaft

(Note “telescoped” temporary casing, CSL tubes, standard pump truck line.)

Figure 4
Portable Equipment Required for Slurry (i.e. “Wet”) Placement Method

(Desanding units in foreground, slurry tanks in background.)

Figure 5
Shaft Inspection Device & Sample Test Output

(Excavation bottom tested at five locations as shown.)

Figure 6
Sample Drilled Shaft Volume vs. Depth Chart

Figure 7
Impulse Response (IR) Testing Schematic & Sample Test Output

(Uncased shaft in partially weathered rock; results do not fit classic velocity wave pattern.)

Figure 8
Crosshole Sonic Logging (CSL) Schematic & Sample Test Output

(“FAT” and “waterfall” plots exhibit classic, uniformly vertical patterns.)

Figure 9
16 MN “Statnamic” Rapid Axial Load Test – Before & After

(Note position of weight stack in each photo.)

Figure 10