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Advances in Auger Pressure Grouted Piles: Design, Construction and Testing

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Abstract: There have been several interesting new advances in the design, construction, and testing of auger pressure grouted (APG) piles recently. The new construction techniques involve the addition of displacement piles (no spoils) and partial displacement piles (some spoils) that utilize many of the same installation procedures as traditional APG piles. These new pile types are more applicable where soft to firm clay and loose to medium dense sand soil conditions are present. The new testing and quality assurance procedures available include (1) automated pile installation monitoring equipment, (2) non-destructive integrity testing and (3) grout maturity strength testing procedures. In addition to new design methods, analytical methods are now used that can utilize the installation effort or drilling resistance measured when these piles are installed to provide a much higher level of confidence of the capacity and integrity of these pile types.

Introduction

Auger Pressure Grouted (APG) and Auger Pressure Grouted Displacement (APGD) piles continue to see significant growth in the market due to their unique combination of speed of installation and high capacity that results in one of the most cost effective deep foundation systems available. APG piles were first introduced in the 1940s, and are also known by a variety of names including auger cast and continuous flight auger piles. In the 1990s the Deep Foundations Institute came up with the generic term Augered Cast-In-Place (ACIP) piles to describe this type of deep foundation system.

Engineers in the past have had two main concerns with using APG piles on a widespread basis. The first area of concern had been where soft or loose soils conditions exist at a site and there is a potential for necking or removal of excessive soils when using a continuous flight auger to install the piles. The second area of concern had been associated with a perceived lack of quality control and quality assurance procedures for the pile installation process overall. In the past few years there have been several important new processes that have essentially eliminated both of these concerns.

Pile Types and Soil Conditions

The pile types used in practice today range from non soil displacement APG piles (and the low headroom application) to partial soil displacement and full soil displacement APGD piles. The soil conditions at a specific site dictate which pile type will be most appropriate.

APG Piles. The APG pile is the traditional, industry standard pile that is installed using a hollow stem continuous flight auger to pump fluid grout under pressure during auger withdrawal to form the pile. APG piles are constructed by rotating a hollow stem, continuous flight auger into the ground to the desired tip elevation. When the required depth is reached, a high strength, fluid grout is pumped under pressure through the hollow stem of the auger exiting through the tip (or bit). A pre-established amount of grout is pumped prior to lifting the auger to build up a "grout head" around the outside of the auger. The auger is then withdrawn in a controlled manner slowly rotating clockwise as the pumping continues to both maintain the head of grout and avoid any intrusion of water or soil into the grout column.

The upper portion of the pile is then screened of any debris that may have fallen in while the spoils are removed from the pile location. Reinforcing steel is then placed through the fluid grout column, and the pile top elevation is established by either dipping out or adding fluid grout to the pile.

Pumping fluid grout under pressure (as opposed to pumping or tremieing concrete) results in a higher pile capacity, reduces quality control questions, allows for fast reinforcing steel installation, and results in a overall quicker total installation time. This same basic process is used for all the pile types described in this paper.

APG piles can be installed with diameters ranging from 12-inches to 24-inches in 2-inch increments, plus 30-inch and 36-inch. The lengths of APG piles up to 24-inch-diameter can extend up to 130 ft. The lengths of 30-inch and 36-inch diameter piles can extend up to 100 ft.

LHR APG Piles. The low headroom (LHR) APG pile is a special application of the APG pile installation method that allows for pile installation in areas where there are overhead or lateral space constraints. Auger sections are added as the pile is drilled and removed as the pile is grouted. The lengths of the auger sections can be varied depending on the specific overhead limitations. LHR APG piles can be installed with diameters ranging from 10-inch to 24-inch in 2-inch increments. The lengths of LHR APG piles can extend up to 80 ft.

APGD Piles. APGD piles utilize a Berkel patented system that laterally displaces all the soil within the pile diameter to the area surrounding the pile. The lateral displacement action improves the load carrying capacity of the pile and greatly reduces the volume of spoils generated during installation. The full displacement APGD piles can be installed with 12-inch, 14-inch, 16-inch and 18-inch diameters with lengths up to 80 ft. The partial APGD pile is a Berkel designed system that displaces a portion of the soil within the pile diameter to the area surrounding the pile. The partial lateral displacement action improves the load carrying capacity of the pile, but to a lesser degree than full displacement piles. The volume of spoils generated is greater than that of the full-displacement system, but less than the neat volume of the pile. The partial displacement APGD piles can be installed with 12-inch, 14-inch, 16-inch, 18-inch and 20-inch diameters with lengths up to 55 ft.

Applicable Soil Conditions. Table 1 summarizes in general the most applicable soil conditions for each pile type. This information is meant to provide general guidelines only and would not necessarily preclude any of the pile types from being used for specific conditions. Please note that APG piles can be installed in any of these soil conditions but full displacement APGD piles cannot be installed in stiff clay/dense sand profiles.

To reduce the potential for ground loss, excessive depressurizing of the soils and the associated quality related issues, priority should first be given to full APGD, then to partial APGD, and then to APG piles when selecting the most appropriate pile type for the specific soil conditions.

Table 1. Applicable Soil Conditions

Pile Type	Main Soil Condition	Soil Layer/Pile Diameter Limitations
APG	Medium dense to very dense sand ; soft to hard clay ; soft rock	If a loose sand layer is present, diameters should be limited to 24-inch; if the loose sand is more than 20 ft thick, the diameter should be limited to 16-inch.
Partial APGD	Loose to dense sand with SPT blow counts less than 50 blows/ft	For any diameter, stiff, firm and soft clay layers should not exceed 15 ft, 20 ft and 30 ft thick, respectively
Full APGD	Loose to medium dense sand with SPT blow counts less than 25 blows/ft	For any diameter, stiff, firm and soft clay layers should not exceed 5 ft, 10 ft and 20 ft thick respectively; dense sand layers should not exceed 10 ft

Note: Thin soil layers (or seams) less than 3 ft thick are not significant enough to factor into the selection of the most applicable pile type.

Drill Rig Types and Equipment Specifications

The types of equipment used to install the piles are a very important factor in the installation process. Usually, the selection of the specific equipment to install the piles is left up to the contractor, which is then sometimes reviewed in the submittal process. However, not enough attention is typically given by the designers to the selection and approval of adequately sized equipment for the particular application in practice.

In general, APG pile drill rigs cannot typically add crowd (or a down force) to the auger while drilling. The only down force is simply the weight of gearbox and auger. This is the case whether the leads are fixed (pinned) to the crane boom or hanging (swinging) from the boom tip. However, the drill rigs used for APGD piles must be able to produce both a large down force and a high gearbox torque. Therefore, a traditional lattice boom crawler crane supporting a gearbox and leads cannot be used for these piles. APGD piles must be installed using hydraulic fixed mast rigs specifically built for these types of drilling applications. There is a wide range of equipment commercially available, and much of it would be considered too small or undersized for most applications. The types of drill rigs used by Berkel and their main specifications are summarized in Table 2.

Table 2. Equipment Types and Specifications

Pile Type	Gearbox Torque	Crowd/Gearbox Weight	Drill Rig Horsepower
Typical APG	36,000 ft-lbs	5,000 lbs (wt)	350 hp
Large/Deep APG	88,000 ft-lbs	10,000 lbs (wt)	750 hp
LHR APG	21,000 ft-lbs	3,000 lbs (wt)	200 hp
Partial APGD	150,000 to 180,000 ft-lbs	15 to 40 tons	250 hp
Full APGD	150,000 to 180,000 ft-lbs	15 to 40 tons	250 hp

Design Methods

Published Methods. There have been several new methods and reviews of existing methods published in recent years on the static capacity of APG and APGD piles. In some cases the existing methods were then modified to provide more accurate results. These modified existing methods and new methods were developed specifically for APG or APGD piles from pile load test databases.

APG Piles. The main previously published methods that have used for APG pile design include: (1) Neely (1991) which was specifically for APG piles in sand, (2) API (1993) which was developed for driven piles, and (3) FHWA (Reese and O'Neill, 1988) which was developed for drilled shafts. Several papers have been published recently that evaluate these main design methods (in addition to others) and the authors have come to different conclusions on which works the best. These recent studies used somewhat to completely different databases. They also used different failure criteria to establish the ultimate measured load used in their database. They always attempt to separate skin friction from end bearing but the number of fully instrumented test piles in the databases ranged from none to a very small percentage. Nonetheless, some interesting conclusions can be made by comparing the results of these recent studies.

The most recent evaluations of APG pile design methods can be summarized into four main studies: (1) Zelada and Stephenson (2000) for sands, (2) O'Neill et al (1999 and 2002) for mixed soils (sand and clays), (3) Coleman and Arcement (2002) also for mixed soils, and (4) Moss and Stephenson (2004) for clays.

Zelada and Stephenson concluded that the best design method for sands was a combination of a modified FHWA method for skin friction and modified Neely method for end bearing. They determined that both the FHWA beta (β) factor for skin friction and Neely's end bearing factor should be reduced slightly. This results in a much higher end bearing component than would be calculated using FHWA method. Their design equations for unit skin friction (f_s) and unit end bearing (q_b) are listed below.

f_s = β p', where p' is effective overburden stress

β = $1.2-0.108z^{0.5}$ $0.2<\beta<0.96$, where z is in feet

q_b = 1.7 N (tsf), limited to 75 tsf, where N is the SPT blow count

O'Neill et al (1999 and 2002) reviewed a database of mixed (sand and clay) soil conditions primarily as part of a research program funded by the Texas Department of Transportation (TxDOT). Thus the unique TxDOT (1972) methods for soil testing and design were included. O'Neill concluded that for all clay profiles the TxDOT alpha (α) method provided the best results. This is similar to the FHWA α method except α is equal to 0.70 rather than 0.55, and there are no exclusion zones. O'Neill also concluded that the TxDOT Houston District limiting (ultimate) end bearing of 4 tsf was too conservative, and the FHWA end bearing for clay would be more applicable. For mixed soil conditions and all sand profiles O'Neill concluded the FHWA methods worked the best.

Coleman and Arcement (2002) reviewed a database of primarily mixed soil conditions. They concluded that the API method worked the best. They further concluded that the FHWA method could also be used effectively, provided adjustments to both the α and β factors were made to improve the accuracy.

Moss and Stephenson (2004) reviewed a database for APG piles in all clay profiles. They concluded that the API method worked the best followed closely by the FHWA method.

In summary, either the API method or the modified FHWA methods can be used for mixed and all clay profiles with the best state-of-the-practice accuracy currently available. For all sand profiles the Zelada and Stephenson modifications to the combination of the FHWA and Neely methods provides arguably the best prediction method.

APGD Piles. The design methodology for capacity of APGD piles as installed by Berkel has been developed internally. A complete description of the development of the design process is described by NeSmith (2002). Both shaft and toe resistances are calculated based on either SPT or CPT results, along with modifiers (w_s for shaft and w_t for toe) based on the characteristics of the granular material penetrated. Unit shaft resistances (f_s), based on either CPT tip resistance (q_c) or SPT blow count (N) corrected to N_{60} efficiency, are calculated as follows:

$f_{s,cpt} = 0.01*q_c + w_s$ where $q_c \leq 19$ MPa (200 tsf)

$f_{s,spt} = (0.005 \text{ MPa} * N_{60}) + w_s$ or
$f_{s,spt} = (0.005 \text{ tsf} * N_{60}) + w_s$ where $N_{60} \leq 50$

For uniform, rounded materials with up to 40 percent fines, the shaft resistance modifier w_s is taken as zero, and a limit shaft resistance of 0.16 MPa (1.7 tsf) is applied. For well-graded, angular materials with less than 10 percent fines, w_s is taken as 0.05 MPa (0.5 tsf) with a limit shaft resistance of 0.21 MPa (2.2 tsf). Interpolation between these values is used for intermediate materials.

Unit toe resistances (q_t), based on either average CPT tip resistance (q_{cm}) or average corrected SPT blow count (N_m) is calculated as follows:

$$q_{t,cpt} = 0.4 * q_{cm} + w_t \qquad \text{where } q_{cm} \leq 19 \text{ MPa (200 tsf)}$$

$$q_{t,spt} = (0.19 \text{ MPa} * N_m) + w_t \qquad \text{or}$$
$$q_{t,spt} = (1.9 \text{ tsf} * N_m) + wt \qquad \text{where } N_m \leq 50$$

Average CPT tip resistance or SPT blow count are calculated from four diameters above and below the pile toe. For uniform, rounded materials with up to 40 percent fines, the toe resistance modifier w_t is taken as zero, and a limit toe resistance of 7.2 MPa (75 tsf) applied. For well-graded, angular materials with less than 10 percent fines, w_t is taken as 1.34 MPa (14 tsf) with a limit toe resistance of 8.62 MPa (89 tsf). Again, interpolation is used for intermediate materials.

Feedback from Drilling

The concept of relating the effort required to advance the tooling for cast-in-place piling systems to the subsurface materials through which the tooling is penetrating has been around for some time. Van Impe (1988) described the development of "Specific Energy" for screw piles, which included crowd, torque, and the vertical penetration rate and rotational speed of the tooling. De Cock and Imbo (1993) provided additional insight into the process and comparisons of specific energy and cone tip resistance. Increased interest in this concept in the United States has coincided with the increased use of automated monitoring equipment (AME) on the drill rigs used in the U.S. deep foundation industry. New, simplified approaches have been developed for both conventional APG and APGD piles.

The approach for each of the systems is similar in that only the hydraulic pressure developed in turning the tooling and the rate at which the tooling penetrates are considered. These units are normalized by dividing them by a reference value of the same units and then combined to create a non-dimensional index. This index an indicator of the instantaneous effort required to advance the tooling and is analogous to the number of hammer blows required to advance a driven pile. The higher the drilling resistance, the harder the material is that is being penetrated. Automated monitoring equipment can be set up to directly compute, display, record and print this information. Plots of drilling resistance versus depth can be developed to both quantify and compare the subsurface stratigraphy along the length of a pile and between different piles. In applicable subsurface conditions, this can provide a readily apparent and quantifiable means to infer where bearing materials were encountered or where harder zones where drilled. Although the installation parameters used for evaluation of APG piles and those used for APGD piles are the same, the methodologies have evolved differently. The approach for APG piles is termed “Drilling Resistance” and is described in Brettmann (2004). The methodology for APGD piles is termed “Installation Effort” and was put forth by NeSmith (2003). A summary of the data gathering and processing methods is contained in the following sections.

Drilling Resistance. The three pieces of data collected to compute drilling resistance are depth, time, and hydraulic pressure. For each depth interval there is a corresponding time interval and average pressure. Drilling resistance for each unit depth interval is fundamentally defined as an interval time multiplied by average hydraulic pressure.

The units for these terms are time (seconds) and pressure (psi). To provide a basis for uniform comparison and change drilling resistance to a non-dimensional parameter, these values must be “normalized”. Normalizing these values is simply dividing them by a reference value with the same units. These two normalized values are called the time factor and the torque factor.

The reference value for the time interval would be taken as a typical (or average) interval time to drill a unit depth. For example, a typical reference interval time value for an 18-inch-diameter pile would be 10 to 20 seconds per foot. The time factor is computed by dividing the measured interval time by the reference interval time. Thus if the reference interval time is set at 10 seconds and the it took 10 seconds to drill one foot, then the time factor for that interval would be 1.0.

Similar to the method used to compute the time factor, the reference value for the hydraulic pressure would be taken as a typical (or average) hydraulic pressure used during the pile installation. A typical reference hydraulic pressure for a power unit could be 1,400 psi. The torque factor is computed by dividing the average measured hydraulic pressure by the reference pressure. Thus, if the average hydraulic pressure was 1,400 psi and the reference pressure was set at 1,400 psi, then the torque factor would be 1.0.

The drilling resistance for each interval is then computed by multiplying the time factor by the torque factor. If an interval were drilled at the reference time interval (say 10 seconds) with the reference hydraulic pressure (say 1,400 psi) then the drilling resistance would be 1.0. If the next interval were drilled in 8 seconds with the same average hydraulic pressure then the drilling resistance would be 0.8. Similarly, if the next interval took 20 seconds to drill but the average pressure was only 700 psi, the drilling resistance would be 1.0.

Using this combination of non-dimensional time and torque factors to compute drilling resistance allows each factor to “balance” one another. An unusually high time factor could be the result of an operator not allowing the auger to penetrate. However, the corresponding hydraulic pressure should be relatively low in that case because the gearbox would not have been loaded heavily during that time.

Drilling resistance is currently meant to provide only a relative comparison of subsurface strength from which one can infer stratigraphy. Normalizing factors could either be preset for "standard" conditions or could be site specific based on the test pile information.

Examples presented by Brettmann (2004) show that drilling resistance helps explain somewhat unusual test pile results. Comparing the drilling resistance and grout takes of the test piles to the production piles would provide the means to infer the capacity of all the production piles. This is similar in concept to counting blows for a driven pile. Every APG pile would then become a "verified" pile.

There is still much more data to be gathered on this subject before many specific and final conclusions can be made. It is also important to understand that this type of information is likely only useful for a limited number of soil conditions when applied to APG piles. This is primarily where piles need to be socketed a minimum distance into a soft rock, like a shale. However, it is apparent that this concept shows significant promise to advance the practice of APG pile installation, and increase its understanding and use in the deep foundation industry.

Installation Effort. The installation platform used for Berkel's displacement pile system is typical of those used for European CFA installation, and recently, U.S drilled shaft work. It includes a vertical mast with an attached turntable capable of producing about180,000 ft-lbs of torque, and a system of cabling that allows a downward force (crowd) of about 40,000 pounds to be placed on the tools. Monitoring and collection of installation parameters with these platforms has been common in Europe for many years, and the data acquisition systems needed to gather the information necessary to assess installation effort is now standard on most of this equipment. From the early implementation of the displacement system it was clear that torque (fluid pressure of the motors driving the turntable) was a reasonably good indicator of the strength of the subsurface materials being penetrated, and torque was (and still is) a part of the drilling termination criteria on many projects.

While torque-related criteria are applicable in some conditions, it became clear that it was not applicable to more complex conditions, and that differences in operator techniques could be a significant variable. After several iterations, it was found that the combination of torque (t_{fp}) and tool penetration rate (PR) provided a relatively simple, reliable indicator of subsurface conditions. The data base from which the process described evolved is for pressure grouted displacement piles installed by Berkel & Company Contractors, using the procedures described by NeSmith (2002). The piles considered in this work were installed primarily in granular deposits of Tertiary age or younger. The majority were in relatively recent alluvial deposits adjacent to major rivers. Installation platforms were either Bauer BG 25s or Casagrande 220s. Data was acquired and processed using Gamperl & Hatlapa hardware and software, with readings at 1-second intervals. The penetration rate index (PRI) is calculated as the inverse of the square root of the penetration rate (PR) normalized by a base penetration rate (PRBase), typically 20 feet per minute.

$$PRI = 1/(PR/PRBase)^{0.5}$$

The torque index (TI) is calculated from the measured fluid pressure of the motors driving the tools (t_{fp}), normalized by a base torque level (TBase) of 100 bars.

$$TI = 2.78(t_{fp}/TBase)^{1.36}$$

The product of PRI and TI gives the Installation Effort (IE). The relationships for PRI and TI arise from the determination that neither penetration rate nor torque is linearly related to soil strength.

It has been demonstrated on numerous projects that instantaneous installation effort (IE) is a good indicator of subsurface stratigraphy, and can serve as an adjunct to subsurface information gained through normal exploration methods. It has also been determined that total installation effort (the sum of the instantaneous values) can, on a project-specific basis, be related to ultimate pile capacity.

Quality Control and Quality Assurance

Several new trends for quality control and quality assurance of APG and APGD piles have emerged in the last few years. These new technologies can effectively supplement existing quality control procedures. In the past, quality control of APG piles has been performed manually; that is, by a person. This “inspector” is typically an engineer or technician working for the project’s geotechnical engineering firm or testing laboratory. These manual quality control procedures were developed over many years and were published as industry standards by the Deep Foundations Institute (DFI, 1994 and 2003).

It is important to understand that these new technologies do not replace the inspector, they simply provide: (1) accurate and automatic records of key aspects of the pile installation process and (2) additional quality assurance tests and data for verification.

Automated Monitoring Equipment. Automated monitoring equipment (AME) has been developed in recent years that will automatically monitor and record key aspects of the pile installation process. Typical systems will measure: (1) time, depth and hydraulic pressure during drilling, and (2) time, depth, grout volume, and grout pressure during grouting. The systems will provide a real time graph for the operator to watch during installation, a hard copy print-out of the data, and a digital record that can be stored on a computer.

The grout volume is measured with an in-line magnetic flowmeter and is reported by the manufacturer to be accurate within 2 percent. Since the flowmeter measures volume directly it does not depend on or need to know the pump calibration. Time is measured to the nearest second and depth is measured to the nearest tenth of a foot (0.03 m). This equipment results in a remarkably precise and accurate record on the pile installation compared to the manual techniques.

Nondestructive Integrity Testing. Even though piles installed in accordance with the specifications would be considered acceptable, there are times when additional testing is considered beneficial. This is especially true in new markets where the past use of APG or APGD piles has been limited. The designer may want to have some additional quality assurance that the methods described in the specifications do indeed result in a good quality, continuous pile that does not contain defects such as necking, soil inclusions or poor quality grout.

Nondestructive testing (NDT) of the structural integrity of the pile can be performed using sonic methods. These tests are performed after the grout has sufficiently cured. The main NDT methods used in practice for APG or APGD piles can be divided into two groups: (1) surface reflection, and (2) direct transmission.

Surface reflection methods (or pulse echo) involve striking the top of the pile with a hammer to generate a stress wave that is partially or completely reflected by changes in the pile. The reflections are monitored with an accelerometer at the top of the pile and, with a few assumptions, the depth to this reflection can be estimated. This method is not recommended for piles with a length to diameter ratio of more than 30. The reliability of this method is further reduced when there are multiple changes in the cross-sectional area of the pile.

Unfortunately for this method, APG piles typically have a length to diameter ratio of greater than 30. Also, the process of pumping grout under pressure through various soil layers produces piles with multiple changes in cross-section (bulges in softer layers). These unique properties of APG piles limit the usefulness of surface reflection techniques.

Sonic integrity logging is a direct transmission technique that was developed to overcome both the length to diameter and multiple changes in cross section limitations. Singlehole sonic logging (SSL) is typically used for APG or APGD pile applications. In this method, a pair of ultrasonic source and receiver probes is lowered down a single access pipe in the center of the pile. The travel time and signal strength of an ultrasonic pulse that travels between the probes (though the pile) is measured along the full length of the pile. Changes in travel time or signal strength identify the depth where a soil inclusion or poor quality grout is present.

SSL is considered to be much more reliable than surface reflection methods. Typically only 10 to 20 percent of the piles on a project are selected for SSL, and it is usually performed at the very beginning of the project to verify the pile installation procedures are resulting in good quality piles. NDT should only be used to supplement detailed installation records and soils information, not as a stand alone, pass/fail type of test.

Grout Maturity Method. The rate of strength gain for grout is essential information when determining the load carrying capacity of a recently placed pile. As described by Brettmann et al (2004), new technology in maturity meters allows for embedded components to be used in lieu of external recording devices, making the use of these instruments more feasible on job sites. This nondestructive means of determining in-place grout strength is referred to as the maturity method. The maturity method provides real-time grout strength information with the use of embedded sensors and a portable handheld reader. The sensors record time and temperature during hydration and curing to determine the rate of strength gain as compared to a specific mix design. The method has a published standard, as described in ASTM C 1074.

Grout maturity testing consists of implanting a time/temperature sensor in the actual pile and then using a readout gauge to obtain an empirical maturity reading. Having previously established a calibration curve for the specific grout mix, the maturity reading can then be correlated to an actual compressive strength. Any number of maturity (or strength) readings can be obtained from each sensor starting the day after the pile is installed, since the sensors are not destroyed during testing. Only one strength reading can be obtained with a grout cube since it is destroyed during testing. Although this wouldn't completely replace traditional cube strength testing, this new method would certainly reduce the amount of cube testing required on APG or APGD pile projects.

Summary and Conclusions

Several new advances in the design construction and testing of APG and APGD piles have emerged in recent years. These advances involve the development of new design methods, pile types and increased quality control techniques. Specific conclusions about these new trends are summarized below.

1. In addition to traditional non displacement APG piles, full and partial displacement piles can now be better utilized in soft soil conditions.
2. To reduce the potential for ground loss, excessive depressurizing of the soils and associated quality related issues, priority should first be given to full APGD, then to partial APGD, and then to APG piles when selecting the most appropriate pile type for the specific soil conditions.
3. Pumping fluid grout under pressure (as opposed to pumping or tremieing concrete) will provide for a higher pile capacity, reduce quality control questions, allow for fast reinforcing steel installation, and result in a overall quicker total installation time.
4. Automated monitoring equipment (AME) for all applications (except for limited headroom) can now be used to provide real time monitoring of key aspects of the installation for the operator and inspector, and provide a digital record of the installation.
5. New design methods for both APG and APGD piles has dramatically increased the ability to accurately estimate pile capacities for a wide range of soil conditions.
6. Nondestructive testing (NDT) of the structural integrity of the pile can be performed using sonic methods. Singlehole sonic logging (SSL) is considered to be much more reliable than surface reflection methods. NDT should only be used to supplement detailed installation records and soils information, not as a stand alone, pass/fail type of test.
7. The grout maturity method provides for better quality control and quality assurance of the grout used on APG and APGD pile projects, and can be used to supplement and reduce the amount of traditional grout strength testing needed.

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