Pile Testing – State-of-the-Art

Garland Likins, P.E., Senior Partner
Pile Dynamics, Inc., 30725 Aurora Road, Cleveland Ohio 44139, USA
glikins@pile.com

Abstract

Foundations are a critical element of any structure. They must carry the desired loads safely both structurally and geotechnically. If there is a structural weakness the pile foundation might fail catastrophically. If the soil cannot support the applied loads with a sufficient margin of safety then significant settlements of the structure will occur, rendering the structure unsafe and causing either expensive remediation or possibly even demolition of the structure. The risk of failure is therefore unacceptable.

In general, the piles are also buried so direct inspection of the in-place element is impossible. Fortunately, in recent decades a growing list of test methods has been made available to indirectly evaluate the structural integrity and load bearing capacity of piles. These test methods are now routinely applied. The relatively modest cost of testing, compared to the cost of the foundation, is justified in reducing risk of foundation failure and in many cases results in an overall reduction in cost of the total foundation.

Introduction

A brief history of testing is given in Hussein and Goble (2004). Prior to about 50 years ago, methods to evaluate pile foundations were generally limited to inspection during installation (not further discussed in this paper) and static load tests. Static load tests were applied only to a small sample of piles on any project due to time and cost constraints. When the static test was run to geotechnical failure (not always the case), results of these tests were then correlated to soil borings to improve static analysis methods which could then be used for smaller projects where a static load test was not justified. Of course, if static analysis methods alone were used to assess capacity, large safety factors were needed to minimize the risk of failure if no confirming static load test was performed, since correlation of soil strength to SPT "N-values" has a high coefficient of variation.

For driven piles ("Estacas cravadas"), dynamic formula (also called "pile driving formula" or "energy formula") could be applied. However, a large study in the 1930's led to the conclusion that dynamic formulas in any form were unreliable and the recommendation was to evaluate capacity only by static load tests (Likins, 2012a). The blow count for each unit of penetration was generally recorded for the full length of the pile installation; if the "blow count log" was unusual, such as a sudden decrease in value or significantly different penetration from typical neighboring piles or from the expected penetration based on the soil profile, then the engineer was often forced to use "judgment" when considering if the pile was acceptable.

For bored piles ("Estacas escavadas", also called "drilled shafts"), inspection of the construction process was the only realistic general alternative; coring was possible but not generally cost effective. For piles which could be drilled in self-supporting soils, visual inspection of the open dry hole was possible. In some cases, humans entered the hole to inspect the pile bottom, although today this dangerous practice has been eliminated. If the hole was drilled with a fluid (e.g. slurry) to maintain the walls, or if the pile was installed with a continuous flight auger ("Estacas hélice continua"), inspection was generally lacking other than knowing if the reinforcing cage could be installed to the design depth. Small diameter bored piles could be statically tested.

Structural Integrity Evaluation

For driven piles ("Estacas cravadas"), structural integrity is indirectly evaluated from the "blow count log" or if the static load test carries the desired design load with a satisfactory net settlement, or directly from dynamic testing measurements (discussed in a subsequent section).

For both driven piles ("Estacas cravadas") and bored piles ("Estacas escavadas"), evaluation of structural integrity is essential to detect structural weaknesses prior to completing the structure. If a weakness is detected, it can be repaired or replaced at a relatively low cost. If some form of integrity testing is eliminated, the risk of an undetected weakness increases, and remediation costs for a failed foundation due to such a weakness are orders of magnitude larger than the cost of basic integrity testing.

Low Strain Integrity Testing

Structural integrity is most often of concern for bored piles ("Estacas escavadas"). Other than visual inspection, when possible, several different methods are used indirectly evaluate the shaft structural integrity. One of the earliest methods is low strain integrity testing (Rausche et al, 1988). The method of data collection is specified in ASTM D5882. Once the bored concrete pile is installed and the concrete has sufficient strength, the pile top is struck by a hand-held hammer which generates a small force that travels down the pile shaft, reflects off the pile toe or other cross section changes and then back to the pile top. An accelerometer is attached to the pile top by a thin coupling compound and records both the input and reflections. The acceleration is integrated to velocity for interpretation. Generally, the soil dampens the traveling wave so a magnification of the signal with time is needed to view the reflections. Other signal enhancing techniques are often required to evaluate these small signals (Likins and Rausche, 2000). Cross section increases like a bulge will create a compression reflection, causing a change in velocity of opposite sign as the input. Cross section decreases (defects) will create a tension reflection, causing a change in velocity of the same sign as the input. Figure 1 shows low strain integrity for two neighboring piles of 25 m lengths. The top graph shows an input and only one reflection at the expected return from the pile toe within the 25 m length. The lower graph shows an addition reflection (arrow) prior to the toe at 25 m; since this reflection is positive (like the input) this is a defect.

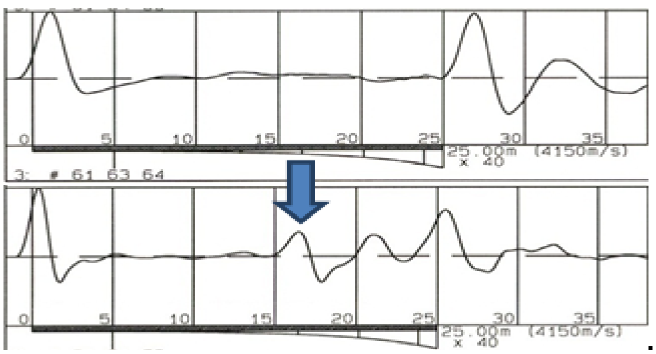

Figure 1: Low strain records of two piles. Top pile is acceptable. Bottom pile has major defect.

The method is generally limited to evaluating the shaft above the first major non-uniformity. For relatively uniform shafts, the length to diameter (L/D) ratio may be limited to 30. In strong soils or if the pile is very non-uniform, an L/D ratio of 30 is sometimes not possible, while in weaker soils with relatively uniform piles L/D ratios even exceeding 50 have been be successfully tested. Even if a reflection from the toe cannot be observed, the test is useful to inspect for major critical defects in the upper portion of the pile shaft, where the full axial or lateral load effects are applied. If a defect is located very near the pile top, the reflection can superimpose on the input to create what appears to be an abnormally wide velocity input compared with similar tests on other piles on the same project; if the input "force" is also measured with the hammer, the velocity pulse width can be compared with the force pulse width for a more direct confirmation of a near-top defect.

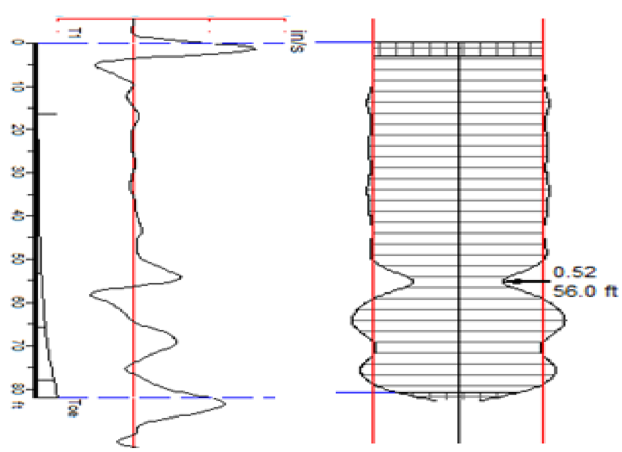

Figure 2: the "profile of the defective pile

If a clear reflection from the toe is present, the velocity data can be further analyzed to provide an “impedance profile”. If the concrete quality is relatively uniform along the shaft then this profile can be considered as the potential cross sectional area (the “profile of the data in Figure 1b is shown in Figure 2).

It should be noted that the reflections from the pile toe or cross section changes are measured in time relative to the input. The observed time is then converted to a length using the formula TIME = 2L/c where L is the pile length and c is the material wave speed. Since the wave speed in concrete can vary with concrete strength and age (typically ±10 percent), the wave speed is often assumed and there is then an uncertainty of length. If the length is accurately known for some piles, then the typical wave speed for that site can be obtained if the toe reflection is observed, and length uncertainty then reduced.

Since the method only requires access to the pile top, and many piles can be tested in a short time at a reasonable cost, no pre-planning is required and every pile of a project could potentially be tested. Often several piles are randomly selected and tested; if defects are detected then neighboring piles are evaluated. It is possible to require testing of every pile for a project since integrity testing fifty or more piles per day can be realistically achieved.

An alternate to evaluation in the time domain is to convert the velocity and force data using Fast Fourier Transforms (FFT) into the frequency domain. However, interpretation of the result is more difficult and the effective depth of investigation is even more limited since magnification with time is not possible.

Cross-hole Sonic Logging

If project specifications call for and multiple “access tubes” are attached to the reinforcing cage of a bored pile (“Estacas escavadas”), the concrete inside the reinforcing cage can be evaluated for uniformity by Cross-hole Sonic Logging (Likins et al, 2004), commonly called “CSL”. The procedures for this method are contained in ASTM D6760. Typically one access tube is recommended for every 300 mm of pile diameter. Little extra benefit is gained by installing more than eight tubes. This author’s recommendation is for a minimum of four tubes since that allows for some evaluation through the pile center. A probe with a transmitter is lowered in one water-filled access tube while a different probe with a receiver is lowered in a different tube. Starting at the bottom, the probes are pulled simultaneously to the pile top. Signals from the transmitter are captured by the receiver typically every 50 mm. The probes are then repositioned to different tubes and the test repeated until all tube combinations have been tested. From the “First Arrival Time” (FAT) of the signal and the spacing between access tubes, the wave speed can be calculated. Any abnormal delay in FAT observed, or decreased wave speed, would indicate a potential defect in the concrete.

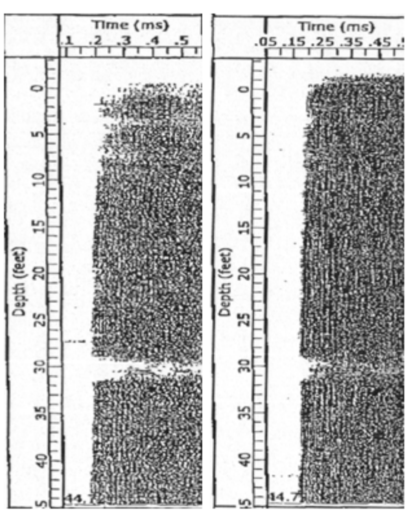

Figure 3: shaft with defect

Figure 4: core of the defective shaft

Figure 3 presents the waterfall diagram which is a “nesting” of the raw data. The left edge is the “First Arrival Time” (FAT) and is the most important feature of the waterfall diagram. The intensity of the graph reflects the signal strength; the “white” band at 9 m (30 ft) indicates a defect. The left half of the plot is the initial test performed a few days after casting. Since basically the same graph was obtained for all tube combinations, the defect was evaluated as a layer of reduced strength through the entire section. A core was then specified and is the resulting core confirming the defect is shown in Figure 4. The shaft was pressure grouted using the core hole, and the shaft was tested again (right half of Figure 3). While the defect is still observed, its severity was greatly reduced and the shaft was accepted.

The method is not sensitive to the surrounding soil and is not limited by length. Unfortunately, it also gives no indication of the quality or quantity of the concrete outside the reinforcing cage and sometimes may falsely indicate problems due to “debonding” (often associated with using PVC access tubes) or “bleed water channels” for some mix designs (and particularly if there is a permanent casing). If low wave speeds are detected in some tube combinations then a “tomography” analysis can be useful to estimate the extent of a possible defect.

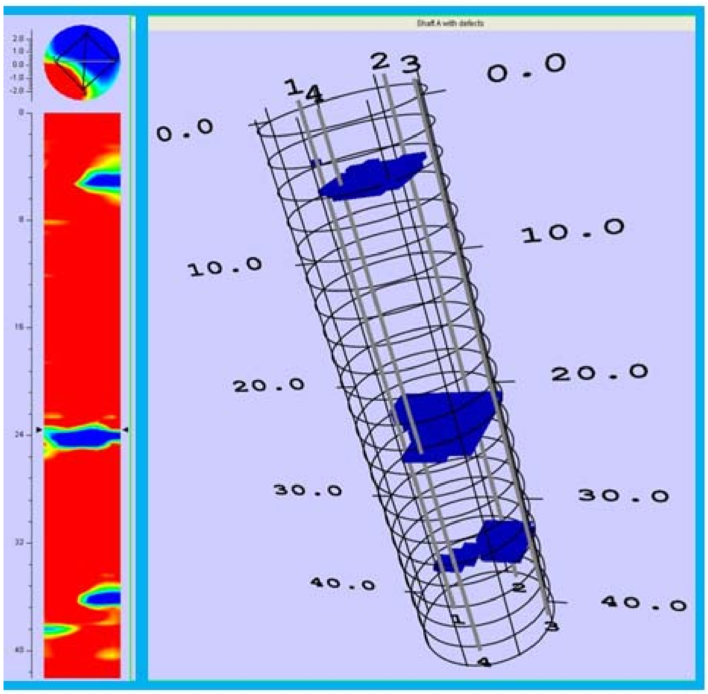

Figure 5: Tomography analysis of shaft with purpose-built defects

To demonstrate a tomography analysis, a shaft was purposely cast with defects. The shaft defect plan included a “half-moon” Styrofoam sheet at about mid depth, and inclusions both near the top and the bottom. For this shaft, Figure 5 shows a horizontal slice (circle near top left) showing the Styrofoam sheet defect covering half the section (defects show up as blue areas in the figure), a vertical slice (lower left) and a 3D full body diagram at right showing all planned defects were found.

CSL access tubes are often specified to be installed for every bored pile constructed with a slurry. Even if only some randomly selected shafts are actually tested by CSL, the contractor generally will usually take more care in construction of every shaft simply because any shaft could be tested. Of course, if any unusual event is detected during construction of any particular pile then that pile should be tested.

Calipers

After completion of the drilling and prior to placing concrete in the bored pile (“Estacas escavadas”), the shape or profile of the hole can be measured. It is then assumed that no further changes in the hole occur after this measurement and the concrete fills the hole exactly as measured by the calipers. Some caliper devices use mechanical arms. More modern devices use an ultrasonic pinging technique. If the log has sufficient resolution, a good estimate of the required volume of concrete can be generated. If the slurry is not properly controlled, particulates in the slurry can affect the detection of the sidewalls; of course, if the side walls are not detected because of particulates in the slurry, this is also useful information that something is wrong with the control of the slurry. These devices can also assess verticality of the hole as an additional benefit. A sample scan of one depth is shown in Figure 6.

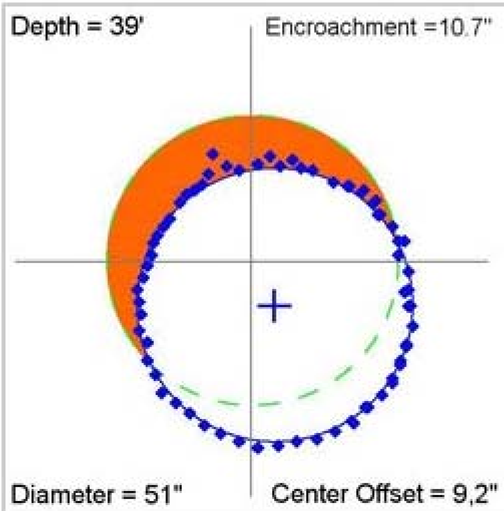

Figure 6: Caliper output at one depth

Automated Monitoring Equipment

During the drilling of augercast piles ("Estacas hélice continua"), measurements can be taken by Automated Drilling Equipment (AME) of auger depth versus time during drilling, yielding a drilling rate. The auger rotation rate can also be recorded. Concrete or grout is pumped under pressure as the auger is withdrawn, preferably at a relatively uniform rate. The concrete or grout volume is measured by Faraday's Law using a magnetic flow meter and can be correlated with auger depth, yielding a "volume pumped versus each depth increment". In the USA and England, the magnetic flow meter is required since counting pump strokes has proven to be inaccurate and unreliable (Piscsalko et al, 2004). Figure 7 illustrates both a proper performance and a faulty performance of the same pump on the same pile. The concrete volume delivered to the pile, verified by the flowmeter measurements during the seven faulty pump strokes, was considerably less than the volume calculated from simply counting the pump stokes.

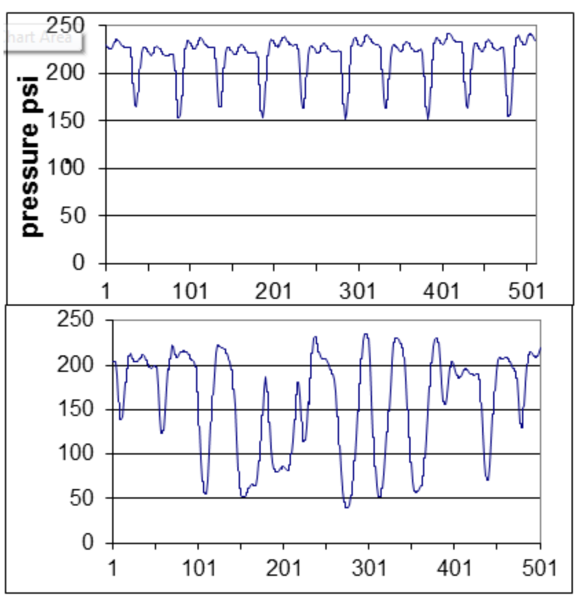

Figure 7: normal (top) and faulty pump (bottom)

Figure 8: AME display for rig operator

The recommended depth increment is 500 to 600 mm. The withdrawal rate can be controlled so that the incremental volume installed for each depth increment exceeds the theoretical volume for the nominal hole diameter by a specified ratio so that no rapid withdrawal causes a vacuum and hence necking of the shaft. Since the concrete or grout is still fluid, there may be some minor redistribution of grout along the length, and excess grout will travel up the auger flights and result in an early "return" of this excess grout at the ground surface prior to the auger tip leaving the hole. Records of both the drilling and concreting phase of the operation are presented on a screen viewed by the drilling rig operator (Figure 8), guiding him into piles which are more uniform along the length as well as more consistent pile to pile. If any pile record shows a deficiency in incremental volume anywhere along the length, the pile can immediately be easily drilled again while the concrete is still fluid and concreted again, thus avoiding a potential structural defect. AME also allows for better control of total concrete volume, potentially minimizing grout overruns and saving costs (both direct cost of grout as well as indirect cost of removal of excess grout on ground surface)

Quality of augered piles ("Estacas hélice continua") is greatly improved by Automated Monitoring Equipment (AME). AME is specified in many codes including the United States' Federal Highway Administration GEC#8 (Geotechnical Engineering Circular No. 8) (Brown et al, 2007). GEC#8, for example, requires 2 ft (61cm) depth increment accuracy and a magnetic flowmeter to measure volume.

Thermal Integrity Profiling

Cement produces heat during the curing process. This phenomenon is the basis for thermal profile evaluation of the entire cross section of bored piles ("Estacas escavadas") and is described in more detail by Piscsalko et al (2013). During the curing of a bored pile, the center of the pile has the highest temperature while the perimeter has the lowest temperature since it is adjacent to the soil and the heat is flowing from the pile into the soil. The more cement content in a concrete mix, the higher the temperature created. Conversely, if any section of the pile has a significantly lower temperature it is due to a lack of cement content surrounding the sensor (e.g. perhaps a necking, inclusion, or contaminated concrete). Figure 9 shows a test shaft with purpose built defects (inclusions attached to the reinforcing cage), respectively covering about 7% (at 20 ft, 6m) and 5% (at 47 ft, 14m) of the cross section, which are clearly identified by the sharp drop in temperature (at 20 and 47 ft; 6 and 14m), as shown by the low temperature orange lines at depths noted by red arrows.

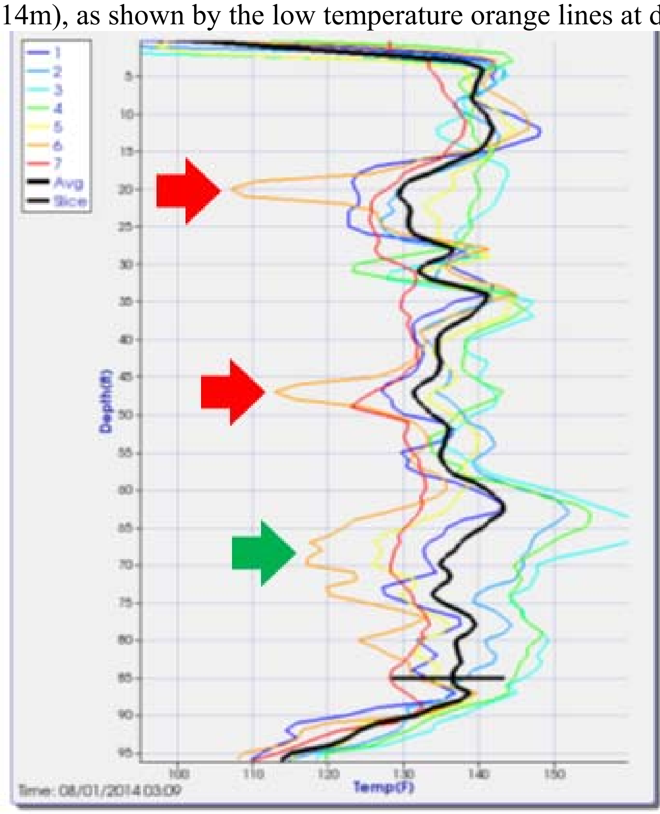

Figure 9: shaft with defects (arrows)

The Thermal Integrity Profiling method procedures are governed by ASTM D7949. It is convenient to measure the temperature at the reinforcing cage by attaching cables with thermal sensors to the reinforcing cage. One such instrumented cable is installed equidistantly around the cage for each 300 mm of pile diameter. The average temperature of the shaft can be correlated to the effective average shaft radius. Local deviations from the average shaft temperature can then be related directly to deviations from the average shaft radius, allowing a 3D evaluation of the effective concrete cover outside the reinforcing cage. Figure 10 shows evaluation of the thermal data transformed into effective radius (left) and a 3D image (right – this image can be rotated to any view angle). This shaft had an oversized casing for the top 28 ft (8.5 m), and thus an increased effective radius, accounting for the higher temperatures near the top. Data also confirms the radius exceeded the design radius ("Shaft") so sufficient concrete cover around the reinforcing "Cage" was assured. It should be noted that evaluation by thermal testing can often be completed within 24 hours of casting concrete, and this can greatly speed up the approval process for construction.

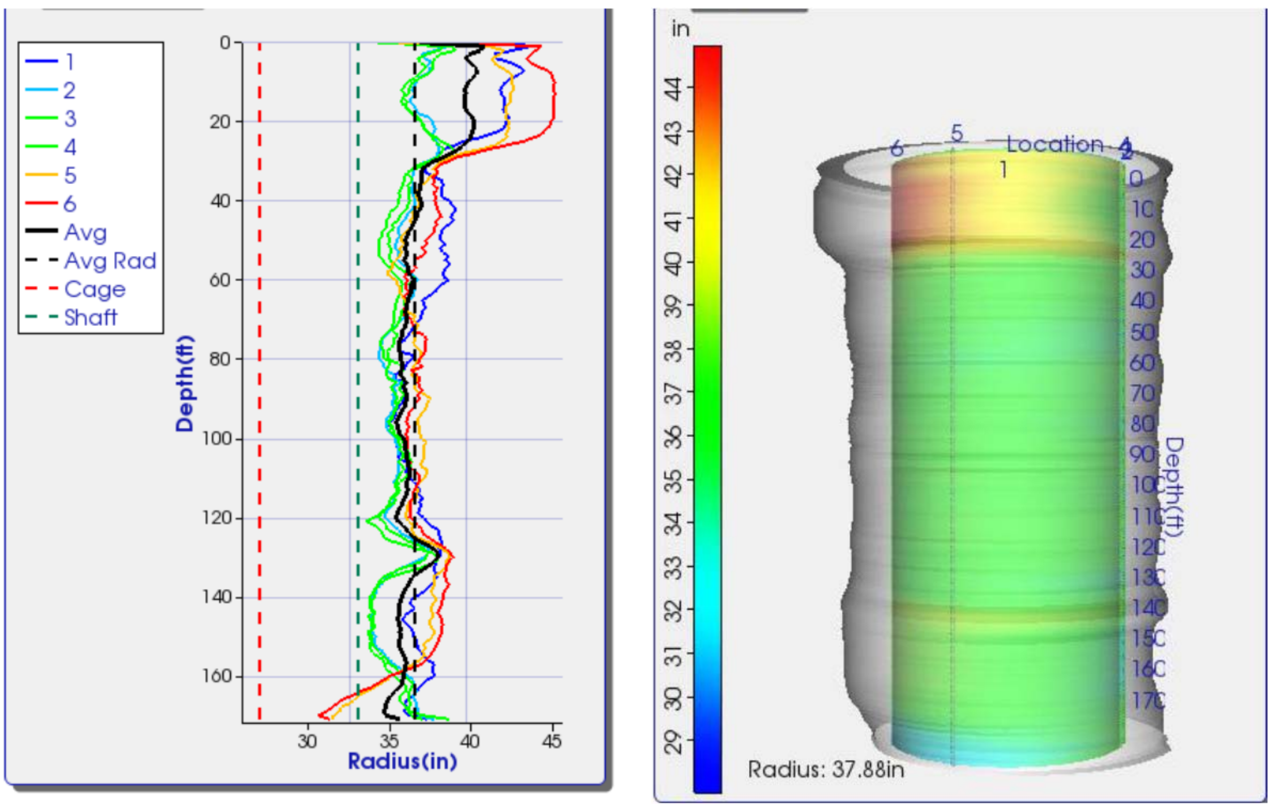

Figure 10: Radius versus depth (left) and 3D image right from thermal measurements

The reinforcing cage alignment can also be evaluated since if the reinforcing cage is not concentric with the hole, then one side of the cage will be closer to the center, and thus warmer, while the diagonally opposite side is closer to the surrounding soil, and therefore cooler. This is demonstrated by the green arrow in Figure 9 at depths 60 to 75 ft (18 to 23 m), and in the top cased section shown in Figure 10, where the oversized casing is not concentric with the main shaft (wire 6 is warmer than average (AVG) while opposite wire 3 is cooler).

<u>Geotechnical Capacity Evaluation</u>

<u>Inspection devices</u>

For bored piles ("Estacas escavadas"), when end bearing is considered in the design, the condition of the bottom of the drilled hole is important and must be "clean", meaning loose sediment removed, so that end bearing is activated at a relatively small displacement rather than first compressing a weak debris layer. This is particularly important in rock sockets. While not directly measuring pile capacity, there are devices with a thin measuring rod which penetrate the soft sediments (rod penetration into the sediment is viewed with a camera) used to inspect the cleanliness of the bottom surface. More advanced inspection tools actually measure the force and distance required to penetrate any potential debris layer and also the resistance in the bearing layer using one or more 10 cm^2 instrumented cones (Figure 11). When the hole is confirmed as clean and the measured bearing force is adequately confirmed, the end bearing may be included in the design. Such a device can be particularly cost effective to minimize the depth of a rock socket by determining when the rock is of sufficient strength.

Figure 11: Shaft bottom quantitative inspection tool (cone probes measure force [arrow 1] while the bottom moveable plate measures deflection [arrow 2])

Wave Equation Analysis

Intuitively there is a relationship of capacity to observed blow count and hammer energy for driven piles ("Estacas cravadas"). This relationship is the basis for all discredited dynamic formulas. However, dynamic formulas do not consider pile type or dimensions, hammer configuration or actual hammer efficiency (Allin et al, 2015), driving system components, or soil type and profile. An approach to rationally include these various components of the pile driving process was developed initially by E.A.L. Smith (1960) and is now commonly referred to as the "wave equation". The hammer and pile are modeled by a series of masses and springs and an initial impact velocity of the ram imposed. The soil is modeled with both springs, representing the static capacity, and dampers, representing viscous effects. Using short time increments the resulting forces in the springs and motions of all element masses are computed as time progresses. Thus, the maximum stresses at every location in the pile can be evaluated for any modeled situation, and the final net displacement calculated for any assumed capacity. Generally several capacities are assumed and the corresponding blow counts computed and the resulting relation of input capacities to computed blow counts is known as a bearing graph. If the soil profile is accurately modeled, capacity can be calculated for any depth by static analysis and the installation analyzed at various depths of penetration as a check for pile driveablility to assure the selected hammer is capable of installing the pile to the desired depth or capacity while keeping driving stresses within acceptable bounds. While not a "test" in the strict sense of the word, the wave equation has proven to be valuable in assessing compatibility of the hammer with the pile for a specific soil profile, and to evaluate the driving stresses in such scenarios to prevent pile structural damages.

Static Load Testing

The load carrying capacity of both driven piles ("Estacas cravadas") and bored piles ("Estacas escavadas") has traditionally been evaluated by static load tests (SLT). Using either dead weights or reaction piles (Figure 12), the test pile is jacked against a frame either in compression (ASTM D1143) or in tension (ASTM D3689). For larger test loads ASTM D1143 requires an instrumented load cell. Because of safety concerns, the test frames must be designed and approved by professional engineers, and it is better to incorporate electronic displacement devices with an automatic recording system so that operating personnel do not approach the actual test pile.

Figure 12: Static Load tests by dead weights (left) and reaction frame (right)

Although setting up a load test takes considerable time and effort, the test can be run with load increments applied at short time intervals, with tests sometimes completed in a few hours, or maintained for longer durations, perhaps requiring multiple days and overnight presence (Figure 12). Testing requires considerable effort, time and cost to perform and thus testing is limited to a very small percentage of piles. Static tests are most common on larger projects, and often in a special test program (e.g., in advance of the final design, and perhaps in a completely separate contract) so that results can be incorporated into the design and into the bid documents for the contractor. The maximum applied loads ideally cause a significant net settlement so that the ultimate capacity is determined, allowing the foundation design to be optimized. Applying lesser loads simply produce lower bound "proof loads" that confirm adequacy for the design capacity. Because it is accepted as the true measure of pile capacity, this test is generally awarded with the lowest safety factor. Unfortunately, the cost of a static test is prohibitive compared the overall foundation cost for most smaller projects.

Bi-directional Load Testing

Bi-directional load testing is a variant of static load testing and was developed first in Brazil by Pedro Elisio Da Silva (1983) and independently by Dr. Jorg Osterberg (Osterberg, 1984, 1994). Rather than placing the jack at the top of the pile, the jack is attached to the reinforcing cage and inserted in the pile (Figure 13).

While the embedded jack is often placed near the bottom of the shaft, it can be placed at any location along the length. When pressurized it then exerts a downward force below the jack (resisted by end bearing and any shaft resistance below the jack) and an upward force above the jack (resisted by the pile weight and the shaft resistance above the jack). The test is run until either the soil fails (continued movement with little increase in load), either the shaft resistance above the jack or the total resistance below the jack (end bearing plus shaft resistance, if any), or the maximum pressure of the jack is reached, or the jack achieves it maximum expansion. The force is computed solely from jack pressure; because the test rarely fails the soils simultaneously above and below the jack, the test result is then generally conservative, and thus any error in determining the force soley by jack pressure is not usually of serious concern. Strain readings are often additional measurements taken at various locations along the pile shaft. Displacements are read both above and below the jack and are plotted against the applied jack load.

Figure 13: hydraulic jack attached to bottom of reinforcing cage of a bored pile

The upward and downward force components are combined into an equivalent applied load using strain compatibility principles. Advantages are the improved safety of this system and the applied load is reduced, typically to half the equivalent mobilized capacity. However, the jack is not recovered and no load cell is reasonably possible (not considered a serious concern as noted previously). Further, unlike a conventional load test where the load is applied at the pile top as in service conditions, in a bi-directional test the maximum load is applied at the load cell, often near the pile bottom, and the pile top usually has zero applied load (so structural integrity of this critical pile location remains unchecked by this test method). Therefore, some form of integrity evaluation is recommended in addition bi-directional tests to confirm the structural adequacy of the test shaft.

Dynamic Load Testing

Based on a twelve year research study at Case Western Reserve University starting in 1964, a procedure was developed to measure force and velocity on driven piles ("Estacas cravadas") using strain and acceleration sensors (Figure 14a). The force can also be measured with a "top transducer" (e.g. instrumented heavy-wall pipe, as in Figure 14b), which avoids uncertainties in concrete modulus of bored piles. ASTM D4945, "Standard Test Method for High Strain Dynamic Testing of Deep Foundations", specifies how dynamic monitoring is to be accomplished. The initial purpose of the research study was to evaluate these measurements to determine pile capacity for driven piles.

Figure 14: (a) Acceleration (right) and strain (left) transducers on pile to measure force and velocity, with wireless transmitter. (b) instrumented "top transducer"

The force and velocity data are analyzed by closed-form solutions of wave propagation which are called the "Case Method" after the University where they were developed (Rausche et al, 1985). The capacity evaluation (R) is done immediately on site blow-by-blow using Equation 1:

$$R = (1-J)(F(t) + Z\,V(t))/2 + (1+J)(F(t+2L/c) - Z\,V(t+2L/c))/2 \qquad (1)$$

where F and V are the measured force and velocity respectively at times t and t+2L/c, where L is the pile length below sensors and c is the pile wave speed. The pile impedance, Z, is the product of pile material density, wave speed, and pile cross sectional area. The damping factor, J, is related to the soil type, typically ranging from 0.4 for coarse grained soils to 1.0 for cohesive soils. This capacity equation is searched over the duration of the impact for the maximum value of resistance R. The force and velocity data can be further analyzed in a rigorous "signal matching" program (Case Pile Wave Analysis Program or "CAPWAP") to extract wave equation-like soil model results, including resistance distribution along the shaft and at the toe (Rausche and Goble, 1972). CAPWAP has demonstrated good correlation with static load tests (Likins and Rausche, 2004). Signal matching is considered "state-of-practice" and most codes require this more rigorous CAPWAP© signal matching for the final capacity evaluation. An instant real-time signal matching (iCAP©) is another alternative to assist evaluation of data as the pile driving is progressing (Likins et al, 2012b).

It was recognized the measurements could also provide insight into driving stresses, inspect shaft integrity, and evaluate the pile driving hammer performance to assess better ways to install driven piles. The energy transferred to the pile can be determined from the equation

$$E(t) = \int F(t)\,V(t)\,dt \qquad (2)$$

and the maximum value of E(t), often referred to as EMAX or ENTHRU, is the best performance indicator and should be compared with the manufacturer's rating.

Stresses at the pile top are directly obtained from the strain measurements. Using one-dimensional wave propagation theory, the average compression at the pile toe and the maximum tension at any location along the shaft can be evaluated from the pile top measurements. Keeping these stresses below the recommended limits based on structural material properties reduces the possibility of pile damage (Hannigan, 2006). For piles with uniform cross section the force should always increase relative to the velocity during the first 2L/c after initial contact. If this is not the case it likely indicates a reduced cross section or pile damage. Figure 15 shows a 41 m long 380 mm square section concrete pile. The upper graph shows the force increasing relative to velocity in the first 2L/c (vertical lines), while the lower graph shows a relative velocity increase about mid pile length (starting at middle dashed vertical line), indicating a damaged to the spliced pile joint. The extent of damage (BTA) and depth are estimated from the data. Further discussion of this valuable additional benefit of dynamic testing can be found in Rausche and Goble (1979) and Likins and Rausche (2014).

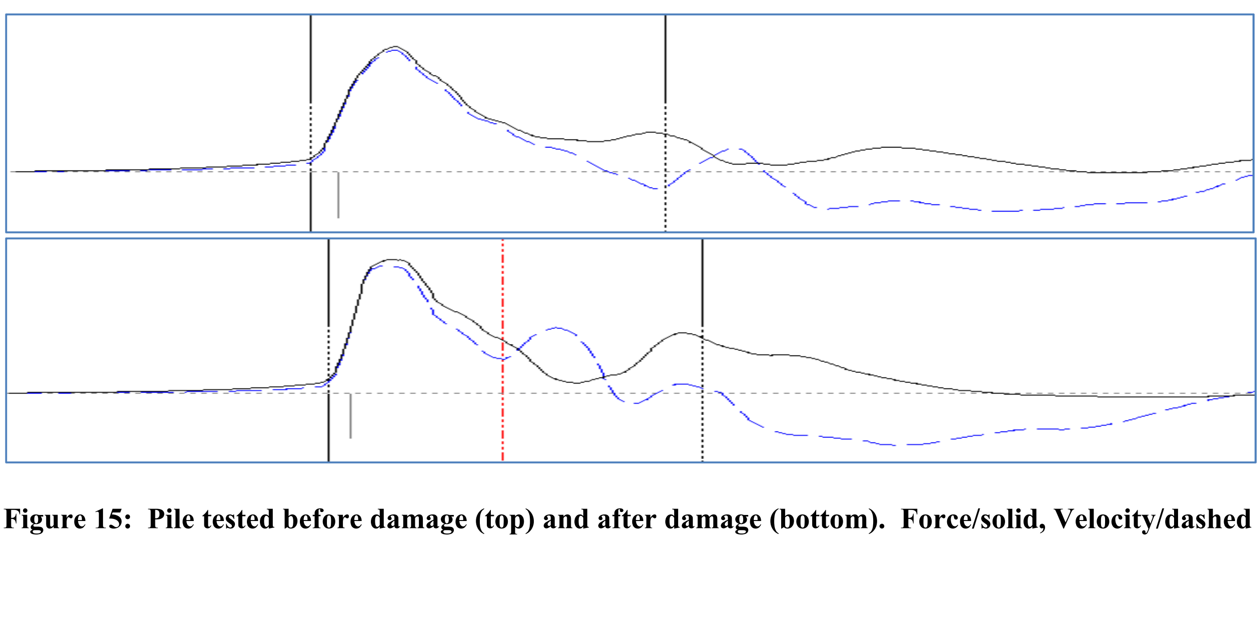

Figure 15: Pile tested before damage (top) and after damage (bottom). Force/solid, Velocity/dashed

The convenience and relatively low cost of this dynamic test method allows testing during the entire installation, and in restrikes, and is usually applied to several piles on site to evaluate site variability and aid in selection of the driving criteria for production piles. By knowing the driving stresses, pile damages can be reduced. By measuring the energy transferred to the pile, efficiency of installation can be improved.

Using large drop weights, the method has been applied successfully worldwide also to bored piles ("Estacas escavadas") for many decades (Rausche and Seidel, 1984, and Seidel and Rausche, 1984). Typical required drop weights are 2 percent of the desired ultimate capacity.

Benefits of Load Testing

The low ratio of testing cost to benefit of reduced cost foundation through a lower safety factor has resulted in more testing, and specifically worldwide acceptance of dynamic testing. The value of testing can be illustrated by an example. The more confidence is given to any particular method of capacity evaluation, the lower the assigned factor of safety can be. Suppose we have a 40,000 kN load to support and that the ultimate capacity of each pile is 2,000 kN. Dividing the pile capacity by the factor of safety (F.S.) for each method of capacity determination yields a design load per pile and dividing the design load into the total load yields the number of piles required to support that load. The results are shown for the AASHTO (American Association of State Highway and Transportation Officials) Allowable Stress Design (ASD) factors in Table 1.

Table 1. Number of piles required for example case for AASHTO ASD method (used prior to 2007)

Determination method	F.S.	Design load kN/ pile	# of Piles required
Dynamic formula	3.5	571	70
Wave equation	2.75	727	55
Dynamic testing	2.25	889	45
Static testing	2.0	1000	40
Static & Dynamic testing	1.9	1053	38

Fewer piles are needed for better testing. These ASD factors of safety produced successful designs for several decades of highway bridge construction. There was no specific guidance for the amount of static or dynamic testing. Since 2007, AASHTO has used a Load and Resistance Factor Design (LRFD). As shown in Table 2, using their specified LRFD resistance factors (phi), LRFD load factors (1.25 dead and 1.75 live), and assuming a typical dead to live weight ratio of 3 for bridges, the required numbers of piles are similar to Table 1. (The equivalent Factor of Safety is shown). Note that testing 100% of the piles dynamically is given the same benefit as testing a pile statically. Guidance has been given for the suggested minimum amount of testing.

Table 2. Number of piles required for example case for AASHTO LRFD

Determination method	Φ	Equiv F.S.	Factored resistance kN / pile	# of piles req'd
Gates formula	0.40	3.44	800	69
Wave equation	0.50	2.75	1000	55
Dynamic_test (min.2% or 2#)	0.65	2.12	1,300	43
Static_test **or** 100% Dynamic test	0.75	1.83	1,500	37
Static_test **and** >2% Dynamic test	0.80	1.72	1,600	35

The main cost of a foundation is in the material and installation of the piles. Thus reducing the number of piles dramatically, as shown above by testing, reduces foundation costs. The cost of testing the piles is then almost incidental, and actually lowers the overall cost of the foundation as seen in the following example. The Ohio Department of Transportation tracked driven pile costs and testing costs over a six year period from 2005 through 2010 (Narsavage, 2011). Total cost for the driven piles was $123,600,000. The testing costs (mainly for dynamic testing) for the same period were $2,556,000, or roughly 2% of the piling costs. With higher confidence than AASHTO, Ohio Department of Transportation uses a higher, more-advantageous resistance factor (0.70) for dynamic testing. Table 3 presents relative pile costs and projected savings compared to just using a dynamic formula to determine capacity. Based on the LRFD resistance factors (phi), a 43% cost savings was achieved for the 2% investment in dynamic testing cost, resulting in a greater than 20:1 benefit to cost ratio. Obviously, testing is beneficial to reducing costs in addition to improving the foundation quality. Testing also reduces risk since measurements replace uncertainty.

Table 3: Estimated savings based on assigned LRFD resistance factors

Method (LRFD)	AASHTO PHI	Relative cost of piles	Savings
Formula (Gates)	0.40	1.00	0%
Wave Equation	0.50	0.80	20%
2% PDA	0.65	0.62	38%
2 # PDA Ohio DOT	0.70	0.57	43%
100%PDA or SLT	0.75	0.53	47%
PDA + SLT	0.80	0.50	50%

Force Pulse (Rapid) Load Testing

By applying a temporary force pulse, a significant compressive force can be applied to the pile top. This can be achieved by either a large drop weight on a highly cushioned pile top (e.g. Fundex Pseudo Static tester uses large stiff springs as per Schellingerhout, 1996: or using a dynamic testing system with very thick plywood cushions as per Rausche et al, 2008), or from a burning of combustive fuel lifting a heavy reaction mass (e.g. Statnamic) (Bermingham and Janes, 1989). Figure 16 shows typical alternate systems.

Figure 16: (a) Fundex mass with heavy spring cushion, (b) cushioned dynamic test, (c) Statnamic with catch mechanism, (d) Statnamic with gravel containment housing (for larger loadings)

Procedures for rapid load testing are outlined in ASTM D7383. Typical required drop weights or reaction masses are 5 to 10 percent of the desired ultimate capacity. The advantage of a relatively long duration pulse (typically only 0.1 second duration) is tension stresses in the pile are of little concern. However, since the force pulse is still a fraction of a second, dynamic resistance forces and inertia forces must be considered, and unless the pile is further instrumented (e.g. using "sister bars") resistance distribution cannot be deduced from this test. There are widely differing opinions on how to deduce the equivalent static test from the basic measurements, particularly in cohesive soils (e.g. Middendorp, 1992; Matsumoto, 1994; Hajduk, 2000; Schmuker, 2005; Weaver, 2010; Brown, 2013). If there is a significant net settlement (minimum 3 percent of pile diameter), then the determined capacity is "fully mobilized" and is perhaps considered more reliable (Miyasaka et al, 2009). AASHTO has not assigned LRFD resistance factors (phi) for rapid load testing. In addition to axial tests, the Statnamic device has been deployed to apply lateral impacts, which help model ship impacts for example.

Summary

Testing plays an important role in the success of any deep foundation. Lack of structural integrity or insufficient pile capacity can result in failure of the foundation to properly carry the load, and ultimately then to high remediation costs or demolition of the structure. By nature, however, deep foundation elements are buried in the ground and cannot be inspected visually after installation. Certainly proper installation techniques and inspection during installation are important, but not by themselves sufficient. Early methods of evaluation included static load tests which are time consuming and expensive, but still a viable option for a limited sample of piles and still recommended for large projects. Static analysis of soil boring information is another option, but the resulting needs to consider a very conservative approach to avoid failures. For driven piles, the blow count can be inspected, but reliance on dynamic formulas also carries an unacceptably high risk and is therefore avoided by most knowledgeable engineers.

Modern indirect methods for evaluating the structural integrity and the geotechnical capacity have been developed and are discussed in brief detail. References are given for further study of each method presented. Each method has advantages and disadvantages. Clearly any testing helps to reduce risk. Codes often give economic incentives such as lower safety factors (or larger resistance factors in LRFD projects) which can result in significant cost savings as a result of testing, and therefore overall significantly lower costs for the installed foundation as well as less risk for failure and expensive remediation.

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