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NON-DESTRUCTIVE EVALUATION OF DEEP FOUNDATIONS

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ABSTRACT

Non-Destructive Testing (NDT) or Non-Destructive Evaluation (NDE) of a great variety of materials and structures has become an integral part of many manufacturing processes. The same tendency towards more testing for improved quality assurance is also apparent in the deep foundation industry. However, the process of testing long piles, deeply embedded in the ground, is more complex than the NDE of other materials: the product can only be accessed from its smallest side and the material is often concrete or timber, which are rather heterogeneous materials with unreliable properties. The greatest difficulties, however, are presented by the intimate contact between pile material and soil, causing dissipation of the NDT energies to varying degrees.

Nevertheless, there has been progress made with improving these methods and they have been employed in a wide variety of scenarios. This paper summarizes the most common non-destructive test methods and gives a few examples of applications with an emphasis on a demonstration of their benefits and limitations.

INTRODUCTION

In deep foundations the term Non-Destructive Evaluation is usually associated with “Integrity Testing” in other words with methods of checking the structural condition of the foundation members. Geotechnical quality, i.e. sufficient bearing capacity and limited settlements, have to be checked by analysis and load testing. Actually, even a load test may also fall within the general classification of NDT, since load testing does not cause any removal or destructive investigation of any material, even though the soil may fail during this test. With few exceptions, the foundation will be able to perform its function after a load test, however, with slightly changed load bearing characteristics. For this reason and adhering to the common convention, load testing will not be considered a non-destructive testing method and therefore will not be considered in this paper.

There are two basic applications for NDT of deep foundations:

(a) quality assurance of a newly constructed foundation element and
(b) evaluation of an existing foundation

For quality assurance of new foundations it is generally wiser to carefully monitor the installation process itself than to check the finished product. This is possible for driven piles using a Pile Driving Analyzer® (Rausche et al., 1976) and for Augered Cast-in-Place (ACIP) piles with a Pile Installation Recorder™ (Likins et al., 2002). However, for Cast in Drilled Hole (CIDH) piles, more commonly called drilled shafts, no simple objective installation monitoring method exists and follow-up integrity testing is therefore often desirable.

For existing foundations, often the length has to be ascertained for a check of the pile bearing capacity by standard geotechnical methods. This task is complicated by a lack of direct access to the pile top due to structural members attached to the pile, which prevent its free vibration and therefore limit the applicability of the dynamic method. A review of available methods for the assessment of existing foundations has been made by Olson, 2003 and some case studies have been described by Hussein et al., 1992. The literature contains many other papers that summarize a variety of methods and present case studies. Among those papers are Davis et al., 1991; Rausche et al., 1992; Baker et al., 1993.

A number of countries have included references to NDT methods for foundation piles in their building codes among them Australia (AS 2159, 1995), China (CABR, 2002) and the United Kingdom (ICE, 1988). Other countries have standardized the test methods. Examples are France (Norme Francaise NFP94-160-1, 2,3), Germany (DGGT, 1998) and the United States of America (ASTM 4947-00, 2000; ASTM 5882-00, 2000; ASTM 6760-02, 2000).

DESCRIPTION OF AVAILABLE METHODS

Frequently referenced and/or utilized Integrity Test Methods for driven piles, cast-in-place piles or drilled shafts are listed and evaluated in the following.

Pulse Echo Method (PEM or PIT, Pile Integrity Test)

The test method was first utilized in the 1970s in Europe and in the US (Steinbach et al., 1975). However the method became widely accepted only after it became possible to apply digital signal processing methods (Reiding et al., 1984; Rausche, et al., 1988). PEM is probably the most commonly employed NDT method for concrete piles, both driven and drilled, and many case studies can be found in the literature. Using a small hand held hammer, the pile top is lightly hit and the ensuing pile top motion is measured with an accelerometer or geophone.

- Advantages: little pile preparation needed, therefore, spot checking possible; quick and inexpensive; gives information about major defects both as far as severity and vertical location.

- Disadvantages: records require experienced interpretation; not all records are conclusive; length limitation of 60 diameters under good circumstances; multiple defects or those below the limiting length cannot be detected; defects of small extent in vertical or horizontal direction cannot be detected; does not give information about horizontal location of a defect; accuracy of length or distance results depend on assumed wave speed.

- Output is a plot of the filtered, amplified pile top velocity vs. time. Records can be analyzed by signal matching to yield an indication of defect size or by the Impedance Log (Paquet, 1991) or Pile Profile Method (Rausche et al., 1992), however, these more advanced analysis methods require assumptions as to the effect of soil resistance on the recorded signals.

Vibration Method, VM

The pile is excited with a variable frequency oscillator installed on top of the pile (Davis et al., 1974). Measurements include the applied force and the velocity response of the pile top as frequencies are varied.

- Advantages: Provides quantitative results

- Disadvantages: Difficult and relatively costly to implement because of the requirement that an oscillator is firmly connected to the pile top and has sufficient energy to cause resonance in the pile; does not provide more information than PEM or TRM; requires experience for interpretation. There is no unique relationship between resonance frequency and type of pile impedance variation (such as increase or decrease of cross section).

- Output includes Mobility and Pile Stiffness: After Fourier transform, the Mobility is calculated as the ratio of velocity divided by force and plotted as a function of frequency. Resonance frequencies yield pile length or distance to a change in pile impedance. The slope of the mobility at zero frequency is a relative measure of the (static?) pile stiffness.

Transient Response Method (TRM)

A concept very similar to both PEM and VM, the method requires an impact of a handheld hammer. In fact, this method provides for the same results as VM but requires much less costly equipment and effort (Rausche et al., 1991). The impacting hammer is instrumented thereby allowing for the measurement of the pile top force in addition to motion. The analysis is similar to that of the Vibration Method with limiting frequencies a function of the hammer weight. Additionally, display of both velocity and force vs. time is also useful.

- Advantage: collects more information than PEM, which helps with identifying defects near pile top; provides all quantitative outputs of VM.

- Disadvantage, slightly more expensive equipment needed than for PEM; has all limitations of PEM.

- Output as discussed in VM includes mobility and dynamic stiffness, however, all of the results of PEM also can be obtained.

Two Accelerometer Method (TAM)

This method is also related to PEM and is particularly useful for the testing of existing structures where at least 1.5 m of the deep foundation is exposed. Rather than measuring the motion at the pile top surface it is measured at its side (Rausche et al, 2002). Yields additional information and therefore allows for more reliable interpretation of records which are influenced by reflections from a structure existing on top of the deep foundation.

- Advantage: signals from two accelerometers at two locations allow for a back calculation of the wave peed of the pile material; as simple and inexpensive a method as PEM; two signal recording provides for separation of reflections from the lower pile portion from those of the pile top or anything that is attached to it.

- Disadvantages: requires that a section of pile is exposed; unless sufficient free distance between accelerometers is available, results may be difficult to interpret; other disadvantages as for PEM

- Output includes two velocity records vs. time as for PEM. In addition, the two velocities can be used to calculate the downward wave, i.e. any effects of non-impact caused downward waves such as reflections of a structure attached to the pile top, can be eliminated by analysis from the plotted output.

Bending Wave (BWM)

Similar to TAM, this test measures the pile top bending response to an impact applied perpendicular to the pile axis thereby producing flexural waves (Douglas, et al., 1993). Not much information exists though the method is being used in the South-Eastern United States.

- Advantage: perhaps a greater wave energy in pile than compressive waves by PEM or TAM.

- Disadvantage: since bending waves are a dispersive medium, they have different wave speeds depending on their frequency. This reduces the accuracy of the method and makes interpretation complex. Pile length limitation is more severe than for PEM.

- Output may identify pile length in either a time and/or a frequency plot. Analysis is done by the so-called Kernel method, which extracts pile length by finding reflections that match he impact.

Case Method (HSM for High Strain Method)

In conjunction with a dynamic load test or on driven piles this is an economic solution; frequently conducted during the pile installation process, it allows for quantifying pile defects (Rausche et al., 1976). Can also be used on CIDH piles, however, testing setup is more involved than for other methods (Likins et al., 1995).

- Advantages: not much extra cost for driven piles; applicable to any type of pile material; provides for bearing capacity evaluation at the same time as it provides integrity information.

- Disadvantage: as an NDT method rather involved for cast-in-place piles; vertical resolution on drilled piles not as clear as PEM; requires experienced personnel and interpretation.

- Output includes a so-called β factor which is the ratio of reduced impedance to pile top impedance. Thus, $\beta=1$ for an undamaged pile.

Cross Hole Sonic Logging (CSL)

This method is frequently used for drilled shaft evaluation; requires that water filled tubes (typically 50 mm diameter) be installed in the test piles over their full length. Test equipment includes an ultrasonic transmitter and a matched receiver. Test procedure requires insertion of the transmitter and receiver in these tubes, lowering and raising them simultaneously while the transmitter continuously sends out and the receiver acquires the ultrasonic signals. Plotting elapsed times between transmitted and received signal vs. depth yields an assessment of the concrete quality located between the tubes.

- Advantages: clear resolution in vertical direction; no pile length limitation; compared to other NDT methods, somewhat simpler interpretation.

- Disadvantages: test needs some interpretation by experienced personnel; piles have to be prepared with tubes prior to pouring of concrete; for smaller piles or those with no rigid reinforcement cage, well-aligned tube installation difficult; checks only concrete between tubes; properties of concrete cover outside of reinforcement cage are usually not checked.

- Standard output is the First signal Arrival Time (FAT) which can be converted to wave speed, assuming that the distance between the tubes is constant along the full pile length. Recently added analysis extensions include the energy of the signal received and a calculated tomography result, which can produce a 3-D display of the perceived shaft quality. Tomography requires that at least 6 scans of the pile are made and, in cases where both horizontal and vertical extent of a defect must be delineated, scans with transmitter and receiver at different levels.

Single Hole Sonic Logging (SHSL)

SHSL uses the same equipment as CSL, however, it requires only a single tube installed in the pile and is therefore better suited than CSL for testing of small diameter piles such as augered, cast-in-place piles. Amir, 2002 has investigated technical details and limitations of this method.

- Advantages: can be used on smaller diameter drilled piles; single tube means less expense than two or more tubes; no pile length limitation.

- Disadvantages: in addition to the CSL disadvantages a lesser resolution of vertical extent of defects and uncertainty about the horizontal extent of the volume of concrete subjected to the test.

- No particular analysis techniques exist other than plotting of signal arrival time or signal energy vs. depth.

Gamma-Gamma Logging (GGL)

Replaces the ultrasonic signal source of SHSL with a radioactive source. Also, requires installation of test tubes in the drilled shaft. The result is a count of photons received, which is inversely related to the density of the material that the radioactive material penetrated.

- Advantages: test involves some volume of concrete surrounding the test tube. Therefore, GGL can be used to draw conclusions on the quality of the concrete surrounding the reinforcement cage; estimates are that 75 mm of concrete surrounding the probe are checked.

- Disadvantages: requires tube installation; test needs some interpretation by experienced personnel and calibration for wet and dry conditions; for smaller piles or those with no rigid reinforcement cage, accurate tube installation becomes difficult; resolution in vertical direction and assessment of horizontal extent of defect not clear; requires handling, storing of nuclear material.

- Output is practically a concrete density vs. depth plot.

Parallel Seismic Testing (PST)

PST requires drilling a hole parallel to the existing deep foundation, filling it with water and lowering a hydrophone in this borehole. The foundation is hit repetitively with a handheld hammer and the hydrophone is simultaneously lowered in the borehole recording pressure changes when the wave arrives. The wave to arrive first is one that travels through the foundation. Delayed arrivals are from stress waves which travel through soil or water. An increasingly delayed signal arrival indicates that the hydrophone has descended below the bottom of the foundation.

- Advantages: accuracy of length result is not dependent on an assumed wave speed.

- Disadvantages: requires a borehole near the foundation and an estimate of the depth to which this borehole should be taken. Typically not effective for embedded pile length much greater than 10 m. For steel piles the effective depth would be much more limited.

- Output is a plot of numerous hydrophone records vs. time, plotted over an appropriate depth scale. The pile length becomes clear from a change of slope of the signal arrival time in the time depth plot.

Parallel Inductive Field Test (PFIT)

Very similar to PST except that instead of sensing a sonic wave in a bore hole near the foundation, this method utilizes a metal detector to sense the proximity of steel piles.

- Advantages: can be used for steel piles, including steel sheet piles, of unlimited length as long as the borehole has been chosen deep enough and less than 750 mm away from the foundation; very simple interpretation.

- Disadvantages: not practical for concrete piles except when reinforced with significant amount of steel; not applicable for timber piles. Also, test meaningless, if distance between borehole increases beyond the limiting value (e.g. due to unplanned deviations from vertical).

- Output can either be an audible signal or a voltage, which can be recorded and used to construct a signal strength vs. depth plot.

EXAMPLES

PEM For Existing Tower Foundation

In recent years, many transmission towers had to be reevaluated to demonstrate their structural soundness. A

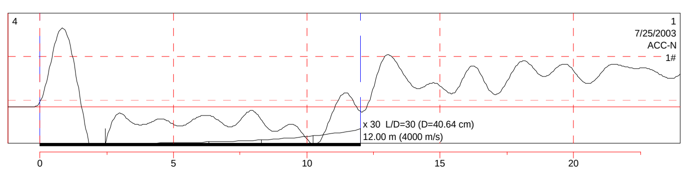


Fig. 1: PEM record from a tower foundation

common design is one which utilizes 3 legs and 3 drilled shafts. In one recent example, the record of Fig. 1 was acquired by the PEM method.

This record shows a clear reflection at a time that corresponds to a length of 12 m, a length that was calculated under the assumption of a wave speed of 4000 m/s. If the wave speed were in reality lower or higher, the resulting calculated length would be proportionally higher or lower. The 4000 m/s wave speed is a frequently used average value with variations of ±5 to 10%, on the same site, a common occurrence. The conclusion therefore should be that the pile length is probably between 11.4 and 12.6 m. In this case, all three legs had shafts of the same apparent length.

PEM For An Existing Bridge Foundation

Rausche et al. (2002) describe results from tests on an existing bridge foundation. Certain piers had settled and the PEM clearly identified piles with only 6 m length (Fig. 2 middle) while others, were 50 or 100% longer (Fig. 2 top, bottom). In this case a 3800 m/s wave speed had been chosen, yielding a somewhat more conservative length result than the 4000 m/s wave speed assumption of the previous example. The settlement pattern of the piers correlated very well with the lengths of the piles determined by PEM.

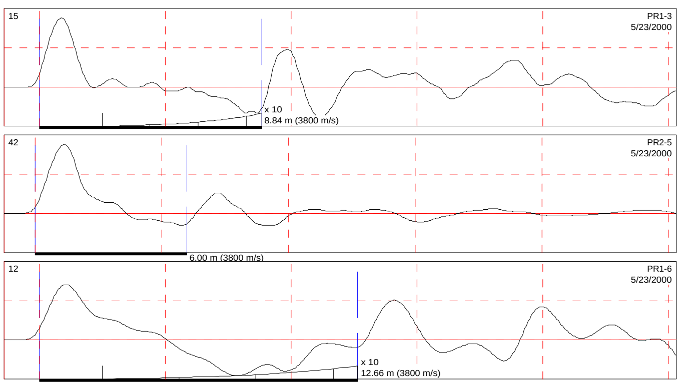
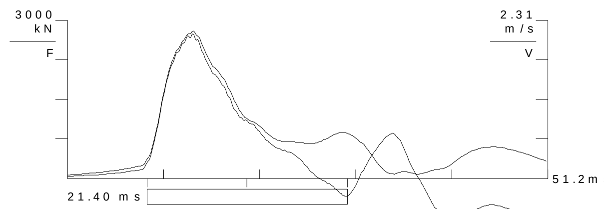


Fig. 2. PEM records from concrete piles under an existing bridge

Obviously, better monitoring during construction could have avoided this bridge failure. Also, it should be pointed out that a test on an existing bridge is less reliable than on a pile with a free top, because pier and deck cause downward reflections, which make the data interpretation difficult. In fact, in the present example, a frequency analysis (e.g. TRM) would not have yielded as clear a result as the evaluation in the time domain.

HSM During Construction on Spliced, Precast Concrete Pile

Concrete piles have to be driven carefully both when low soil resistance and high soil resistance conditions exist. When the soil resistance is low or of low stiffness, tension cracks often develop which affect the shape of a tension reflection. The example of Fig. 3(a) and 3(b) demonstrate, respectively, the performance of a 42 m long, 375 mm square, precast concrete pile, mechanically spliced at mid-length. The records of pile top force and velocity were plotted at a proportional scale. The shape of the impact signal on the left (where the pile top is indicated by a rectangle representing the pile) corresponds well with the shape of the pile toe reflection on the right (where the pile toe is indicated). Between these two signals, the two curves behave proportionally to about the center of the pile. There the velocity rises very slightly relative to the force, i.e. a small tension wave that is the result of a slight flexibility of the splice. The force then rises strongly relative to the velocity due to shaft resistance near the toe.



Top Toe

Fig. 3(a). Early PDA record of pile top force and velocity

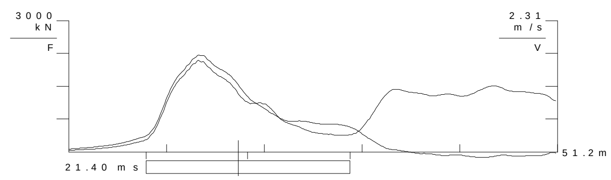


Fig. 3(b). Late PDA records

In Fig. 3(b), which was recorded approximately 300 hammer blows after Fig. 3(a), the relative increase of the velocity in the middle of the pile is now much greater, suggesting that the splice developed a greater flexibility and a β = 0.83. Beta values above 0.80 are generally classified as minor damage. However, such minor damage could be a tension crack over the complete pile cross section. Furthermore, the reflection from the pile toe now occurs over a much longer time and does not match the impact wave in shape. This shape difference can be attributed to cracks, probably in the neighborhood of the splice, which were caused by tension stresses and which filter the tension reflection wave from the pile toe, thereby altering its shape. Note that the PDA has drawn a short vertical line in the rectangular pile sketch where the splice reflection occurs; this line indicates the position of the tensile reflection.

HSM During Construction Of An H-Pile

Driving of high capacity steel piles has become a very competitive alternative to other foundation solutions. Driving stresses are easily in the 300 MPa range and while the strength of the steel is sufficient to sustain such stress levels, welded splices may crack during driving, if not carefully executed. Monitoring during pile driving using a PDA normally shows very clearly when a welded splice has failed, as it would indicate any other defect in the pile as long as it allows for a greater flexibility than the intact pile. Fig. 4 shows a record of force and velocity taken at the pile top of a 24 m long HP14x89 pile at the end of driving when the pile toe encountered a high resistance. At a time that corresponds to a depth of 11 m below sensors (or 12 m below pile top) a velocity increase, relative to the force, shows that a failure has occurred. The PDA calculated β = 0.84 for this cracked splice. However, the experienced test engineer would realize that the damage might involve the complete weld. It should be noted that none of the low strain methods (PEM, TRM, etc.) would work well on a steel pile. The reason is their low impedance (i.e. mass and stiffness) to shaft resistance ratio which causes the low strain energy imparted to the pile to be dissipated too quickly and not generate clear reflections from the pile bottom. The only method that can show a crack in a weld of a steel pile of significant length is the high strain

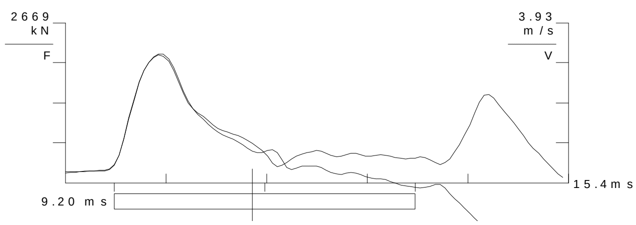


Fig. 4. PDA records of force and velocity from an H-pile with a cracked welded splice

method. However, it is an economical integrity test method only, if used during pile installation monitoring.

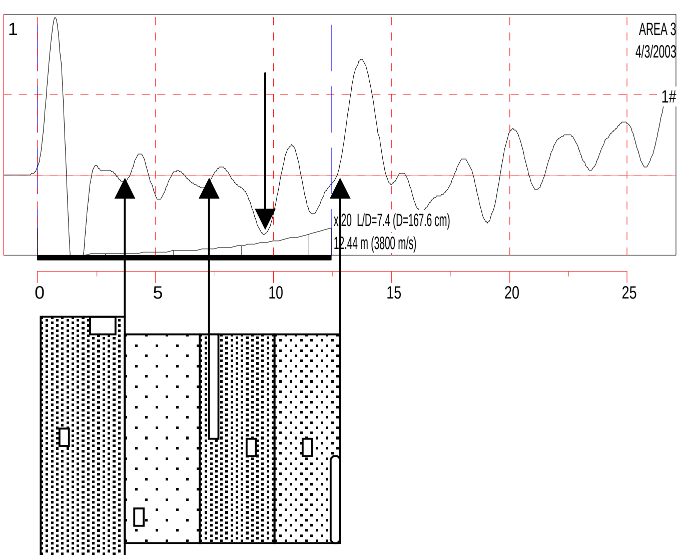


Fig. 5. PEM record and sketch of shaft with planned defects

<u>Comparison of PEM and CSL Methods on a CIDH Pile With Designed Defects</u>

An in-house funded research project by the author's company had as one goal the investigation of relationships between concrete strength and PEM and CSL results. A drilled shaft of 1.5 m diameter (roughly 1.65 m diam. over the upper 3.3 m due to a larger, temporary casing) was installed and subjected to repeat CSL and PEM tests over a 4-month time period. The shaft was poured with concrete qualities from the bottom up of 28, 42, 21, and 42 MPa, the latter mix was a so-call Self-Consolidating Concrete (SCC) whose properties were achieved by means of a superplastisizer. A horizontal sketch of the shaft, on the left side its top, is shown in Fig. 5 together with a PEM record. The rectangles in the sketch point out locations of defects. From the bottom upwards they include

- an air filled bucket plus a plastic cone (tip downwards) between1.8 and 2.1 m below the top;
- a styrofoam panel outside of the rebar cage at 2.4 m below the top;
- foam and duct tape wrapped around one of the tubes at 4.8 m;
- a horizontal styrofoam half-moon, 150 mm thick inside the cage at 6.7 m;
- a sand filled cone (tip downwards) at 8.5 m;
- a sand filled bucket at 10.8 m;
- a soft toe, i.e. loose sand dumped into the shaft bottom.

The buckets, cones and the outside styrofoam occupied only between 1 to 5% of the cross sectional area of the shaft and were therefore not expected to show up in the PEM records. The cross sectional reduction below the temporary casing (from 1.65 to 1.5 m diameter) amounts to almost 20% and therefore is expected to show up with a small reflection (as it does, where the left most arrow in Fig. 5 points to the beginning of the upward reflection). The stryrofoam half-moon whose area is approximately 35% of the shaft area and which was approximately 150 mm thick also caused a reflection, though not as clear as one would hope for, because of its limited vertical extend. Below the half-moon a strong negative reflection (downward arrow) is attributed to an increased soil resistance. At this point the number of reflections is already complicating the record too much for clarity. However, the pile toe signal is clearly apparent.

Fig. 6 shows for the same shaft the CSL results from the 4 major diagonal logs consisting of two interpreted curves vs. depth, wave speed and signal strength and then the so-called sonic map. The first, third and fourth scan clearly show the half-moon defect at a depth of 7.3 m (the second scan was going parallel to the edge of the insert and therefore shows the defect only faintly. The fourth scan also shows the air bucket and cone at 2 m and the sand bucket at 11 m. In the same scan, the defect at approximately 3.5 m depth is an unwanted one; it occurred when one of the PVC access tube connections inadvertently failed and leaked a fair amount of water into the fresh concrete. The second scan from the left also indicates the effect of the foam wrapped tube at 5 m depth.

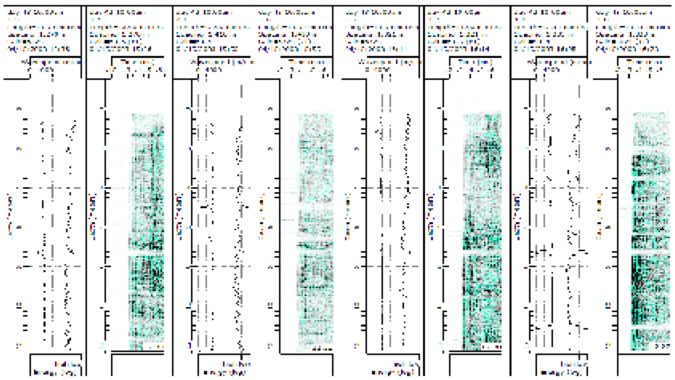


Fig. 6. Four main diagonal scans from CSL for PDI research shaft.

The profession often expresses the desire to receive more easily understandable NDE results. For that reason, efforts have been made to use tomography technology with the data from CSL measurements. An example of such a qualitative, graphical presentation is shown in Fig. 7. This tomography was based on 28 records including those shown in Fig. 6.

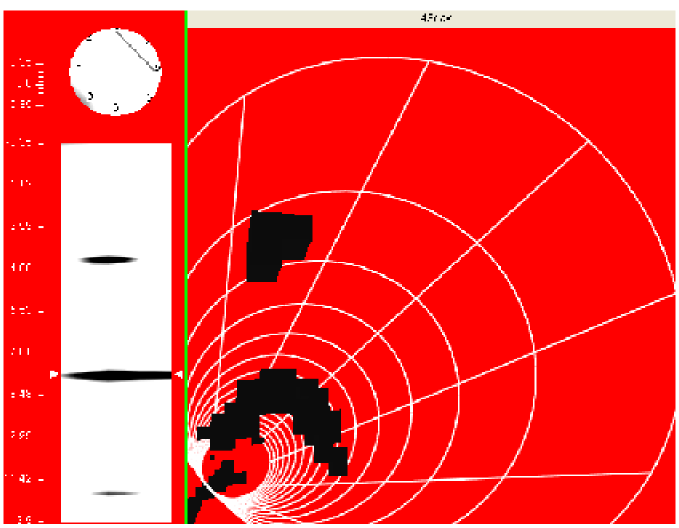


Fig. 7. Tomography of research shaft

The surprising and maybe disappointing result from this study is the lack of a clear relationship between wave speed measured by CSL and the design concrete strength. However, the actual concrete strength development was quite different from the expected values. As shown in Fig. 8, the normal 42 MPa mix only reached 36 MPa after 56 days while the SCC strength practically matched that of the 28 MPa mix. On the other hand, the 21 MPa mix did better achieving almost 24 MPa. Disappointingly, the strength values of the four different mix designs achieved only a spread of 50% not the 100% hoped for. On the other hand, the test very clearly shows that a 50% spread in concrete strength does not necessarily yield appreciable differences in wave speeds. It should be mentioned that the data of Fig. 6 was collected 43 days after the concrete installation by pumping.

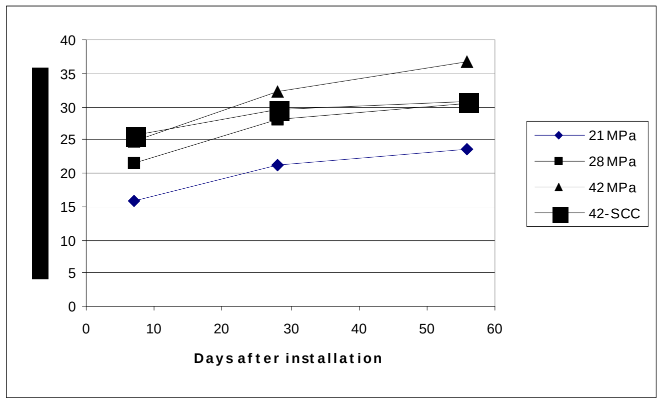


Fig. 8. PDI research shaft: concrete strength development.

SUMMARY

A variety of NDE methods are available to the foundation engineer for either checking the length of installed piles or the quality of new foundations. All of these methods have their individual benefits and limitations.

For driven piles, monitoring during installation provides clear evidence of even small defects at a modest cost without causing construction delays.

For drilled shafts, the most commonly used method is PEM, which has obvious limitations. However, for those experienced with this method, it allows for an inexpensive screening to detect the most seriously flawed shafts.

The CSL method is widely used for QA of major shafts. While it is very powerful in detecting most flaws between the inspection tubes, it does not detect those on the outside of the pile. The Gamma-Gamma method is the only method recognized to provide such information.

Other NDE methods, although frequently mentioned in the literature, are only occasionally used and their experience base is still limited.

Attempting to assess concrete strength from measured wave speeds may be a futile effort.

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