Pile Damage Prevention and Assessment Using Dynamic Monitoring and the Beta Method

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ABSTRACT

Driven piles are subjected to high stresses during installation. It is, therefore, important not to exceed acceptable stresses along the pile shaft and at the toe to prevent damage. Dynamic monitoring has been used for decades to evaluate not only the installation stresses, but also check test piles for signs of structural damage. The Beta Method (ß-Method) for evaluation of the *location and extent of a potential damage* was developed over thirty years ago and has proven effective as a QC and QA tool. As an aid in the process of pile rejection or acceptance, the ß-Method also offers a rating scale which translates the automatically determined ß-number into a helpful pile integrity assessment tool. The reliability of this algorithm has been proven by numerous extracted piles. However, one limitation of the ß-method concerned detection of damage near the pile toe where high toe resistance effects and/or stress wave reflections reduce the effectiveness of the traditional ß-Method. In the past, therefore, near-toe damage was determined by the testing engineer, not only by visual inspection of the dynamic monitoring data, but also by reviewing the pile toe compressive stresses throughout the monitored driving history and the strength and stiffness of the soil response from the pile toe. This approach has now been automated and subjected to tests on existing data. After a review of the existing methods of pile stress and damage calculations, the paper presents the new method, illustrating its effectiveness by examples from measurements on both concrete and steel piles.

INTRODUCTION

Driven pile foundations are very popular because they are both cost effective deep foundation solutions and provide by their installation history assurance that the pile has been installed to sufficient bearing with good structural integrity. Usually the maximum loads that can be applied to a driven pile foundation are limited by the resistance that can be applied to the supporting soil in shaft resistance and end bearing, although any weakness in the structural strength of the shaft may then be the limiting factor. Under very favorable conditions, such as when the pile toe is driven to a hard rock or the pile is long and attains significant resistance through soil setup, the maximum load can be limited by the structural strength of the pile. The pile shaft may be subjected to various loading conditions, including lateral and uplift which impose additional structural strength requirements.

Dynamic monitoring of driven piles on a routine basis has its origin as a research project at Case Western Reserve University (Goble et al., 1975) with the goal to replace static load testing by determining the pile capacity using the hammer impact. The basic method employed by the commonly used Pile Driving Analyzer® (PDA) system measures strain and acceleration with reusable sensors making the cost to instrument a pile very economical.

These measurements were analyzed in closed form solutions called the Case Method and by signal matching in rigorous numerical models called CAPWAP® (Rausche et al., 1972). Numerous correlations were made with static load tests over many decades of testing with generally good agreement when the times after installation for both tests were similar (Likins and Rausche, 2004).

Early tests by the researchers at Case Western Reserve University were generally done on closed-end steel pipe piles considered as friction piles and no pile damage was observed. A later research project was performed on H piles driven to end bearing on rock which did identify damage at the pile toe and noted "If the velocity increases sharply relative to the force at any point earlier than the 2L/c time it indicates damage has weakened the pile" (Goble et al. 1977). The test piles were extracted and visual inspection confirmed the pile damage. An example of the pile toe damage for these research project piles is shown in Figure 1. Although pile damage was detected, no specific attempt was made to estimate the magnitude of the damage, other than to note the capacity was greatly reduced in both dynamic and static load tests after damage occurred when compared with undamaged piles in the study. However, Teferra (1977) analyzed the force

Figure 1. Damaged H pile.

and velocity records of the test piles in closed form, yielding pile toe resistance versus pile toe displacement and, therefore, pile toe stiffness. Today this analysis method is incorporated in CAPWAP as the Pile End Bearing Wave Analysis Program (PEBWAP). For end bearing piles it shows very clearly the reduction of stiffness for piles with toe damage.

The ß-Method for evaluation of the *extent of damage* was developed and published (Rausche and Goble 1979) after dynamic measurements at a large commercial construction site clearly indicated pile distress on prestressed concrete piles of up to 240 ft (73 m) length. The piles were assembled using multiple mechanical splices, and detected damages were located at or near the splices. Rejecting piles with indications of only minor damage was considered unreasonable, particularly since the damage could have been merely a sheared mechanical connector, allowing the pile sections to slightly separate at the joint. But some rating scale was needed. An equation was derived from wave propagation theory which related the stresswave reflection magnitude from the potential damage to the magnitude of the impact force, taking into consideration the effects of soil resistance above the damage. This approach resulted in an integrity factor β indicating the theoretically remaining cross section (100% representing no damage and 0% a completely missing lower section). It was recommended that if, in theory, the thus estimated remaining cross section was greater than 80%, that then the piles were "slightly damaged" and would be accepted. If ß was between 80 and 60%, then the pile was rated as "damaged" and all information available would be reviewed before rejecting the pile. For a β less than 60%, reflections from the pile toe were usually not apparent from the dynamic records and the pile was considered "broken" and rejected. Rausche and Goble (1979) also gave an example of calculating the gap at the splice between sections. These methods were applied to many test piles during a restrike test program where as many as 44 piles were tested in one day, and a correlation was established between the ß-Method and the penetration per blow. Later installed production piles used this β-value versus pile penetration resistance relationship during pile restrikes for a quick decision on pile acceptance or rejection. All parties involved in the project accepted this approach and the project was completed successfully. While no piles were extracted for verification of the β-Method at this particular site, the subsequent 35 years have produced a wealth of independent assessments of detected damaged piles which were positively confirmed by extraction of those piles. For example, Rausche et al. (1988), Hussein and Morgano (1993) and Goble and Hussein (1994) described several case studies. Also, Verbeek and Middendorp (2011) reviewed the β-Method approach and generally agreed with it. They also proposed replacing the rating scale terminology of "damaged" or "broken" with "issues" of various significance.

Implementation of the Case Method

Conventionally, only selected test piles are monitored, and this information provides the basis for an economical and safe installation criterion for production piles.

While the basic methods of data interpretation are based on solid theory, the variety of hammer, pile and soil conditions require that the user has a thorough understanding of foundation construction methods, geotechnology, and the basic theory behind the various dynamic monitoring results. Therefore, training and continued education are important to correctly implement and use the dynamic monitoring results; this is certainly true when evaluating pile damage.

Allowable Driving Stresses

The pile installation process generally results in the highest stresses the pile will experience. Since pile stresses during installation are key to limiting damage to driven piles, determining allowable stresses and then confirming they are not exceeded is critical to preventing pile damage. Measuring the strain and acceleration during pile driving is now a routine practice, and by using these measurements the critical stresses in the pile are either directly obtained or can be calculated from the measurements and wave propagation theory. Recommended limits on tension and compression driving stresses have been established based on decades of experience and observations of the conditions which lead to structural failures of driven piles. These limits relate to the structural strength of the pile material, e.g., the yield strength of steel, and the prestress and strength of the concrete. Typical limits, such as proposed by AASHTO (2010) are as follows.

- 90% of the steel yield strength for steel piles in either tension or compression
- 85% of the concrete compressive strength minus the effective prestress for prestressed concrete piles in compression, and
- 100% of the effective prestress plus an allowance for the concrete tensile strength which is typically 0.2 to 0.3 ksi (1.4 to 2.1 MPa) for prestressed concrete piles in tension.

It should be noted however that once the cross section has suffered a crack the tension strength component of the concrete will be lost. . Limiting the tension stress to the prestress level would therefore be a more conservative approach.

Dynamic monitoring of the stresses in the piles, and developing a driving protocol which maintains production pile stresses below these recommended limits, are the most effective means of reducing the likelihood of pile damage. Such a protocol would specify maximum hammer fuel setting or energy output to blow count. For example, to limit tension stresses for concrete piles the hammer output is often reduced at low blow counts. Limiting toe compressive stresses would potentially require reduced energies at high blow counts.

Case Method Closed-Form Solutions

As mentioned, measuring the strain and acceleration during pile driving using reusable bolt-on sensors is now routine practice. Any pile can be thus tested if installation records raise doubts about pile integrity without the need for preplanning specific pile instrumentation. Strain measurements are converted to force, F(t), by multiplication with the material modulus, E, times the pile area, A. Velocity, v(t), is obtained by integrating the measured acceleration. Until reflections occur from soil resistance or a change in pile cross section, the force and velocity are proportional by the pile impedance, Z, which is the product of modulus E and cross sectional area A divided by the material wave speed, c. Stress waves due to hammer impact can be separated from reflections by separating the measurements into downward and upward travelling wave components. The force in the downward wave can be obtained from

$$Fd(t) = \tfrac{1}{2}\,[\,F(t) + Z\,v(t)\,] \tag{1}$$

And the force in the upward wave is

$$Fu(t) = \tfrac{1}{2}\,[\,F(t) - Z\,v(t)\,] \tag{2}$$

Considering that superposition of the forces in the stress waves yields the total force at any point, the above two expressions can be conveniently used to derive a variety of soil resistance, stress and other results of interest. The following provides a brief summary of what can be readily calculated for uniform piles by a PDA system during pile installation or restrike of any pile type using only the standard bolt-on sensors.

Bearing capacity. Based on the above basic equations, the closed-form capacity solution is computed from the downward wave at time t and the upward wave at time t + 2L/c, where L is the pile length below sensors and c the material wave speed. The basic Case Method equation for the maximum total resistance RTL which includes static and dynamic resistance components of shaft resistance and end bearing is

$$RTL = Fd(tx) + Fu(tx + 2L/c) \tag{3a}$$

Where tx is a time chosen to yield the maximum total resistance value. Static capacity RMX can then be calculated under the assumption of a damping factor J (Rausche et al. 1985).

$$RMX = (1 - J)\,Fd(tx) + (1 + J)\,Fu(tx + 2L/c) \tag{3b}$$

The component and distribution of resistance acting on shaft can be evaluated from the upward wave as a function of time during the initial 2L/c time period. The toe resistance can be deduced from subtracting the shaft resistance from RMX.

Stress extrema. Although dynamic testing was originally developed to evaluate pile capacity, it became apparent that other uses were also beneficial. Using only the standard force and velocity measurements, usually taken near the pile head, methods were developed from wave propagation theory to evaluate the driving stresses in compression at the generally critical pile head and toe locations, and maximum tension somewhere along the pile, which is generally of great importance for concrete pile installations where tension stresses can cause cracks that may develop into more serious pile damage. At the pile head, the maximum stress can be directly obtained from the measurements. The maximum pile toe force is

$$R_{t,toe} = RTL - R_{t,shaft} \quad (4)$$

The maximum tension force requires that the record is searched for the minimum of Fu(t) (which is the peak of the upward travelling tension wave) and superimposing it with an earlier occurring minimum value of Fd(t).

Hammer performance. The performance of the hammer can be assessed by measuring the energy transferred to the pile from

$$E(t) = \int F(t)\, v(t)\, dt \quad (5)$$

Considerable variability was found by comparing the maximum energy transferred to the manufacturer's energy rating (Rausche et al. 1986). Identifying poorly performing hammers is important as the energy transferred influences the blow count, which generally is used as a driving criterion.

The β-Method

The β-Method looks for an early tension reflection caused by a reduced cross section along the pile shaft. The maximum local reduction in the upward wave from this early reflection is related to the extent of the damage. The local reflection caused by damage along the pile shaft can be computed as

$$\Delta = Z\, v(t3) - F(t3) + R \quad (6)$$

where t3 is a time (between the initial impact and reflection from the toe) of the local minimum in upward wave that corresponds to the damage and R is the total shaft resistance, including damping, above the damage location. From wave theory, Rausche and Goble (1979) showed that the term

$$\alpha = 0.5\, \Delta \,/\, [\, F(t1) - R \,] \quad (7)$$

where t1 is the time of the initial force input peak and α can be used to quantify the section reduction by inclusion in the expression

$$\beta = (\, 1 - \alpha \,) \,/\, (\, 1 + \alpha \,) = Z_2 / Z_1 \quad (8)$$

where Z_1 is the original cross section and Z_2 is the reduced section at the damage. Thus, β expresses the percentage of remaining section compared to the original pile cross section. If F(t1) and Fd(t1) are nearly identical, as is usually the case, this can be put in simplest terms as

$$\beta = [\, Fd(t1) - 1.5\, R + Fu(t3) \,] \,/\, [\, Fd(t1) + 0.5\, R - Fu(t3) \,] \quad (9)$$

It is generally more accurate to look at the time between the beginning of the input wave and the beginning of the local minimum in the upward wave and calculate the damage location x from the wave speed c. The wave speed is well known for steel piles (16,800 ft/s or 5,123 m/s), but for concrete piles the wave speed depends on the concrete mix design and the age of the concrete. It typically ranges between 10,500 and 15,500 ft/s (3,250 and 4,800 m/s). While this is a large range of potential wave speeds, it should be understood that, within certain geographic areas, concrete wave speeds typically vary only 5 to 10% because of the predominate and consistent use of certain types of aggregates and mix designs by the local concrete pile suppliers. In any event, for concrete piles, the best estimate of wave speed comes from evaluating the time of the toe reflection from the earliest easy driving when the pile is known to be undamaged.

The closer the apparent impedance reduction is to the pile head, the earlier the reflection from the damage will arrive. Although the engineer should visually inspect the data for a premature tensile reflection, the PDA system automatically does such a search and calculates β and the distance below sensors, if the reflection occurs at a time more than one rise time (defined as the time between initial onset of the impact and the first input peak) before the pile toe reflection. For tensile reflections occurring later, but before the toe reflection, a toe damage is identified and a ß-evaluation is not performed. Further discussion on toe damage assessment is given below.

Example of a Damage Evaluation Along the Shaft for a Spliced Pile

Figure 2 shows measurements of force (F) and velocity times impedance (Z v), and the calculated upward wave force (Fu) and the top displacement (D) as a function of time for a 16.5 inch (420 mm) octagonal prestressed concrete pile. The pile has total length below the sensors of 134 ft (40.9 m) with a splice located 64 ft (19.5 m) below the sensors. The two arrows indicate the beginning of the rise time at impact and the beginning of the expected toe reflection. The time period between the arrows is equal to the wave travel time, 2L/c. After 202 blows the PDA system first automatically detected a very small tensile reflection (β of 96%) occurring just above the splice.

It should be noted that, because of the needed development length, the prestress is not developed at the pile segment ends, so the tension at the splice has to be carried by the dowel bars that connect the sections. In such case a tensile failure therefore happens most likely at the end of either the upper or lower dowel bars, or due to a failure of the splice connection if it is a mechanical connection. Figure 2 shows that the damage had gradually progressed to a β of 81% by blow number 692; the damage is marked by the vertical dashed line about midway between the two solid vertical lines marking the beginning of initial impact and 2L/c later. Even though the prestress was designed to be 1.0 ksi (6.9 MPa), the computed tension from the measurements of only a maximum of 0.65 ksi (4.5 MPa) was sufficient to start the damage since, most likely, the full prestress was not available at the end of the dowel bars. After blow number 692, the ß-value then rapidly regressed to 64% by blow number 700 (Figure 3) and to 50% by blow number 703 (Figure 4). Driving was halted at blow number 713. The damage location was indicated 2 ft (0.6 m) above the splice location. It can be observed in Figures 2 and 3 that the reflection from the pile toe is still apparent (strong increase in velocity relative to force, and a reduction in Fu, after the second vertical line). In Figure 4 the toe reflection (after the third vertical line) is vastly different from that in Figure 3 and it is not certain that the toe reflection can be reliably ascertained.

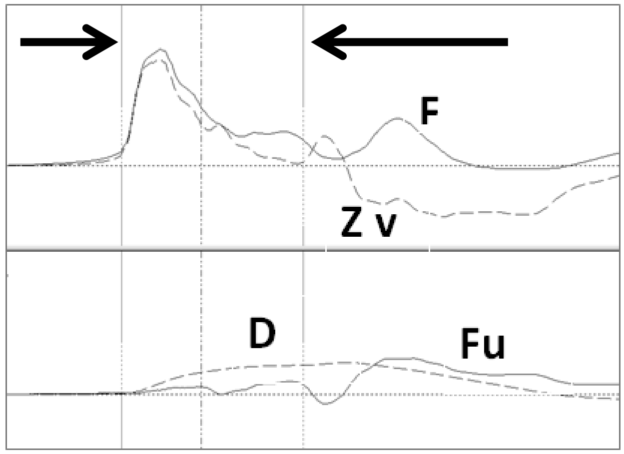

Figure 2. Spliced concrete pile, blow number 692.

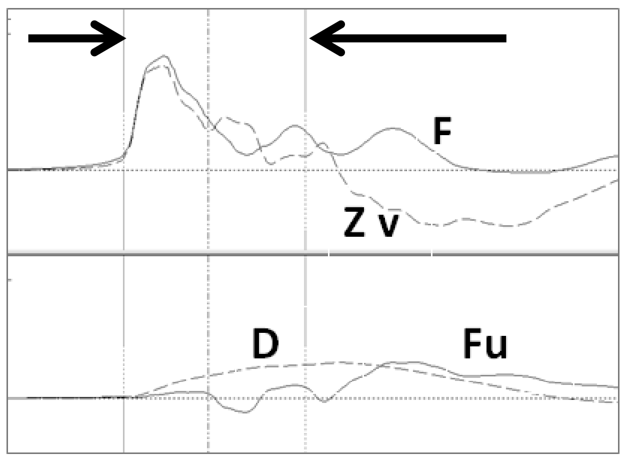

Figure 3. Spliced concrete pile at blow number 700.

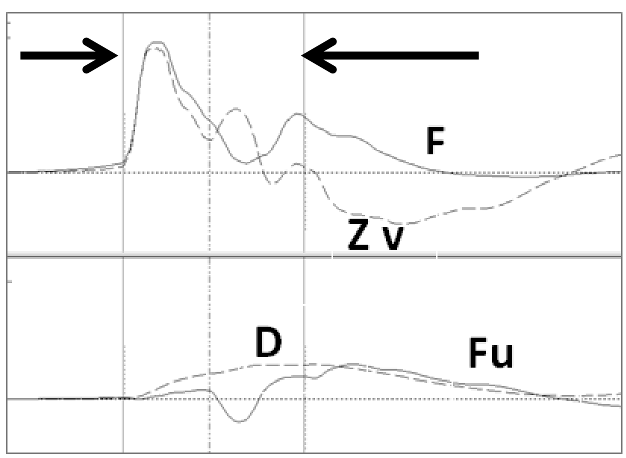

Figure 4. Spliced concrete pile at blow number 703.

It should be noted that there had been ample opportunity to cease pile driving when the damage was still relatively moderate (β above 81%), and even during the last few blows before β dropped below 60% when the pile broke completely. Such a broken pile is unreliable in the service condition. Many extracted piles show a shear failure with the upper section likely sliding off the lower section. The resistance against the broken end might include forces associated with the prestressing strands which are unreliable in the long-term service condition because of probably high stress concentrations and corrosion potential.

Damage Near the Pile Toe

A major defect near the pile toe is classified as toe damage, although there is no strict definition as to the distance of the damage from the toe. However, in such a case, the damage reflection will occur only slightly earlier than the toe reflection and careful observation is required to differentiate between them. Monitoring the pile installation continuously helps determine the time of pile toe reflection of the undamaged pile from early driving and, therefore, allows for toe damage detection should it actually happen. However, for restrike testing, if the concrete wave speed is not accurately known, toe damage assessment may not be possible.

If the damage is located within a distance above the toe that corresponds to one rise time, the reflection will include tension waves from both damage and pile toe and compression waves from the end bearing plus shaft resistance near the toe. Unfortunately these effects all superimpose, and it is not possible to sort out the three individual components. So while it is possible, given an accurate wave speed, to detect the early wave arrival, and thus diagnose existence of pile damage near the toe, it is not possible to clearly assess the magnitude of such a defect. Instead, the pile tester has to evaluate the toe condition from the pile driving behavior and the strength and stiffness of the toe resistance, as may be apparent from soil information and static and dynamic analysis. In addition, a signal matching analysis, such as CAPWAP, can help shed further light on the extent of the distress.

Example of Near-Toe Damage for a Concrete Pile

A 24 inch square solid section pile with length 91 ft (27.7 m) was driven by an APE D50-42 (5 metric ton ram weight) single acting diesel hammer. For hammer blow No. 1160 at a penetration depth of 68.5 ft (20.9 m), Figure 5 shows measured time histories of force and velocity times impedance, Z, plus the force in the upward traveling wave, Fu, together with the pile head displacement D. The toe reflection is obvious from either the relative increase of

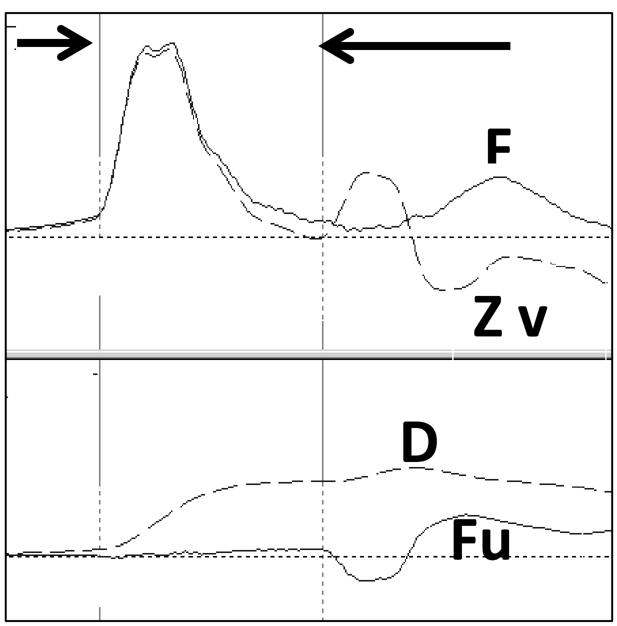

Figure 5. Early record for concrete pile, blow number 1160.

velocity compared to force, or the local reduction in the upward wave, Fu. The material wave speed can be determined from the measured time and known pile length as 13,500 ft/s (4,110 m/s). Figure 6 shows a record taken on the same pile a few blows later. The extra continuous dashed line, pointed to by the dashed arrow, just prior to the 2L/c reflection shows the automatically detected near-toe defect and its location is 2 ft (0.6 m) above the undamaged toe position. Significantly, the critical compressive stress condition in prestressed concrete piles is not at the toe but rather where the full prestress is first achieved, which is some small distance above the toe since right at the pile toe the prestress is zero. Due to the development length for the prestress, this critical distance is typically at two to three feet (0.6 to 1.0 m) above the toe, making it possible to detect near-toe damage for concrete piles using the standard pile top measurements. That distance will be longer for piles where the "stress transfer length" has become longer due to improper strand release in the casting yard.

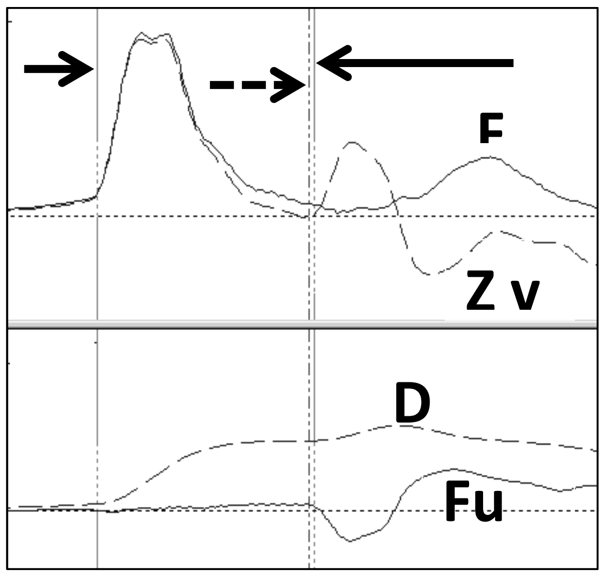

Figure 6. Prestressed concrete pile, a few blows after first damage detection.

The damage is detectable, yet minimal and, if it met other driving criteria, the pile would almost assuredly perform satisfactorily in the service condition if the driving had been terminated at this first detection.

Figure 7 shows a record for the same pile taken about 300 blows later. The damage is now more progressed and clearly indicated by the early reflection, most easily observed in the reduction in upward wave (Fu) just prior to the time of wave return (2L/c). The damage location is now 3 ft (0.9 m) above the original toe. This prestressed concrete pile was extracted and Figure 8 shows the toe condition of this extracted pile. The bottom of the pile is basically still intact, and connected to the main pile shaft by the prestressing strands. It is therefore obvious that the damage did not start at the bottom and progress up the pile shaft, but rather the damage began at the critical section about two feet above the bottom, as shown by the standard measurements at the pile head. It is also likely that the extraction process has caused additional harm to the pile.

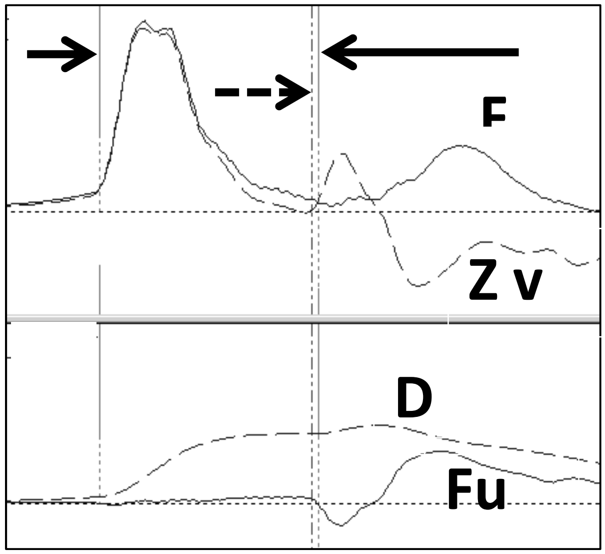

Figure 7. Concrete pile at blow number 1420,
300 blows after first damage detection.

Figure 8. Extracted concrete pile
showing the pile damage.

Example of Steel Pile Damage

The original ß-Method is useful for any pile type which can be dynamically tested, including concrete piles, prestressed or not, timber piles, and all steel pile types. Steel piles may suffer defective splices, bends, tears or twists when encountering obstructions during driving. Similarly near-toe damage can be detected from pile head measurements for any pile type. Steel piles, for example, can suffer collapsed pile toes when improperly trying to seat them with too many hammer blows into a hard material or on bedrock. Exceptions are near-toe deformations that leave the pile impedance virtually unchanged. An example for this type of damage is an ovalization of an open-end pipe.

Large 14x117 (360x174) H-piles were driven by a Delmag D46-32 (4.6 metric ton ram) to a karst limestone. Pile lengths for two test piles were 90 ft (27.4 m) and were equipped with high strength cast-iron shoes. Although the piles were driven in relatively close proximity to each other, the final elevations and the installation records were quite different. Because the shaft of an H pile cannot be visually inspected after installation, dynamic testing was requested to explain the differences. Figure 9 shows measured pile head force, velocity and the associated forces in the downward and upward traveling waves for a blow when one of the piles reached refusal at 59 ft (18 m) penetration. The toe reflection comes at exactly the time predicted based on pile length and steel wave speed. The force F after 2L/c is quite large, and the upward wave Fu after 2L/c is also in compression and a significant percentage of the input downward wave. The dynamic measurements clearly show that this pile has high end bearing and is not damaged.

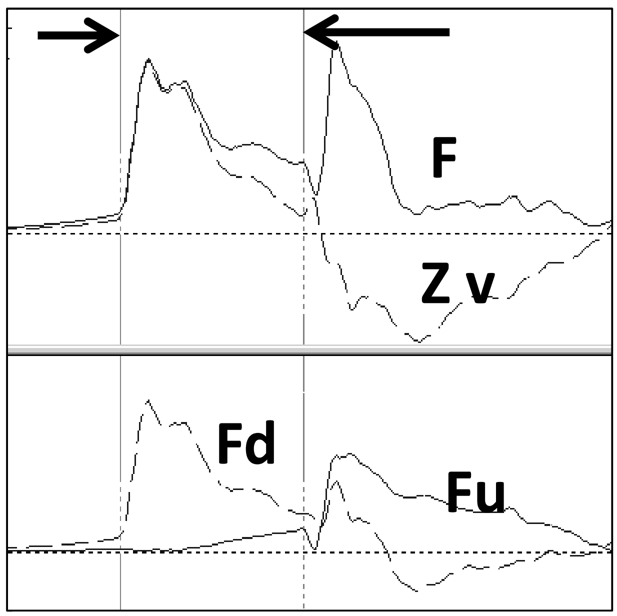

Figure 9. H pile driven to refusal at penetration of 59 ft (18 m).

Figure 10 shows the record of a second pile which was driven on the same project site to 78 ft (23.8 m) penetration. The early portion of the upward wave Fu which is a reflection from the shaft resistance is similar to the upward wave from the previous pile, a similarity that might be expected for neighboring piles in the same soil profile. By contrast, prior to the time of wave return from the toe, both the force F and the upward wave force Fu show an obvious and early reduction. The beginning of the reflection, defined by the relative increase of velocity to force or the beginning of the reduction in upward wave Fu, and depicted by the early full dash-dot line prior to 2L/c, starts too early when compared with the expected time of wave return. This is a clear indication of damage; a photograph of the extracted pile is shown in Figure 11. The pile also originally had a cast-iron shoe that is now missing. The sharp bend about 8 ft (2.4 m) above the pile toe causes a reduced axial stiffness from this near-toe damage. The compressive force in the upward wave after 2L/c is significantly smaller indicating considerably reduced capacity compared to the undamaged pile.

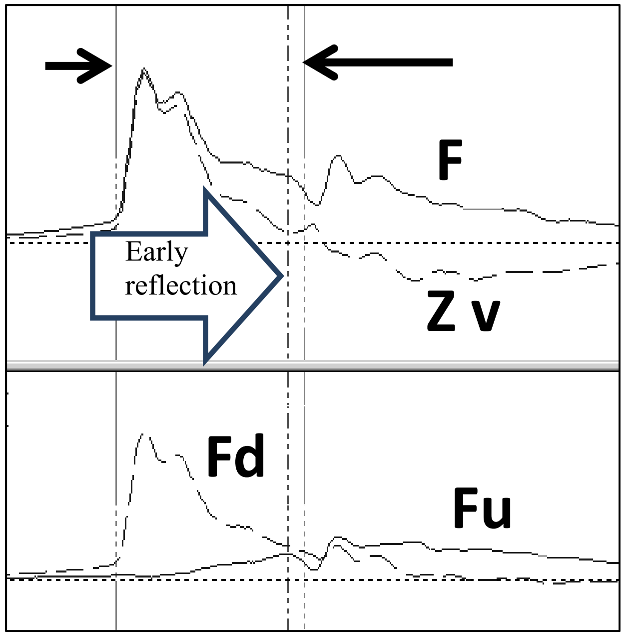

Figure 10. Pile driven to 78 ft (23.8 m) demonstrates near-toe damage.

Figure 11. Extracted pile with near-toe damage.

Discussion of Pile Toe Damage Assessment

Considering the many different pile types and failure modes, an automatic assessment of the severity of damage is always problematic. For this reason, advice has always been given to pile testers to visually inspect the data for an early stress wave arrival which could indicate near-toe pile damage. As mentioned earlier, this has been the advice for decades as noted in the early research project report that included H-pile damage: "If the velocity increases sharply relative to the force at any point earlier than the 2L/c time it indicates damage has weakened the pile" (Goble, et al. 1977).

Over more than three decades, numerous independent engineers all over the world have used the ß-method to detect damaged piles and the major damage has been independently verified by extraction, confirming the validity of the original recommendations derived from theory. In general, the reflection from the pile toe and thus a positive indication of a continuous pile over its full length cannot be observed when the β-value is less than 60%. This is a further justification of the aforementioned damage scale. Since frequently the bearing capacity and pile integrity are closely related, the Case Method bearing capacity determination combined with the β-Method has proven to be a very powerful tool.

For dynamic measurements, the typical data sampling frequency of 10,000 Hz gives a resolution in distance to damage location evaluation of 0.2 m. Given the wave speed from pre-damage pile monitoring, near-toe damage can be investigated automatically by a PDA system by checking for relatively small local reductions in the upward traveling compression wave just prior to the expected time of the wave return from the toe. While this method can detect toe damage and its distance above the toe, it cannot assess the extent of the damage. However, both the traditional β-Method and the near-toe damage detection are applicable not only to prestressed concrete piles but also to all other pile types including steel H-piles and pipes. In general, the only piles that need to be instrumented and tested are test piles. Production piles that do not meet the installation criteria should also be instrumented at their pile head and tested during a restrike. Conveniently, this can be done remotely, i.e., by having field personnel attaching the sensors to the pile and sending the data from the PDA via internet to the experienced test engineer.

To evaluate toe damage in concrete piles, Verbeek and Middendorp (2011) propose to measure changes in strains in all piles near their toe with expendable sensors. Although this promises a direct measurement of pile integrity, diagnosing significant pile damage from an apparent strain variation requires establishing an arbitrary criterion which could lead to rejecting piles that meet the driving criteria and would perform acceptably in long-term application. Experience shows that slipping prestressing strands frequently produce some prestress loss near the pile ends, even though the concrete of the pile has not been damaged.

It is important to consider the day-to-day operations on a pile driving site and the consequences of pile acceptance and rejection. A pile which has suffered significant damage will not likely achieve the same blow count as a neighboring undamaged pile and therefore its installation criterion will not be met. A damaged pile which does not meet the driving criteria, and has been dynamically tested and confirmed damaged, is most likely rejected and so are untested production piles which do not achieve the driving criterion. Test piles with damage near the toe, as determined by dynamic testing, exhibiting high resistance from the bearing layer may be analyzed with a slightly shorter length and may still be deemed acceptable as are all untested piles which meet the driving criterion. Similarly, a closed-end pipe pile with a collapse in the vicinity of the toe may still be filled with concrete and become serviceable.

Best practice is to perform initial dynamic pile tests with the actual driving equipment intended for the project and for all contemplated pile types to establish a driving sequence and installation criteria with acceptable stresses that avoid damage. This is a time honored procedure which provides a known quality foundation while being both economical and construction time sensitive. It should be noted that the initial test piles on a site are often driven harder, deeper, and to higher loads than required, resulting in a higher chance for pile damage. This is particularly true for concrete piles when the reason for the test program is to establish casting lengths for the piles. Further, the driving system and in particular the plywood cushion for the pile head are usually selected by a wave equation analysis. However, after measuring the stresses by dynamic monitoring, the cushion thickness might be increased or the energy output of the hammer reduced to decrease the stresses for subsequent production piles.

The greatest problems on a site occur when excessive time is spent on meetings trying to decide if an apparent pile damage is serious enough to require pile extraction or replacement. If the damage cannot be detected from the dynamic pile head measurements alone and the pile meets the installation criteria, then it is unlikely that the pile's ability to sustain the required load is compromised. With the availability of automatic methods of damage detection the pile testing engineer is well equipped to make the proper judgment within a short time after installation.

CONCLUSIONS

Dynamic pile monitoring according to the Case Method, as a quality control and assurance method, has gained worldwide acceptance because it provides construction professionals with a complete evaluation of a foundation's geotechnical and structural adequacy. Together, calculated bearing capacity, pile stresses, hammer performance, and the ß-integrity factor provide the basis for rational decisions for pile acceptance or rejection. The ß-Method was developed for detection and estimation of damage along the pile shaft and, over the subsequent more than three decades, numerous independent engineers have used the ß-method to detect damaged piles. Major damage has then been independently verified by extraction, confirming the validity of the original recommendations derived from theory.

The ß-Method only requires instrumentation near the pile head to detect potential damage at any location below this point of measurement. This single point of measurement makes instrumentation reusable and therefore economical since it can be employed for any driven pile, even after-the-fact, and avoids extra instrumentation permanently embedded in the pile. Additional economy can be achieved with remote testing technology. A further advantage of the ß-Method is its applicability to any uniform cross section pile, including steel and concrete piles, covering the vast majority of all driven piles. For non-uniform piles, unless continuous monitoring is performed, which would show from record characteristics developing damage from blow to blow, a more rigorous analysis such as CAPWAP is needed to evaluate pile integrity.

Where near-toe pile damage is indicated, the ß-Method cannot evaluate the extent of the damage due to superposition of the closely following reflection from the pile toe. However, damage near the pile toe can be detected automatically by the PDA system by looking for a reflection occurring too early for the correct 2L/c. In any event, near-toe pile damage should always be evaluated along with an assessment of pile bearing and its stiffness. Dynamic testing, including a rigorous signal matching analysis, provides information on the magnitude and stiffness of the pile toe response which are invaluable when deciding on the acceptance of a pile with near-toe damage.

Pile damage is best prevented by establishing an installation protocol based on a wave equation analysis and then confirmed by dynamic measurements. Such a driving prescription may, for example, require a minimum cushion thickness and replacement schedule and hammer energy setting as a function of driving resistance.

Based on measurements near the pile head alone, piles that lack any sign of damage either along the shaft or near the pile toe will perform satisfactorily in service conditions provided they also meet the established driving criteria.

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