Mastering the art of pile testing

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ABSTRACT: The State of the Art of the Application of Wave Speed Measurements to pile design and construction control is impressive. The number of tests performed; the quality of equipment, measurements and analyses; the general acceptance of the test as evidenced by various codes and specifications; and the personalities involved in this specialty segment of the deep foundation industry all testify to a healthy and vibrant profession. Unfortunately, those involved in this activity also know that serious problems can exist. Reasons for these problems range from the simplistic expectation that tests only require attaching sensors and the pushing some buttons to the complex expectation that each field test becomes a research project.

This paper examines some of the causes for misjudgments and what can be done to avoid them. In order to provide an understanding for the analysis difficulties, examples are shown for the variability of the dynamic soil resistance parameters.

While basic and continued instruction in proper testing and data interpretation procedures is definitely a must, it is also important to create the proper working environment for the pile tester. Although some of these conditions are already specified in a variety of different documents, this paper additionally proposes more detailed analysis related guide lines. This addresses the problem of several pile and soil model options which are relatively unknown because they have only recently been introduced but may have a decisive effect on the results. It is hoped that these recommendations will create a more uniform basis for pile tests and results.

1 INTRODUCTION

Looking back, the first applications of high strain dynamic testing were done in a research environment, allowing the testers great flexibility as to testing and analysis methods. The early clients and sponsors saw the potential of this new technology for more and better information and, most importantly, great savings and generally agreed to invest time and money in hope of future benefits. As the testing and analysis systems matured, construction specifications were developed which included dynamic testing. Today ASTM D4945 is often referenced as a minimum test requirement. Another helpful document which is frequently referenced is the Sample Specification for High Strain Dynamic Pile Testing which can be found at www.pile.com. Many other specifications list the requirements which the testing firm has to meet but to a lesser degree the provisions on the constructions site.

Testing requirements as documented in codes and standards have developed in many countries (Beim et al, 2008). For example, taking advantage of Remote Testing Technologies (Likins et al., 2008a) on a modern Swedish test site, only a single hammer blow is required to establish a dynamic load test. On the other hand, the Florida Department of Transportation (DOT) in most instances requires Dynamic Monitoring throughout the installation of several prestressed concrete test piles and, if questions as to bearing capacity remain, a so-called “Set-Check”, which is another term for a Restrike Test. This Set Check typically involves driving the pile for 10 hammer blows or several 25 mm penetrations and typically happens within 3 days after pile installation. The Ohio DOT, on the other hand, prefers End-of-Drive (EOD) Tests of predominately pipe piles and authorizes restrike tests only when the end of drive capacity is unexpectedly low. The Oklahoma DOT requires the testing of all of their H-piles on standard bridges at EOD and sometimes also upon restrike. The Oklahoma contractors have perfected the art of testing, performing 40 pile tests in less than a day. Of course, the specifications of the various owners result from different geologic conditions, pile types and test objectives. While for concrete piles driven to a hard bearing layer damage check is as important as capacity testing, H-piles driven into weathered rock or silt shales often have to be checked for relaxation.

Problems with wrong or inconclusive results sometimes cause disputes. Normally they can be resolved given good quality dynamic test records. However, poor test preparations often render the pile test records inconclusive. Proper test preparation can avoid problems and in the past major emphasis has been placed on assuring proper training and knowledge for the testing and analysis engineer (tester). Unfortunately, civil engineering education at best allows for a few hours of instruction in deep foundation testing and text books describing the dynamic tests and analyses sometimes confuse the dynamic methods. Fortunately, good texts such as Salgado, 2008 also exist. Thus, someone involved in the testing has to undergo extensive initial training and continued education should occur at regular intervals. Certification of testers through an independent professional service is now possible (www.FoundationQA.com), however, to date this has been successful only for those geographic areas where both testing firms and clients have been positive about its value.

While the testing itself can be demanding, the analysis is challenging too. Both a wave equation analysis which simulates the whole process and a signal matching analysis, based on collected records, (e.g. CAPWAP) are not simple processes. Dynamic soil parameters which are important for a good match and a good correlation defy clear relationships with standard soil properties. Furthermore, it is important that the analyst clearly presents the various results in the test report so that the intended reader of the report has a clear understanding of the tested or expected pile and soil performance.

2 PROBLEM STATEMENT

Likins et al., 2008 have described what constitutes a good PDA test and that paper is a basis for the following discussion as to what can be done to avoid poor test results and what can be reasonably expected from a good test. Let us look at potential problems in the deep foundation analysis and testing arena.

2.1 Static analysis and static test problems

Civil engineers like to analyze information, create a design and then construct the project. Why is it so different with deep foundations? We have loads and static formulas to estimate pile length from soil investigation information. In certain geographical areas, geotechnical engineers get good correlations with static tests as long as their experience continue to match the soil testing methods, pile types, and pile installation procedures. However, when analyzing piles in an unknown or unfamiliar geology, the

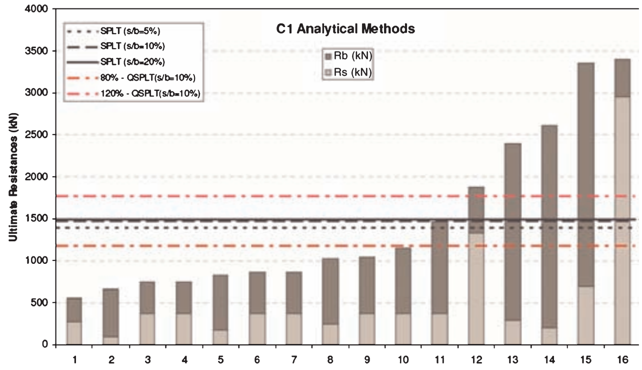

Figure 1. Comparison of capacities from analysis of driven pile and static load test on companion pile – after Viana da Fonseca and Santos (2003).

same geotechnical engineers struggle to achieve good correlations. Unfortunately, even in a familiar environment, the soils often vary enough to cause surprises. Fig. 1 shows the analytically predicted capacity by 16 participants for a driven pile installed and tested for a prediction event conducted at the University of Porto in 2004 (Viana da Fonseca et al, 2005). The predictions varied by a factor 5 even though the soil parameters were carefully evaluated and documented. Only 1 out of 16 predictions was within 20% of the static load test capacity (SPLT). Using recommended static analysis programs, e.g. free software such as DRIVEN (www.fhwa.dot.gov) or proprietary codes and methods such as UNIPILE (www.unisoftltd.com) does not necessarily yield better predictions if there is a lack of local experience factors. Major reasons why experience in a particular geology with a particular pile type and installation procedure is necessary include:

(a) The in-situ or laboratory test results usually have a great scatter and can be interpreted differently by different analysts.
(b) The pile installation process greatly changes the soil leading to loosening, densification, liquefaction, remolding and setup, scaling effects of pile vs. in situ tests, etc.
(c) Due to site variability, the borings may not reflect the soil properties near the tested piles.
(d) The analysis methods are based on experiences in certain pile and soil types and may not be applicable to differing conditions.
(e) Differing installation methods (e.g. jetting or pre-boring) or sequences for multiple pile installations may affect the result.

Because of these uncertainties with static analysis methods, dynamic testing and analysis and static testing are frequently recommended. In the past, for the driven pile, the simplest test was always the blow counting either combined with dynamic formula or wave equation analysis while for the bored piles static tests had to be done.

Static tests have their own problems, primarily because of money and time constraints and, therefore, can only be reasonably done on one or at most a few piles. In addition, like any other pile test, static testing has to be done after a sufficient waiting time and with sufficient reaction load capacity. Also, there should be no disturbance of the piles after they are installed. For example, installing reaction piles after the test pile installation may lead to severe inaccuracies. Interpretation of the capacity result may result in widely divergent answers from the same load-movement curve (Duzceer et al, 2000). Fellenius, 1980 also has addressed problems with both interpretation methods and measurement for static load tests.

2.2 *Dynamic analysis problems*

Enough has been said about the unreliability of dynamic formulas and this subject is not further discussed in this paper. The wave equation analysis (PDI, 2006) has (or should have) replaced the dynamic formula. It does not require dynamic testing and helps selecting an adequate pile driving system. For reliable results, the analysis requires reliable hammer, pile and soil information as an input. We have discussed the problem with reliable and adequate information from soil borings. The advantage of the dynamic analysis over the static analysis is that feedback from the installation of the driven pile (e.g. hammer energy provided and set per blow achieved) is considered in the capacity prediction process.

Randolph, 1992 suggested that the Smith parameters of quakes and damping factors are not easily derived from standard soil mechanical parameters. The point is well taken and particularly when we do drivability analyses it may be better to derive the quake and damping factors from well performed laboratory tests or from accurate in-situ tests rather than using standard recommendations which are at best a good average over the whole world's soil conditions. For example, Randolph recommends calculating the quake based on shear modulus and pile diameter with the caveat that dynamic and residual stress effects would require additional adjustments. Based on typical shear strength and shear modulus values, Randolph estimates shaft quakes to vary between 0.2 and 0.5% of diameter (D/500 to D/200) and toe quakes between 1 and 2% of diameter (or D/100 to D/50).

Short of more accurate laboratory or in-situ test results, simulation analyses can be improved if energy, stress and soil resistance parameters are determined by measurement and signal matching. This leads to the so-called "Refined Wave Equation" analysis (Hannigan et al., 2006). Discussion of this subject is beyond the scope of this paper, but it should be mentioned that a tester familiar with simulation analyses both for test preparation and post analysis has a distinct advantage through expanded knowledge.

To illustrate the problems with dynamic soil resistance parameters, a study was made where the calculated shaft and toe damping factors, J_s and J_t, have been summarized for 55 load test cases. The data investigated was taken from a data base maintained by GRL. It includes results from static and dynamic tests, performed on a variety of pile types, pile sizes and soil types. Much of this data was used in a correlation study performed by Likins et al. (2004). A few cases were omitted for which, for example, the static and restrike test times were too different or where the restrike energy was insufficient for full resistance activation. The correlation of the remaining original dynamically calculated capacities with the static load test capacities, evaluated by the Davisson Offset Criterion (Fig. 2), was good for the restrike results, with the ratio of dynamic to static capacity having a Coefficient of Variation (COV) of 0.16. Given such a good correlation, it can be expected that the associated quakes and damping factors also are reliable.

Fig. 3 shows for shaft and toe, respectively, damping factors calculated from beginning of

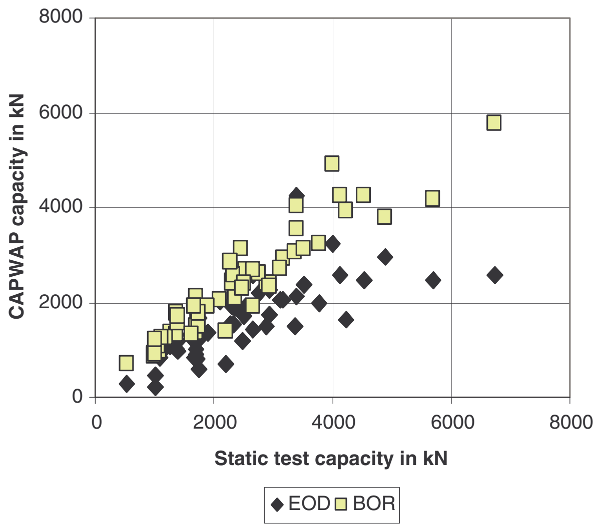

Figure 2. Correlation of CAPWAP capacity with static load test capacity for 55 cases selected for the damping and quake study.

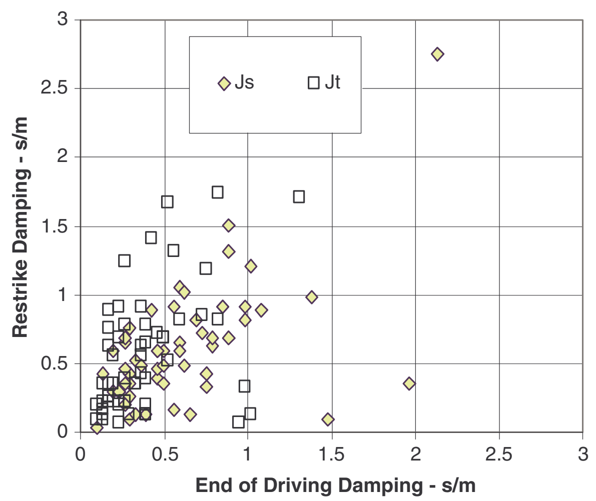

Figure 3. Shaft and toe damping factors: EOD vs BOR from 55 tests.

restrike (BOR) vs. those calculated from end of drive (EOD) records. These damping factors are of the Smith-viscous type, i.e., with these factors, damping is calculated as:

$$R_{d,\,si} = R_{si} J_{si} v_{si} \tag{1a}$$

$$R_{d,\,t} = R_t J_t v_t \tag{1b}$$

where $R_{d,si}$ and $R_{d,t}$ are, respectively, the shaft (at segment i) and toe damping forces, R_{si} and R_t are the shaft (at segment i) and toe ultimate static resistance forces (for the toe also the variable static resistance with time rather than the ultimate value is sometimes used for R_t) and v_{si} and v_t are the velocity at shaft and toe. J_{si} and J_t are the damping factors for shaft segment i and toe, respectively.

Ideally, if the damping factors were the same at all times for a certain soil, since each point of Fig. 3 pertains to the same soil material (though tested at different times), all points would fall on the 45 degree lines. Clearly, there is quite a scatter and that is expressed by the relatively high coefficients of variation of the ratios of the BOR to EOD damping factors shown in Table 1. The scatter is somewhat reduced if, as shown at the bottom of Table 1, an overall or weighted average damping factor is computed according to the following formula.

$$J_{comb} = (J_s R_s + J_t R_t)/(R_s + R_t) \tag{2}$$

where R_s, is the sum of all R_{si} values. The resulting BOR versus EOD combined damping factors are shown in Fig. 4.

Table 1. Damping factors and quake values from 55 End of Drive and Beginning of Redrive tests analyzed by CAPWAP

Damping Results	Average s/m	COV	Average Ratio J_{BOR}/J_{EOD}	COV Ratio
EOD Shaft damping	0.61	0.68		
BOR Shaft damping	0.62	0.71	1.18	0.58
EOD Toe damping	0.38	0.69		
BOR Toe damping	0.56	0.78	1.72	0.69
EOD combined damping	0.49	0.66		
BOR combined damping	0.57	0.74	1.26	0.48
Quake Results	Mm		D /quake	
EOD Shaft quake	2.8	0.44	181	0.53
BOR Shaft quake	2.6	0.41	201	0.61
EOD Toe quake	8.3	0.58	71	0.67
BOR Toe quake	6.6	0.66	89	0.61
Stiffness Results	kN/mm		$k_{t\text{-}BoR}/k_{t\text{-}EoD}$	
EOD Toe Stiffness	125	0.85		
BOR Toe Stiffness	211	1.22	2.27	1.15

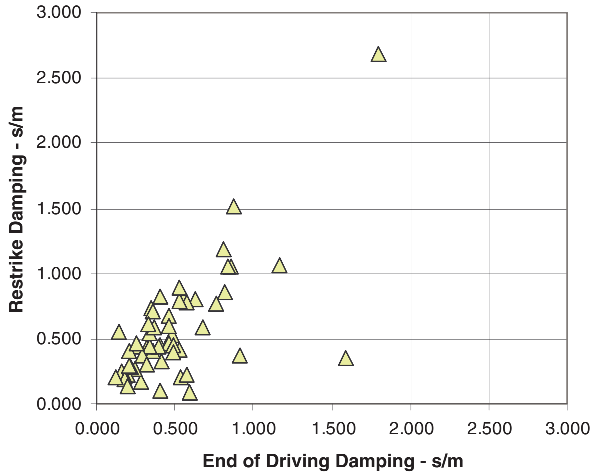

Figure 4. Combined damping factor J-comb (BOR vs EOD) for the 55 selected cases.

Averaging the damping factors we see in Table 1 shaft values near the usually recommended value for clay of 0.65 s/m. This suggests that, on the average, the GRLWEAP, 2005 recommended shaft damping values of 0.16 s/m for sand and 0.65 /m for clay are on the low side since the 55 cases shown were driven in both granular and cohesive soils. For the toe, the average damping factor increases from 0.38 s/m at EOD to 0.56 s/m at BOR (recommendation is 0.50 s/m) or an increase of nearly 50%. The average of the ratios of BOR to EOD damping factors show an even greater increase of up to 72% for the toe (Table 1). The higher BOR damping factors can be in part explained by lower restrike velocities. Gibson et al., 1968 had shown that the effective Smith damping factor would be expected to be greater for smaller pile velocities (that is why they suggested the use of an exponential damping law). However, the change of soil properties certainly has an even greater influence and it is obviously very difficult to choose the proper damping factors for a pre-construction analysis. Too many different phenomena, including energy losses, inertia effects, viscous effects, incomplete resistance activation, radiation damping etc. have to be covered by the damping forces in a dynamic analysis.

The shaft quakes, q_{si}, at a segment i along the shaft and the toe quake, q_t, express the inverse of the soil stiffness, k_{si} and k_t of the material in the pile-soil interface at shaft and toe, respectively. For pile displacements less than the quake values the relationship between stiffness and quake is given by

$$q_{si} = R_{si}/k_{si} \tag{3a}$$

$$q_t = R_t/k_t \tag{3a}$$

Like the damping factors, the quakes affect the calculated capacity. For wave equation analyses, the GRLWEAP, 2005 recommendations are a shaft quake of 2.5 mm and a toe quake between D/60 and D/120 where D is either the pile width or diameter. Table 1 shows that, on the average for the 55 CAPWAP results investigated, the calculated shaft quake is indeed close to 2.5 mm (or on the average 1/200 to 1/180 of the diameter) both for EOD and BOR. The toe quakes, on the other hand, display higher averages of 7 to 8 mm, corresponding to 1/70 to 1/90 of the average diameter or width, D, for EOD and BOR, respectively. This indicates that the soil at the pile toe behaves somewhat stiffer during the restrike than during installation. Fig. 5 also shows that toe quakes can reach 25 mm. The scatter of the toe quakes is substantial and the correlation with the pile size is weak. This can be attributed to the quake values being not only a function of pile size, but also of soil type, densification or loosening due to driving, soil stiffness and soil strength. Calculating the toe resistance stiffness according to Eq. 3 we see in Table 1 that the average of the toe stiffness ratios of BOR to EOD increases by a factor greater than 2. Again the scatter is significant owing to the many pile sizes and soil types in this study. For stress predictions the toe quake value is important and the scatter shows why simulation analyses are potentially inaccurate.

This study of different pile types and soil types suggests that the dynamic shaft resistance parameters are reasonably close to those normally expected and are not much affected by the time of testing. On the other hand, toe resistance factors differ more between EOD and BOR. While the toe damping factor may be chosen lower than recommended for EOD, it increases significantly during setup and is then close to the recommended value. While toe quakes are on the average D/71 and therefore slightly lower than the recommended D/60 for looser than very dense soils or softer than hard soils, they tend to the very dense/hard recommendation (D/120) when testing is done during restrike (D/89). The scatter is great and much can still, and should, be done to improve the recommendations, particularly for the pile toe; on the other hand the shaft values need not as much research attention.

While it is instructive to consider a variety of pile and soil types, it is also of interest to investigate the variability of dynamic soil resistance parameters at the

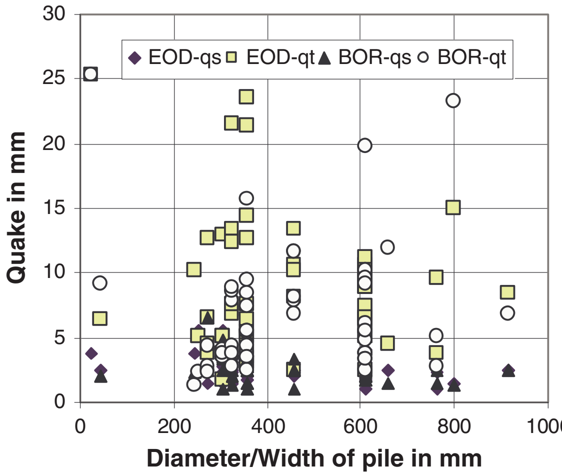

Figure 5. CAPWAP calculated toe quakes vs pile diameter or width for the 55 selected cases.

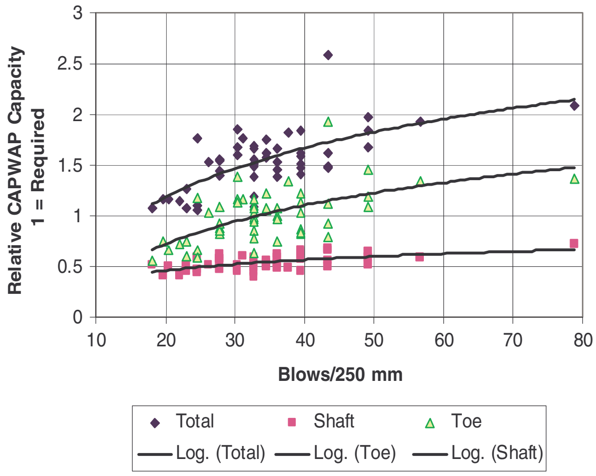

Figure 6. CAPWAP capacity results for 42 piles at the same site.

same site. As an example, consider Fig. 6 which shows CAPWAP calculated capacities relative to the required ultimate capacity as a function of blow count. The results were taken from 42 H-piles (10 to 12 m long) driven in two abutments of two adjacent single span bridges. The piles were driven by a 1.6 tonnes diesel hammer to generate a minimum capacity of 730 kN in shale. After reaching the desired capacity by the Case Method, the piles were driven another one to two feet, i.e., not much effort was made in stopping the piles at exactly the same capacity. Correspondingly, the figure shows a clear trend of capacity and end bearing increase with blow count while the shaft resistance, as would be expected for not much variation in pile penetration, was rather constant. Table 2 summarizes various dynamic soil resistance parameters obtained by CAPWAP for the piles at this site.

There is, once again, quite a bit of scatter in the capacity-blow count relationship. One of the reasons is the variability of the hammer energy transferred to the pile tops which, apparently, was independent of driving resistance (Fig. 7). An even greater scatter exists in the calculated shaft and toe damping factors (Fig. 8) which have been normalized by standard recommendations. The combined damping factor, again calculated as per Eq. 2, is more stable. For the shaft quake the trend may be coincidental. In contrast, the toe quake (Fig. 9) shows a definite, decreasing trend relative to blow count. Apparently, as the toe bearing increased so did its stiffness, but not

Table 2. Statistics of results from 42 CAPWAP analyses for the same site

Quantity	Quantity Reference Value	Average of Ratio	COV of Ratio
Total capacity	730 kN	1.54	0.19
Shaft capacity	730 kN	0.54	0.14
Toe capacity	730 kN	1.00	0.26
Shaft damping	0.65 s/m	1.07	0.22
Toe damping	0.50 s/m	0.54	0.40
Combined Damping	0.50 s/m	0.84	0.19
Shaft quake	2.5 mm	0.46	0.32
Toe quake	2.5 mm	4.15	0.23

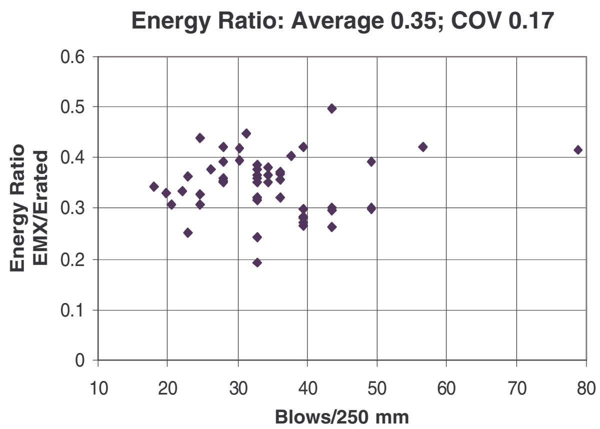

Figure 7. Energy transfer ratio vs blow count for 42 piles at the same site.

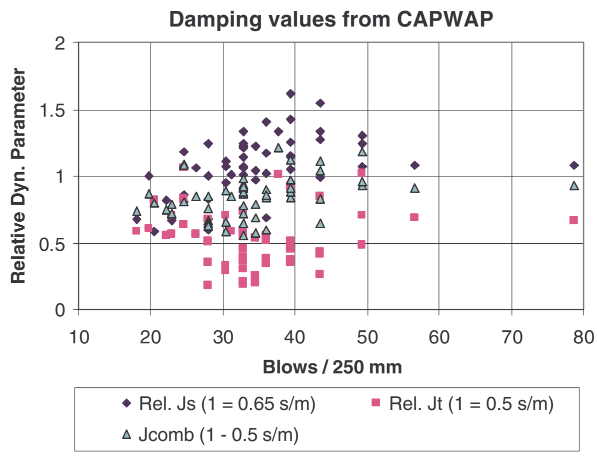

Figure 8. Damping values for 42 piles at the same site.

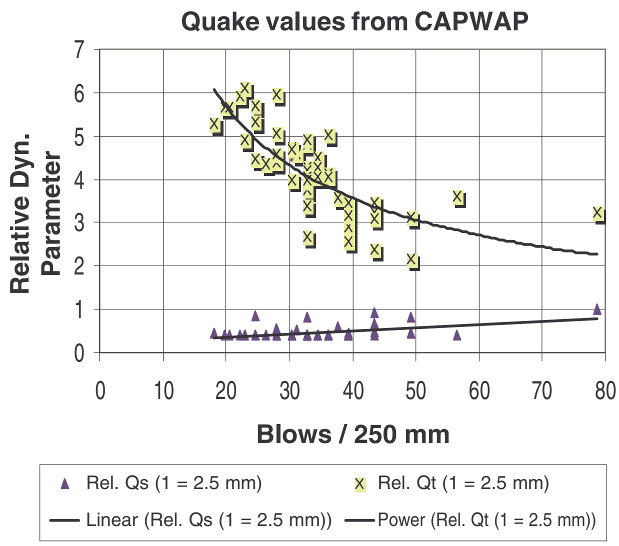

Figure 9. Quake values for 42 piles at the same site.

only because of the resistance increase itself, but also because of the reduction in its elastic limit (Fig. 10). Furthermore, the toe quake was much larger (3 to 6 times) than would ordinarily be expected for an H-pile driven into shale – or into any other soil for that matter. This example suggests that when determining the Smith soil model parameters, soil type is not a sufficient information, because of the great variations within the same soil layer.

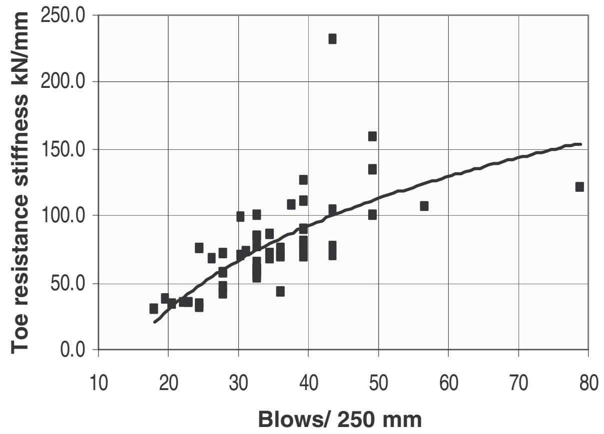

Figure 10. Toe resistance stiffness in shale for 42 piles at the same site.

2.3 *Dynamic monitoring problems*

Dynamic monitoring requires that the pile is instrumented during all or a part of its installation. The result is an assessment of hammer performance, pile stresses, pile integrity and capacity at the time of testing. Dynamic monitoring is often required where long piles are driven with many hammer blows through layers of variable resistance and/or to or through hard soil or rock layers. It is particularly useful for concrete piles where high tension stresses may develop during early pile driving or compressive stresses can cause damage at either top, along the pile or at the bottom. Potential problems include:

1. Erratic/erroneous measurements possibly exist due to (a) sensor attachment problems, faulty sensors or cables, or sensors out of calibration; (b) pile top problems including poor quality pile top material, cracks in concrete, and yielding steel or (c) excessive magnitudes of hammer eccentricity, stresses, frequencies or accelerations. The test engineer can be blamed for the conditions in (a), but conditions (b) and (c) require better quality materials or pile top preparation (e.g. flat, perpendicular to pile axis), or better hammer-pile alignment by the contractor for improvement.
2. Improper pile property input produces erroneous results: this condition may be due to a mistake of the test engineer, but it may also be based on wrong information provided to the engineer; for example, the pipe wall thickness is not easily verified once the pile is in the leads. Also the pile size and material strength is frequently unknown. For example, tolerances of steel pipes, rolled steel sections or concrete piles may be higher than nominal by 10% and that would mean that energy and capacity results (but not stresses) could be low by as much as 10%.
3. Required capacity wrongly assessed; this is a common problem because of the difficulty of

doing an accurate capacity assessment during driving when there is little time to perform a rigorous signal matching analysis. Ideally, specifications would allow for some time during the pile installation for the engineer to do some analysis. Hopefully, in the future more frequently than is currently the case, with on-site internet access, data will be immediately sent to office personnel to help with the signal matching analysis while testing goes on at the site.

4. No clear relationship between blow count and calculated capacity. If only a few tests are conducted on a site, a clear trend may not be apparent. The example of Fig. 6, discussed above, showed a clear trend of capacity versus blow count, however, it also showed great scatter. Reasons for the scatter were discussed earlier. Considering this scatter, if only 4 instead of 42 piles had been tested, the results could have been bewildering. Specifications should, therefore, require sufficient numbers of tests to assess site variability and confirm consistency of conclusions.
5. Pile damage recognized late or not at all; unfortunately damage is always recognized too late because it takes a few hammer blows until it is certain that damage has indeed occurred, and once damaged the pile cannot be fully restored. Not recognizing existing damage at all is another question; it may happen because the pile is non-uniform, the records are inconsistent, damage is very close to the pile toe or the wave speed is not accurately known. Furthermore a damage such as an ovalization of a pipe may not produce a discernible reflection and similarly a twisted H-pile, embedded in rock (the pile in Fig. 11 was driven into karst) may not always cause a clear damage reflection, because the axial pile impedance does not abruptly change. In the dynamic pile test record, a damage reflection would be indicated by a clear reduction of the gap between force and velocity prior to the pile toe reflection. In Fig. 11 the time of the pile toe reflection is indicated by the second major vertical line and before the time there is no clear reduction of the difference between force and velocity. It may also be debatably as to what constitutes pile damage. For example, a hairline crack of a prestressed pile would, under certain circumstances, be a non-issue. In a corrosive environment, however, it may not be tolerated and considered a pile damage. In the absence of tension stresses during the test, the hairline crack may not be recognized by standard dynamic monitoring.

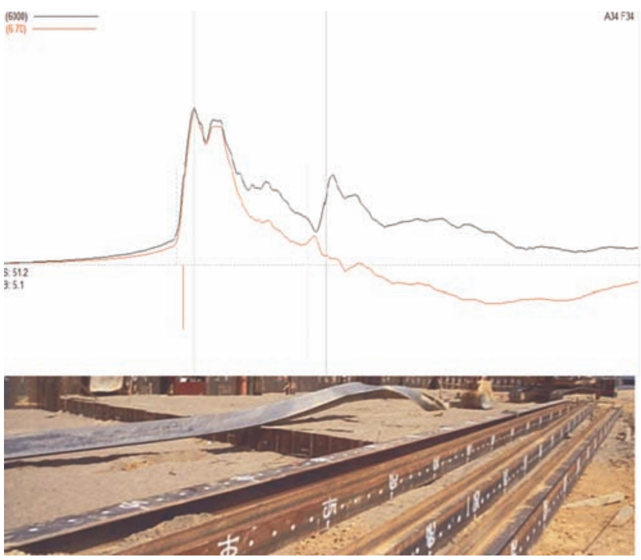

Figure 11. Example of a twisted H-pile (below) and the associated EOD record of force (upper curve) and velocity vs time.

6. Pile erroneously classified as damaged; this may occur when measurement problems exist, a splice exists and it may cause a reflection even though it is not cracked or broken, or when the pile material wave speed is unknown.
7. Stresses not maintained below safe levels; this may be due to a lack of knowledge of the pile material strength or, for concrete piles, an inaccurate wave speed which affects the measured stress level. Bad strain and/or acceleration measurements are an obvious reason for misjudged stresses. Sometimes the tester knows that stresses are high yet communication with the contractor is lacking or delayed; it is important, therefore, that communication be established between inspector or foreman and tester to prevent damage from high stress conditions.
8. Inadequate hammer performance not or erroneously recognized; acceptable hammer performance has to be reasonably defined in the specifications so that it is not a judgment call. Such definitions have to be inspired by knowledge of what can reasonably be expected from a certain hammer type. A wave equation analysis may provide insight.
9. Unhappiness develops when the pile driving progress is slow due to delays caused by construction site problems, such as mobility constraints or equipment breakdown and repairs; relatively minor interruptions caused by sensor attachment are then blamed for the slow progress even though these are only incidental time delays.
10. Initial conclusions later revised and/or final report late; Revision of initial field results, particularly capacity, must be expected after subsequent signal matching. Reasonable conditions for the report should be placed on the engineer. For example, requiring monitoring on a continuous 12 hour/day schedule and then requiring results within 24 hours is unreasonable unless testing and analysis can be done by two

engineers. Also, after the field work is finished, the tester has to travel back to the home office. Note that some delays can be minimized if the tester is given basic hammer, driving system, pile, bearing capacity and soil information without delay. Problems with a late report can also be reduced if the tester can access the internet and have a second engineer at the home office do the analysis work. Remote testing (Likins et al 2004b) will further alleviate this problem.

2.4 *Dynamic load testing problems*

Bearing capacity determination is often the most important objective of dynamic pile testing because this is where the most money and time can be saved. Unfortunately, these economic and time advantages have caused a push for even lower testing cost and shorter construction interruptions. As a result, quality of testing and reliability of results suffer. On the other hand promising too much (no preparation needed; we will be in and out in no time; any hammer will do; analysis is unnecessary and only adds cost; etc.) is the fault of the testing houses in their quest for getting a job. Potential problems, for example described by Likins et al. (2008b), include insufficient waiting time between pile installation and test, and insufficient hammer size. The latter problem is aggravated if more capacity has to be activated than possible due to a limited pile size (i.e., the required capacity divided by pile cross sectional area exceeds approximately 80% of pile strength.) Other potential problem areas include an excessively soft cushion (because of fear to damage an installed and intact pile), or poor pile top condition for measurements and energy transfer (Fig. 12).

For bored piles, the concrete has to have achieved sufficient strength prior to testing. If the pile top is not square with the pile axis, and flat, the data can be compromised. Furthermore there has to be sufficient and competent working space around the pile both for proper sensor attachment and for impact device placement.

2.5 *Signal matching problems*

The signal matching analysis, based on measurements, is currently our best means for dynamically assessing capacity. Since the late1980s the CAPWAP program includes a reasonably accurate automatic procedure (Likins et al., 2004); CAPWAP is the most commonly used signal matching program and employs the soil model proposed by Smith in the 1950s with some modifications (PDI, 2006). The unknown parameters of this model are the static ultimate resistance forces (both positive during loading and negative during unloading) along shaft and toe, the associated damping and quake values, the latter both for loading and unloading, radiation damping for shaft and toe, damping options (Smith and viscous), residual stress analysis option and soil mass related resistance forces. Occasionally, the automatic procedure fails to properly separate the end bearing from the shaft resistance near the toe, however, it is doing well in predicting total capacity.

Figure 12. Example of a pile unsuitable for testing.

Signal matching is frequently criticized of being non-unique. Indeed it is possible to achieve differing results. Viana da Fonseca and Santos (2005), reported the analysis results prepared by six different analysts for records from one or more hammer blows for a Class A prediction; analysts were given the same records for one test pile and the participants analyzed up to five different hammer blows. Fig. 13 shows the results from Signal Matching including the calculated ultimate values of end bearing (Rb), shaft

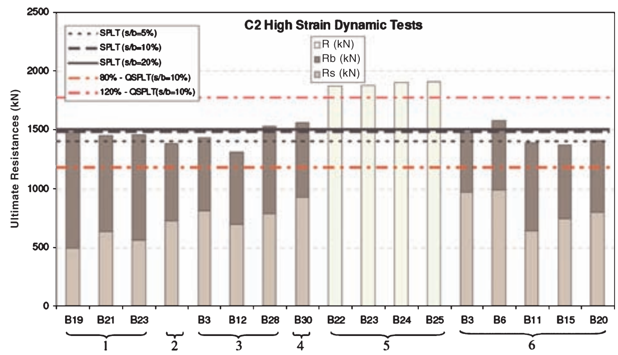

Figure 13. Comparison of capacities from signal matching of dynamically tested pile and static load test on companion pile after Viana da Fonseca and Santos (2003).

resistance (Rs) and total ultimate capacity (R). Five analysts had rather comparable R values, as is commonly true, although their Rs and Rb showed some variability. One tester obviously predicted high, but still close to the upper limit of static test interpretation (QSPLT) of a companion pile. It is interesting that Fellenius, 1988, reported similar predictions from a variety of analysts at the 3rd International Stress Wave Conference.

Admittedly, non-uniqueness of damping and quake factors in addition to the separation between end bearing and shaft resistance can be a problem. Reducing the number of unknowns by simplifying the model would be one possible solution, however, the number of unknowns actually increases as more experience is gathered, owing to the large variety of conditions which must be represented with the same soil model. One way of reducing the number of unknowns would be pre-assigning damping and quake values or at least to limit their possible ranges and that has lead to some success. But this requires extensive experience for a variety of pile and soil types and should be done under consideration of the particular pile type tested. For example, a drilled shaft test conducted with low velocities requires different factors than a driven steel pile test. In any event, if the number of unknowns is severely reduced then the model may be incorrect, the signal match may not be satisfactory and the capacity might not be correctly determined.

As for the suggestion to use static analyses results as a guide for the signal matching process, only when the signal match is indifferent to capacity changes would this be of help. In most cases such a process would be counter to the idea of a "Best Match". Comparing the results shown in Figs. 1 and 13, it is obvious that static analysis results generally do not help improve the results obtained by signal matching.

2.6 *Simplified analysis vs. signal matching and automatic signal matching*

The simplified method, having only one damping factor as an unknown produces a unique result fo one damping factor. In fact, it is easy to see the influence of the damping: the higher the factor, the lower the static capacity. However, if no "calibration analysis" is performed, a temptation to "negotiate" a certain damping factor may develop since results for different assumed damping factors can vary widely. Years of practice (since 1970) show that much more reliable results are obtained if damping factors for a site are established by signal matching. In the example case of 42 piles driven at the same site, the CAPWAP and the Case Method capacity with a damping factor $J = 0.8$ (Fig. 14) shows good correlation over a wide range of capacity values.

There are two schools of thought when it comes to signal matching. One school strongly advocates an automatic solution which has not been modified by the

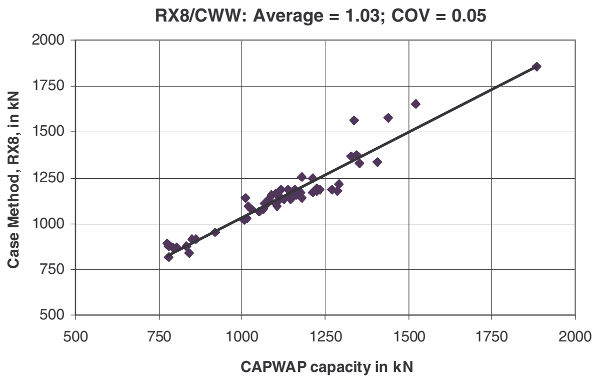

Figure 14. Simplified vs Signal Matching for 42 cases at the same site.

analyst and, therefore, should represent an unbiased capacity prediction. The automatic procedure minimizes the differences between calculation and measurement, and includes various restrictions on the selection of the model parameters and in so doing produces a unique solution. The disadvantage of the automated procedure is that its solution may not be the most enlightened and depends on the particular computer program which in turn is the result of more or less experience. The second group of CAPWAP analysts therefore prefers a solution which is derived only by interaction between computer and an experienced engineer to adjust the soil model parameters. The present CAPWAP combines the advantages of both approaches by requiring engineer interaction while offering powerful automatic options.

3 RECOMMENDATION FOR DEVELOPING A DRIVING CRITERION

a) After all testing and analyzing is completed, the tester often has to develop a pile installation criterion and that is again an activity that can lead to confrontation. The driven pile has an advantage over all other foundation elements: it is being tested as it is being installed, even if testing is only an observed blow count. This advantage exists only if the piles are driven to a reasonable installation criterion. Test piles can be from a special preconstruction test program, or they may be simply some of the first production piles on smaller projects. The driving criteria for the test piles must be decided in advance or problems will quickly develop. The installation criteria may be based on depth of penetration (high setup soil), blow count (when no measurements are taken) or simplified method which, within a short time period, should be backed up by signal matching.

b) Making sense out of the resulting, potentially huge volume of data is not a simple matter. The data may include a possibly large number of installation and restrike dynamic tests which have to be evaluated for capacities by signal matching, stress levels at top and along the pile, the potential for or actual pile damage and the hammer performance. Refined wave equation analyses, based on measured energy and stresses and soil resistance from signal matching, are also sometimes required and these are particularly time consuming.

Unfortunately, the advantage of preconstruction tests is optimal if the production pile driving criterion is issued immediately after testing has been completed. Then the production pile installation can continue without interruption. These considerations lead to construction site pressures for an immediate formulation of a final driving criterion and unhappiness if a later review of the data indicates that the piles did not quite reach the capacity and corrective action may have to be taken. Unhappy clients and/or contractors will definitely blame the tester. Of course, time savings can be realized if a conservative criterion is issued and this may be the preferable solution for small jobs; the test program, therefore, should not exceed the time and effort for the actual foundation installation. However, for larger projects, because of the complexity of the testing and analysis work and potential for large savings, the tester should receive some reasonable time period in which to develop the optimal recommendations to obtain significant savings.

Obviously, the best solution to this time pressure problem is having better testing tools for quick and error proof data processing. Remote monitoring obviously greatly helps, because it allows simultaneous field measurements and office analysis. Alternatively, equipping the field engineer with a broadband internet hook-up also allows for immediate engineering support from the office. In either case an independent result review in real time is then possible.

The proposed driving criterion has the potential for disagreements over several details; one common point of contention is requiring both a minimum blow count per unit penetration (or maximum set for a certain number of blows) and a minimum pile toe penetration. Yet the latter is important when soil setup must provide a major portion of the bearing capacity.

Another point of disagreement is the so-called refusal criterion. The tests may indicate additional capacity at very high blow counts while hammer manufacturers and contractors want to limit blow counts to at most 250 blows/250 mm (1 mm/blow). The benefits and problems caused by a higher criterion have to be clear to all parties. Prolonged pile driving at very high blow counts can lead to costly damage of pile and driving equipment.

4 TESTING AND ANALYSIS RECOMMENDATIONS

The following is a list of recommendations compiled to avoid problems for the pile tester on the site. This list is neither complete nor unique and should complement what is already contained in testing codes, standards (e.g. ASTM D4945), government specifications or trade association recommendations.

(a) Hammer, hammer energy setting, cushion: must be chosen for possible capacity activation and safe stresses. An improper hammer may render the test useless. Low energy systems may be safer but not able to prove the required capacity. Slowly rising energies during a restrike may cause capacity losses of the set-up gains before sufficient energy has been generated to activate the required capacity. Thus it is recommended to
- perform a wave equation analysis prior to the test. The analysis should show that the desired capacity can be mobilized with a set per blow of more than 2.5 mm and tension and compression stresses allowed for driven piles (Hannigan et al., 2006) or the strength of the concrete (bored piles).
- utilize a cushion which is pre-compressed prior to performing the restrike test,
- carefully check the hammer-pile alignment,
- check that the hammer has to be in good working condition (warmed up) or_mobilize a special test hammer, for example, when high soil setup generates high shaft resistance.

(b) Stress and Integrity Control: to avoid problems tester and pile drivers have to communicate during the monitored pile installation. Then
- if for xx (e.g. 10) consecutive hammer blows tensile or compressive stresses (averaged over the cross section) exceed allowable values, corrective action must be taken,
- if non-uniform stresses exceed allowable stresses by xx% (e.g., 20%), corrective action must be taken,
- if consistent indications of pile damage with a $\beta <$ xx (e.g. 85: or 15% loss of impedance) become apparent, stop pile driving and take corrective action.

(c) End-of-drive or Restrike Test: This is a complex issue because of conflicting interests and it should not be left to the tester to make this decision upon arriving at the site. For example, the engineer may prefer a dynamic restrike load test while the contractor would rather require that the pile reaches capacity during the installation. Driving the pile to full capacity eliminates waiting for setup to develop, but may require longer piles. The following are possible scenarios from which to choose:
- Drive the test pile until the Case Method, with damping factor x, exceeds the required capacity for at least xx (e.g., 20) blows (this implies that experience exists for the soil behavior at this site),
- Drive the test pile until the maximum Case Method with damping factor x, exceeds xx%

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(e.g., 90% for granular soil and 66% for cohesive soils) of the required capacity for at least xx (e.g., 20) blows; restrike after xx hours (e.g., 4 hours in sand) or xx days (e.g., 3 to 7 days in cohesive soil) to confirm that the required capacity has been achieved; immediately after the restrike test, analyze the data by signal matching to confirm capacity is adequate (time must be allowed to do the analysis).

- Drive the pile to the wave equation indicated blow count for the required ultimate capacity and take dynamic measurements at least during the last 1 m of pile driving; immediately analyze the EOD data by signal matching and adjust the driving criterion, for example, by performing an updated wave equation analysis; if necessary drive the pile further to achieve the required capacity.
- Drive the test pile to the penetration as calculated by a static analysis based on information from the borings; perform testing at least during the last 1 m of driving; perform restrike tests at least xx days (hours) later and confirm adequate capacity. Modify minimum pile toe penetration or establish a minimum blow count or both.

(d) Selection of records for analysis: At the end of driving, records are usually consistent as far as hammer energy delivered and bearing capacity and any one of several records may be chosen. For hard driving situations a high energy record should be selected for analysis. When driving into hard rock, the last or, if that is a shot-off blow, the second to last record should be selected for analysis. During restrike testing when the set per blow is less than 2.5 mm, or when testing a bored pile, the transferred energy and therefore also the activated capacity change (usually increase) from blow to blow. At the same time the shaft loses set up capacity. These effects lead to disagreements as to which record to select for analysis, which can be avoided if a procedure is established prior to the test.

- The dynamically predicted failure load shall be taken from that test blow whose maximum temporary dynamic toe displacement plus the sum of the permanent sets of all previous test blows exceeds the pile diameter divided by 60 (this is recommended when a permanent set is realized under the impacts and shaft and/or toe resistance are sensitive to dynamic action) (Rausche et al., 2008).
- The dynamically predicted failure load shall be taken as the highest value obtained during the dynamic test. (This is recommended for refusal situations, i.e. a very small pile set).
- The dynamically predicted failure load shall be taken as an average of the predictions from all blows (except the very first blow) necessary to advance the pile xx mm (e.g. 25 mm). (This is recommended when the set per blow is relatively large and the soil is not very sensitive to dynamic effects).

(e) Analysis method requirement: Simplified method or signal matching – actually there is no question, for a proper dynamic load test, signal matching has to be done. However, when a large number of piles are tested under comparable soil conditions, only 25 to 50% of the piles need to be analyzed by signal matching for long term bearing capacity.

- At least xx% (e.g., 25%) of the piles tested during installation and xx% (e.g., 50%) of the records taken during restrike shall be analyzed by signal matching. The minimum number of signal matching analyses is one for each type of test.
- The simplified method shall be "calibrated" to match the capacity from signal matching; capacity shall be calculated by this simplified method for all data not analyzed by signal matching.

(f) Superposition analysis: Activating full capacity generally requires a net displacement of more than 2.5 mm per blow. It may not be possible to activate the total capacity because of either limited hammer energy or because full capacity activation could damage the pile. When the net displacement is less than 2.5 mm per blow during restrike, superposition (e.g., end bearing from the end of drive plus shaft resistance from early restrike blows) may be an acceptable way to calculate the total restrike capacity (Hussein et al., 2002; Stevens, 2000). Where relaxation of the end bearing is a potential problem, superposition should not be allowed. Because of its potential for over prediction, peer review of superposition analyses and results is strongly recommended. The specifications could include the following option.

- For restrike tests with early sets per blow less than xx (e.g. 2) mm/blow superposition analysis shall be performed and reviewed by an independent, experienced dynamic test engineer.

(g) Radiation damping analysis: Likins et al. (1996) have shown that the radiation damping model for shaft resistance has the potential for greatly improving the predictions of signal matching. However, thus far no generally acceptable recommendations exist since this option tends to predict higher capacities than standard analyses. The option should be restricted to essentially granular soils (i.e., when the bearing occurs in non-cohesive soils) and pile sets per blow less than 10 mm (blow counts greater than 25 blows/25 cm). The radiation damping option is preferred for bored piles, for driven piles where damping factors would otherwise exceed recommendations, and for piles with sets per blow less than 2 mm (blow counts greater than

125 blows/25 cm). The option may also be reasonable for open profiles (pipes or H-piles) with potentially slipping plugs. Peer review is recommended.

- For records with sets less than xx mm/blow (e.g., 10 mm), signal matching shall be done with shaft radiation damping if failure to do so would raise shaft damping factors above 1.3 s/m.

(h) Residual stress analysis (RSA): this analysis option is reasonable for long flexible piles with significant shaft resistance. Even for BOR in soils with high setup and low shaft resistance at EOD may RSA provide more realistic answers than the standard analyses. RSA is less advantageous for short piles, particularly short concrete piles. Some clients prefer, and therefore should explicitly require, this analysis type. It may produce a more realistic prediction of resistance distribution, however, it is more time consuming and may require more experience than standard analyses.

- Signal matching shall (shall not) be done with residual stress analysis option. When residual stress option is used, the negative shaft resistance limit (UN for CAPWAP) has to be at least 0.2.

5 OUTLOOK

There is still more that can be done for hardware and software development and also as far as the preparation of the tester for future testing services is concerned. The following are a few suggestions.

5.1 *Sensors*

Wireless sensors are now available, reducing the potential for failure of the main cables. However, better protection of the sensors themselves is not a completely solved problem. Furthermore, anchors in concrete piles are still prone to failure if not properly installed. Both problems could be easily solved with the cooperation of the pile casting and pile driving industry by casting protective recesses with anchors already installed in the test piles or by using leaders with sufficient distance between guides to reduce the likelihood of sensor damage. Under favorable circumstances, force transducer testing, as commonly used in the 1960s and early 70s will be a better solution. A relatively small, ruggedized pile top force transducer would be very a very useful device, particularly for concrete piles.

Another problem is that the industry has been producing hammers that can be operated without cushioning. Just when acceleration measurements became very reliable for cushioned hammers, harder hitting, uncushioned models appeared, generating high frequency, high acceleration (in excess of 10,000 g's) signals which can lead to electronic and/or mechanical overloads and/or fatigue failure of the sensors. Further ruggedizing of accelerometers for these higher acceleration levels, therefore, has to continue. This development cannot, however, be at the expense of the transducer's electronic stability, in other words, double integration should still lead to accurate velocity results and acceptable displacement measurements.

5.2 *Processing equipment*

The signal conditioning and data processing equipment has reached great maturity and provides the knowledgeable tester with a huge amount of valuable information for each hammer blow. It warns of possible sensor malfunctions, excessive stresses and pile integrity problems and displays numerically and graphically information that is invaluable to the experienced tester. For the less experienced person, some of the available information is beyond their understanding and daily needs. Simplifying the equipment could simplify life for those testers who are involved in only a limited scope of work; results from inexperienced testers should be peer reviewed by those with more experience.

5.3 *Software*

The foregoing discussion on signal matching clearly suggests that the best solution would be an automated analysis which can reliably be used for standard tasks. Additionally, it would help, if the software were able to automatically recognize pile model problems such as caused by cracks, plugging, non-uniformities (particularly for drilled shafts), soil model difficulties as with radiation damping, highly plastic soil types which do not follow the elasto-plastic static and linearly viscous soil model. Other complex issues include the need for a more reliable extrapolation from tests with insufficient energy/pile top force to ultimate capacity, a better understanding of the influence of hammer type, energy, and blow rate on capacity reduction due to pile driving, and developing a better procedure for calculating plugged end bearing static resistance from dynamic tests. Many of these non-standard issues cannot be dealt with by testers with a limited experience base and meeting these research and development goals will require substantial additional time and effort.

5.4 *Tester and other participants*

As hardware and software become more and more sophisticated, the testers have to be able to take advantage of these developments. This means for

- The pile tester: Dynamic pile testing provides an exciting and challenging opportunity with great job satisfaction, but it requires personal involvement and commitment. The tester, therefore has to tolerate occasionally rough site conditions. The tester should review the pile installation and testing specifications carefully and if they are incomplete, address areas of concern prior to beginning the test. The tester should understand the stress wave theory on which the dynamic testing results are based, the pile material limitations, the hammer's operating principle, the projects and the site and soil conditions. Also the tester should come to the site well prepared, be attentive and accommodating on site. Asking colleagues for support and peer review is not a sign of inexperience, but quite the opposite, it shows that the tester understands the complexities of the job. Above all, be safe.

- The testing house management: understand the requirements and pressures that are put on the tester. Create a reasonable working environment and do not separate the field work from the office work. For the tester it is important to realize the challenges of the analysis and the analyst has to have a thorough understanding of the field work. Provide sufficient time for the tester to produce the project report in a timely manner. Do not promise immediate results if they cannot be properly reviewed. Do not require excessive working and travel times from your tester. Tired testers do not perform well and pose a safety problem. Provide for peer review.
- The specification writer: be clear as to what goals you want to achieve with the test. This paper has pointed out a number of problems that can be avoided with proper test specifications and preparation. Include an allowance for peer review if conditions are expected to be complex. Require that the tester issues the report directly to the owner or its representative, and preferably be retained by them to avoid pressure from the contractor. Include in the specifications what level of certification or experience, if any, is required for the testing house.
- The owner: dynamic pile testing provides quality assurance and quality control at very reasonable costs, and therefore helps improve the value of the foundation. Measurements provide solid answers which reduce risk, and provide solutions to installation problems. However, the testing service cannot be "cheap". Selection of testing houses solely on cost comparison is an invitation to shortcuts and improper conclusions by inexperienced or unknowledgeable testers. Properly done by experienced engineers, substantial savings in foundation cost can be realized. The lowest total cost of a foundation does not come from eliminating testing costs, but rather by increasing the testing to optimize the foundation for the applied loads.

6 SUMMARY

This paper described some of the problems which face the dynamic pile tester and analyst. It was shown that both field work and analysis pose challenges. The current models include dynamic resistance parameters which are not easily defined by standard soil tests because they even vary in the same soil strata. Toe resistance parameters display a greater scatter and require more research than the relatively stable shaft parameters.

The evaluation of measurements by signal matching is the preferred and most reliable method for capacity assessment. Lack of uniqueness is less of a problem than the proper selection of options and careful and objective analysis execution. Peer review is recommended in difficult situations.

During monitoring, stress control, damage detection and pile bearing capacity assessment are challenges even for the experienced tester. Sufficient time should be allowed to properly evaluate the results. It may be better to collect less data and do a careful evaluation than generate a lot of information and limit resources for analysis. Dynamic load testing needs careful preparation. The tester cannot be expected to produce reliable results under impossible conditions such as poor pile material properties, insufficient loading devices, short waiting times etc.

While the equipment and software has reached a high degree of sophistication, further development is recommended to simplify the testing and analysis process, thereby reducing the possibility for errors.

Last but not least, the construction industry will benefit the most from this technology if good communication is maintained between all construction professionals concerned and the pile tester before, during and after the deep foundation project.

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