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Pile driving in calcareous sediments

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ABSTRACT: Pile installation is a costly undertaking regardless of pile size or soil type and it becomes even more expensive if driving equipment and pile type are not well selected for the type at hand. Unfortunately, prediction of pile driveability, a demanding task under any circumstances, is an even greater challenge in calcareous soils, particularly in cemented sandy layers, limestone or coral deposits. The performance of piles driven into calcareous materials is therefore often disappointing. Considering the amount of money spent on deep foundations, efforts to improve our analytical and experimental capabilities have a powerful incentive.

This paper discusses available installation equipment and pile types used both offshore and on land and experiences gathered with procedures frequently adopted for construction control and quality assurance. These procedures include preconstruction analyses, construction monitoring, and bearing capacity testing. Among these methods are also soil exploration, static soil analysis, driveability analysis and dynamic analyses by finite difference and finite element methods. Sometimes restrike tests are also performed some time after initial pile installation for an assessment of "long term" bearing capacity.

1 INTRODUCTION

Calcareous soils, those with a significant content of calcium particles, exist in many parts of the world where deep foundations are required to sustain substantial loads, particularly in the offshore environment. These soils often exhibit a dynamic and static behavior that is distinctly different from that observed in other soils, especially when piles are installed by hammer impact. Loss of bearing capacity due to the pile installation process and either full or only partial regain of capacity by setup, are phenomena affected to a great deal by the mineral contents or structural properties of the soil grains. Often, these dynamic soil properties are not predictable by laboratory or in-situ soil investigations. However, prior to installing a pile foundation, driveability must be checked by analysis and after installation the pile integrity and bearing capacity have to be assessed. Therefore, engineers or owners frequently use wave equation analyses and they adopt dynamic testing during installation or restrike to check the quality of the installed foundation element (*e.g.*, Dutt *et al.*, 1986).

For pile installation preparation a preconstruction wave equation driveability analysis is often performed. This analysis is based on the concept developed by Smith (1960) and was later adapted to driveability analyses (Goble *et al.*, 1997a). For this analysis to be meaningful, an accurate assessment has to be made of both long term static soil resistance (SSR) and temporary, static resistance to driving (SRD). While SSR is usually obtained from standard geotechnical methods, SRD can also be obtained through dynamic testing and test record analysis by CAPWAP (Goble *et al.*, 1997b) of data recorded under similar circumstances. The wave equation driveability analysis may also be used in hind-cast analyses of existing long pipe piles (e.g., Dutt, *et al.*, 1997; Alm & Hamre, 1998) for an assessment of soil properties. For calcareous cemented sands, this analysis may be more difficult than for other soils since the ratio of SRD/SSR (its inverse is called the setup factor in the GRLWEAP documentation) is not well known. Thus even if SSR is accurately determined based on in-situ tests or laboratory soil analyses, SRD is still only estimated. The reduced SRD in cemented calcareous sands is generally attributed to a crushing of the soil particles (Murff, 1985). For other soils, the degradation of resistance may be caused by pore water pressure changes, soil remolding, soil fatigue, sensitivity or thixotropy or other reasons described in the literature and is therefore considered reversible. In other words, while for most soils SRD can be estimated from SSR and also SSR from SRD, because it is a reversible process, this may not be the case for calcareous, cemented sands where the soil resistance may not reach full capacity even after long waiting times because of a permanent change of the soil structure.

Related to the degradation of the SSR is strain softening in calcareous sands and a variety of clays. This is not considered in the standard Smith soil model but has often been described to occur in calcareous sands (e.g., Murff, 1985; Nauroy *et al.*, 1988).

Dynamic testing of piles was originally developed in the mid 1960's and adapted to offshore pile monitoring in the 1970's where static testing is extremely expensive and at best technically difficult. Procedures have been established which allow for a rational assessment of the quality and acceptance of installed foundation piles include CAPWAP analysis which extracts the static and dynamic soil resistance parameters from pile top force and velocity records by signal matching. The static parameter values thus calculated may be used to simulate a static pile load test. For large diameter offshore pipe piles in calcareous soils, few if any correlations of dynamic with static load test capacities exist. The authors have in their data base (Rausche *et al.,* 1997) a number of correlation data sets for onshore piles driven into soils with at least some calcareous content. Analyzed by CAPWAP which is based on the traditional Smith Model, these results, see Figure 1, suggest that CAPWAP tends to be somewhat conservative, a conclusion that has also been made for general soil types (Likins *et al.,* 1996). An extension of the Smith soil model that includes radiation damping model and/or MBA (Rausche *et al.*, 1996), improves the correlation of CAPWAP results with static load tests. The conservatism of the CAPWAP approach may be caused by an insufficient hammer energy during the test, or a short waiting time after pile installation while longer waiting times are usually allowed before the static test is performed.

Important for the assessment of the bearing capacity of open pile profiles such as H-piles and pipes is the plugging effect. In general, soil plugs in large pipe piles slip during pile driving and develop full end bearing, like a closed ended pipe pile, during static load applications. Both GRLWEAP and CAPWAP treat the pile like a slender rod and therefore cannot differentiate between a hollow or a solid pile. Thus, for an assessment of the plugging effect of open ended pipes a finite element analysis may be a better approach. One such program, called TIPWHIP, specifically developed for signal matching, driveability studies and static analyses has been developed and described by Abou-matar *et al.*, 1996.

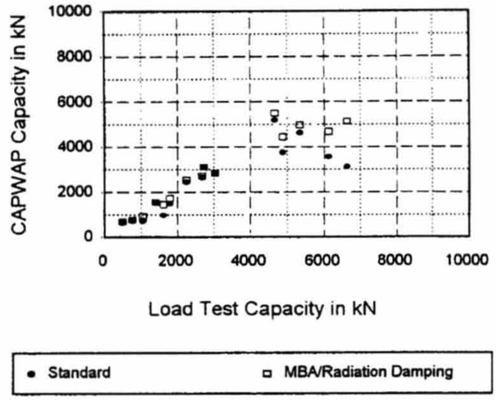

Figure 1. CAPWAP calculated and static load test capacities for large pipe piles

2 SOIL TYPES

For pile driving, calcareous soil types present special problems and in the context of this paper, the following soil types are particularly important.

- Calcareous cohesive soils, including deep deposits of calcareous clays in the oil and gas producing offshore environment. In the Southeastern United States the so-called Cooper Marl also falls under this category.
- Calcareous sands, dense or loose, with brittle sand grains and weak to strong cementation. Again, these materials are often encountered in the offshore environment. Calcareous sands produce difficult foundation conditions and the literature contains several publications on this subject (e.g. Murff, 1985, Jewell *et al.*, 1988).
- Coral and/or limestone rock, often as relatively hard layers between softer soil layers may have brittle properties, may be soft or very hard, or become soft under the dynamic effects of driven piles.

Though cemented calcareous sands are difficult, soil types with no or relatively low percentages of carbonate soil content also may present problems for pile installations. For example, dense sands or gravels sometimes produce pile capacities much lower than anticipated from in-situ soil tests.

3 PLUGGING AND INTERNAL FRICTION

Open profile piles, i.e. open ended pipes and H-piles may or may not develop a soil plug in the bottom part of the pile thereby either experiencing partial or full end bearing. If a plug develops then the end bearing forces are transferred from the soil to the pile somewhere along the inside of the pipe pile. The shear forces between soil plug and inside pile can be much higher than the shear strength of the soil due to arching of the soil confined inside the pipe, a subject that has been investigated by Paikowsky, 1990. Prior to the pile installation, for example when performing a driveability analysis, it is difficult to predict whether or not the pile will plug during driving. Particularly in dense sand layers, plugging may lead to early refusal blow counts and would require removal of the soil plug by drilling or jetting if the pile has not reached the required minimum penetration. In a related study Raines *et al.*, 1992 concluded that the shape of the steel annulus at the pipe bottom may have a significant influence on the driveability of an open ended pipe. In clayey soils the question of plugging is generally not as critical as for dense sands.

For large pipe diameters (say greater than 600 mm or 24 inches) it is a reasonable assumption that the piles do not plug during driving unless the pile is driven into a very dense sand. It may also be assumed that open ended pipes will plug after soil setup has occurred in a static loading condition. Exceptions are soils with a very low frictional resistance (relative to the full end bearing) and/or those soils that are too weak to support high arching stresses within the pipe. These arching stresses generate a high effective stress against the inside pile wall and thus friction forces that are much higher than on the outside.

One of the primary reasons why a pile will not plug during installation even though it plugs under static loads is the inertia force of the soil plug. Consider two piles, one with 300 mm diameter the other one with 1500 mm diameter both with an assumed soil plug length of 3 m (it would probably be longer in the larger pile). At a 50 g acceleration the plug inertia would be 190 kN and 4800 kN for the smaller and larger pile, respectively. For a 100 kPa sand the internal friction (not considering arching) would be 280 and 1400 kN, respectively. Thus while the plug has a chance not to slip in the small pipe, it definitely would slip in the larger one even in the absence of end bearing. Another reason why plugging does not occur during pile installation is a reduced frictional soil strength due to dynamic effects on the soil.

In calcareous sands plugging may not always occur under static load conditions because the weak grain particles cannot sustain high arching stresses. For that reason pipe piles are sometimes driven closed ended (see the section on Piles below) for an increased pile bearing capacity. However, Dutt (1985) reported that the soil plug caused an internal pipe friction much higher than the external friction because of the densification of the calcareous soil plug during driving. Randolph (1988) also concluded that the densification of the soil plug inside driven pipe piles would generate enough frictional support for the end bearing forces.

Because of the tendency of the soil plug to slip under impact hammer blows, assessing the full capacity of open ended pipes in sands is difficult. The dynamic tests tend to indicate internal plus external friction only, in other words, they do not always indicate the full end bearing or the friction forces due to arching. For a better chance to activate full end bearing in the dynamic test and to avoid losing the plug, restrike tests should be performed with low impact velocity hammers (this means a relatively heavy hammer with a low stroke) such that inertia forces are kept at a minimum. However, even if the plug does not slip under the first few blows of a restrike test on a very large pile, it cannot be expected that full end bearing is activated in a dynamic test that corresponds to a static bearing at a pile set of 10% of pile diameter as is often suggested since the dynamic pile set typically reaches not more than 25 mm.

4 SETUP BEHAVIOR

Installation of piles by impact pile driving always causes changes in pore water pressures and/or in soil structure. Increased pore water pressures reduce the effective stresses in the soil and therefore the shaft resistance. In sandy soils with good drainage there may only be a small temporary increase of pore water pressure, however, well graded sandy soils with some silt and/or clay content may exhibit very significant increases in pore water pressure. It is generally agreed that the loss of strength affects primarily the shaft resistance. However, for open ended pipes the friction between the inside of the pile and the soil plug is similarly affected and that limits the end bearing that can be transferred from the soil to the pile.

In cohesive soils, temporary soil strength losses occur because of pore water pressure changes and/or soil remolding. Cemented sandy soils and calcareous sands lose soil strength because of the crushing of soil grains and loss of cementation in the pile-soil interface. The effective stresses are then temporarily reduced since the cemented soil will transfer lateral pressures in the unaffected cemented soil around the pile-soil interface. Actually, Chow *et al.*, 1997 have shown that not only cemented sands but also dense marine sands develop such arching mechanisms around the pile which explains why sometimes there is a relatively low friction in sands during pile installation.

Once the excess pore water pressures dissipate, the frictional resistance regains its strength, a process that is called setup. Similarly, creep deformations reduce the arching effect and lateral effective stresses increase in sands after installation leading to an apparent setup behavior. Soils with thixotropic properties will also regain strength and realize setup when at rest. Restrike tests will indicate how much strength has been regained. For example, Hussein *et al.*, 1988 reported that restrike tests conducted on main piles of North Rankine A indicated setup gains up to 10 times above the soil resistance to driving under the first restrike blow. However, for the second restrike blow the soil strength was only one half and after the 20th blow practically all of strength regained by the soil during the waiting period was lost. It may be hypothesized that the first restrike blow not only encountered setup strength but also a peak frictional strength; later blows experienced a reduced setup strength and only the residual soil strength. Setup strength assessment by restrike tests should always be done with instrumentation. Purely relying on a restrike blow count may indicate a so-called "false setup" since hammers usually do not perform well under the early restrike blows.

Where bearing capacities assessed by dynamic methods are less than required, it is common practice to restrike piles several times with longer and longer waiting times to include additional strength gains in the capacity assessment. However, in calcareous sands, because of their brittle behavior and the likely crushing of soil grains during testing, the restrike test may actually cause additional and possibly permanent soil strength losses.

5 PILES

There are no particular preferences for certain pile types in calcareous soils. Foundation engineers should always consider all possibilities that may lead to the most economical foundations.

In the offshore environment, pipe piles are usually the only possible solution for reason of driveability and also, of course, for structural considerations. These pipes are either spirally welded or rolled with axial seams. Spirally welded pipes may be manufactured to less stringent tolerances than rolled piles and their locked-in welding stresses tend to produce piles with a somewhat lower strength. In addition they produce inconsistent strain measurements which therefore should be taken with at least four strain transducers at the same cross section and with gages placed sufficiently far away from the welds. Pipe piles are uniform for land or near shore applications where they are of moderate length. For offshore platforms they are often nonuniform with a heavy section near the mudline (to resist bending) and also with a so-called internal shoe, Figure 2a, which is a short section of pipe at the bottom with a reduced inside diameter (greater wall thickness) and often a beveled bottom surface. The internal shoe reduces internal friction in the pipe during driving and protects the pile toe against local damage.

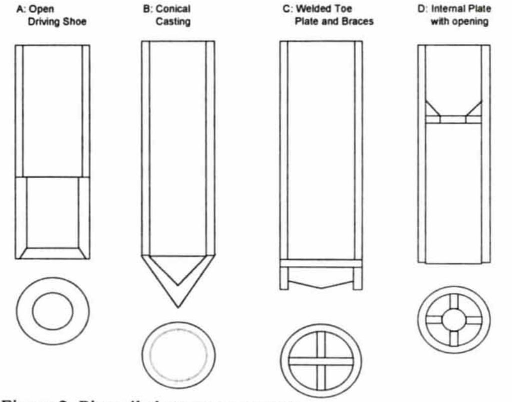

Figure 2. Pipe pile bottom treatments

While pipe piles are usually driven open-ended, a few exceptions have been reported. For example, Mello and Galgoul, 1992 described installations in the Brazilian Campos Basin with 66 inch (1680 mm) and 80 inch (2030 mm) outside diameter pipes closed at the bottom with cone shaped pile bottom sections. The cones, Figure 2b, were rather elaborate castings whose purpose was to increase end bearing while at the same time allowing for relatively easy driving. The authors reported a 20% increased shaft resistance due to densification and a reliable end bearing. Progressive densification was noted as more and more piles were installed. However, the driving effort was also increased by the closed pile toe effect.

Another example of a closed ended pipe of 48 inch (1219 mm) diameter is a project where piles had to be driven into porous limestone in the Carribean. First test indicated a very low resistance for open ended pipes. The pile toes therefore closed with a plate and cross braces were added, Figure 2c. Obviously the purpose of this closure was both a protection of the pile tip and a reliable end bearing. Similar pipes in the northwestern United States where coarse grained soils offer relatively low frictional resistance, 48 inch diameter closed ended pipes are often driven.

Closed ended pipes have also been tried at other sites where loose sands, with cemented calcareous layers created an unreliable end bearing (Paikowsky, 1992). Where these soils are competent enough to prevent installation of the piles to the necessary penetration (*e.g.*, to assure uplift resistance) the end plate might be installed at some distance above the pile toe, Figure 2d. It is then necessary to cut a center hole into the bearing plate so that soft soils or water can escape from the lower pile portions.

On land, in the southeastern part of the United States, prestressed concrete piles are often installed in calcareous soils. Because of the relatively low strength of the soils these driven piles may have a size of 18, 24 or even 30 inches square (457, 610, 762 mm). In most case, the 30 inch piles have a center void of 18 inches diameter and a closed bottom. Concrete piles can be driven into calcareous soils, soft limestone or coral without any particular protection of the pile tips. However, specially manufactured steel pile tips are available.

6 HAMMERS

Pile driving hammers are the same for the driving of piles in calcareous and non-calcareous soils. Typically, for larger pipe piles driven above water, the hammers are open ended diesels, or they are air, steam or hydraulically powered.

Hammer sizes have to match the pile size and the soil resistance for efficient blow counts and safe installation stresses. In the United States, average dynamic compression stresses during driving are usually limited to 90% of yield for steel piles or 85% of strength minus prestress for concrete piles. Bending or hammer eccentricities can add additional stresses. For example, for battered piles the static bending stresses due to hammer weight must be added to the dynamic stresses. Thus, even though the strength of the pile material probably exceeds the nominal - static - material strength for blows with a short rise time, higher allowable limits would undoubtedly lead to frequent incidences of pile damage. A commonly adhered to maximum blow count criterion is 100 blows for 250 mm penetration. Greater blow counts would easily lead to refusal if either the hammer is slightly less efficient than anticipated or if the soil resistance is slightly greater than expected. To avoid using equipment that is either too large or too small, specifications often include the requirement that wave equation driveability analyses be performed for equipment selection so that blow counts and stresses can be checked.

For the proper modeling of hammers in a wave equation analysis, a hammer efficiency value has to be estimated and used as an input in the program. For traditional hammers such as steam hammers, manufacturers ratings were based on the hammer's potential energy, i.e., ram weight times fall height. Of course, some of this potential energy is lost during the fall of the ram. The hammer efficiency which is the kinetic energy just before impact divided by the potential energy therefore includes an allowance for losses such as guide friction in the hammer. Typically, the hammer energy for single acting air hammers is assumed to be 67%. Other losses, as they occur during impact and due to helmet motion or cushion compression further reduce the energy that is ultimately transferred to the pile (transferred energy). The ratio of transferred energy to manufacturers' rated energy is called transfer efficiency, transfer ratio, or global efficiency. Figure 3 shows a graph of transfer efficiencies that were measured at the end of driving on numerous pile driving sites when air/steam hammers where driving steel piles. For the offshore industry it might be interesting that among the hammers included in the set of 376 tests, 85 had rated energies in excess of 68 kJ (50 kip-ft). Their transfer efficiencies were slightly higher than those of total population (57 vs 54%). It has been the authors' experience that the transfer efficiencies of repeatedly tested hammers show improvement with time. The reason is probably an improved hammer maintenance by contractors who use the measurements advantageously.

Modern hydraulic hammers are rated differently than traditional air/steam hammers. Rather than rating the maximum potential energy, sensors in these hammers actually measure the ram impact velocity which can be displayed or printed. This so-called net energy excludes any friction or other losses that the ram experiences prior to impact and the hammer efficiency is therefore higher if it is based on this reduced energy. When analyzing such a hammer, its energy setting must be known. The hammer efficiency that is then applied to the monitored pre-impact kinetic energy only has to consider losses that occur immediately prior to and during the impact and therefore should be much higher than the efficiency of air/steam hammers that reduces the highest possible potential energy to the kinetic energy. For hydraulic hammers, with built-in impact velocity monitoring, the efficiency is usually assumed to be 95%. A statistical summary of transfer efficiencies measured for these types of hammers is not as meaningful as for traditional hammer types since the hammers are often run at reduced settings.

Hydraulic hammers are frequently used for underwater pile installations. Unless the kinetic energy readout is available, hammer efficiency considerations must include the effect of high pressures within the hammer and the effect of water inside the pipe pile, underneath the helmet (pile cap). It is recommended that the hammer manufacturer be consulted for advice.

Somewhat anecdotal evidence suggests that soil resistance to driving is a function of a hammer's blow rate and the magnitude and duration of the impact pulse. As a consequence the blow count of a hammer not only depends on hammer energy and

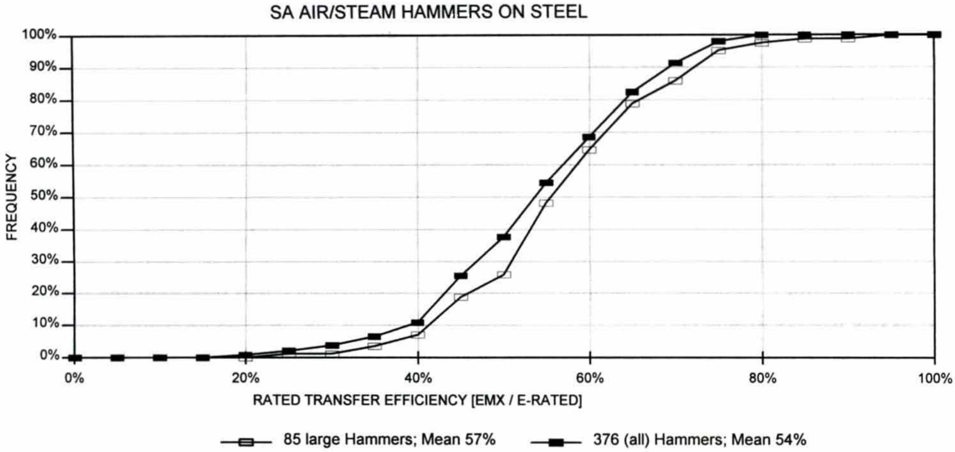

Figure 3. Measured transferred energies for single acting air/steam hammers, on steel piles

momentum but also on the hammer's ability to reduce the soil resistance during driving (SRD). These effects depend on the hammer's force pulse shape, blow rate, the set per blow and other variables and because of their complexity still require further study.

The actual impact velocity of the hammer ram during pile driving can be monitored by utilizing radar technology. The Hammer Performance Analyzer (HPA) is a field system that is used to monitor the operation of impact hammers (with visible rams) for evaluation of kinetic energy and hammer efficiency.

Large vibratory hammers are now being built (e.g. 3600 kN eccentric force) and they may also be useful in the installation of large diameter pipe piles, particularly in calcareous sands. For example, in their retrofit programs for several bridges, the California Department of Transportation (CALTRANS) has designed open ended pipe piles with up to 3660 mm (144 inch) diameter and 100 mm (4 inch) wall thickness and considered installing them with vibratory hammers. Also, Likins *et al.*, 1992 reported the use of a vibratory hammer for 1524 mm diameter (60 inch) mooring piles in clayey fine sands in the Gulf of Mexico.

Although the mechanics of the vibratory hammer lends itself to pile driveability analysis using the wave equation approach, it is still difficult to estimate the resistance to driving based on standard soil properties. For that reason, prediction of the rate of penetration of a pile driven by a vibratory hammer is still more an art than a science. It is known however, that there are important relationships between hammer frequency and the rate of penetration, not only because higher frequency are usually associated with higher driving forces but also because of certain resonance phenomena in pile and soil and because of the way in which the soil resistance changes (Massarsch *et al.,* 1992).

7 DRIVE SYSTEMS

Drive systems are mechanical devices inserted between hammer and pile that align hammer and pile and spread the impact forces as uniformly as possible over the pile top surface, see Figure 4 for terminology. For steel piles they include a hammer

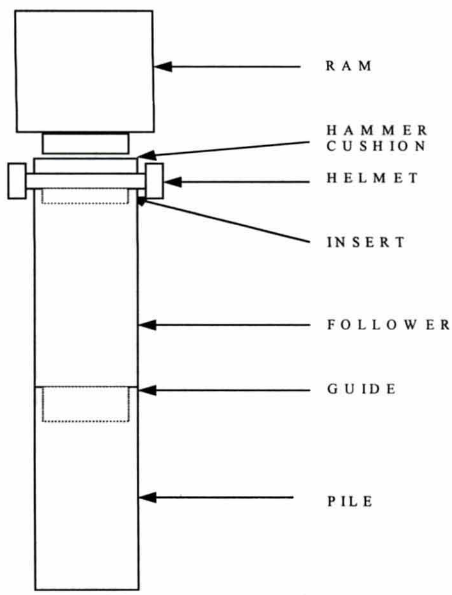

Figure 4. Driving system components

cushion and a cap. For concrete piles an additional cushion is inserted between cap and pile top.

In general, hammer manufacturers make certain hammer cushion recommendations and for large hammers there is very little choice but use their recommendations. If there is a choice among different cushion materials then consideration of the life of the cushion may be more important than price where cushion exchanges could require driving interruptions at critical times in the installation. For example, cushions made of wood typically require an exchange every 1500 blows. If a cushion exchange is required when driving resistance is high then the interruption could possibly allow for enough soil setup to produce refusal blow counts after the cushion exchange.

Followers are drive system elements which allow a pile to be driven below water when the hammer cannot be submerged. Generally, engineers and contractors employ followers only hesitatingly because of a concern for follower damage. To avoid damage, and for an efficient energy transfer, followers should have dynamic properties that are similar to the pile itself. For pipe piles the best followers are sections that are identical to the pile. For concrete piles a steel section has to be built whose product of cross sectional area and elastic modulus matches that of the pile. Actually, the fear that followers would dissipate or reflect valuable energy is unfounded if the follower impedance matches that of the pile. A greater problem is the limited fatigue life of the follower. Since it has to withstand at least the same stress level as the pile which is often near yield, and since it is reused for many piles it has to withstand many more high stress cycles than a pile. Welded connections are particularly vulnerable.

8 DRIVEABILITY ANALYSIS BY WAVE EQUATION

Originally, the wave equation method was developed to replace the dynamic pile driving formula by establishing a rational relationship between pile penetration and bearing capacity at the final pile embedment. This so-called bearing graph indicates either the minimum blow count (the inverse of the pile penetration per blow) for a required capacity or the capacity for an observed blow count. The wave equation analysis has two major advantages over the dynamic formula: (a) the method includes realistic models of hammer, pile (even very long or non-uniform piles) and soil, and (b) it is capable of accurately predicting pile stresses even for piles with complex profiles.

The driveability option in GRLWEAP (Goble *et al.*, 1997) calculates blow count and pile peak stresses at a suitable number of pile penetrations based on an accurately calculated static resistance and several dynamic resistance parameters. Since the calculated static soil resistance (SSR) usually represents long term conditions, the GRLWEAP driveability analysis also includes a means of calculating the static resistance to driving (SRD) from SSR. The difference between SRD and SSR is due to remolding, increased pore water pressure and therefore decreased effective stresses, crushing of particles, breaking up of cementation, or other soil strength changes caused by the dynamic action of the pile rapidly moving through the soil. In general, the soil regains its strength with time after pile installation. Exceptions are soils which suffered structural changes such as the crushing of calcareous soil particles during the pile installation.

Practically, the driveability analysis requires that unit shaft resistance and end bearing, together representing SSR, are input for each soil layer. If these values have been obtained by static analysis from geotechnical soil properties, then together with the known length of the pile, its circumference and bottom area, the static pile bearing capacity can be calculated at any pile penetration. Additionally, for a wave equation analysis, the dynamic soil parameters shaft and toe quakes and shaft and toe damping factors have to be known for all soil layers. To calculate SRD from SSR, the so-called Setup Factor, f_S, is introduced which is the ratio of SSR over SRD and thus SRD = SSR/f_S. The setup factor may be established from laboratory or in-situ soil tests (full shear strength divided by remolded strength), by experience in similar soils, or by either static load test or dynamic restrike test together with the dynamically determined end-of-driving (EOD) capacity.

Optionally, the GRLWEAP driveability analysis can be made even more realistic by considering what happens during waiting times, *e.g.* during pile add-on and welding operations, hammer cushion exchanges or between the end of driving and a restrike test. Figure 5 shows how in the model the resistance decreases during driving and increases during the driving interruption, based on a so-called Soil Setup Time, *i.e.* the time required for the soil resistance to increase from SRD to SSR. GRLWEAP calculates an intermediate static resistance value if the waiting time is less than the full Setup Time. This calculation is based on the assumption that the static soil resistance increases logarithmically with time. Of course, for wait times greater than the setup time, SRD is equal to SSR. The reduction of full capacity SSR to SRD is considered in GRLWEAP by accumulating the energy dissipated in a soil segment and comparing it with the so-called Relative Energy which is an input and which is the energy necessary (or roughly the distance over

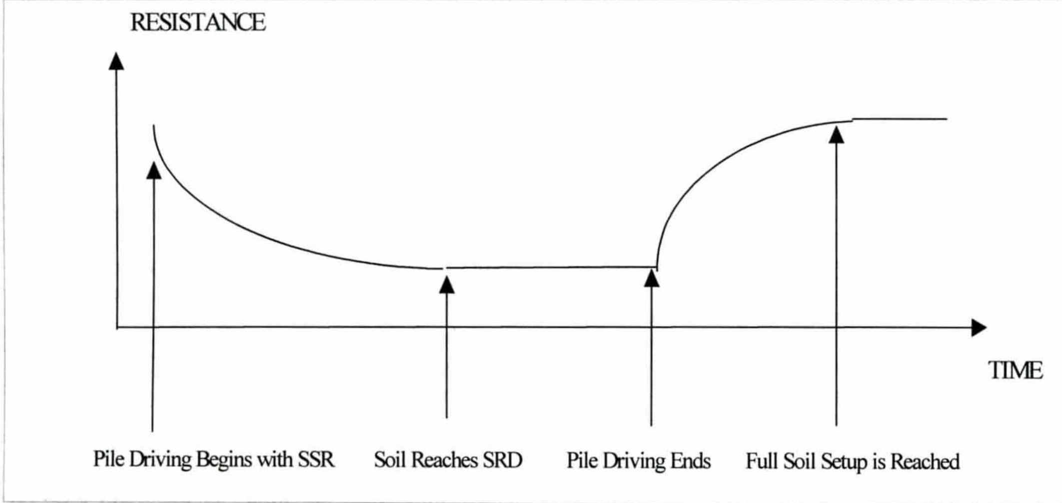

Figure 5. Variation of Static Resistance to Driving in GRLWEAP

which a pile segment has to be driven) to reduce SSR to SRD.

For driveability predictions of piles with deep penetrations and significant skin friction, residual stresses build up in the pile between hammer blows. This effect had already been described by Holloway, 1975 and a Residual Stress Analysis (RSA) was built into GRLWEAP by Hery, 1983. Comparison analyses show that RSA predicts lower blow counts than the standard Smith analysis which assumes that the pile is in a zero stress state for every hammer blow. However, either because of the concern over non-conservatism or because of a lack of correlation cases in the offshore environment, RSA has generally not been widely accepted in the offshore practice; however, it appears as though this phenomenon will be given greater consideration in the future and this will affect driveability analyses of piles in all soil types (Choe, *et al.*, 1997).

9 PILE MONITORING AND DYNAMIC LOAD TESTING

Dynamic testing during pile driving has the following main objectives:

Hammer performance checks,
Dynamic pile stress assessment and control,
Pile integrity monitoring, and
Assessment of static soil resistance to driving (SRD)

Dynamic monitoring of piles requires the measurement of force and velocity near the pile top using strain transducers and accelerometers. Measurements can be made both above and under water using a Pile Driving Analyzer® (PDA). Since the First International Conference on Calcareous Settlements in 1988, major progress has been made with this technology and integration problems of the motion measurements encountered previously when non-cushioned hammers were used (Hussein *et al.,* 1988) have now been solved even for the most severe metal to metal impact conditions, both by digital data processing and improved sensor technology. Further improvements also have been made with the methodology of data analysis.

Static resistance to driving can be calculated by a simple approach (called Case Method) which is, however, limited to relatively uniform piles. A more accurate soil resistance analysis is CAPWAP which is a signal matching or system identification process based on the records of force and velocity taken near the pile top under a hammer blow. One of these records is considered the input, the other one the output and the CAPWAP analysis determines the unknown soil resistance model that transforms the input to the output. The static component of the calculated soil model is equated to the static soil resistance SRD, if the data was taken during driving.

Dynamic load testing requires that a pile is restrike tested while PDA measurements are taken and evaluated by CAPWAP. It is recommended that a dynamic load test is done after a sufficiently long waiting time following installation. Exceptions are tests in some coarse grained soils which can be tested at the end of driving. Correlation of CAPWAP results with static load tests have always been based on restrike records (Likins *et al.,* 1996).

The literature contains many examples (*e.g.* Dutt *et al.*, 1986; Mello *et al.,* 1992; Chow *et al.,* 1998) where CAPWAP results were used for estimates unit shaft resistance and end bearing in calcareous soils in lieu of static test results. These results are particularly important for the offshore industry where direct static load testing is very difficult or prohibitively expensive. Unfortunately, not all of these results are as meaningful as they should be. If they are derived from dynamic records obtained under refusal conditions then the calculated resistance values are only lower bounds of the potentially available capacity. Furthermore, if no restrikes were performed or only after a very brief waiting period then the calculated resistance values may not reflect the long term soil strength.

10 MULTIPLE BLOW ANALYSIS - MBA

Restrike analysis by CAPWAP is often difficult because of variations of both hammer energy and soil resistance. In the beginning of the restrike test, the hammer energy typically increases from blow to blow, starting at low values which only partially activate the resistance. Thus, calculation of resistance using records of an early hammer blow, tends to under predict the SSR. However, the partial resistance activation will already cause reductions of soil resistance in the upper pile portion. The soil resistance will further decline under the following hammer blows. Thus, once the hammer energy has reached the level at which it can fully activate SSR, the soil resistance has already been reduced to some value between SSR and SRD. Thus, for a correct result, all blows have to be analyzed up to and including at least the first blow that has sufficient energy to activate the remaining soil resistance. The soil resistance values calculated for different blows then have to be superimposed.

In addition, while the restrike process occurs, residual stresses build up in the pile. Residual stresses are particularly important for long and flexible piles. Actually, residual stresses, generated during pile driving, probably already exist when the restrike test begins. (These preexisting locked-in stresses are not considered in the present version of MBA.) Thus, single blow analyses, repeated on the records from several consecutive blows may not correctly interpret the residual stresses because of the variable nature of hammer energy and soil resistance.

For this reason, CAPWAP has been expanded to allow for a Multiple Blow Analyses using force and velocity records under successive hammer blows. While MBA is performed with the same quakes and damping values for all blows analyzed, the capacity is assumed to change from SSR to some value between SSR and SRD or to SRD if enough blows are analyzed. The resistance reduction is governed by a Capacity Reduction Factor, f_R, which may vary along the pile and which determines the lowest capacity value that can be reached during the MBA. Between the full capacity, available when the first blow is applied, and the final analysis, the capacity is reduced based on the plastic deformation occurring along the pile. Typically, it takes cumulative pile displacements of more than 2 to 10 quakes before the soil resistance is fully reduced, Figure 6. This concept is not unlike the assessment of residual soil strength under static conditions where the residual soil strength is also assumed to be reached at a certain displacement (MURFF, 1987). The CAPWAP MBA procedure actually allows for an estimation of this displacement value, however, under dynamic load conditions.

The MBA procedure requires that a best match is achieved while also considering the pile penetrations per blow measured during the test. This signal matching can be rather difficult and time consuming when a large number of blows is analyzed. Of course, the increased number of unknowns (Capacity Reduction Factors) in the analysis is compensated for by the added information of additional records from blows with variable energy and soil resistance.

11 FINITE ELEMENT ANALYSIS

Dynamic finite element analysis has been investigated by several researchers (see Deeks *et al.*, 1992) for the assessment of pile stresses and soil resistance to driving. A new program called TIPWHIP, is a rotationally symmetric finite element analysis that can solve the same problems that the simpler one-dimensional programs, GRLWEAP and CAPWAP, analyze. In addition it can simulate a static test based on either static geotechnical properties from soil borings or parameters gained from dynamic signal matching. The static analysis is particularly useful for open ended piles which, depending on their diameter, wall thickness, and soil

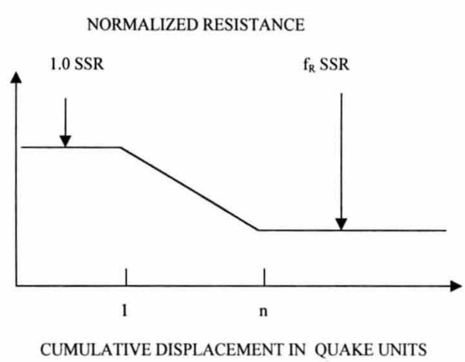

Figure 6. Strain softening in the CAPWAP MBA option

properties may not plug during pile driving but can plug during static loading where they behave like closed ended piles with full end bearing.

The pile-soil interface is represented by thin elements which obey either the Mohr-Coulomb or the van Mises failure criterion. The soil properties, *i.e.* friction angle, cohesion, elastic modulus, Poisson's ratio and specific weight, for the static analysis may either be obtained by TIPWHIP signal matching of records from dynamic restrike testing or from in-situ soil investigations such as CPT. The TIPWHIP program automatically sets up a grid including thin interface elements around the pile and soil elements to approximately 3 diameters around and underneath the pile. The main result of the static analysis is of course the load-set curve. Important for a demonstration of the load transfer is also the deformed grid.

12 EXAMPLE 1: SETUP BEHAVIOR IN CALCAREOUS MARL

This example was described by Camp *et al.*, 1992 and shows how dynamic testing helped to resolve a puzzling phenomenon an a large 4.8 km long bridge project in the southeastern United States. Below some 2 to 9 m of clay, silt or loose sand the bearing layer consisted of overconsolidated, calcareous, firm to very stiff clayey, slightly sandy silt of medium plasticity with an undrained shear strength between 100 and 150 kPa. Extensive tests, both static and dynamic, at two sites along the bridge were performed for the selection of an economical pile type. Steel pipes, steel H-piles, drilled shafts, and prestressed, precast concrete piles (PPC) were included in the comparison. Dynamic tests, correlating within 15% with static tests, indicated strong setup gains with time and pile design was based on the pile capacity achieved after two weeks wait time. Eventually, square PPC piles of 460 mm width and 18 m length were selected and installed.

Dynamic construction control tests yielded disappointing capacities, only approximately one half the anticipated capacity. To further investigate the low capacity values, a total of 22 static and 75 dynamic tests were conducted over a time period of more than two years. Furthermore extensive exploration and laboratory tests were performed to investigate these problems. Field tests included porewater pressure measurements near the test piles.

While it was confirmed that the calcareous bearing layer was as predicted everywhere along the site, piles driven through clay overburden generally had lower capacities at comparable waiting times than those driven through sand. All dynamic tests were evaluated by CAPWAP for unit skin friction in the calcareous marl Figure 7. The following trends were established: piles with sand overburden had average unit skin friction values of 70 kPa one day after installation and 168 kPa after 1000 days of waiting. For piles with clay overburden the average skin friction increased from 50 kPa after 5 days to 120 kPa after 1000 days. It was concluded that the

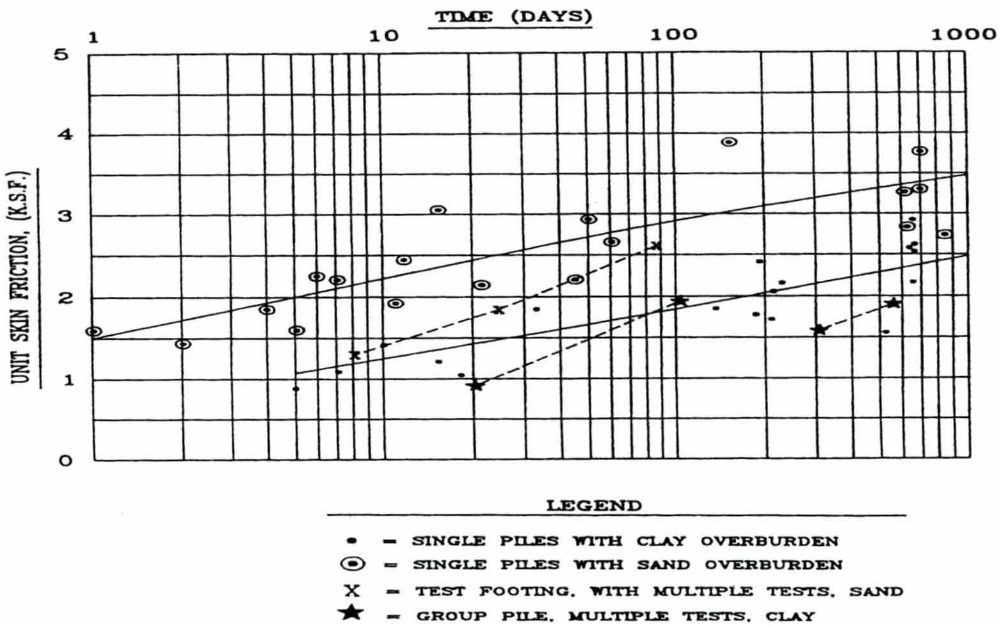

Figure 7. Capacity development with time from Camp et al, 1992

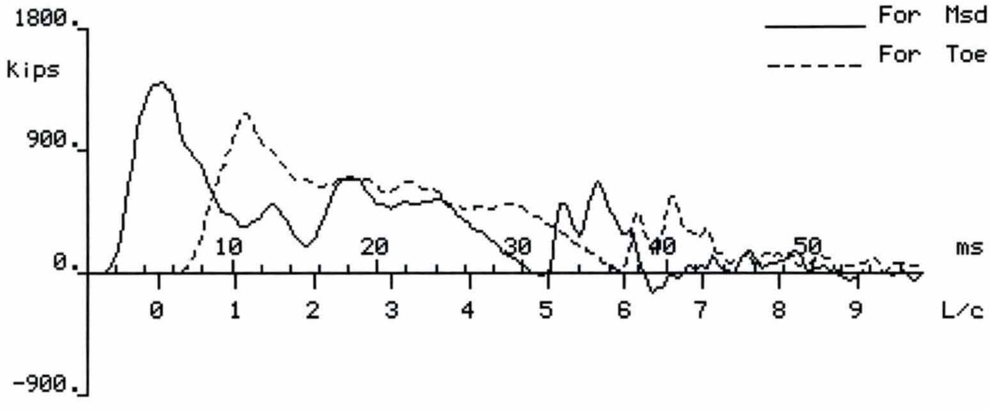

Figure 8. Measured top force and CAPWAP calculated toe force for closed ended pipe in limestone

difference in performance was a result of slower drainage in the case of piles with clay overburden.

13 EXAMPLE 2: PILE DRIVING INTO LIMESTONE

For a port facility, 1220x16 mm (48x0.625 inch) diameter pipe piles had to be driven into a coralline limestone formation. This rock was designated a Recrystalline Fossiliferous Limestone, very porous and fractured; it had greatly varying Rock Quality Designitions (between 0 and 100) and similarly variable Recoveries. Several voids were identified in this formation. Initial pile tests indicated that open ended the piles would not pick up any reasonable resistance because of the brittleness of the rock. The piles therefore were equipped with a bottom plate with braces, see Figure 2c. Using a 134 kN ram weight with 0.9 m drop height, the pile easily reached a depth of 15 m. Dynamic monitoring then indicated sufficient capacity and satisfactory toe stresses; as indicated in Figure 8 which shows the recorded pile top force and the calculated pile toe force. A CAPWAP analysis also indicated a capacity of 4200 kN of which 2900 kN acted at the pile toe. Damping at the toe, according to the Smith model, was lower than the normal 0.15 s/m value. The toe quake was 12 mm or D/100 which compares well with the usual assumption of D/120. The low damping factor and the normal quake lead to the conclusion that the limestone behaved like a stiff granular soil. For the production piles a bearing graph was developed using the CAPWAP determined dynamic soil parameters. The bearing graph of Figure 9 summarizes these results.

14 EXAMPLE 3: DRIVEABILITY ANALYSIS

The project involved driveability analyses, pile installation using a steam hammer, and dynamic monitoring of an offshore platform leg pile. The calcareous soil consisted of alternating, layers of clays and sands. Cone penetration test data (p_c values) were available.

For the driveability analysis the following assumptions were made when calculating unit shaft resistance and end bearing as an input to GRLWEAP, V. 1997-2:

- In the calcareous clay, shaft resistance (SSR) is 2% of p_c, end bearing is 40% of p_c. To calculate the shaft resistance to driving (SRD) a setup factor 2.5 was used. Thus, it was assumed that the calcareous clay had 40% of its static resistance during driving.
- In the calcareous, cemented sand, shaft resistance (SSR) was assumed to be 0.5% of p_c, limited to at most 30 kPa. Unit end bearing was set to 100% of p_c, limited to 150 MPa. Shaft resistance to driving was calculated based on a setup factor of 1.5. This is equivalent to assuming that the calcareous sand had only 67% of its static resistance during driving.
- Only external friction was considered.
- Only end bearing against the steel annulus of the pipe pile was considered. End bearing increasing linearly from the beginning of a layer to full value at ½ pile diameter.
- All quakes were set to the standard 2.5 mm value.
- Shaft damping was set to standard values of 0.15 and 0.65 s/m for sand and clay, respectively.
- Toe damping was set to 0.5 s/m (standard).

Of course, the SRD could have been evaluated

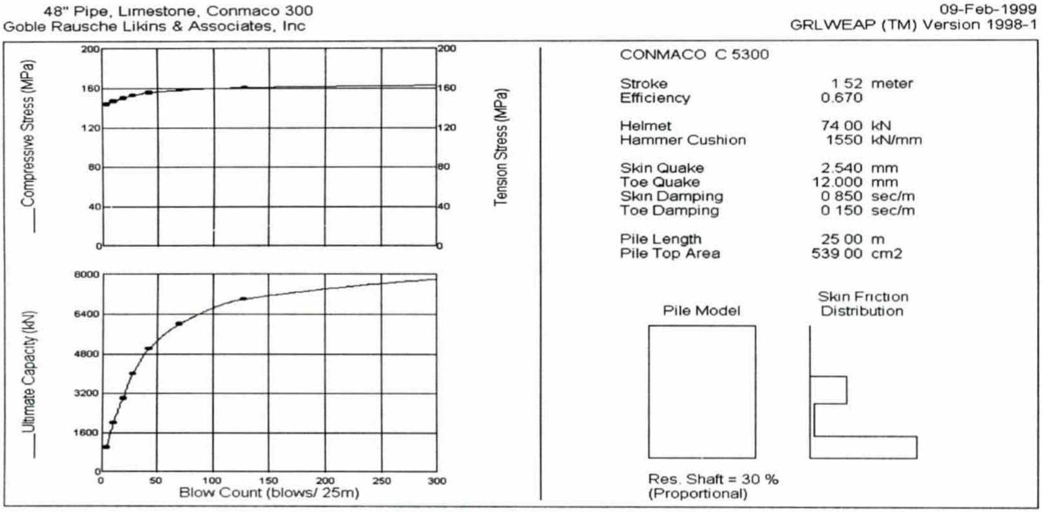

Figure 9. GRLWEAP bearing graph for closed pipe pile in limestone

with a different combination of p_c multipliers and setup factors. However, a restrike test was performed and the blow count match was reasonably good for both installation and restrike, with SRD calculated according to the above rule for the initial installation and the associated SSR for the restrike.

Figure 10 indicates good agreement between the trends of the observed and computed blow counts during driving. However, there are minor spikes in the measurements, probably due to local soil conditions, which were not predicted, or which were predicted at slightly offset penetration values.

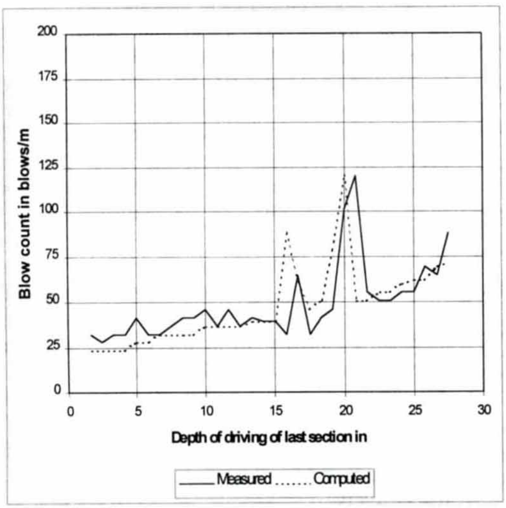

Figure 10. General trends of computed and measured blow counts

15 EXAMPLE 4: MULTIPLE BLOW ANALYSIS

For the pile of Example 3, the records of the first ten blows of a restrike test were analyzed by CAPWAP using the Multiple Blow Analysis. This was considered beneficial since the soil indicated a relatively quick loss of setup capacity. On the other hand, the restrike blow count was so high (permanent set per blow less than 2.5 mm per blow) that full capacity activation was unlikely to have occurred under the first few blows when full resistance was present. Figure 11 shows how the SRD decreased from blow to blow and how the activated capacity increased in the Multiple Blow Analysis.

For a best match of all 10 records, shaft quakes of 4 mm were required (somewhat higher than the 2.5 mm that would normally be obtained by standard CAPWAP analysis for a single blow) and the shaft static resistance values at individual points along the piles reached full reduction after the displacements of the individual pile segments had exceeded 7 quake values. The loss of soil resistance was relatively slow compared to the more rapid loss seen sometimes in cemented sands or in very soft cohesive soils and after the tenth restrike blow the SRD was still twice as high as at the end of driving. The CAPWAP-MBA results also indicated a relatively high end bearing of 6.5 MN implying that CAPWAP identified internal friction forces near the toe as end bearing. The analysis can explain why CAPWAP is, on the average, conservative, relative to load test results. However, it must be recognized that MBA may be non-conservative, particularly in calcareous sandy soils, if the soils are also sensitive to large displacements during static load applications.

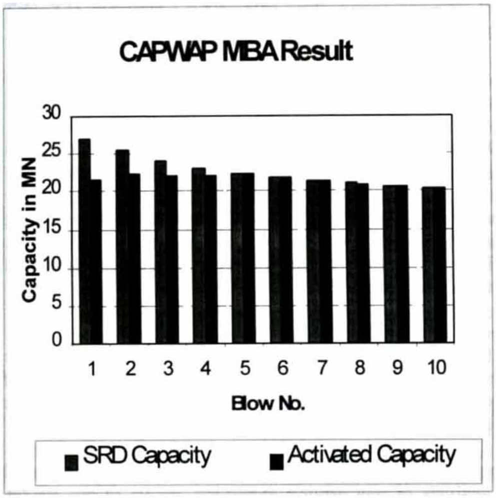

Figure 11. SRD and activated capacities from CAPWAP for first 10 blows

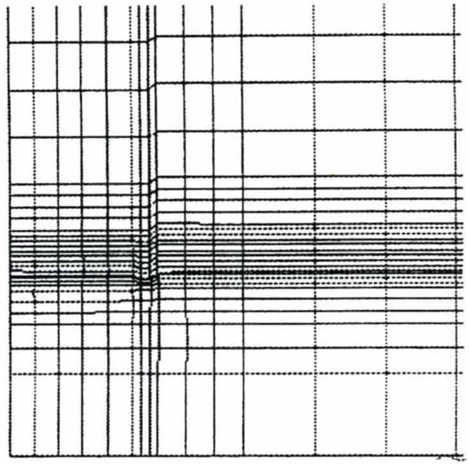

Figure 12. TIPWHIP calculated deformed grid.
Note: left edge is axis of symmetry

16 EXAMPLE 5: STATIC ANALYSIS BY TIPWHIP

To check the load transfer in the static loading case, the TIPWHIP analysis was performed for Example 3. The deformed grid is shown in Figure 12 (only the part around the pile toe is shown with the left side of the graph being the pile axis. The closely spaced vertical lines represent the pile wall.) The deformed grid shows clearly the outside interface elements deformed. On the other hand, the inside segments move together with the pile and therefore a soil plug formed inside the pile. The forces acting on the inside of the pile and against the steel annulus of the pile sum up to approximately 3,500 kN or slightly more than ½ of the bottom friction indicated by CAPWAP. The outside friction is approximately 29.5 MN and therefore higher than the dynamic test result. The TIPWHIP results are based on cone penetrometer results which may not reflect the exact local conditions that the dynamic test encountered. On the other hand, the dynamic test may not indicate the long term conditions (restrike test performed too early). Another source of inaccuracy in the dynamic test is its inability to distinguish resistance effects over small distances (e.g. resistance on the shaft near the toe vs resistance at the toe). In any event, it can be concluded that the pile plugs during static loading and/or in the restrike condition and that this plugging effect vanishes during pile driving.

The predicted load set curve is shown in Figure 13. Different from CAPWAP which is based on a purely elasto-plastic force deformation model, the TIPWHIP analysis shows strain hardening effects with large deformations. This method therefore is also less conservative than the standard CAPWAP analysis and should be used with caution where strong strain softening effects are expected for the shaft resistance.

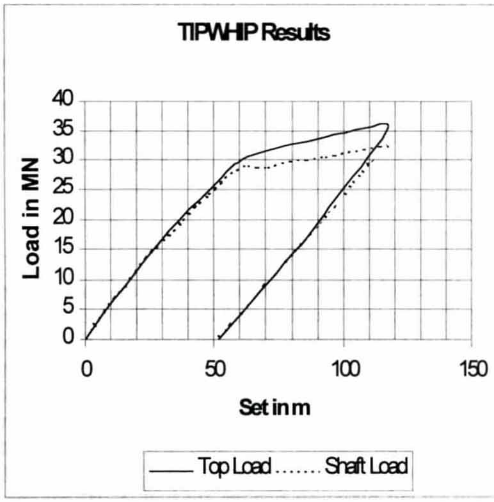

Figure 13. Load-set curve predicted by TIPWHIP

17 SUMMARY AND CONCLUSIONS

Pile driving in calcareous soils is not very different from pile driving in other subsurface conditions. The same pile types, hammers, driving systems, test and analysis methods are used. However, differences in the soil behavior both during pile driving and after the installation make driveability predictions and capacity determination much more complex. Wave equation driveability analyses using variable setup concepts, can be a helpful tool in the preparation of pile driving in calcareous soils. Agreement between observed and calculated blow counts are, of course, at best as good as the soil exploration and the calculated static soil resistance. Calculation of SRD from SSR requires high quality geotechnical data and experience for the individual characteristics in such soil types.

Instrumented restrike testing is a valuable tool to assess soil strength changes occurring after pile installation. This is true for both calcareous sands and calcareous clays. However, particularly for sands restrike testing may cause non-recoverable soil strength losses. Strength gains which are quickly lost again during the restrike may also be lost under static loads.

Analysis of restrike tests may be difficult when restrike energies are low under the first hammer blows in soils that are susceptible to strength losses, even after only a few low energy hammer blows. In that case. Multiple Blow CAPWAP Analysis helps to assess full SSR.

Dynamic finite element analysis can simulate a static pile test based on either conventional geotechnical soil properties or parameters obtained from dynamic testing. This is particularly useful in studying the plugging of open-ended piles.

18 RECOMMENDATIONS

For more economical foundations in calcareous soils, studies are recommended which would better identify the relationship between SSR and SRD as a function of driving energy and blow count. Such studies would require that instrumented restrike tests are performed with several waiting times of both short and long duration. It is also recommended that the plugging in calcareous sands and driveability of closed ended pipes be further studied. Furthermore, it is important that future geotechnical investigations classify in a more meaningful manner calcareous rock hardness, brittleness and the difference between calcareous soils and rocks.

19 ACKNOWLEDGMENTS

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