ON THE PREDICTION OF LONG TERM PILE CAPACITY FROM END-OF-DRIVING INFORMATION

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ABSTRACT: During the past 50 years extensive research has been directed at improving the accuracy of bearing capacity assessment by dynamic methods at the time of testing. It has been learned that the change of capacity with time after installation depends on a number of parameters, both soil and pile type dependent. The long-term ultimate pile capacity may range between 50 and 1000% of the end-of-driving capacity. Sometimes relaxation occurs and then the long-term bearing capacity is less than 100% of the end-of-driving capacity. Usually the capacity increases with time due to "setup".

While the accuracy of capacity prediction at the time of testing has considerably improved due to electronic measurements and more detailed analysis procedures, reaping the full benefit from this development requires restrike testing with waiting times after pile installation varying from a few hours to several weeks. Unfortunately, this puts a significant scheduling and time burden on the construction process and thus eventually on the owner. It is clear that the full benefit of a fast and inexpensive dynamic test can only be realized if better test procedures can be developed to predict long term capacity from end of driving information. Such procedures would reduce the need for restrike testing with long waiting periods after pile installation.

This paper examines the promise and shortcomings of methods that are proposed for service load capacity predictions from end-of-drive tests and compares them with standard dynamic test procedures. A quantitative assessment of the dynamic resistance is also done based on CAPWAP® analyses and these values are compared

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to setup gains considering soil type along the shaft and at the toe. Based on a review of new and existing data bases it is concluded that the most economical test procedure would include a 24 hour restrike test.

INTRODUCTION

Formulas

Historically, pile driving formulas have provided simple estimates for the engineer: an end-of-drive blow count together with the hammer energy and one or two experience parameters would approximate the useful design pile capacity without much complex arithmetic. The energy formula determines a soil resistance, averaged over the time during which the pile moves downward. This soil resistance was compared with static load test results and certain formula dependent parameters were adjusted to make the capacity result match the load tests. Because the formulas were empirically adjusted to yield reasonable results based on end-of-driving information, and because it was known that setup effects would affect the long-term pile capacity, there was hesitation to use formulas with restrike blow counts[4].

Methods developed under the leadership of Professor George Goble, based on the measurement of dynamic forces and velocities and calculations by the Case Method, (Goble et al. 1967, Goble et al. 1970) and CAPWAP, (Rausche et al. 1972) are capable of calculating accurately the static bearing at the time of testing. Likins et al., 1996, have shown that correlations are very good if the static load test and dynamic test are performed at similar waiting times after pile installation. Also, it was learned that methods based on the analysis of measured force and velocity of one particular restrike blow, rather than on an average blow count and energy, produce much more accurate results than those that are blow count dependent (e.g., the Wave Equation approach). The main reasons are the unreliability of blow count, variable hammer energy as the hammer restarts, and reversal of setup or relaxation effects during the early restrike.

It has also been recognized that substantial savings in foundation cost can be achieved if measurement methods together with setup capacity gains are considered in foundation design[5]. On the other hand, mixing accurate methods of capacity assessment based on dynamic measurements with the historical approach of using the end-of-driving information yields unsatisfactory results (Paikowsky et al., 2001). Yet the cost of restrike testing (due primarily to the delays from wait time) is sometimes prohibitive, particularly in the offshore environment, and has occasionally led to disillusionment with the modern approaches that require restrike tests after considerable waiting times for accurate long term pile capacity assessments.

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[4] Chellis, B., D., 1951. Pile Foundations, p. 47: "... Occasionally these penetration values [referring to those from a restrike test] are substituted in pile-driving formulas for those occurring at the cessation of the continuous driving, but this should not be permitted without approval of the engineer."

[5] Komurka, V.E. and Piefer, S. J., 2003. Determination of support cost distribution for the design of driven piles exhibiting set-up. PDCA Winter Round Table: "Accounting for set-up in pile design offers substantial benefits, such as the use of smaller hammers, smaller pile sections, shorter piles, higher capacities, and therefore more economical installations (lower support costs) than otherwise possible."

Damping Modeling

There is a basic problem in the modern approach of capacity determination from measurement of dynamic force and velocity: the static resistance at the end-of-driving is often less than the total resistance at the end-of-driving and the long term capacity after a setup period is higher than the static resistance at the end-of-driving. So, why not just use the total resistance at the end-of-driving and set it equal to the long-term capacity? In reality, that is the basis for the way formulas have been used in the past.

Smith's concept of dynamic resistance assessment considered a static, pile-displacement dependent component, Ru, and a dynamic, pile-velocity dependent resistance component, Rdyn.

$$R_{total} = R_u + R_{dyn} \tag{1}$$

This approach has also been widely accepted in other modern dynamic testing and analysis methods. It states that the ultimate static capacity during a hammer blow is less than the total resistance. Although this was a more rational concept than the formula approach, early wave equation proponents still preferred to calculate bearing capacity from end-of-drive blow counts[6].

Another method of assessing the dynamic resistance would consider a ratio of total resistance to static capacity; let us call the following ratio the dynamic resistance factor

$$f_{dyn} = \frac{R_{total}}{R_u} \tag{2}$$

As discussed below, the tests by Gibson et al., 1968 indicated ratios of total to static capacity of normally loaded triaxial samples roughly between 1.5 and 2.5. For velocities greater than 0.3 m/s, these ratios appeared to be independent of general soil type.

Reasons For Static Resistance Changes With Time

If setup is defined as a gain of bearing capacity occurring after installation, then the following reasons are often given for these changes:

- Positive pore water pressure changes in granular and fine grained soils, which decrease the soil's effective stress and therefore shaft resistance during installation.

[6] Hirsch, T. J., Lowery, L.L., Coyle, H. M. and Samson, C. H., 1970. Pile-Driving analysis by one-dimensional wave theory: State of the art. Highway Research Record No. 333, pp33-54: "If the soil resistance is predominantly due to cohesionless materials such as sands and gravels, the time effect of soil setup that tends to increase the pile bearing capacity will be small or negligible. If the soil is a cohesive clay, the soil setup might increase the bearing capacity as discussed earlier. .... It can also be conservatively disregarded ...."

- Arching in dense or very dense sands around the pile shaft and therefore a reduced effective stress immediately after pile installation. Recuperation of full resistance may take weeks (Seidel and Kolinowski, 2000).
- Liquefaction in loose granular soils due to the dynamic pile motions and thus a greatly reduced effective stress.
- Soil remolding as is frequently found in clays or thixotropic materials; the remolded geo-material has greatly reduced shear strength during pile driving.
- Soil fatigue, i.e. a loss of unit friction from the pile, practically smoothing the surface of the hard cohesive soil.
- Loss of cemented structure in calcareous soils.

Loss of soil resistance after installation is normally called relaxation and usually attributed to one of the following phenomena:

- Negative porewater pressure changes, caused by soil dilation or soil volume increase in very dense granular soils. As a consequence, an increased effective stress occurs during pile installation, primarily at the pile toe.
- Chemical deterioration of the soil at the pile toe due to the presence of water introduced during the pile installation.
- Gradual cracking of rock underneath the pile toe due to very high contact stresses under the pile toe.

Quantifying the Setup Time Relationship

Obviously, there are many possible reasons for soil resistance changes with time and it is therefore not surprising that an analytical method of soil setup prediction is still elusive. Formulations exist that attempt to predict how capacity will be regained as a function of time after driving. For example, Skov et al. (1988) presented an equation that assumes a linear capacity increase with the logarithm of time. Adapted to the designations of this paper, with the static capacity a function of the waiting time since pile installation, t_w, and a reference capacity $R_u(t_o)$ determined at a relatively short time, t_o, after installation, their equation would be

$$R_u(t_w) = R_u(t_o)[\ 1 + A \log_{10}\{t_w/t_o\}] \qquad (3)$$

The factor A is the relative increase of the static capacity during a 10 fold waiting time increase. The authors suggested A= 0.2 for sand (t_o = ½ day) and A = 0.6 for clay (t_o = 1 day). In the case of clay, for example, if after 1 day of waiting, the reference capacity $R_u(t_o)$ is 100 tons, then it will be 160 tons after 10 days, and 220 tons after 100 days. Note that R_u(1 day) is probably significantly higher than $R_{u\text{-}eod}$. However, the authors felt that basing predictions on $R_{u\text{-}eod}$ would introduce scatter in the prediction. Also, extrapolation to capacities with large t_w values should be done with caution. While 10 day waiting times are rather well documented, restrike tests for waiting times greater than 30 days are quite rare.

Quantifying Setup Gains

More important than describing the functional relationship between capacity increase and waiting time after pile installation is the total long term capacity that eventually will be achieved. The soil setup factor, f_{setup}, is defined as the ratio of long term capacity over end-of-driving static resistance.

$$R_u = f_{setup} R_{u\text{-}eod} \tag{4}$$

Actually, it would be more reasonable to have two factors available: one for the shaft, and one for the toe. However, while it is indeed more correct to separate the setup factors for shaft and toe, this is normally too difficult to accurately assess and therefore little accuracy would be gained with this more sophisticated approach. We will, therefore, work with Eq. 4; combining it with Eq. 2 for an end-of-drive case, which leads to

$$R_u = \left(\frac{f_{setup}}{f_{dyn}}\right) R_{total,\,eod} \tag{5}$$

Coincidentally, for sands fsetup is often (wrongly) expected to be near 1 and so is fdyn. For clays, on the other hand, fsetup is expected to be approximately equal to 2 and fdyn is also expected to be near 2. Thus, in many instances the ratio fsetup / fdyn appears to be 1 and the long term capacity Ru is roughly equal to Rtotal,eod. It is because of this coincidence why formulas based on end-of-drive observations occasionally appear to work quite well and it is a cruel fact of life that the more accurate methods to predict static resistance based on measurements at the end-of-driving may severely under-predict Ru-final.

Rausche, et al., (1996) have determined set-up factors in a variety of soil types (characterized by grain size). These setup factors were included as an example in the FHWA Manual on the Design and Construction of Driven Pile Foundations (Hannigan, et al., 1998). Their Table 9-19 is reproduced in Table 1 and clearly demonstrates the uncertainty and possible range of setup factors. For example, for clay, the range is 1.2 to 5.5, for sand-clay mixtures, 1.0 to 6.0 and for sand, 0.8 to 2.0. Unfortunately, there is considerable scatter and a single value for a particular soil type may lead to either accurate or inaccurate long term setup predictions.

TABLE 1. Soil Setup Factors

TABLE 9-19 SOIL SETUP FACTORS OF THE FHWA MANUAL (after Rausche et al, 1996)			
Predominant Soil Type Along Pile Shaft (1)	Range in Soil Set-up Factor (2)	Recommended Soil Set-up Factors[a] (3)	Number of Sites (Percentage of Data Base) (4)
Clay	1.2 – 5.5	2.0	7 (15%)
Silt – Clay	1.0 – 2.0	1.0	10 (22%)

Table 1. Soil Setup Factors Continued

TABLE 9-19 SOIL SETUP FACTORS OF THE FHWA MANUAL (after Rausche et al, 1996)			
Silt	1.5 – 5.0	1.5	2 (4%)
Sand – Clay	1.0 – 6.0	1.5	13 (28%)
Sand – Silt	1.2 – 2.0	1.2	8 (18%)
Fine Sand	1.2 – 2.0	1.2	2 (4%)
Sand	0.8 – 2.0	1.0	3 (7%)
Sand – Gravel	1.2 – 2.0	1.0	1 (2%)

[a] Confirmation with Local Experience Recommended

Investigation of Damping

Gibson-Coyle

In 1968 Gibson et al. described tests in which soil samples in triaxial cells were subjected to impact loads with speeds between 0.3 and 3.6 m/s. Three types of sand and four types of clay were investigated with a few different water contents. It is important to note that these tests gave an indication of maximum damping values and not a functional relationship between velocity and damping force. The tests revealed a very characteristic behavior: the total resistance increases quickly for low impact velocities up to approximately 0.5 m/s and at a much lower rate for higher velocities. Ignoring the low velocity portion, a straight line relationship would therefore be a reasonable approximation. Gibson-Coyle proposed an exponential relationship Rdyn = J vN; however, for a number of reasons, that approach does not improve the accuracy of predictions and therefore has not seen practical applications. Figure 1 shows straight-line approximations of the Gibson-Coyle results, which roughly covered an impact velocity range between 0.3 and 3.6 m/s, extrapolated to cover a range of velocities between zero and 6.6 m/s. In addition three lines corresponding to the normally recommended Smith shaft damping factors for sand (0.16 s/m) and clay (0.65 s/m) and for the toe (0.50 s/m) are shown. Probably only the Smith toe line is meaningful in this comparison because of the way in which the Gibson-Coyle tests were performed.

The fdyn values at 3.3 and 6.6 m/s on the Gibson lines are of particular interest, because they roughly correspond to the impact velocities of low and high stroke impact pile driving hammers. Table 2 shows these ratios for the soils investigated. A few conclusions may be drawn.

- For the 3.3 m/s velocity, the f_{dyn} values vary between 1.7 and 2.4 for sand.
- For the 3.3 m/s velocity, the f_{dyn} values vary between 2.0 and 2.9 for clay.
- There is no clear difference between Ottawa sand and clay results.
- The clay results show some tendency of increased f_{dyn} values for higher water contents.
- Doubling the velocity values from 3.3 to 6.6 m/s increases f_{dyn} values by roughly 15%, a relatively low percentage considering that the f_{dyn} values are typically 1.5 to 3.0.

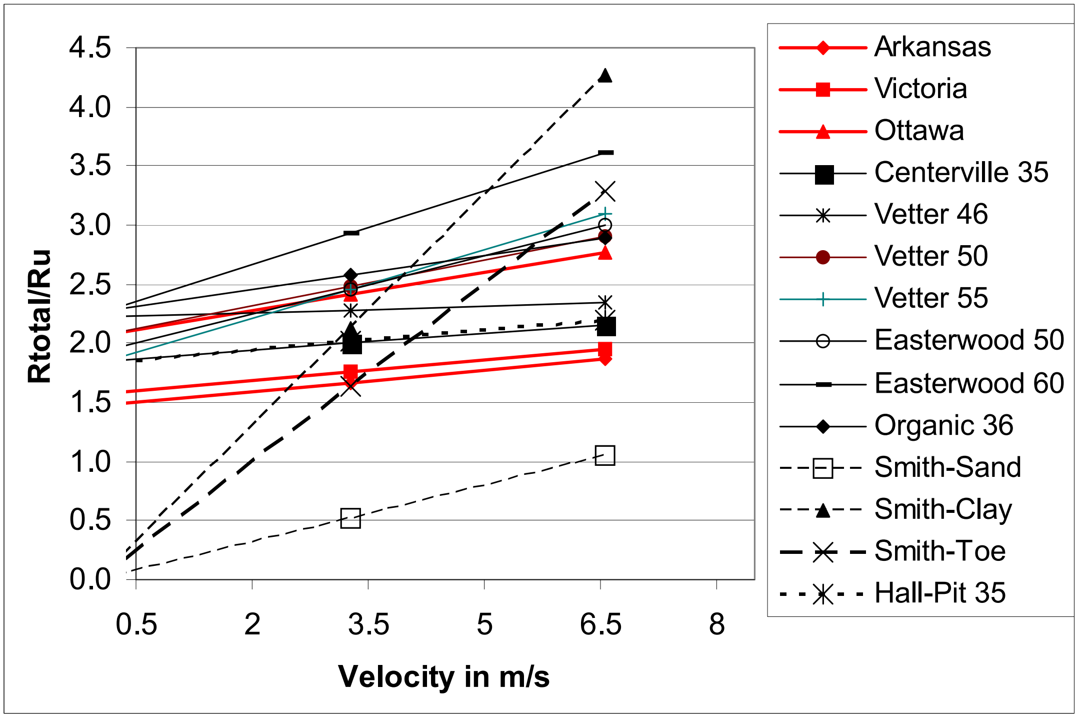

FIG. 1. Dynamic Resistance According to Gibson-Coyle and GRLWEAP

- The Smith toe damping values would be 2.4/3.3 = 0.72 s/m for the 3.3 m/s pile velocity in clay. For a 6.6 m/s velocity in clay the factor would be 0.42 s/m. For the sands, the corresponding values are 0.59 and 0.33 s/m, respectively. The usual assumption of J_{toe} = 0.5 s/m is therefore reasonable.
- For refusal situations with low pile toe velocities (e.g. 0.5 m/s), the pile toe damping factor may easily reach 2.0/0.5 = 4.0 s/m. Thus, contrary to expectations, if EOD is at refusal, the ratio f_{setup}/f_{dyn} may be less than 1.0 and one should not be too optimistic about additional pile capacity being available.
- The Gibson Coyle study also supports f_{dyn} values in the range of 1.5 to 3.

TABLE 2. R_{total}/R_u from Gibson-Coyle Using Straight Line Extrapolations

Soil Type (1)	Soil Source (2)	Water Ct. % (3)	f_{dyn}-0 (4)	f_{dyn}-3.3 (5)	f_{dyn}-6.6 (6)
Sand	Arkansas	Saturated	1.47	1.67	1.87
	Victoria	Saturated	1.57	1.76	1.94
	Ottawa	Saturated	2.05	2.41	2.77
Average			1.70	1.95	2.19
Clay	Centerville	35	1.84	2.00	2.16
	Vetter	46	2.22	2.28	2.35
	Vetter	50	2.06	2.48	2.91
	Vetter	55	1.82	2.46	3.10

TABLE 2. R_{total}/R_u from Gibson-Coyle Using Straight Line Extrapolations Cont.

Soil Type (1)	Soil Source (2)	Water Ct. % (3)	fdyn-0 (4)	fdyn-3.3 (5)	fdyn-6.6 (6)
	Easterwood	50	1.92	2.46	3.00
	Easterwood	60	2.25	2.93	3.61
	Organic	36	2.26	2.58	2.90
SandyClay	Hall-Pit	35	1.83	2.02	2.20
Average			2.02	2.40	2.78

Assessment of Dynamic Resistance

Modern methods of bearing capacity determination allow for a separation of velocity dependent resistance from the static components. For example, the Case Method determines the total resistance (RX0) and reduces it to a static capacity by subtracting a damping resistance. In CAPWAP, the total resistance is calculated at each pile segment as the sum of a velocity dependent and a displacement dependent quantity. CAPWAP allows for an output of the maximum total resistance at each segment. The sum of the individual segments total resistance peaks may be considered the total pile resistance, even though these peak values do not occur at the same time. It should be mentioned that the sum of the energies dissipated in the soil at each segment can also be calculated by CAPWAP. It was attempted to evaluate the ratio of damping energy to static resistance energy dissipated during the impact event. As it turns out, the energy ratios are very close to the ratio of peak total resistance to static resistance. For further evaluations of damping effects, working with resistance forces rather than energies appeared to be satisfactory.

METHODS OF PILE BEARING CAPACITY ESTIMATION

Formulas Used in the United States

Three formulas are currently in use or are recommended for use in the United States to estimate bearing capacity of a pile at end-of-drive.

1. The Engineering News formula (based on Wellington, 1893, but modified to yield an ultimate capacity),

$$R_u = FS \frac{(W_r h_r)}{[6(s + s_l)]} \tag{6}$$

   where FS is an assumed factor of safety, W_r and h_r are the ram weight and rated hammer stroke, s is the final set per blow and s_l is a loss term, usually .0025 m).

2. The Gates formula as adopted by the FHWA (Hannigan et al., 1996),

$$R_u = 7(W_r h_r)^{1/2} \log(BlCt) - 550 \tag{7}$$

where the result is in kN with W_r (N) and h_r (m) the ram's weight and actual drop height and BlCt the Blow Count (Blows/250 mm).

3. The measured energy formula (Broms et al., 1988; Paikowsky et al., 1992),

$$R_u = \frac{2EMX}{(s + DMX)} \tag{8}$$

where EMX and DMX are the measured maxima of transferred energy and pile top displacement, respectively.

The EN formula will not be considered in this study; for a variety of reasons it gave results which were too widely scattered to be of any use. The Gates formula requires an observation of the actual hammer stroke while the measured energy formula actually requires pile top instrumentation to measure energy plus observation of blow count. An alternate of that method is

4. the simplified measured energy formula R_u = 2EMX/DMX, where EMX and DMX are again the measured maxima of transferred energy and pile top displacement, respectively. This formula is equivalent to 3 in a refusal case and would yield higher capacities than Formula 3 when blow counts are low.

Case Method

Given measured pile top force $F_t(t)$ and velocity $v_t(t)$, (of a pile of area A, modulus E, length L, and material wavespeed C), the bearing capacity of the pile at the time of the test can be calculated from the so-called RMX (Rmax) Case Method formula as:

$$RMX = ½ \{(1-J_c)[F_t(t_x) + Zv_t(t_x)] + (1+J_c)[F_t(t_2) – Zv_t(t_2)]\} \tag{9}$$

where t_x is chosen such that RMX is maximum and $t_2 = t_x + 2L/c$. The pile impedance (EA/c) is Z. The dimensionless Case damping factor, J_c, is usually chosen between 0.4 and 1.0 with higher values more appropriate for finer grained soils. If the damping factor is set to zero, then the Case Method formula yields the total capacity:

$$R_{total} = RX0 = ½ \{[F_t(t_x) + Zv_t(t_x)] + [F_t(t_2) – Zv_t(t_2)]\} \tag{10}$$

Wave Equation Analysis

This approach much more accurately models hammer, pile and soil behavior. In fact, the soil resistance is assumed to consist of a static and a dynamic component. For the shaft, higher damping factors are normally chosen to represent finer grained soils. The wave equation calculates a so-called bearing graph which relates bearing capacity to blow count. Applied to end-of-driving blow counts, the wave equation generally underpredicts the pile bearing capacity. Applied to restrike situations, the wave equation approach generally yields rather unreliable results because of the uncertainty of the exact blow count (or set per blow) and hammer performance.

CAPWAP

CAPWAP is a signal matching program that separates static, displacement dependent resistance components, from those that appear to be dynamic or velocity dependent. The analysis is based on Smith's static soil resistance model combined with viscous damping. CAPWAP calculates an R_{total}-value (see examples below), however, this value is the maximum of simultaneously occurring resistance values and only approximates the sum of individual maximum R_{total} values occurring at different times along the pile.

Case Studies

As a first check on the performance of commonly employed dynamic capacity methods the somewhat expanded database of Likins, et al., 1996 was utilized to calculate EOD based capacity values. Consistent with the presentation of restrike results in the 1996 study, capacity results from different methods were divided by static load test capacities and these ratios evaluated for Mean and Coefficient of Variation for EOD based CAPWAP, Gates, Paikowsky and Simple E/D (Table 3). They are shown together with results listed in Likins, et al., 1996.

TABLE 3. Database Results For EOD and BOR for GRLWEAP and CAPWAP and EOD For Formulas

Method (1)	Number of Samples (2)	Mean (3)	CoV (4)	Remarks (5)
CAPWAP-BOR	99	0.92	0.22	1996
GRLWEAP-EOD	99	0.82	0.44	1996
GRLWEAP-BOR	99	1.22	0.35	1996
CAPWAP-EOD	106	0.72	0.47	2003
GATES	139	1.02	0.47	2003
PAIKOWSKY	98	1.02	0.39	2003
SIMPLE E/D	98	1.34	0.38	2003

Obviously, none of the EOD based methods (2003 results) resulted in a CoV less than 0.3 and therefore, must be used with high factors of safety if they are considered for construction control. It should be noted that this data base represents a large variety of soil and pile types; unfortunately, an attempt to improve the EOD based predictions for certain soil types was not successful.

In order to further study the relationship between end-of-drive resistance and set-up effects, additional data sets were investigated. Those additional cases were situations with substantial setup and at least two restrike tests after a substantial waiting time.

Case 1

In the early 1970s Professor Goble and his research team at Case Western Reserve University conducted a large number of static and dynamic pile tests at many different locations in many states under the sponsorship of the Ohio Department of Transportation and the Federal Highway Administration. The research reports frequently included the maximum simultaneously occurring resistance. For example, Goble et al., 1972 described the test of a closed ended pipe pile, driven into a clayey silt which would practically turn liquid during pile driving and cause the pile to slowly rebound after driving due to buoyancy of the empty pipe pile. The pile was statically load tested, failing at 970 kN.

Table 4 shows that at the end of restrike, the sum of static and dynamic resistance approaches the static pile capacity. It may be argued therefore that at the end-of-driving R_{total} was also equal to the long term static resistance. However, it also clearly demonstrates that both R_u and R_{dyn} are highly variable during the restrike with R_{total} to R_u ratios ranging from 1.5 and 3.6. Reversely, it may also be expected that the R_{total}/R_{dyn} ratio varies with time after pile installation. The variability of these resistance components must be attributed to continuously changing soil conditions in addition to variation of energy from blow to blow.

Is it a coincidence that R_{total} at the end of the restrike, i.e. in the EOD condition, approximately equaled the long term static capacity of the pile?

TABLE 4. Oneonta Load Test Pile Results

Restrike Blow Number (1)	R_u (kN) (2)	Max Damping R_{dyn} (kN) (3)	$R_{total}=R_u+R_{dyn}$ (kN) (4)	R_{total}/R_u (5)
Load test	**970**	-	-	-
1	**908**	414	1322	1.46
2	841	730	1571	1.87
4	712	957	1669	2.34
8	396	1028	1424	3.60
122	289	690	**979**	3.39

Case 2

In early 2002 dynamic pile tests were conducted on 27 m long H-piles with a remotely operated PAL-R Pile Driving Analyzer®. The test pile was driven by a Vulcan 506 hammer through soft silts, clays and into a silty sand. Driving was rather easy and the final blow count was 17 blows/0.25 m. The design load was 445 kN and, with a factor of safety of 2.25 for construction control by dynamic tests an ultimate capacity of 1000 kN was needed. Added to that, an estimated downdrag of 547 kN yielded a total required capacity of 1547 kN. The local geotechnical engineer cautioned that the end-of-driving (EOD) capacity would be low and that the soils would set up very slowly and required a minimum undisturbed waiting time of 7 days for full soil setup. The contractor was impatient and performed restrike tests after both 2 hours and 64 hours. However, the required capacity could not be demonstrated at that time. After an additional 7 days of waiting, the beginning of restrike testing (BOR) indicated a blow count of 72 blows/0.25 m and sufficient capacity according to CAPWAP, which was in agreement with the expected capacities based on static methods.

It is instructive to check the bearing graph obtained for this case both with damping factors of 0.0 and for the 0.65 and 0.5 s/m damping factors at the shaft and toe that would normally be used for a cohesive soil (Figure 2). Applying the observed blow counts at EOD and during the last restrike to these bearing graphs yields the bearing capacity results of Table 5. The associated CAPWAP results are also shown in this table.

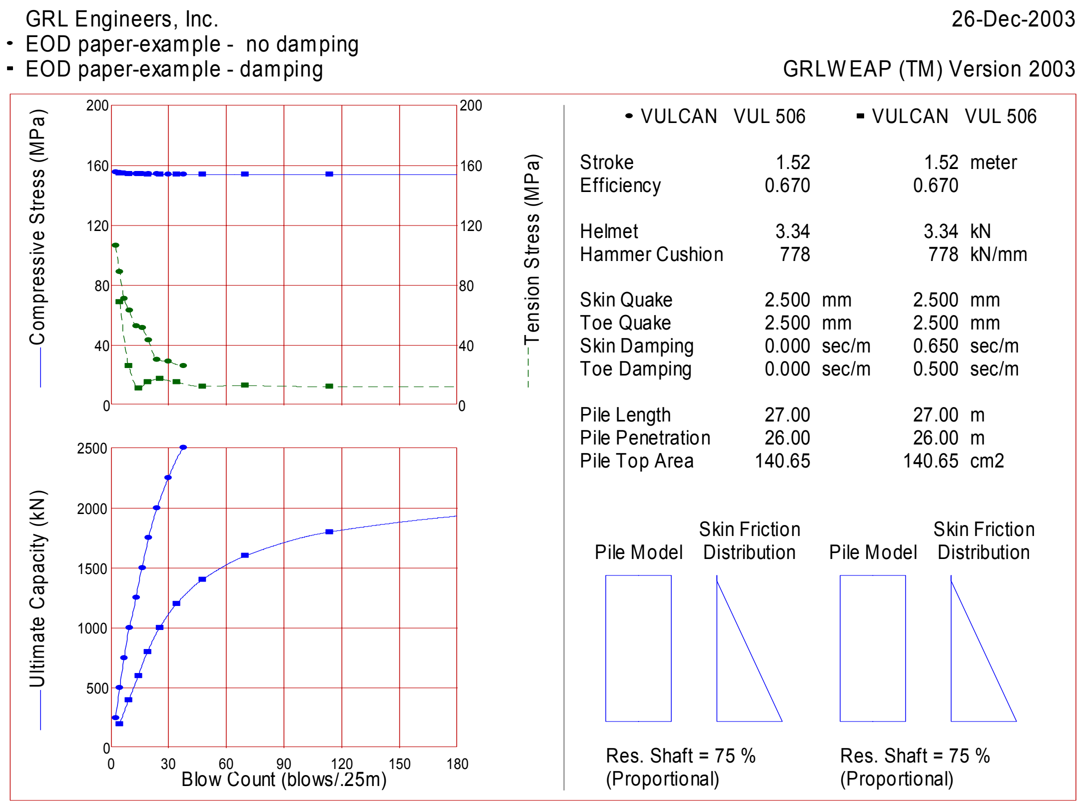

FIG. 2. Bearing Graphs For Case 2 Pile

TABLE 5. Case Study 2 Results; Required Capacity 1547 kN

(1)	End-of-Drive (2)	9–Day Restrike (3)
Blow Count (Blows/0.25 m)	17	72
GRLWEAP capacity - No damping (kN)	**1550**	3050
GRLWEAP capacity - Normal damping (kN)	700	**1580**
CAPWAP capacity (kN)	950	**1535**

Is it just a coincidence that the zero-damping, or total resistance approach applied to the end-of-driving blow count yields almost exactly the capacity that both GRLWEAP and CAPWAP yield after a waiting period of 9 days?

Cases 3 Through 23

Case 1 and 2 showed a striking similarity between the EOD total resistance values and the restrike static resistance components. It was therefore of interest to check whether or not these relationships could be confirmed for other cases where high setup was found to develop after pile installation. The data of Cases 3 through 23 were provided by and analyzed by GRL for the Louisiana Department of Transportation and Development (LA DOTD) between 2001 and 2003. The soils in many parts of Louisiana are known for developing a large setup capacity during often rather long waiting periods following pile installation. For this reason, the LA DOTD performed dynamic tests during the end-of-driving, after 24 hours, and then again after typically 14 days, sometimes after having performed a static load test. The value of this data is therefore the consistency and completeness of its testing efforts in soils expected to exhibit substantial setup gains. Table 6 shows both the properties of the previously discussed Case 2 and those of the LA DOTD cases. In the latter 21 cases the soils generally consisted of clays with some sand and silt. While in many instances static load test results were available, the studies presented here depend on the CAPWAP capacities for consistency of calculated capacities at EOD, and restrikes R1 and R2.

As a check on the hypothesis that total resistance in a fully remolded soil equals static capacity after a long setup period, the following calculations were made and listed in Table 7 together with EOD and R2 (second restrike) blow counts and available static load test results.

1. Formula based on EOD blow count and stroke (Gates ED)
2. Formula based on EMX, DMX and blow count at EOD (Paikowsky-ED)
3. Simplified EMX, DMX formula
4. Case Method without damping reduction at EOD (RX0-ED)
5. CAPWAP total resistance at EOD, i.e. R_{total} (CW-D-ED)
6. CAPWAP R_u at EOD (CW-S-ED)
7. CAPWAP R_u at R1 (CW-S-R1)
8. CAPWAP R_u at R2 (CW-S-R2); adjusted for high blow counts as described below

GRLWEAP and Engineering News formula depend on a reasonably accurate energy input and since hammer energy was varied to reduce tension stresses, these methods were not applicable.

While the GRL data base was restricted to cases where the blow counts are below refusal, such restriction would have limited the usefulness of the present study. Thus, instead of eliminating cases with excessive blow counts, the CAPWAP – R2 capacity was adjusted for very high blow counts by an "underprediction factor", which

TABLE 6. Properties of Cases 2 Through 23

Case No. (1)	Pile Type Nominal Size (2)	Pile Length (m) (3)	Penetration (m) (4)	Hammer (5)	Soil Shaft[a] (6)	Soil Toe[a] (7)
2	HP 12x74	27	26	V 560	sft Silt, Clay lrs	si Sand
3	Pipe 24x1/2"	33.5	16.2	D 46-32	sa Clay	si Sand
4	PSC 24x24"	20.7	11.9	D 46-32	sa si Clay	sa Clay
5	PSC 24x24"	20.7	14.3	D 46-32	Clay, Silt lrs	si Sand
6	PSC 24x24"	18.3	10.2	D 46-32	si Clay	Sand w/ si Clay
7	PSC 30x30"	28.0	23.8	V 025	Clay	Clay w/ nodules
8	PSC 16x16"	23.5	9.9	ICE 60S	Clay, Sand, Silt	Sand w/ Clay
9	PSC 16x16"	16.8	11.0	ICE 60S	Clay, si Clay	Sa Clay
10	PSC 16x16"	18.3	8.2	ICE 60S	sa Clay	cl Sand
11	PSC 14x14"	13.0	13.1	ICE 60S	cl Silt, si Clay	Clay
12	PSC 16x16"	28.0	71.0	D 16-32	Unknown	Unknown
13	PSC 14x14"	18.0	12.0	ICE 60S	Clay, cl Sand	cl Sand
14	PSC 14x14"	18.0	11.5	ICE 60S	sa Clay, Sand, Clay	cl Sand
15	PSC 14x14"	18.0	13.2	ICE 60S	si Clay, Sand	Si Clay w/ sand lrs
16	PSC 16x16"	21.3	7.0	ICE 60S	Clay	cl fine Sand
17	Pipe 24x1/2"	48.4	17.4	IHC S90	sa Clay	Clay w/ sand lrs
18	PSC 24x24"	23.8	11.0	ICE 120	Silt + Sand	Sand + Gravel
19	PSC 16x16"	22.9	30.3	Fairch. 32	Ln Clay	LnClay, trSand
20	PSC 14x14"	16.8	14.2	ICE 60S	Clay	Si Clay
21	PSC 14x14"	16.8	19.1	ICE 60S	Clay	Si Clay
22	PSC 24x24"	26.0	15.5	D 30-32	Clay	Clay
23	PSC 24x24"	26.0	17.1	D 30-32	Clay	Clay

[a] sft ... soft; lrs ... layers; si ... silty; sa ... sandy; cl ... clayey; ln ... lean; tr ... trace

depends on the blow count (BC). In other words, the capacity calculated by CAPWAP was increased between blow counts of 120 blows/0.3m and 480 blows/0.3m by a factor, linearly increasing between 1.0 and 1.2 and then remaining at 1.2.

$$\text{f-adjust} = \begin{cases} 1.0 \text{ for BC} < 120/0.3\text{m}; \\ 1.0 + 0.2*(\text{BC}-120)/360) \\ 1.2 \text{ for BC} > 480/0.3\text{ m} \end{cases}$$

This data set specific adjustment reduced the CoV of the CW-S-R2/Load Test ratios from 0.24 to 0.22.

The first four methods rely only on EOD information; Methods 4 and 5 yield a total resistance which, like the formula methods 1, 2 and 3 are to be applied to EOD situations. The calculated results are shown in Table 7 together with available static load test (LT) values.

The results of Table 7 were reduced to average values of CAPWAP static and total resistance. The average setup factors as well as the dynamic resistance factors were calculated as averages of the individual factors and entered in Table 8. Obviously, the ratios of the averages would be different (lower).

TABLE 7. Calculated Capacity Results

Case No. (1)	ED-Blow Count (Bl/0.3m) (2)	R2-Blow Count (Bl/0.3m) (3)	Gates-ED (kN) (4)	Paikowsky EMX-ED (kN) (5)	Simple ED 2EMX/DMX (kN) (6)	RX0-ED (kN) (7)	CW-D-ED (kN) (8)	CW-S-ED (kN) (9)	CW-S-R1 (kN) (10)	Adj. CW-S-R2 (kN) (11)	Load Test (kN) (12)
1	21	88	1277	1428	2430	1500	1838	952	1113	1535	
2	10	48	1477	1816	3315	2719	2367	1228	2198	2198	
3a	26	108	2452	2060	3364	2519	2292	1335	1958	1958	1344
3b	25	132	1869	1028	1384	1139	886	534	1108	1568	
3c	68	436	3062	2381	2848	2118	2519	1647	2025	2485	
4	20	192	2176	2585	4116	2795	3097	1420	2959	4050	3204
5	29	48	1798	1420	2336	1526	1807	1019	1264	1647	
6	20	58	1371	886	1433	668	708	352	846	1046	
7	36	400	1918	1437	2256	948	1233	846	1308	1774	
8	9	100	730	280	561	383	378	227	890	912	997
9	7	200	498	312	627	369	481	178	579	781	961
10a	11	204	761	303	574	338	352	245	1153	1584	>1202
10b	19	216	1282	641	1148	685	899	441	1224	1523	>1202
10c	7	84	556	383	743	516	476	223	935	1219	>1202
11	522	277	4143	1691	1731	1362	1393	1202	1602	2613	
12	93	1000	3418	4779	5465	6016	8304	3151	5002	7204	
13	36	120	4050	2852	4499	3022	3831	1424	1469	2661	
14	27	180	1424	1126	1802	1148	1393	748	1308	1508	
15b	10	60	828	503	952	583	627	441	1001	1219	997
15c	40	300	1602	748	1246	792	894	605	1357	2340	>1210
16b	190	804	4058	3044	3778	2821	3453	1420	3066	2296	2648

TABLE 8. Average Results of Cases 2 Through 23

(1)	Units (2)	EoD (3)	Rest. 1 (4)	Rest. 2 (5)
Wait time after EOD	days	0	1.2	14.2
CAPWAP-R_u	kN	934	1562	1861
CAPWAP-R_{total}	kN	1857	3129	4072
$f_{setup} = R_{u\text{-}BOR}/R_{u\text{-}EOD}$		----	2.13	2.62
$f_{dyn} = R_{total}/R_u$		1.89	2.08	2.25

Surprisingly, the capacity increase was rapid in the first day (113% gain) with only an additional 23% of the R1 capacity gained during the next 13 days. The A factor (Skov et al.) of the average pile studied would therefore be only 0.19 (varying between 0 and 0.5) or what the authors expected for sand. More important, in the context of this paper, is the fact that the ratio of total to static resistance at EOD is 1.9 while the setup factor after roughly two weeks is 2.6. Both the average setup factor and the average dynamic resistance ratio are in line with expectations. On the average, therefore, the EOD dynamic resistance information may predict, albeit conservatively, the long term performance of the pile. Unfortunately, the devil is in the details. Figure 3 shows a plot of the individual dynamic resistance factors versus the setup factors and this plot does not suggest that a reliable correlation exists

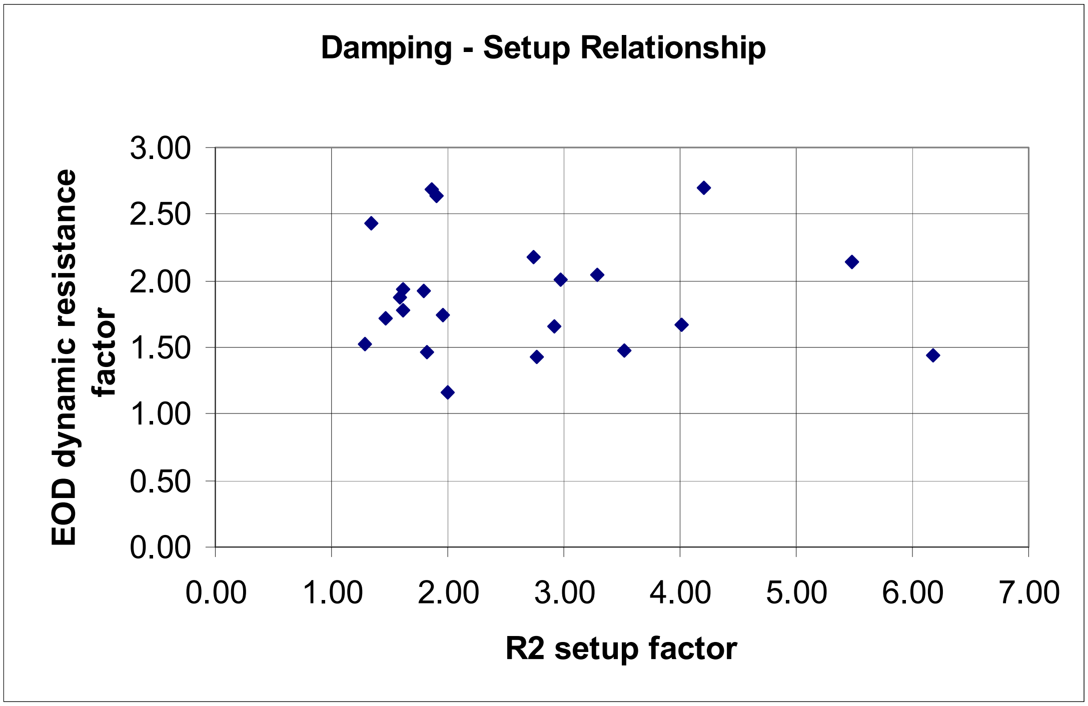

FIG. 3. The Non-Existent Correlation between Setup and Dynamic Resistance at EOD

between dynamic resistance and soil setup. Indeed, while the R2 setup factors range between 1.3 and 6.2, the EOD dynamic resistance factors are much more narrowly grouped between 1.2 and 2.7.

Additional results were obtained by evaluation of the formulas discussed above. Also, the static EOD resistance was adjusted based on the following consideration. The pile driving process causes greater reductions in static resistance for greater movements under each hammer blow. In other words, excessive hammer energies tend to cause greater damage to the soil structure in the interface than lower energies and therefore lower pile sets per blow. It must therefore be expected that EOD determined static resistance values are subjected to a higher soil setup gain if the blow count was very low than if the blow count is higher. The adjustment factor for EOD static CAPWAP resistance was therefore set to

$$f_{setup} = \begin{cases} 3.0 \text{ to } 2.2 \text{ between Blow counts of 0 to 24 Bl/0.3 m} \\ 2.2 \text{ for Blow Count} > 24 \text{ Bl/0.3 m} \end{cases}$$

Obviously, this adjustment is very data set specific and cannot be applied to any other data set, particularly if it does not include soils with similarly high setup factors. In other words, the 2.2 factor which was selected for all blow counts above 24 blows per 0.3 m expresses an average setup factor for the data set investigated. It should not be applied to other geomaterials without extensive review. Also, the increase of the adjustment factor for low blow counts, which causes at most a 36% increase of predicted capacity for the theoretical blow count of zero, is data set specific.

All calculated results were divided by the blow count adjusted (for high blow counts) CAPWAP R2 static resistance which, for this study, is considered the long term pile capacity. The ratios were then evaluated for average and CoV and entered in Table 9. The CoV values mirror those in Table 3 for like methods.

TABLE 9. Summary of Normalized Capacity Results

Pile (1)	Gates-ED (2)	Paikowsky ED (3)	Simple ED (4)	RX0-ED (5)	CW-D-ED (6)	CW-S-ED (7)	Adjusted CW-S-ED (8)	CW-S R1 (9)
Average	0.982	0.688	1.126	0.728	0.823	0.433	1.001	0.803
CoV	0.406	0.428	0.390	0.428	0.444	0.385	0.350	0.211

DISCUSSION OF RESULTS

Without doubt, occasionally setup factors and dynamic resistance factors are similar and the EOD total resistance is then equal to the long term static resistance. However, while averages may work out reasonably, the individual results display significant scatter. For example, the EOD based formulas Modified Gates, Paikowsky, RX0, and CW-D have CoVs between 0.40 and 0.44. Surprisingly, the very simplified 2EMX/DMX formula fares slightly better, i.e. it has a lower CoV, than any of the other formula or no-damping-reduction methods. Only the CAPWAP static EOD resistance is comparable in COV to the simple ED formula. In fact the factored CW-ED shows an additional slight improvement over the ED formula.

Not too surprisingly, the first restrike CAPWAP to second restrike CAPWAP ratio has the best CoV. For this data set, the 24 hour restrike capacity should be increased by 25% for a prediction of long term capacity. With this correction, the 24 hour restrike would probably establish a reasonably reliable and economical testing means. However, sufficient local experience, preferably a local data base, would be needed. On most projects, a 24 hour restrike is still possible and practical.

CONCLUSIONS

Experience with results from dynamic tests performed at the end of pile installation, the review of available data bases, and the analysis of a data set exhibiting high setup behavior support the following conclusions.

- Capacity determination involving restrike blow counts often yield unreliable results because of the variability of hammer energy and soil resistance and therefore set per blow.

- Dynamic resistance factors vary within relatively narrow bounds of 1.5 to 3.0. The dynamic resistance factors determined by CAPWAP analysis agreed well with those expected from Gibson-Coyle tests.

- Setup factors can vary widely between values of less than 1 (relaxation) and probably 10 or more. The ratio of setup to dynamic resistance factor therefore varies widely. However, the average capacity predicted from end-of-drive data may reasonably well predict long term capacity of the average of a large data set.

- A variety of reasons exist for soil setup including the energy imparted by the hammer to the driven pile. It would therefore be desirable to limit driving energies such that blow counts in sensitive soils exceed 24 blows/0.3 m.

- Most of the cases presented in this paper represented soils with high setup potential. General conclusions cannot be formulated from the data analysis results presented in this paper.

- For an economical and reliable capacity test, a 24 hour restrike seems to be much better than any other EOD based method or formula.

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