Analyzing and interpreting dynamic measurements taken during vibratory pile driving

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ABSTRACT: Dynamic force and velocity measurements taken on impact driven piles during installation have become routine during the past 40 years. However, these relatively easily measured quantities have only infrequently been recorded during vibratory pile driving, which has seen increased popularity in the deep foundation construction industry, because of potentially great speed of installation and therefore good economy. Unfortunately, vibratory pile driving has a strong effect on the soil resistance which makes calculations of pile bearing capacity from dynamic measurements difficult. A lack of meaningful measurement and analysis methods for different vibratory pile driving hammers and a variety of soil and pile types add to these challenges.

The paper describes and demonstrates by example analysis procedures and data interpretation methods of vibratory pile driving force and velocity measurements. The paper also includes recommendations for (a) an analytical procedure for data analysis including input parameters for the wave equation approach, (b) a dynamic testing procedure during vibratory pile driving and (c) the minimum information necessary for the development of a meaningful data base. The example includes a correlation attempt between capacity determined by CAPWAP from impact records and from vibratory measurement analyses. A measured power approach shows the most promising agreement for the vibratory data evaluation.

1 INTRODUCTION

While not as extensive as for impact driven piles, the literature on vibratory pile driving includes a number of recent publications dealing with background information and experimental results. Important references are included in Holeyman (2002) and Viking (2002) among others. This work was prompted by the need to improve both the economy of this deep foundation installation technique and its acceptance by the geotechnical profession. However, much of the literature concerns itself with the vibratory installation of piles in sands where it is often surprisingly economical, while driving in cohesive soils is more of a challenge. The two questions to be answered are driveability, i.e. how large a hammer is needed to advance the pile with acceptable stresses in a given soil type and secondly bearing capacity assessment, i.e., what is the bearing capacity of a pile installed by vibratory hammer.

Driveability analyses are conveniently done by a wave equation analysis (Rausche, 2002). Dynamic data taken during vibratory driving also has been reported. Likins et al. (1992), for example, showed good agreement between resistance from vibratory and impact testing. However, such comparisons are rare and because of the lack of a simple and reliable analysis procedure of these measurements, results have been relatively scarce. This paper describes an additional effort to develop a comprehensive analysis method.

2 WAVE EQUATION MODELING

The analysis of vibratory pile driving can be done conveniently by wave equation analysis, even though for most common situations, the frequencies of the hammers are well below the lowest resonant frequency of the pile, which means that there is actually no advantage to a wave propagation analysis compared to a rigid body analysis. However, the wave equation analysis (eg., GRLWEAP, Pile Dynamics 2010) provides a numerical representation of hammer, pile and soil including reasonable model parameters, although the selection of the proper soil resistance constants is still a challenge. Also, this type of analysis is easily understood by the pile dynamic analyst and, for modern high frequency hammers and/or very long piles with standard low frequency hammers, the analysis will faithfully reproduce the dynamic forces and motions of the pile. A shortcoming, however, is a lack of soil mass representation. For that reason, low frequency resonance effects occurring in the soil surrounding the pile cannot be realistically reproduced by the analysis.

The wave equation model of the pile is straight forward, representing mass and stiffness of the components of hammer and pile. The most difficult part of the wave equation model concerns the soil resistance which for both impact and vibratory analyses consists of a static, elasto-plastic part which is a function of pile displacement and a dynamic or damping resistance which is dependent on the pile velocity. The following soil resistance parameters have to be considered.

(a) The ultimate soil resistance, Rui, acting at every pile segment, which may be calculated by static geotechnical methods as the so-called Long Term Static Resistance (LTSR). However, as is known from impact driving, the static resistance acting on a pile during driving may be quite different. It is called the Static Resistance to Driving, SRD, and for the shaft

$$SRD_{shaft} = LTSR_{shaft}*fgl \qquad (1)$$

With fgl being a Gain/Loss factor which in the case of soil setup is less than unity (fgl $\leq$1). The Gain/Loss factor may be thought of as the inverse of the soil setup factor, fs, which is greater than 1 in most instances (fs$\geq$1). For impact driving the gain/loss factor is typically lower for clays than for sands. In fact, fgl = 0.1 (corresponding to fs = 10) has been frequently observed for clays in marine environments. On the other hand, sands generally do not lose much of their shaft resistance during impact driving and an fgl = fs = 1 is frequently appropriate. For vibratory driving the opposite may be true. While sands, particularly when they are submerged, are expected to practically liquefy and therefore lose much of their shear strength due to the vibratory pile motion (thus fgl may be 0.1 and fs =10), for clays the opposite might happen: the clay particles stick to and move with the pile and therefore the shaft soil resistance remains at a high value; an fgl = fs = 1 is then appropriate.

For the toe,

$$SRD_{toe} = LTSR_{toe}*fr \qquad (2)$$

with fr being a relaxation factor (fr$\geq$1). This factor fr covers the relatively rare circumstances where the end bearing is higher during driving than after a waiting period. This may happen for example in a vibratory driving situation with very dense silty fine sands, which could dilate during the dynamic event creating negative porewater pressures and temporarily high end bearing values. After driving is complete this effect may dissipate within a few minutes or hours. In any event, it is necessary to calculate LTSR based on soil borings with an allowance for the type of pile installation and then apply appropriate fgl and fr factors for an estimate of SRD. Reversely, if SRD has been assessed from driving records, LTSR has to be calculated by division with fgl and fr.

(b) The stiffness of the elasto-plastic static soil resistance, ks, is normally expressed in terms of Ru and quake q:

$$ks = Ru/q \qquad (3)$$

The quake is the pile displacement where the soil changes from elastic to plastic behavior. For the shaft during impact driving the quake is usually assumed to be 2.5 mm; this is reasonable since a quick hammer blow does not give time for the soil surrounding the pile soil interface to move under the effect of the resistance forces. For the toe quake a larger value has to be assumed in the case of a displacement pile (concrete pile, closed ended pipe). Considering that sands lose much of their static resistance during vibratory driving, the standard value of 2.5 mm is adequate for vibratory analyses. However, one of the problems of clays is that, as a function of hammer frequency, the soil may either have a low stiffness or it does nor shear and, therefore, moves with the pile which leads to a highly energy consuming soil resistance. This can be approximated with a larger than normal quake. Since vibratory hammers have limited amplitude of vibration, such a condition can quickly lead to refusal. Similarly, for the pile toe in cohesive soils it may be prudent to choose a larger quake than typical for impact driving.

Changing either quakes or SRD changes the soil stiffness and therefore the resonant frequency of the hammer/pile/soil system. A surprising consequence is that higher resistance values may produce easier driving conditions.

(c) Generally it is agreed that the faster the pile moves the higher the damping or dynamic resistance. Studies have shown that damping is not linear and is relatively higher during low velocity pile movements. Since in most common situations the vibratory velocities are lower than for impact driving, higher damping factors (say double the impact recommendations) may be appropriate. The most commonly used damping model for impact driven piles is the Smith model which calculates the damping resistance as the product of Smith damping factor times temporary static resistance times pile velocity [Rd(t)=Js Rs(t)vp(t)]. The multiplication with the temporary resistance, being zero every time the static resistance reverses direction (it happens twice per cycle), strongly reduces the damping effect; replacing the temporary resistance term with the ultimate resistance [Rd(t)=Js Ru vp(t)] makes the approach truly linear (Smith-viscous) and more appropriate for vibratory analyses. Note that the dynamic resistance component is directed against the direction of pile velocity; should the velocity be strongly out of phase with the displacement and, thus with the static resistance, damping may actually decrease the driving resistance.

3 WAVE EQUATION OPTIONS

3.1 Bearing Graph

An impact driving simulation produces a relationship between bearing capacity (actually SRD) and driving resistance or blow count (or the inverse of set per blow). Similarly, a dynamic analysis of a vibratory driven pile produces a relationship between SRD and driving resistance or time per unit penetration (i.e., the inverse of penetration speed). While the bearing graph has the shape of a hyperbola for impact driven piles, it is possible for vibratory driven piles that different capacities lead to nearly the same unit penetration time (the inverse of penetration speed). The capacity vs. penetration time curve then assumes a more complex shape.

3.2 Driveability

Predicting the driveability, i.e., whether or not it is possible to advance the pile with a vibratory hammer at a certain depth appears to be more successful than the bearing capacity determination. The analysis is practically a sequence of bearing graph calculations for increasing depths of penetration. It requires an accurate assessment of the LTSR, fgl and fr for a realistic assessment of the SRD. Obviously, this is not something that is easily accomplished based on crude soil investigation. Fortunately, the analysis is relatively insensitive to errors in shaft SRD although the magnitude of the quake and the end bearing can have a major effect. Also, as the system approaches refusal, small changes in resistance can have a very strong effect on penetration speed.

4 FIELD TESTING

For improved predictability of the pile driving process as well as for determining in real time the actual hammer performance, pile forces, stresses and SRD, measurements have to be taken and then analyzed.

4.1 Measurements

Conveniently, measurements of force and acceleration can be taken with a Pile Driving Analyzer®. A few details have to be observed. The vibratory pile driving produces a steady state, continuous motion event while impact driving produces transient records separated by a time period of little or no motion. Thus, prior to each impact the measured signals can be balanced (set to zero). For vibratory driving this can only be done prior to commencement of driving or during a driving interruption. Also the driving is continuous and a decision has to be made as to how long an individual record should be taken. Obviously, taking records at certain known penetrations helps assess penetration speed. Each individual record length and sampling frequency should be chosen under consideration of the hammer frequency. For sufficient resolution of important record features the sampling frequency should be the greater of 1000 sps or10 times the pile frequency or the hammer frequency. For example, for a 10 m long pile (fundamental frequency 250 Hz) and a hammer with 50 Hz maximum frequency, sampling 2500 sps should be adequate. Also the record duration should be such that at least 5 cycles are recorded which, for a 20 Hz hammer, would require a record length of at least 250 ms. On the other hand the records should not be too long so that, for simplification of the analysis, the assumptions can be made that the penetration speed is constant during the record. Records of ½ second duration are usually adequate.

For low frequency hammers (say less than 30 Hz) acceleration levels are rather low. Depending on the mass of the pile, as a Rule of Thumb the acceleration level may be estimated to be less than 20% of the square of the hammer frequency (Note: for a perfect sine wave, acceleration amplitude is displacement amplitude times the square of the frequency; the Rule of Thumb is based on the assumption of a 50 mm amplitude which would be much more than what is common and, therefore, includes a margin of safety). Thus for the typical hammer with less than 30 Hz frequency the expected acceleration level would be below 200 g's. On the other hand, a high frequency or sonic hammer may produce acceleration levels comparable to those of an un-cushioned impact hammer. Thus for good resolution of acceleration measurements with low frequency hammers, records should be taken with a

10 times more sensitive accelerometer than what is often used during impact driving.

Strain and stress levels in vibratory piles are usually much lower than in impact driven piles. Critical stresses can, however, occur at the clamp and as for impact driven piles, it is important to mount the transducers at a distance away from the clamp that assures uniform strain measurements. For large pipes, four instead of only two sensors (always on opposite pile sides) may produce more accurate results and allow for a determination of the stress uniformity in the cross section. For sheet piles, particularly when double sheets are driven, four sensors are a must because of the lack of double symmetry.

Recording the speed of pile penetration is conveniently done by noting the penetration and time (in seconds) when records are taken. The PDA automatically records the time of each record. If records are continuously taken, then the penetrations at the beginning of some records should be noted. Alternatively, the measurement engineer may take records at certain known penetrations. A very simple way of recording the penetration speed is by video camera, but that requires an additional analysis effort.

Pile properties such as total length, length below sensors, cross sectional area and wave speed are to be input as for any other pile driving job. Other quantities to be recorded and/or data collected would include hammer model and serial number, confirming its total weight, power aggregate details and hammer operational quantities such as hydraulic oil pressure, flow and expected frequency. Important is also to record a crowd force (if any) or alternatively, whether the crane pulled up on the hammer top.

4.2 Analysis of records

Analysis of the vibratory data is generally done in closed form and involves the following steps.

1. Determination of the average penetration speed, v_a, for the record (this is from the time elapsed for a certain unit pile penetration).
2. Determination of the frequency, f, by either Fourier analysis or time record zero crossings.
3. Determination of the beginning and end of the record. Since the data of an individual record begins at some arbitrary time during the vibratory cycle, it is recommended to choose a record length which is an integer number of cycles. A minimum of 5 cycles should be contained in a record to allow for a check on consistency.
4. Calculation of force from strain.
5. Determination or check on force offset or static component, which should be equal to crowd force plus hammer weight plus weight of pile above sensors. This produces the corrected force F(t).
6. Calculation of stress maximum and minimum at the gage location (both from individual and average sensors).
7. Zeroing of the acceleration signal, i.e. assuming that there is no increase of the penetration speed during the record time (penetration speed is the time average of the pile velocity). This produces the acceleration a(t), from which the maximum acceleration amplitude, a_{max}, can be determined.
8. Integrating the acceleration to obtain velocity and adding an integration constant to the velocity to make the average velocity equal to the observed penetration speed. This produces the corrected velocity v(t), from which the maximum velocity amplitude, v_{max}, can be determined.
9. Integrating the corrected velocity to produce the corrected displacement. This produces the displacement u(t), from which the displacement amplitude (maximum per cycle), d_{max}, can be determined.
10. Integrating the product of force and velocity to determine the energy transferred to the pile during the record duration. This produces the transferred energy $E_t(t)$.
11. Dividing the transferred energy by the corresponding record time which yields the average power transfer, P_a.
12. Searching the energy-time record for the maximum slope representing the maximum instantaneous power transfer, P_{max}.
13. With M_p being the mass of the pile below sensors, calculating the positive and negative maximum soil resistance values from:

    a. $R_{rt}(t) = F(t) – M_p\, a(t)$ (4)

    b. and optionally reducing this resistance to the static value by subtracting damping based on a viscous damping factor J:

    c. $R_{rs}(t) = R_{rt}(t) – J\, v(t)$ (5)

14. Calculating the standard Case Method total RCt(t) and static resistance RCs(t) and finding their minima and maxima.

    a. $RC_t(t) = ½[F(t)+F(t+2L/c)+(M_p\ c/L)(v(t) – v(t + 2L/c)]$ (6)

    b. $RC_s(t) = RCt(t)–J[F(t)+(M_p\ c/L)v(t)–RC_t(t)]$ (7)

15. Calculating the soil resistance (SRD) by a power approach which is related to the Hiley formula (analogous to Smart 1969 as discussed by Rausche, 2002). With P_a the average power

transfer, v_a the average penetration speed and d_{max} the maximum displacement amplitude, the following formula may be evaluated.

a. $RP_a = P_a / (v_a + (f)\, d_{max})$ (8)

where P_a, v_a, f and d_{max} are the quantities defined in items 11, 1, 2 and 9, respectively.

Having performed the closed form analyses, a refined wave equation may be done for finding among other parameters the fgl and fr factors.

5 EXAMPLE

The installation of the first 20+ m section of 60+ m long bearing piles for I-90 Innerbelt Bridge in Cleveland, OH was installed by a vibratory hammer. Additional sections were then spliced to the steel H-pile and driven with a large diesel hammer. The example is of interest as it demonstrates the installation of a heavy H-pile section into a hard clay layer which is demanding both for the pile driver and the driveability analyst. Also, PDA impact records, collected when a neighboring pile was impact driven beginning at 18.6 m depth, several days after having been vibratory driven to that depth, were available for CAPWAP® analysis, thus allowing for a comparison of calculated capacities.

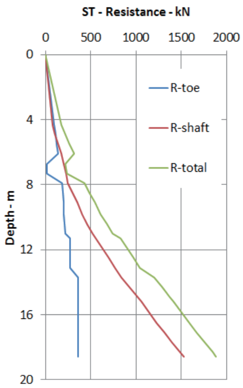
The first 20 m pile section was vibratory hammer driven to a depth of approximately 18.7 m, first through loose sands and then hard clays. A generalized soil profile is shown in Figure 1, together with the SPT-N60 and q_u values (from a handheld penetrometer), where available. GRLWEAP's (see Pile Dynamics, 2010) Soil Type (ST), N and q_u-value (SA) (also documented in Rausche et al., 2000) and soil density/undrained shear strength (API) based static analysis methods were employed, yielding for a depth of 18.6 m the capacities shown in Table 1. Note that both unplugged and plugged end bearing values were calculated and that the shaft resistance was assumed to only act on 4 sides (the perimeter was taken as 1.84 m).

The vibratory hammer was an ICE model 66-80 with 26.7 Hz (1600 rpm) rated frequency and 749 N-m (6620 lb-inch) eccentric moment. This moment and frequency correspond to a rated centrifugal force of 2150 kN (483 kips). The pile was assembled from 20 m long HP 460x304 (HP 18x204) sections, having 390 cm^2 (60 inch^2) cross sectional area (Figure 2).

A driveability analysis was conducted, first with the unplugged ST soil resistance (SRD) and with dynamic soil resistance parameters as per GRLWEAP recommendations. A summary of these parameters is shown in Table 2. The resulting calculated penetration times were much lower than measured (Figure 3) and would have normally led to the non-conservative conclusion that the pile installation should be relatively easy.

A second analysis assuming that the pile would plug in the hard clay produced a good agreement with the measured driving resistance in that part of the penetration. Note that the quake for the plugged analysis was left at the 5 mm value which is strictly speaking only appropriate for the unplugged analysis.

Soil Properties



Soil Description	Depth (m)	N-60	qu (kPa)
Loose Sand	0.9	4	
Loose Sand and Silt	2.7	6	
med dense silty Sand	6.4	17	
Medium stiff silty Clay	7.9	9	240
Medium dense silty Sand	11.0	27	
Stiff to hard silty Clay	13.7	15	250
Stiff to hard silty Clay	16.8	26	480
Stiff to hard silty Clay	19.8		480

Figure 1. Soil properties and ST pile capacity vs depth.

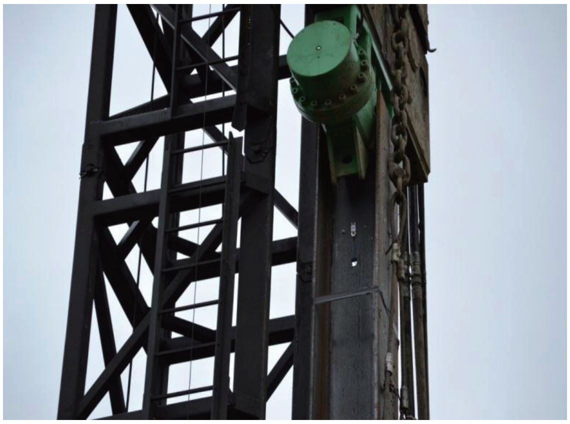


Figure 2. The ICE 66-80 hammer on the instrumented HP 46x304 pile.

It should also be mentioned that the plugged analyses with either the ST or the API resistance resulted in refusal driving resistances in the lower clay.

Table 1. Statically calculated capacities in kN (see Pile Dynamics, 2010 and Rausche et al., 2000).

	Soil type based (ST)		N and qu-value based (SA)		Density and Su based (API)	
	Unplugged	Plugged	Unplugged	Plugged	Unplugged	Plugged
SRD	1360	1650	860	1230	1980	2350
LTSR	1590	1890	1020	1390	2170	2630

Table 2. Summary of assumed dynamic analysis parameters for the driveability analysis.

	Sand	Sand and Silt	silty Sand	silty Clay
Shaft Quakes (mm)	2.5	2.5	2.5	5.0
Toe Quakes (mm)	2.5	2.5	2.5	5.0
Shaft Damping (s/m)	0.3	0.3	0.5	1.3
Toe Damping (s/m)	1.0	1.0	1.0	1.0
Setup Factor	5.0	2.0	2.0	1.0

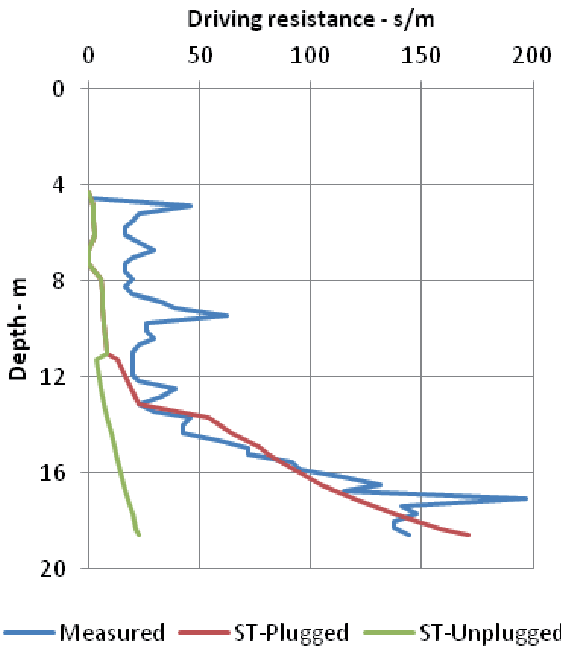


Figure 3. Unit driving time (diving resistance) vs. depth.

Plugging is indeed a distinct possibility as suggested by impact records which, following vibratory installation and splicing, were collected when impact driving started. CAPWAP analysis then indicated capacities of 1780 and 1600 kN for the first and second blow, respectively, with little further resistance decrease after the second blow. The end bearing calculated by CAPWAP was 1160 kN, which corresponds to a unit resistance of 5.4 MPa. The ST end bearing in contrast was only 300 kN. What was also surprising was that the CAPWAP calculated toe quakes were huge: 63 mm for the first blow and 55 mm for the second one. Had the wave equation analysis been performed with the CAPWAP calculated resistance parameters, the result would have been absolute refusal. On the other hand it may be argued that because of the large quake, the limited vibratory amplitude could only activate about 10% of the available end bearing. It may be theorized that the pile was fully plugged during the early impact restrike, but that the plug slipped during vibratory driving, and that the soil therefore behaved stiffer (lower quake). and thus less energy demanding, during vibratory driving.

The PDA pile top measurements of force, F (compressive positive), and acceleration, a (downward positive), are shown for an early easy driving record and for a late one in Figure 4. The acceleration has been multiplied with the pile mass for scaling. For these two records, Table 3 lists results relevant to the present discussion; the results from other records ranged between these two extremes. Results from all records are plotted vs. depth in Figure 5 (force, inertia, velocity and penetration speed) and in Figure 6 (resistance from power, Case and rigid body approaches together with the driving resistance). Note that the maxima of force, velocity, displacement etc. shown are the positive maximum amplitudes (i.e. ½ of maximum – minimum of a symmetric record).

A few comments should be made:

The centrifugal force is greater than the force in the pile, which is not unreasonable particularly for a pile with a large mass (5.9 Mg in the present case). The force is apparently not affected by the soil or driving resistance and that is similar to impact driving where the force peak at impact depends on hammer and pile properties, but not very much on the soil resistance. This may be different in a refusal situation. The power transfer, averaged over several cycles, is less than half of the rating of the power unit. It increases from 208 to 270 kW, as the resistance increases. However, the temporary peak power transfer, happening only at certain short time periods during a cycle, was somewhat higher than the rating. While it is not possible to independently confirm the correctness of the measurements and data interpretations, (slight phase shifts between force and velocity and/or low frequency adjsutments of the motion records), it is intuitively reasonable to expect that the energy transfer for short periods assumes higher (and also much lower) values than the average. Of course, over longer time periods, i.e., several cycles, the output from the power unit cannot be greater than the rated value. The results are, therefore, considered reasonable.

Table 3. Selected numerical results for early and late records.

Quantity	4.1m Depth	18.6m Depth	Quantity	4.1m Depth	18.6m Depth
Drive Res. (s/m)	4.4	134	Centrif. Force(kN)	2150	2120
Amax (g's)	18.3	14.4	Rt Case (kN)	300	560
Vmax (m/s)	1.24	0.85	Rt Rigid (kN)	340	710
Dmax/Cycle (mm)	8.2	5.1	R Power A(kN)	460	1840
Fmax (kN)	1410	1430	Inertia (kN)	1030	808
Average Power (kW)	208	270	Rated Power (kW)	597	597

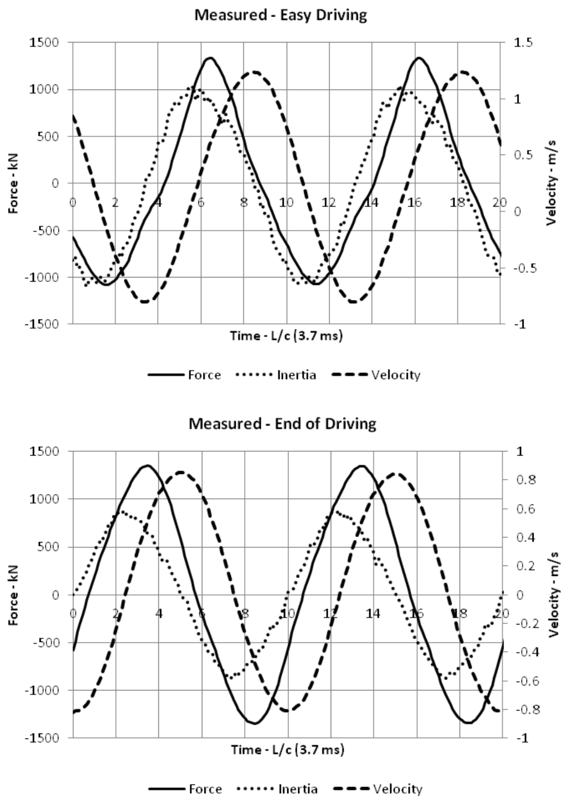


Figure 4. Force, velocity and acceleration times pile mass for records taken at 4.1 m (top) and 18.6 m (bottom) penetration.

The respective unit driving times were 4.4 and 133 s/m, respectively, and that explains the positive shift of the velocity curve for the easy driving record. The velocity peak values clearly show the effect of resistance and so does to some degree the displacement amplitude. A commonly used rule of thumb is Dmax = Me/Wt (eccentric moment divided by total vibrating weight of hammer plus pile). It leads to 6 mm in the present case and agrees quite well with the 8.2 and 5.1 mm values for easy and hard driving. However, comparing that to the GRLWEAP recommended quake of 5 mm for cohesive soils, it is obvious that there is not much margin left for overcoming the soil resistance, since that requires a displacement amplitude greater than the quake. Indeed CAPWAP calculated a much larger quake for the toe resistance.

The resistance results are interesting (to say the least). Case Method (RC) and rigid body(Rr) model yield similar results and that is not surprising since the records (Figure 4) show only very faint wave effects in the acceleration records (note that the time scale is in L/c units and wave effects would be expected at intervals of 2L/c). Probably the Case Method, which is averaging force and acceleration values, would be more applicable than the rigid body formula when frequencies approach the pile's resonance frequency and it is, therefore, considered a safer method for general applications. But the Case resistance values increase from only 300 to 560 kN and that is surprisingly low even though it is expected that some resistance is lost due to the driving process. The power approach shows nearly perfect agreement with CAPWAP and with what would be expected from static fromulas and wave equation. This could be a coincidence and should not be taken as a proof that it would always yield such good results. However, the power approach has the advantage that it includes consideration of the penetration speed (the inverse of the unit driving times) while the Case Method only provides the sum of simulataneously occurring resistance values. Considering that the upward and downward motion of the pile creates positive and negative resistance components and that, for large toe quakes, the toe resistance may not completely unload, leads to the conclusion that the instantaneous resistance will always be a low value. On the other hand, the power approach will identify as resistance not only the effort of overcoming the soil resistance in the pile soil interface but also the effort of keeping the surrounding soil in motion.

6 SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

This paper demonstrated what methods are available and what kind of results can be expected for the analysis and construction control of vibratory pile driving. It was found that

- Not unlike static formulas, resistance results from different evaluation methods of measurements vary quite significantly. The rigid body and Case method results turned out to be much lower than expected from restrike tests or static analysis.

  While lower soil resistance values are generally expected to occur during vibratory driving, this may not be so in cohesive materials where, in the present example, the power approach produced a rather reasonable result. Unfortunately, it cannot be stated with confidence which dynamic testing analysis methods would be most appropriate for all soil types and hammer frequencies. More experience and data has to be gathered.
- Driveability predictions of vibratory driving are fraught with the same uncertainties as those for impact driven piles when dealing with open profiles in hard or very dense soils, i.e., it is more a question of a correct assumption on plugging than of the basic numeric model details whether or not the wave equation approach leads to success. It may therefore be wise to perform both upper bound and lower bound analyses.
- In order to advance the State of the Art, it is recommended that the profession collect as frequently as possible the following information:
  - Force and acceleration records and their evaluation according to the above recommendation;
  - Driving resistance (or speed of penetration);
  - Accurate hammer and power unit details;
  - Soil profiles and strength information;
  - Wave equation driveability prediction;

Comparable impact records and their evaluation (since frequently piles are first driven by vibratory hammer and, after the driving process becomes uneconomical, by impact hammer); favorable site conditions for such measurements often exist.

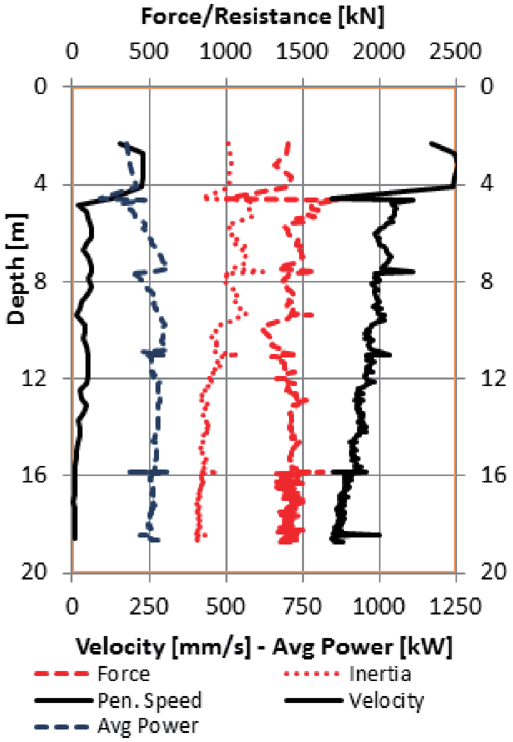


Figure 5. Maxima of force, velocity, inertia and average power together with the penetration speed from video records.

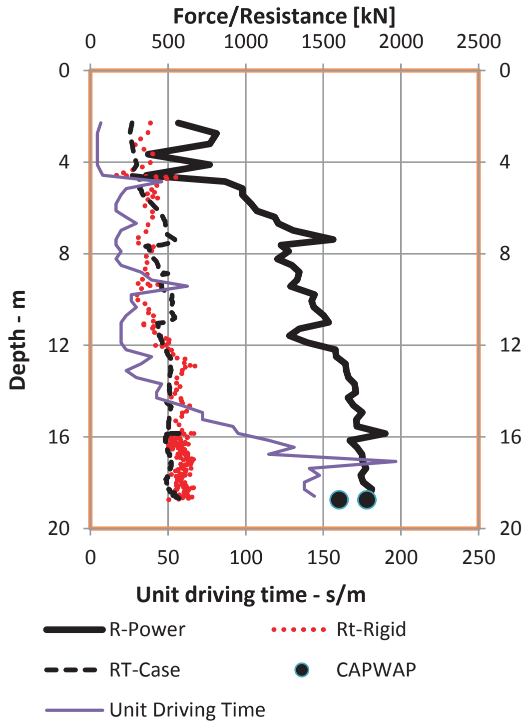


Figure 6. Soil resistance from rigid body model, Case and power approach.

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