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Comments on the Current and Future Use of Pile Dynamic Testing

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ABSTRACT: Impact driving of piles is a complex process which depends on the interaction of the stress wave progressing down the pile and on the dynamic soil resistance along the shaft and at the toe. Pile driving vibrations can adversely affect adjacent structures, and the intensity of ground vibrations depends on the dynamic resistance along the pile rather than on the energy applied to the pile. The viscous damping factor, J_c, is one of the fundamental parameters when the static pile resistance is determined from dynamic tests. Theoretical analyses and field observations show that the J_c-factor is not solely a soil parameter, but a function of the ratio of pile impedance and soil impedance at the pile toe. Consequently, the pile toe damping factor for an open-toe pipe pile would be different when no plug develops in the pile and when the pile toe is fully plugged. Moreover, the mentioned aspect needs to be considered when the results of a test pile of one pile type are applied to a construction pile of a different type.

Viscous damping along the pile shaft and at the pile toe is the source of ground vibrations. The vibration magnitude is governed by the interactive nature of dynamic pile-soil resistance, which is a function of the ratio between the pile impedance and the soil impedance for P-waves. It is shown that the vibration transmission efficacy and the dynamic resistance are inversely linearly proportional to the pile impedance, and that energy transmission efficacy correctly reflects the vibration emission from the pile to the surrounding soil layers.

The literature includes several studies of comparisons between pile capacity determined in a static loading test to that determined in a CAPWAP analysis on dynamic records. Much of the agreement and non-agreement in such papers is due to that many of the comparisons include mistakes and a few sources of such are illustrated and explained. A case of over reliance on blow-count is presented.

1. INTRODUCTION

After forty years and seven stress-wave conferences, the engineering practice still often looks upon a dynamic test as not much more than a low-cost alternative to a routine static loading test, and the engineering practice appears to have reached a plateau with regard to the use of dynamic testing of foundation piles. However, as will be indicated below, the dynamic test data can be drawn on to generate information also beyond pile capacity.

During driving, energy is transmitted from the pile hammer to the pile, and, as the pile penetrates into the soil, both static and velocity-dependent (dynamic) dynamic resistances are generated. The dynamic soil resistance gives rise to ground vibrations which are transmitted through the soil, potentially, causing settlement in some soils, or adversely affecting nearby installations or structures on or in the ground. In this context, the process is more complex than realized by many, but the theoretical format is quite simple, as will be shown below.

Moreover, even when only looking for a capacity value, many unnecessary gaffes are committed. For example, the influence of time on the static pile resistance at different times after installation can have important consequences on how to interpret static and dynamic pile loading tests and must not be overlooked. Some of these aspects are addressed below and some new concepts are introduced showing how the piling industry can benefit from a deeper understanding of dynamic pile-soil interaction.

2. DYNAMIC PILE-SOIL INTERACTION

2.1 General

When the pile driving hammer impacts the pile head, a stress or strain wave—vibration—is created that propagates at certain frequency and amplitude down the pile, into the soil, and in under and into adjacent structures. The main aspects of vibration propagation during impact driving of piles are illustrated in Fig. 1. During pile driving, energy is generated by the hammer (1) in impacting the pile cap (2) and entering the pile (3). A stress (or strain) wave propagates down the pile and this aspect of pile dynamic measurement [**A**] for the complex interaction occurring between the dynamic force in the pile and the soil [**B**], which generates static and dynamic soil resistance along the pile shaft (4) and at the pile toe (5). This aspect of dynamic pile measurement is generally well understood and applied in dynamic pile analysis, and it has been the main focus in the past and used to estimate the "dynamic" and an equivalent "static" soil resistance (bearing capacity). However, the dynamic (velocity-dependent) soil resistance gives rise to ground vibrations, which propagate in the soil [**C**]. The vibrations can adversely affect structures buried in, or founded on the ground [**D**]. For some reason, which is difficult to explain, only little attention has been paid to the interdependence of the pile driving resistance and ground vibration problems. Indeed, there is an almost complete lack of well-documented case histories, where both stress wave measurements and ground vibration measurements are made.

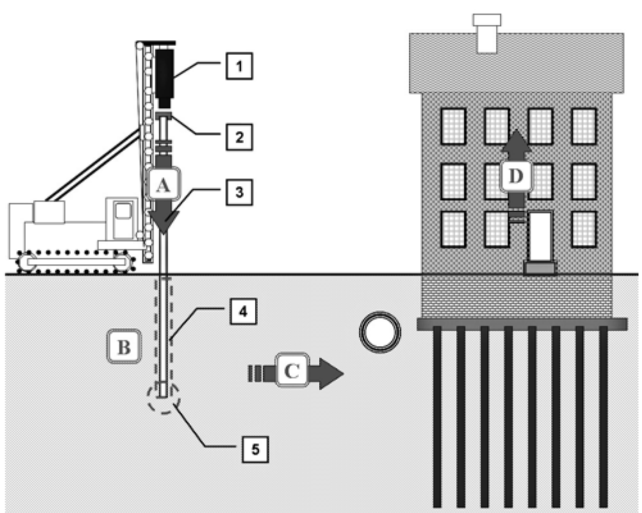

Fig. 1 Transfer of energy from the hammer, through the pile, into surrounding soil, and in under and into adjacent buildings.

The following simple example can be used to illustrate the problem. It is apparent that when a pile is installed by static force (i.e. at a very slow penetration rate, for instance during a static loading test), no ground vibrations will be generated. On the other hand, when the pile is driven, soil resistance created along the pile shaft and at the pile toe is velocity dependent, and ground vibrations will arise. The vibrations will vary as the pile penetrates through different soil layers, and, therefore, ground vibrations will emanate from several locations along the pile. The interdependence of dynamic properties (impedance) of the pile and the soil will be discussed in the following and illustrated by examples.

2.2 The Damping Factor, J_c

Evaluation of dynamic records includes determining the dynamic resistance from the relation shown in Eq. 1 (Goble et al. 1980).

$$R_{dyn} = J_c Z^P v^P \tag{1}$$

R_{dyn} = dynamic portion of the driving resistance
J_c = dimensionless damping factor
Z^P = pile impedance
v^P = particle velocity of pile

The damping factors are separated on resistance along the pile shaft and at the pile toe, as are the values of static resistance. The damping factor is generally assumed to be a soil parameter and independent of the pile. Rausche et al. (1985) indicated ranges of damping factors, J_c, for different main soil types to be as shown in Table 1.

TABLE 1 Damping Factors for Different Soils

Soil Type	J_c
Clay	0.60 – 1.10
Silty clay and clayey silt	0.40 – 0.70
Silt	0.20 – 0.45
Silty sand and sandy silt	0.15 – 0.30
Sand	0.05 – 0.20

Rausche et al. (1985)

The pile impedance is a physical parameter and is determined as shown in Eq. 2.

$$Z^P = \frac{E^P A^P}{c^P} = A^P c^P \rho^P \tag{2}$$

Z^P = pile impedance
E^P = modulus of elasticity of pile material
$E^P = (c^P)^2 \rho^P$
A^P = pile cross section area
c^P = velocity of stress wave in pile
c^P = velocity of stress wave in pile
ρ^P = bulk density of pile material

Moreover, before reflections from the soil resistance have superimposed the impact wave, the pile impedance is also equal to the ratio between the force in the pile and the pile physical velocity, as shown in Eq. 3.

$$Z^P = \frac{F_i}{v^P} \tag{3}$$

Z^P = pile impedance
F_i = impact force
v^P = particle velocity of pile

When values determined in actual full-scale tests deviate from those of Table 1, this is frequently considered to be an anomaly or attributed to natural variations of dynamic soil properties. For example, Fellenius et al. (1989) and Riker and Fellenius (1992) reported dynamic measurements on about 45 m long, closed-toe, heavy-wall pipe piles (9.625-inch/245 mm diameter with a 0.545-inch/13.8 mm wall) and H-piles (12HP63; 310HP93) driven to moderate penetration resistance into a glacial outwash deposit consisting of silty sand. According to Table 1, the damping factor should be about 0.20 — for both piles because they were driven into the same soil. However, while CAPWAP analysis on blow records from the pipe pile gave pile toe J_c-factors of 0.20 and 0.22 at end-of-initial driving, EOID and beginning-of-restrike, BOR, respectively, the CAPWAP analysis on the blows from the H-pile gave pile toe J_c-factors of 0.05 and 0.06 at EOID and BOR, respectively. That is, the ratio between the J_c-factors from the two pile types ranged from about 3 through almost 5. Cleary, the presumption of the damping factor being solely a soil parameter is not fully valid.

Indeed, as indicated by Massarsch (2005) and Massarsch and Fellenius (2008) quoting Iwanowski and Bodare (1988), the damping factor is a parameter that depends on the dynamic properties of both the pile and the soil, as indicated in Eq. 4.

$$J_c = 2\frac{Z_P}{Z^P} \tag{4}$$

J_c = dimensionless damping factor
Z^P = pile impedance
Z_P = soil impedance (from P-wave velocity)

The soil impedance, Z_P, is defined in Eq. 5.

$$Z_P = A_t^P c_P \, \rho_{soil} \tag{5}$$

Z_P = soil impedance for P-waves (P- waves) originating from the pile toe
A_t^P = cross section area of the pile toe in contact with the soil
c_P = velocity of P-wave in the soil
ρ_{soil} = bulk density of soil at pile toe

The velocity of the P-wave, c_P, is a soil parameter, i.e., it is a function of the soil characteristics and whether or not the soil is saturated. The range of values varies close to an order of magnitude depending on soil type and degree of saturation.

Equation 4 considers a pile with the pile cross section (at and above the pile toe), A^P, equal to the toe area in contact with the soil, A_t^P. However, for a closed-toe pipe pile, the two areas are different. The toe damping factor, J_c, for the latter pile is expressed in Eq. 6 by combining Eq. 4 with Eqs. 2 and 5.

$$J_c = 2\frac{c_P}{c^P}\frac{\rho_{soil}}{\rho^P}\frac{A_t^P}{A^P} = 2\frac{Z_P}{Z^P}\frac{A_t^P}{A^P} \tag{6}$$

J_c = dimensionless toe damping factor
c_P = velocity of P-wave in the soil; a parameter that depends on soil type and degree of saturation
c^P = velocity of stress wave in pile
ρ^P = bulk density of pile material
ρ_P = bulk density of soil at pile toe
A^P = pile cross section area
A_t^P = cross section area of the pile toe

For example, if assuming a steel pile driven into a soil with a bulk density, ρ_P, of 2,000 kg/m^3, then, for the wave speed of steel, c^P, of 5,120 m/s, and the pile material bulk density, ρ^P, of 7,800 kg/m^3, Eq. 6 simplifies to Eq. 7.

$$J_c \approx 1\times10^{-5}\, c_P \frac{A_t^P}{A^P} \tag{7}$$

J_c = dimensionless damping factor
c_P = velocity of P-wave in the soil
A^P = pile cross section area
A_t^P = cross section area of the pile toe

Equations 6 and 7 express that J_c is a function of the ratio between the area in contact with the soil and the pile cross section area. Thus, for the example piles, the pipe pile and the H-pile, as the P-wave velocity, c_P, is a soil parameter and the two piles are driven close to each other and into the same soil, the same c_P-value applies to the piles. However, the ratios between their respective pile cross section area and pile toe area are different. For the pipe pile, the ratio is 4.7, whereas it is unity for the H-pile. Thus, the ratio of calculated damping factors, J_c, according to Eq. 6, on applying the actual pile area ratios is 4.7. This value lies within the 3 through 5 range of ratio of the CAPWAP determined J_c-values indicated above. That is, the factual observations and the theory agree, the damping factor is not solely a soil parameter. As a side consequence, the pile toe damping factor for an open-toe pipe pile would be different when no plug develops in the pile and when the pile toe is fully plugged. Moreover, the mentioned aspect needs to be considered when the results of a test pile of one pile type are applied to a construction pile of a different pile type.

2.3 Ground Vibrations from Impact Pile Driving

Massarsch and Fellenius (2008) provide an account of the interactive nature of the pile impedance and the soil impedance which can be used to assess the vibration effect of pile driving, as follows. The fact that the damping factor is a function of the ratio between the pile impedance and the soil impedance for P-waves is verified by a reanalysis of vibration measurements reported by Heckman and Hagerty (1978), who measured the intensity of ground vibrations at different distances away from piles being driven. The piles were of different type, size, and material. Heckman and Hagerty (1978) determined a *k*-factor, expressed in Eq. 8, which is a measure of ground vibration intensity (usually the vertical vibration velocity).

$$v = k \frac{\sqrt{W}}{r} \tag{8}$$

v = vibration velocity (m/s)
W = energy input at source (J)
k = an empirical vibration factor ($m^2/s\sqrt{J}$)
r = distance from pile (m)

The vibration velocity in Eq. 8 is not defined in terms of direction of measurement (vertical, horizontal, or resultant of components). Moreover, the empirical factor, *k*, is not dimensionless, which has caused some confusion in the literature. Figure 2 presents the *k*-factor values of Heckman and Hagerty (1978) as a function of pile impedance and measurements of pile impedance.

The measurements were taken at different horizontal distances away from piles of different types and sizes driven with hammers of different rated energies. Unfortunately, the paper by Heckman and Hagerty (1978) is somewhat short on details regarding the driving method, ground conditions, and vibration measurements and, therefore, the data also include effects of ground vibration attenuation and, possibly, also effects of vibration amplification in soil layers. Yet, as shown in Fig. 2, a strong correlation exists between the pile impedance and the *k*-factor, as the ground vibrations increased markedly when the impedance of the pile decreased. In fact, ground vibrations can be ten times larger in the case of a pile with low impedance, as opposed to vibrations generated at the same distance from the driving of a pile with high impedance (Massarsch 1992).

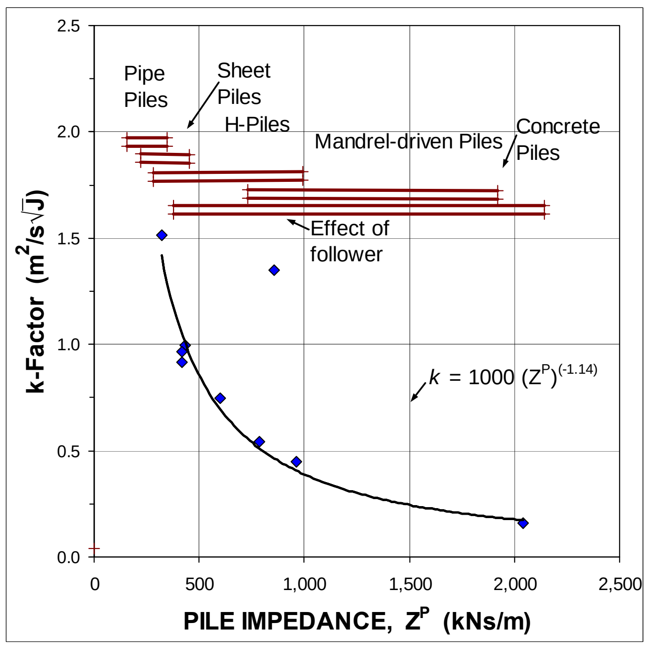

Fig. 2 Influence of pile impedance on the vibration factor, *k* (Eq. 8). (Data from Heckman and Hagerty 1978).

Massarsch and Fellenius (2008) defined "vibration transmission efficacy" as the ratio of the dynamic portion of the driving resistance to the impact force in the pile as shown in Eq. 9.

$$E_T = \frac{R_T}{F_i} \tag{9}$$

E_T = vibration transmission efficacy at the pile toe
R_T = dynamic resistance at the pile toe
F_i = impact force

Equation 1 (with $R_{dyn} = R_T$), Eq. 3, and the definition of Eq. 9, shows that the vibration transmission efficacy is equal to J_c and proportional to the ratio between the pile and the soil impedances (Eq. 10), which expresses the dynamic stress emitted from the pile toe.

$$E_T = J_c = 2\frac{Z_P}{Z^P} \tag{10}$$

J_c = dimensionless damping factor
Z^P = pile impedance
Z_P = soil impedance (from P-wave velocity)

Equation 10 indicates that the vibration transmission efficacy and the dynamic resistance (the velocity-dependent resistance) are inversely proportional to the pile impedance. The Heckman and Hagerty (1978) data prove the linearity, when the data are replotted versus the inverse of the pile impedance as shown in Fig. 3.

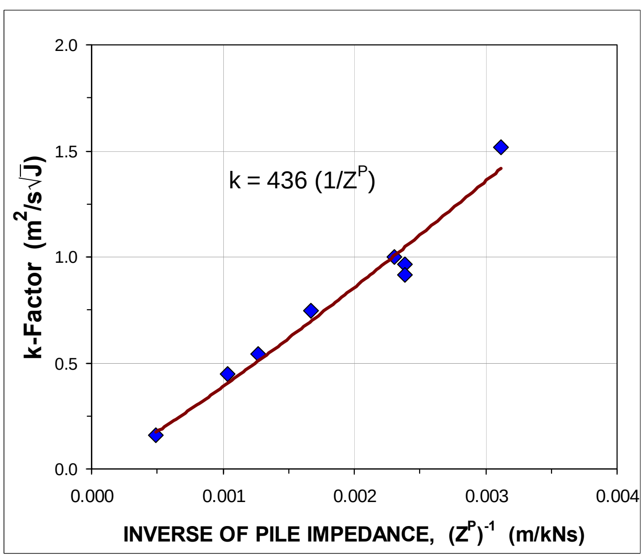

Fig. 3 Relationship between k-factor and inverse of pile impedance. Data from Fig. 2 replotted.

The correlation shown in Fig. 3 is surprisingly good, considering that the measurements were taken in different soil conditions. The data provided by Heckman and Hagerty (1978) indicate that ground vibrations in the reported cases mainly originated from the pile toe. Indeed, the data confirm that the energy transmission efficacy correctly reflects the vibration emission from the pile to the surrounding soil layers. The data also confirm the validity of Eq. 4.

3. SLIP-UPS WHEN DETERMINING STATIC vs. DYNAMIC PILE RESISTANCE

3.1 Introduction

The literature includes several studies of comparisons between pile capacity determined in a static loading test to that determined in a CAPWAP analysis on dynamic records. Some of these show excellent agreement between the two methods of determining capacity, while the agreement shown by others is less good. Notwithstanding that the capacity determined from the load-movement response of the tested pile is a matter of definition, which choice varies between authors and report writers, much of the non-agreement is due to that many of the comparisons are severely hampered by obvious mistakes, for example, the gage calibration is not up-to-date. A common mistake is performing the dynamic analysis on records from an end-of-driving blow as opposed to a restrike blow, which means that the capacity of the static loading test, in contrast to the end-of-driving condition dynamic test, benefits from soil set-up, or, when the dynamic value after all is from a restrike blow, several days lie between the restrike event and the static loading test. Another common mistake comes about when the modulus of elasticity used for determining the force from the measured strain is interpreted from the time of toe reflection, and its value is not the same as the modulus at the force gages. (The former can be affected by micro cracks in the pile which slow down the wave travel). When this is the case, the CAPWAP analysis must be made with different modulus for the *2L/c*-time and the force-from-strain record. Mostly, the dynamic testing industry has learnt to avoid these errors when evaluating pile capacity using the dynamic method. However, frequently other, not always fully recognized, key mistakes or pitfalls impinge on the results. Ways to identify and overcome these will be presented in the following.

3.2 When the Hammer Blow Does Not Fully Mobilize the Soil Resistance

The main task of a pile driving hammer is to install the pile to a desired and safe final capacity. In most soils, the capacity at end-of-driving, EOD, is smaller than the desired capacity; the balance is obtained from soil set-up. Usually, most local practices have learnt to apply a driving formula or other empirical relation to the driving records at EOD and establish the expected final capacity, which is then verified by restriking the pile. The penetration resistance at restrike is then much larger than at EOD. The reliability of such empirical relations is severely limited. However, while the advent of the dynamic testing and analysis methods have vastly improved the reliability of pile capacity determination, one must understand that "dynamics" only measures what the hammer is sending down the pile, or more accurately expressed, what the soil is able to reflect back up to the gages.

The example reported in Figs. 4a and 4b illustrates the point. The data (Riker and Fellenius 1992) are from testing a 500 mm diameter, 41 m long, prestressed concrete pile driven by jetting through loose sand to 25 m depth and underlying sand to siltstone bedrock. The CAPWAP determined capacity on a two-day restrike blow at a penetration resistance (PRES) of 90 blows/25 mm was 3,250 kN. The subsequent static loading test reached a plunging mode capacity of 7,300 kN (Fig. 4a). The dashed load-movement curve shows the results of the static loading test rising from the net movement measured at the preceding restrike test.

The capacities of the dynamic and static tests of the case history by Riker and Fellenius (1992) have by more than one author been used as example of a discrepancy between capacities determined by the two types of test. That interpretation is wrong, and has its roots in ignorance or in an underhanded selection of reference data. When properly understood, the results of the two tests show perfect agreement. The simple fact is that the hammer restrike impact did not fully mobilize the soil resistance along the full length of the pile, only along the upper about 30 m, as evidenced by the large value of penetration resistance, PRES, and that the maximum toe movement for the blow was only 6 mm (Fig. 4b). In contrast, the static loading test mobilized the resistance along the full pile length and achieved a toe movement of 45 mm. The agreement between the two methods is demonstrated by that the load distribution curves are parallel in the upper about 25 m depth.

Most will understand that if the available reaction load for a static loading test is, say, 200 tons, a capacity value larger than 200 tons cannot be demonstrated in the test. But it appears to be much harder to understand that, if the hammer-pile-soil combination is such that, at the most, the system can overcome a resistance of 200 ton, then, when the pile encounters a larger resistance, the result will essentially only result in a large PRES value, and the CAPWAP-determined capacity value will be no more than about 200 tons. One must understand that the CAPWAP value is a measure of what the hammer-pile-soil system has achieved. If the mobilized resistance is smaller than the pile capacity, the capacity will not be mobilized—not be measured. In other words, for the capacity to be determined, the maximum movement (penetration) of the pile must be

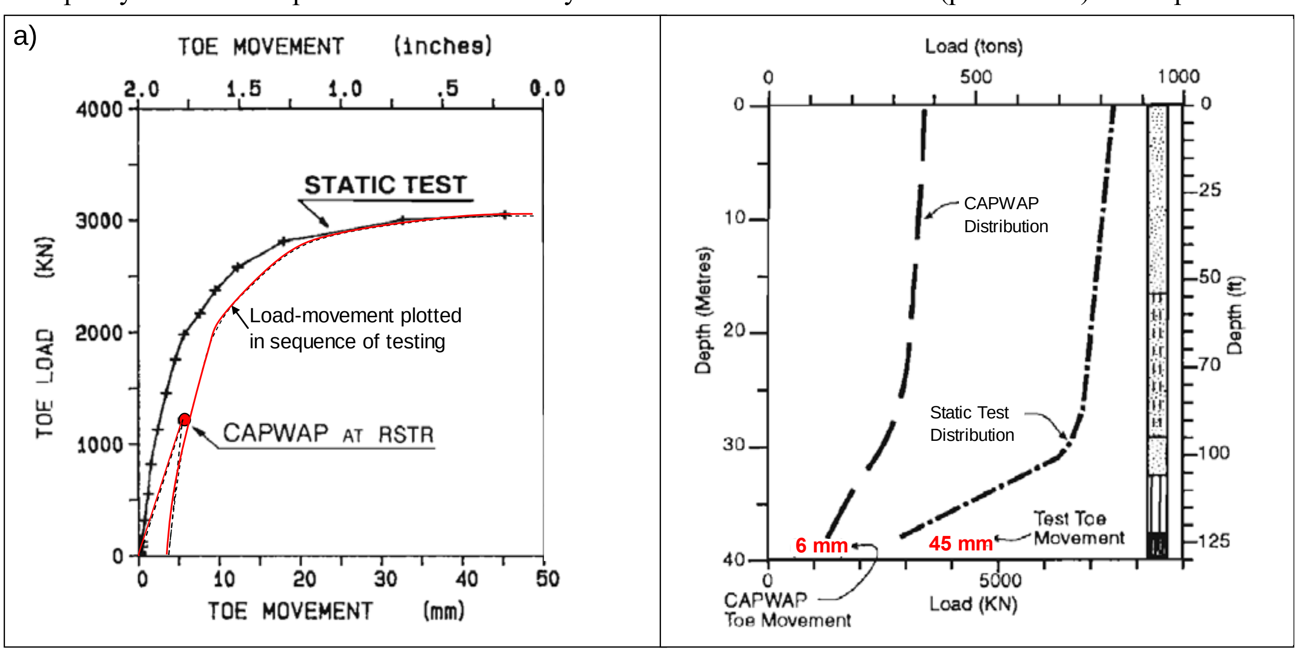

Fig. 4 Results of CAPWAP analysis and from subsequent static loading test. (Data from Riker and Fellenius 1992).

larger than the soil toe quake. While the toe quake is usually about 3± mm, it can reach an extreme of 30 to 40 mm (Authier and Fellenius 1980). Where the blow-count, i.e., the penetration resistance (PRES), reaches 15 to 20 blows/25 mm of penetration, the toe quake is usually not exceeded, and, therefore, the pile capacity is not fully mobilized.

A pile driving hammer is sized to drive a pile and not to be a testing device for restrike conditions after set-up has developed. However, not to include set-up when it is available is costly and poor engineering. In many places, local industry has learnt to count on soil set-up to give the capacity estimate a boost beyond that at end-of-driving. Mostly by applying some quasi rules to estimate the set-up from the "refusal" restrike PRES value in relation to the EOD PRES value. Bringing in a heavier hammer to overcome the set-up is an alternative, of course (e.g., Fellenius et al. 1989), but one that is often considered an unnecessary cost item. If the hammer is unable to fully mobilize the pile capacity at restrike, combining the information from analysis of dynamic tests from the end-of-driving and from the beginning of-restrike will go a long way toward improving the understanding of the results of the tests, as indicated in the following example from driving a 500 mm square prestressed, concrete test pile for the foundations of a near-shore project in Washington, DC. The soils consist of an about 50 ft (15 m) of silt deposited on mixture of sand and clay. The piles were driven to an EOD termination criterion PRES of about 20 blows/25mm. Figure 5 shows the recorded pile driving diagram and the measured maximum (impact) force in the pile and the calculated CMES pile capacity for a damping factor, J_c, of 0.5. The values recorded at Beginning-of-Restrike, BOR, are also shown.

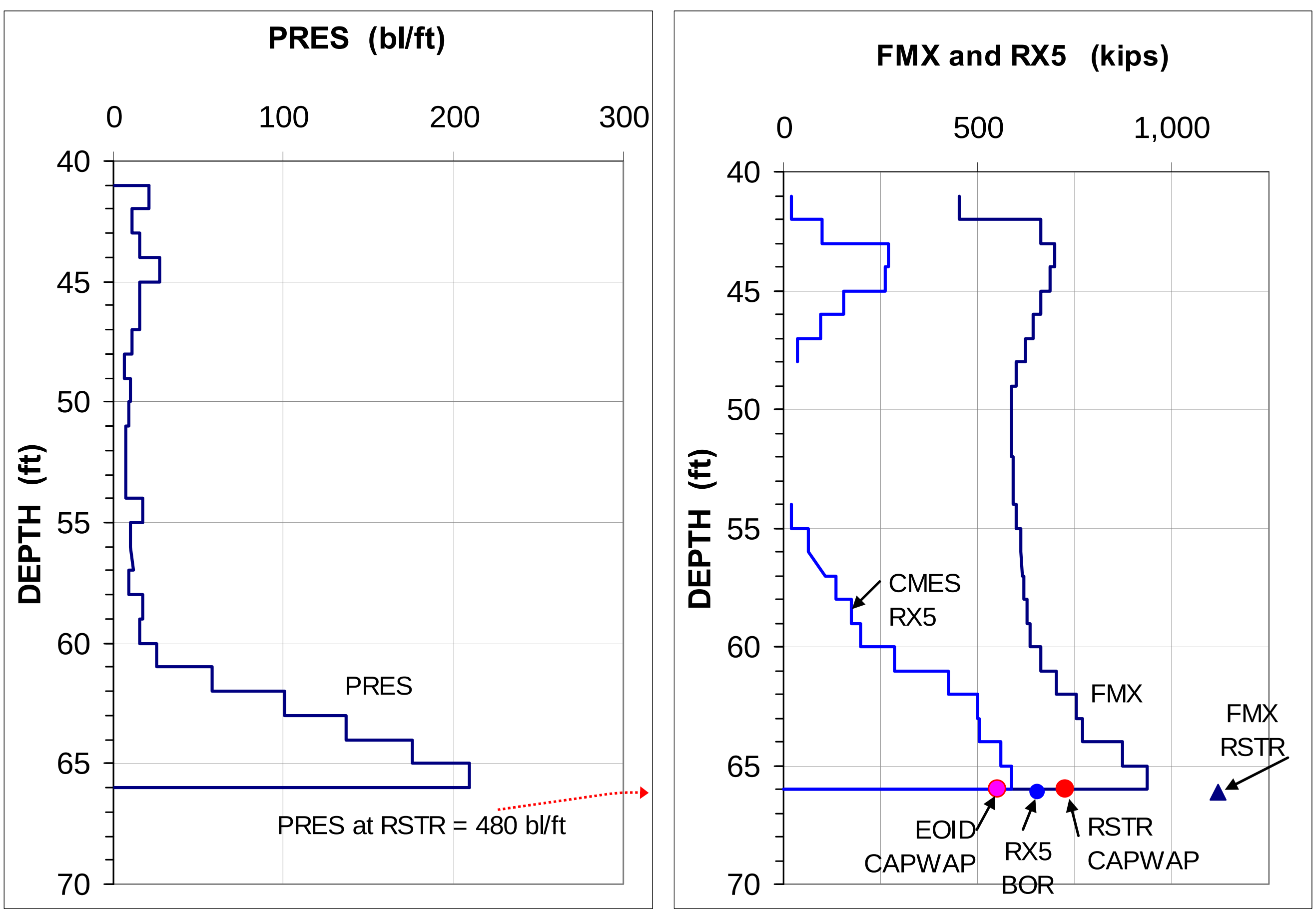

Fig. 5 Recorded PRES, FMX, and CMES values at initial driving, ID, and beginning of restrike, BOR. (Data courtesy of AATech Scientific Inc.).

The CAPWAP-determined capacity at EOID and at 8-day BOR are 550 kips (2,450 kN) and 725 kips (3,200 kN), respectively, indicating a slight set-up effect. The EOID and BOR wave traces shown in Fig. 6 demonstrate that at EOID the toe reflection includes a velocity increase, a sign of a significant toe quake (10 mm), whereas the BOR traces show a good positive toe reflection, a smaller quake (5 mm), and no indication of relaxed toe resistance. Indeed, at BOR, the pile capacity is not fully mobilized and the indicated capacity is smaller than the actual, as already the PRES values, 20 bl/25mm at EOID and 40 bl/25mm at BOR, make clear.

The two quake values are also the maximum toe penetrations for the blows, which suggest that neither blow has fully mobilized the soil resistance. This is confirmed by looking at the Fig. 7 resistance distributions as determined in the CAPWAP analysis. At EOD, the shaft resistance was mere about 100 kips (450 kN), whereas at BOR it was almost six times larger. Moreover, in having to expend the hammer force and energy to overcome that large shaft resistance, the BOR blow was only able to mobilize a fraction of the available toe resistance, about half that of the value mobilized at the EOID blow. It is a safe assumption for these data that the toe resistance at BOR is at least as large as that engaged at EOD. A

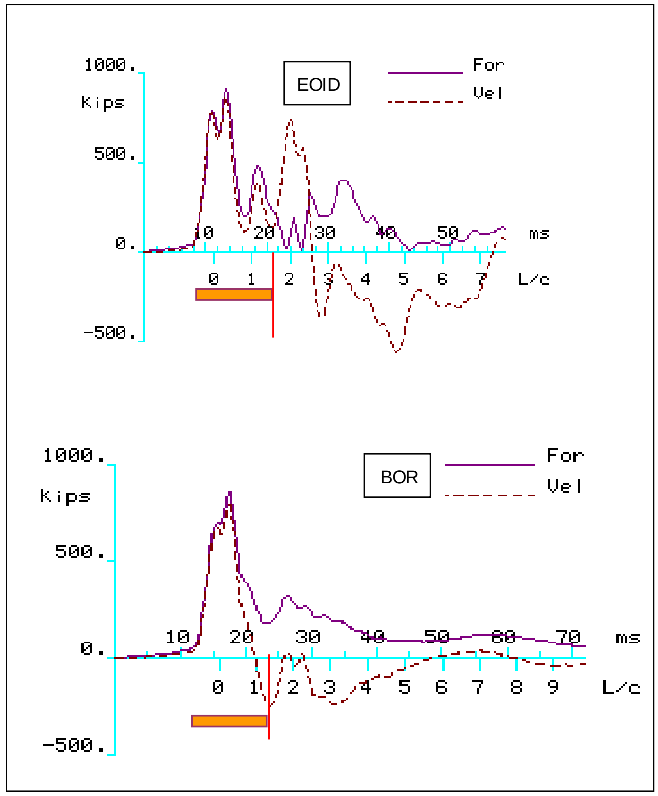

Fig. 6 Wave traces from EOID and BOR blows. (Data courtesy of AATech Scientific Inc.)

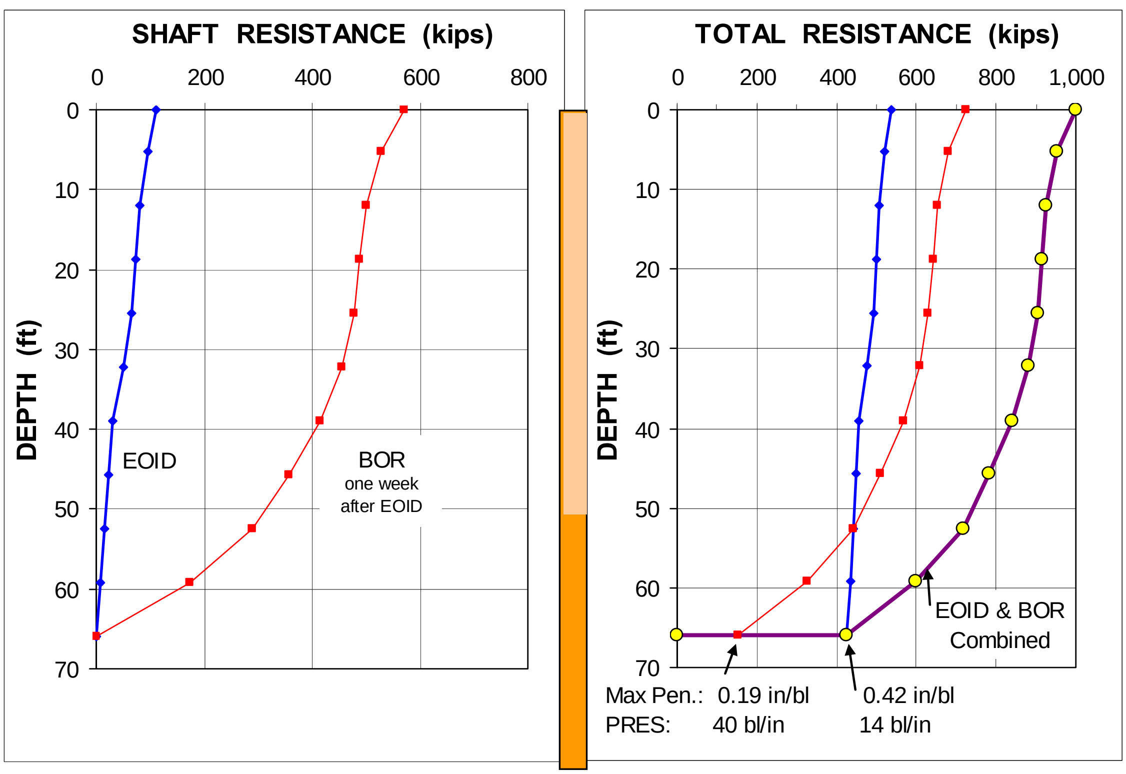

Fig. 7 CAPWAP determined shaft and toe resistances at EOID and BOR blows. (Data courtesy of AATech Scientific Inc.).

lower bound estimate of the pile capacity can therefore be obtained by simply combining the EOD toe resistance with the BOR shaft resistance as indicated in Fig. 7. Thus, by combining the records from EOD and BOR records, the analyses show that the pile capacity at restrike is about 1,000 kips (4,450 kN). As the desirable capacities in compression and tension were about 280 kips (1,245 kN) and 180 kips (800 kN), respectively, the test results demonstrate that considerable savings would be possible by shortening the piles. As an afterthought, a second test pile was driven about 12 ft (3.7 m) shorter to a EOID PRES of 5bl/25mm. Dynamic tests at a two-week restrike (PRES 20 bl/25mm) showed the capacity, now fully mobilized, to be 600 kips (2,670 kN) with a shaft capacity of 400 kips (1,780 kN). The 12 ft shortening represented a considerable savings for the project.

As the mentioned case history demonstrates, a testing programme usually includes test piles driven to the depths and termination PRES perceived as the most probable or safe design. However, such a programme can easily become a self-fulfilling event inasmuch as information may not be available to shorten the piles or easing up on the termination criterion. To alleviate this problem, one of the test piles should be driven shorter than the other and let to set up before a restrike. It should not be driven first because how much shorter it should be depends on how the "main" test piles drive. Immediately on completion of the restrike of the shorter test pile, it can be driven down to any desired final depth and termination criterion (if the test piles, as often happens, are to become part of the construction piles). Such a testing programme provides information that can assist in finding the safe and yet economical design of the piled foundations.

3.3 Who Goes First and Who's on Second

It is not wholly recognized that the sequence of testing is important, that is, it matters whether or not the static loading test is performed before or after the restrike. This is not a trivial matter, because a dynamic test is preferably performed with a pile head well above ground ("stick-up"), while a static loading test is preferably performed after "cut-off" leaving the pile head close to the ground surface and not directly suitable for a dynamic test. The following case history illustrates the importance of the testing sequence. The case is a long-term study of the development of capacity with time for a 324 mm diameter, closed-toe, concrete-filled, pipe pile at a highway bridge site in Alberta, Canada. Details on the case are presented by Fellenius (2008).

The soil at the site is an eroded, transported, and re-deposited glacial-till clay. Pore pressure measurements showed that large excess pore pressures developed from the driving. The excess pore pressures took more than 30 days to dissipate. Static loading tests were performed 15 days, 30 days, and 4 years (1,485 days) after end-of-driving, EOD. Restrike tests were carried out one day after EOD and the day **before** the 30-day static loading test and the same day, but **after** the 4-year static loading test. The 30-day restrike consisted of 5 to 8 blows to a total penetration of about 50 mm. The four-year restrike consisted of a series of eleven blows. In all restrike blows, the full capacity of the pile was mobilized. The results of the dynamic and the static tests are shown in Fig. 8.

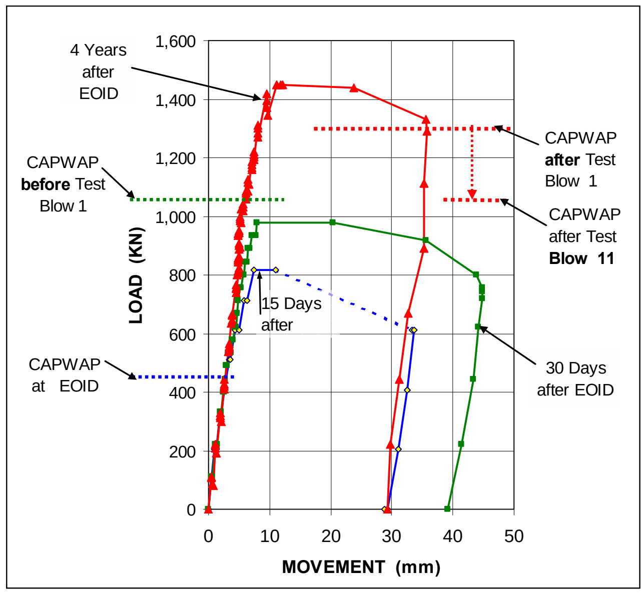

Fig. 8 Load-movement at static testing at different times after initial driving and CAPWAP determined capacity (Fellenius 2008).

The results of the dynamic test at EOD and the 15-day static loading test show that the pile capacity was setting up with time. As reported by Fellenius (2008), the set-up corresponded to the simultaneous dissipation of the excess pore pressures. The CAPWAP capacity at the 30-day restrike was 1,050 kN and the plunging type failure of the next-day static loading test occurred at 980 kN. (The CAPWAP capacity being 7 % higher than that of the static loading test is for all practical consideration as close as one could possibly expect from the two very different methods of testing). The sequence of testing was reversed for the 4-year tests. The capacity found in the static loading test was 1,450 kN and the CAPWAP capacity for the first restrike blow was 1,300 kN, or 12% lower, again a very good agreement between the two methods. However, the agreement is actually better than indicated by the numbers alone. The CAPWAP performed on the 11th blow of the restrike showed a capacity of 1,050 kN. That is, each blow reduced the capacity. One can venture to state that the static loading test might have reduced the capacity somewhat for the first blow. Well, perhaps not significantly. However, there is no doubt that the restrike preceding the 30-day static test must have reduced the capacity found in the static test. The example case suggest that not only should a dynamic and static test be in "temporal agreement", it is advisable to at all times perform the dynamic test after the static test. This, because the static test disturbs the condition less as opposed to a dynamic test (which also always includes several blows).

3.4 Where conventional wisdom failed

In preparation of construction for a hotel building in San Juan, Puerto Rico, a couple of test piles were driven and three of these were subjected to static loading tests (Salem et al. 2008). The site was located in a in-filled lagoon with the upper about 35 ft to 45 ft (about 10 m to 15 m) consisting of fill and bottom peat. Hereunder, and down to limestone bedrock at a depth of about 100 ft (30 m), the soils consist of material washed down from the near-by mountains and costal deposits made up of mixture of silty clays and variable amounts of sands and clayey sands. The cohesive samples are very stiff to hard, while the coarse soils are dense to very dense as indicated by N-indices ranging from a low of 30 bl/0.3 m to exceeding 150 bl/0.3 m. Figure 9 presents the distribution of N-indices and natural water contents. The embedment depth of the test BH-3 is described as mostly sandy clay, whereas the soil in BH-5 is described as mostly clayey sand. The piles driven close to each borehole is indicated. Below the upper fill and peat layers, the soil profile at differences in soil type proportions, however, are marginal.

The driving diagrams for the two test piles are presented in Fig. 10. Despite the high N-indices, the piles did not need any particularly hard driving to reach embedment depths of 71 ft to 65 ft (21.6 m to 19.8 m). Piles TP-3 and TP-5 were subjected to static testing 18 days and 13 days. respectively, after end of driving. Judging from the pile driving diagrams, it would be expected that Pile TP-3 would show the smaller capacity and softer response as opposed to Pile TP-5. Therefore, the pile head load-movement curves for the tests shown in Fig. 11 gave a momentous surprise. Not only is its capacity of Pile TP-5 much smaller than that of the other pile, its load-movement curve shows a significantly softer response.

To investigate, Pile TP-5 was driven down 4 ft after the static loading test. As the driving data provided no explanation to the different response to load for the piles, 20 days later, dynamic restrike tests were performed on both piles. The load distributions determined in the CAPWAP analyses of the BOR blows for each pile are shown in Fig. 12.

Note that the restrikes were made after the static loading test on Pile TP-3 and, therefore, the CAPWAP values represent re-loading conditions. Neither pile showed net penetration for the restrike blows and the resistances are therefore lower bound values.

The flatter slope of Pile TP-3 curve below about 40 ft as opposed to the steeper slope for TP-5, indicates that the unit shaft resistance in the sandy clay of BH 3 is larger than in the clayey sand at BH-5. Judging by the N-indices in BH 5 being about twice those of BH-3 one would have expected the reverse. The details of the CAPWAP analysis indicates the reason for the difference. As indicated in Fig. 13, showing the distributions of shaft quake for the two tests, the quake values are about twice as large for Pile TP-5 as for Pile TP-3. That a large quake makes for seemingly harder driving and smaller capacity has been known a long time (Authier and Fellenius 1980). Rather unexpected, however, it would appear from the presented case record that also the SPT index could be affected by the same conditions that produced the large quake and provide falsely large N-indices. It is unfortunate that the case history does not include the results of a static cone penetrometer, CPTU, as it might have provided some clue to the site conditions.

Further acceptance of the piles for the project were based on the results of a comprehensive extended testing programme of dynamic measurements and CAPWAP analyses.

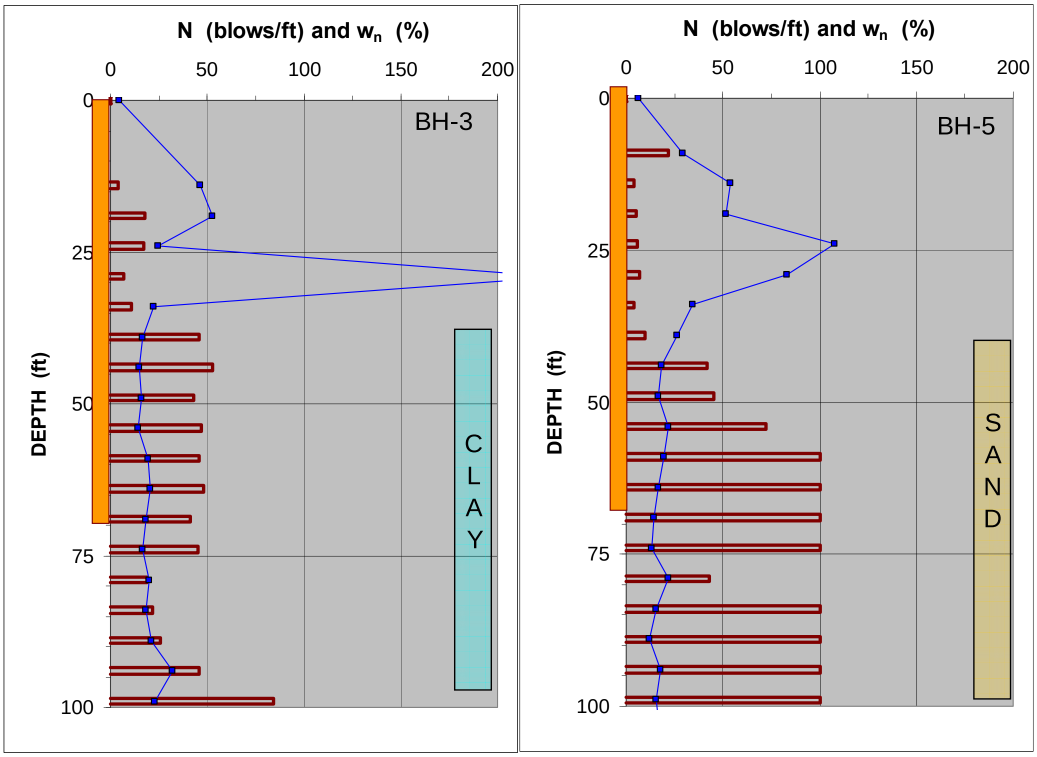

Fig. 9 *N*-indices and water contents with pile embedment depths at two borehole and test pile locations.

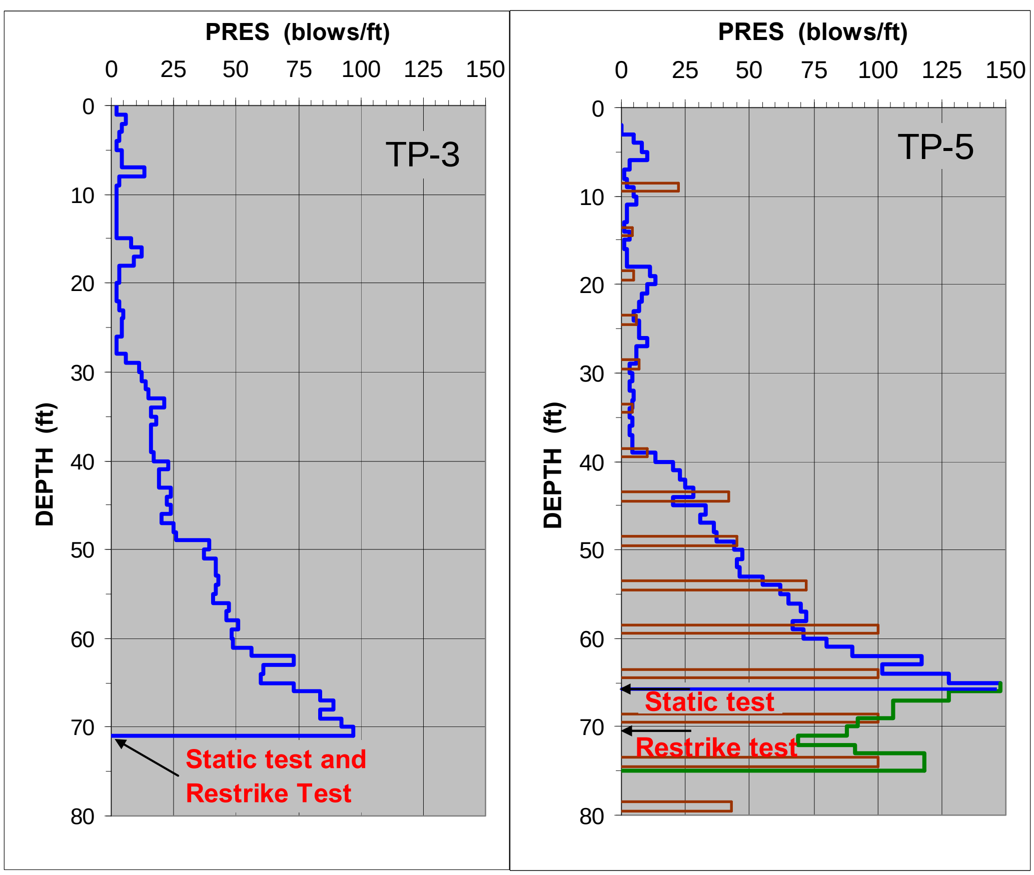

Fig. 10 Pile driving diagram from the three test piles.

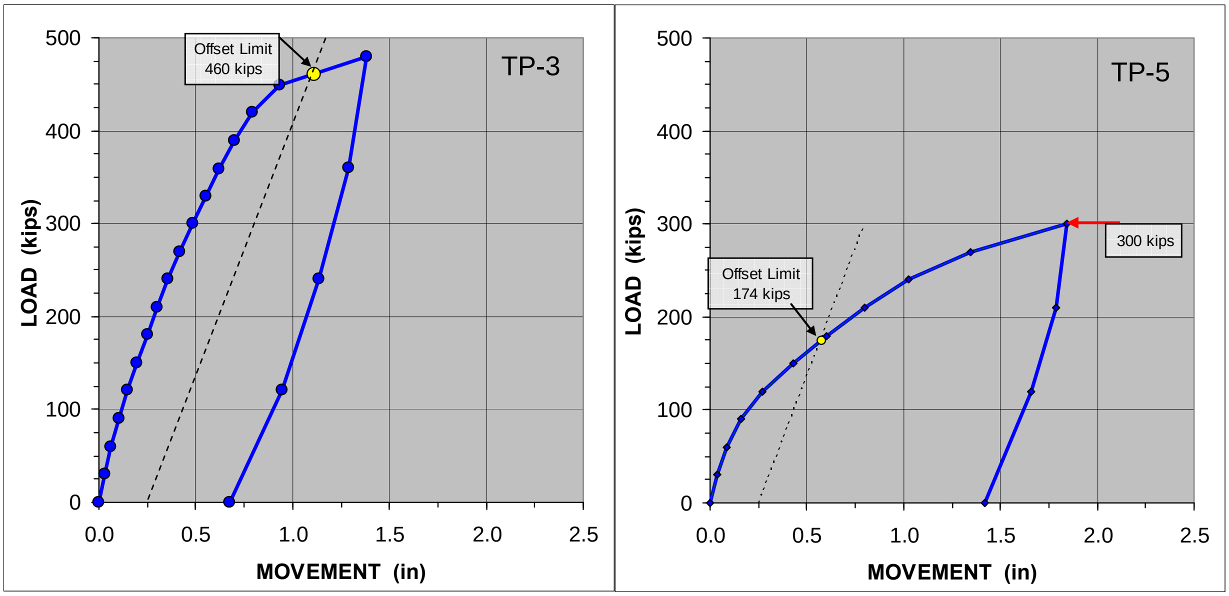

Fig. 11 Load-movement curves from the two static loading tests.

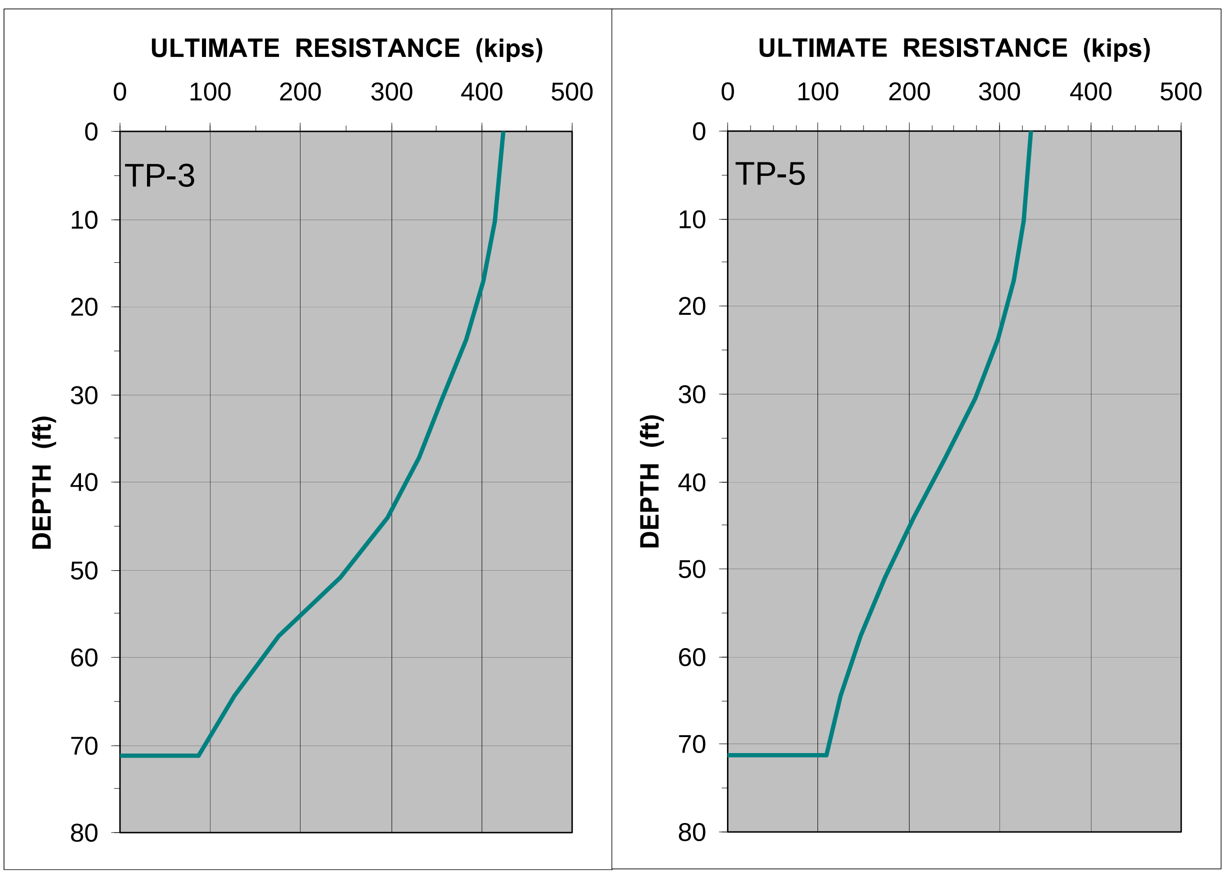

Fig. 12 CAPWAP determined load distributions.

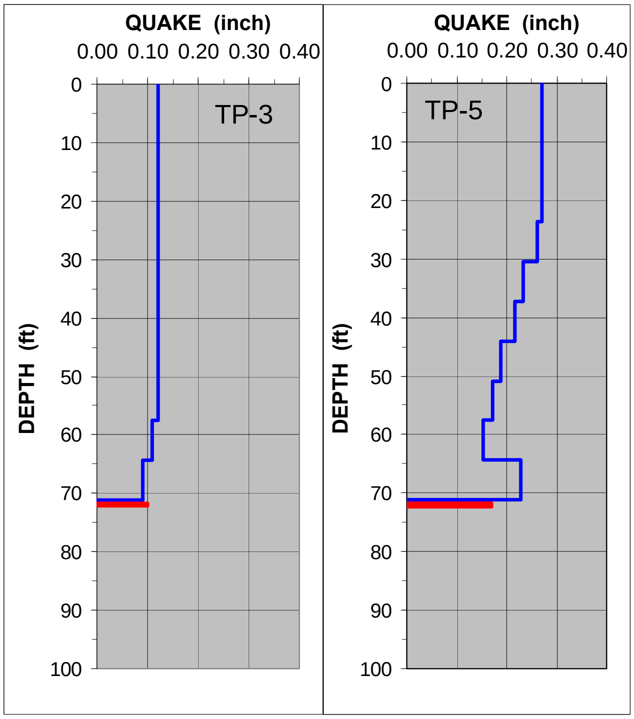

Fig. 13 The CAPWAP-determined quake values.

4 CONCLUSIONS

Considering that dynamic pile tests have been around for more than 40 years, it is surprising that many fundamental aspects which influence the interpretation of measurements are not yet appreciated by the profession. Impact driving of piles is a complex process which depends on the interaction of the stress wave and the soil. Pile driving vibrations can adversely affect structures buried in, or founded on the ground surface, and the intensity of ground vibrations depends on the dynamic resistance along the pile rather than on the energy applied to the pile. The viscous damping factor, J_c, is one of the fundamental parameters when the static pile resistance is determined from dynamic tests. Theoretical conditions show that the pile toe J_c-factor is not solely a soil parameter, but a function of pile impedance and the ratio between pile cross section area and pile toe area in contact with the soil. For example, when applying the theoretical relations to actual measurements reporting different J_c-factors in the same soil, the discrepancy of results disappeared, and the factual observations and theory agree. As a side consequence, the pile toe damping factor for an open-toe pipe pile would be different when no plug develops in the pile and when the pile toe is fully plugged. Moreover, the mentioned aspect needs to be considered when the results of a test pile of one pile type are applied to a construction pile of a different pile type.

The interactive nature of the pile and soil impedances and damping is a function of the ratio between the pile impedance and the soil impedance for P-waves was verified by a reanalysis of the vibration measurements reported by Heckman and Hagerty (1978). The data show that the vibration transmission efficacy and the dynamic resistance are inversely linearly proportional to the pile impedance, and that energy transmission efficacy correctly reflects the vibration emission from the pile to the surrounding soil layers.

The literature includes several studies of comparisons between pile capacity determined in a static loading test to that determined in a CAPWAP analysis on dynamic records. Much of the agreement and non-agreement in such papers is due to that many of the comparisons include mistakes and a few of these are illustrated and explained. A case of over reliance on blow-count is presented.

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