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**SOIL DAMPING AND RATE DEPENDENT SOIL STRENGTH CHANGES DUE TO
IMPACT AND RAPID LOADS ON DEEP FOUNDATIONS**

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ABSTRACT

Measurement of the static soil resistance of deep foundations can be done by either static or dynamic loading tests. The dynamic test applies a load to the pile by impact of a large mass onto the highly or minimally cushioned pile top. Measuring the resulting force on top of the deep foundation (pile) and the associated motion and performing a dynamic analysis of the pile-soil system allows for the separation of static from dynamic soil resistance components and the calculation of an equivalent static load-displacement curve. This result can be compared with the same type of curve obtained by the static loading test. For certain plastic soils, it has been found that the static resistance derived by analysis from the dynamic test may not completely account for the fact that quickly loaded materials exhibit a strength greater than a slowly loaded material. The resulting static resistance should then be reduced by a rate factor which depends on basic soil parameters. This paper reviews different approaches of loading rate and damping effect identification. It also presents several examples of dynamic test results on piles driven or drilled into soils with different levels of plasticity. The effectiveness of two approaches in separating the dynamic component from the total soil resistance in addition to accounting for rate dependent strength gains is assessed by comparison with static loading test results, considering partial resistance mobilization and soil setup. Recommendations are given aimed at both reducing the possibility of overestimations of capacity and the need for accurately knowing the soil characteristics near the location of the dynamically tested foundation.

Key Words: Pile load testing; dynamic pile testing; rate factors; signal matching; dynamic pile monitoring; drilled shafts; driven piles; augered cast-in-place piles; CFA piles

INTRODUCTION

Testing of any type of construction material is done or should be done at standardized loading rates or the strength values obtained from these tests would include undetermined rate effects. These rate effects generally produce a higher apparent material strength the higher the rate of loading. The situation is particularly complex when it comes to evaluating the static resistance from measurements taken during a hammer blow on a pile or impact event on a drilled foundation. There are several effects which interact with each other and which have to be considered by the dynamic testing professional.

1. Soil damping or dynamic resistance, generally considered a velocity dependent resistance component.
2. Soil Setup, an increase in the static soil resistance over time that follows the temporary loss of soil resistance due to disturbances of the soil caused by the pile installation process.
3. Partial resistance activation, caused by insufficient hammer energy during the dynamic test.
4. Rate dependent strength effects, i.e., an apparently higher static strength of the soil material.
5. Creep, a time dependent settlement, which is obviously not something that a dynamic test can assess. Creep adds displacements to the load-displacement curve and therefore reduces the failure load defined by a fixed pile top displacement.
6. Relaxation, a decrease in the static soil resistance over time that follows a temporary increase of soil resistance during pile driving. In general the soil resistance increase at the time of driving does not occur under a few impacts of a pile test and is therefore limited to driven piles.

Overestimation of the static soil resistance due to rate effects causes concerns, because deep foundations supported by primarily shaft resistance in soils with high plasticity tend to, at best, exhibit a plunging failure and, at worst, a strain softening behavior. These piles, therefore, have less of a resistance margin than, for example, end bearing piles in a dense granular material. There are two reasons why rate effects on dynamic testing outcomes cause little concern: (a) piles are in general designed to penetrate through such unreliable materials and into a more capable bearing layer and (b) both traditional factors of safety and the more recent Load and Resistance Factor Design, LRFD, approach [1] cover potential discrepancies between static and dynamic behavior. However, recognizing such shortcomings of the dynamic approach and adjusting for them, may eventually lead to lower and, therefore, more economical overall factors of safety. It is therefore reasonable to look at some of the related literature and study the influence of this problem as discussed in the following sections.

Pile velocities, rise times, accelerations. Before discussing rate effects, it should be helpful to consider the typical pile test penetration velocities. While true static loads (zero velocity) occur probably only once a building is constructed, a near static penetration velocity is that of a maintained static load test where we may assume that a penetration of 100 mm is achieved in one 24 hour time period (since rates of pile penetration depend on pile response in a maintained load test, see for example ASTM 1143 [2], an exact and generally applicable rate of penetration cannot

be given). The resulting penetration speed is approximately 10^{-6} m/s. A Constant Penetration Test or a Quick Test may be done 3 times faster (3×10^{-6} m/s). The cone penetrometer is typically applied at a speed of at least 10^{-2} m/s. Pile driving, finally, applies velocities ranging between 1 m/s and 6 m/s. However, these pile driving velocities are peak values at the top; unless driving is very easy, the velocities quickly diminish to lower values and then become negative. Furthermore, in hard driving, peak velocities in the bearing layers may be much lower. Penetration velocities during both High Strain Dynamic Tests (ASTM D4945, [3]) and Force Pulse (Rapid) Tests (ASTM D7383, [4]) typically vary between 1 and 6 m/s with the latter testing method maintaining such velocities over a longer time than the former test.

A penetration velocity of 10^{-5} m/s will be used as a reference velocity in the following discussion. That means that pile driving reaches velocities of at most 6 orders of magnitude (OM) higher than what may be considered truly static while cushioned pile driving or testing produces velocities about 5 OM higher.

Actually, the effect of the “Rate of Loading” on the material strength has two aspects: one is the velocity of pile penetration discussed above, and one is the rise time, i.e. the time during which the pile velocity increases from zero to its maximum penetration level. For example, in typical pile driving situations of steel piles, the rise time is in the neighborhood of 2 ms; for more heavily cushioned concrete piles it is typically 4 ms and can reach 50 ms or more for Force Pulse (Rapid) Tests.

The acceleration may also cause a high temporary soil resistance, primarily at the pile toe in cohesive soils, during the short initial time period of positive (downward) acceleration. This high inertia effect is comparable to the effect of the impact of a flat object on a water surface. The acceleration may be 6 m/s divided by 0.5 ms or 1200 g's in an uncushioned steel pile. In a heavily cushioned concrete pile it may be 1 m/s divided by 10 ms or 100 g's; in an even more cushioned Force Pulse (Rapid) Test it may be only 1 m/s divided by 100 ms or 10 g's. To the author's knowledge, a relationship between acceleration and dynamic soil resistance on piles has not been studied.

As an example of the relationship between shearing strength and speed of shear, Taylor [5] reports in his text book about laboratory compressive strength test results obtained on remolded Boston Blue Clay (Plasticity Index, $I_p = 17$) samples at different shearing rates. A diagram summarizing these results indicated an apparent shear strength 14% higher for a shearing rate of 1% per minute than for a rate of 10^{-3} % per minute. Extrapolating linearly to a rate 10^{-6} % per minute suggests an approximately 30% lower static shear strength than for the dynamic (1% per minute) rate.

E. A. L. Smith [6] developed a dynamic analysis approach simulating the response of pile and soil to an impact load. His now widely used soil model relates the ratio of total to static soil resistance, R_t/R_s , to the pile velocity v using a damping factor, j_s .

$$R_t/R_s = 1 + j_s v \quad (1a)$$

The total resistance is the sum of the static resistance R_s and dynamic resistance, R_d ; the latter is the product of damping factor times pile velocity times static resistance. The static resistance, i.e., the resistance component which is dependent on the pile displacement, R_s , is not constant. It is thought to increase from zero to a maximum R_{ud} value and when the pile rebounds then it decreases. The so-called Smith-viscous approach [7] substitutes the ultimate resistance R_{ud} for R_s which then makes the damping approach linear.

$$R_t/R_{ud} = 1 + j_s v \quad (1b)$$

R_{ud} is the ultimate static resistance determined in a dynamic loading test after accounting for damping, $R_d = (j_s v)R_{ud}$. In general, the Smith-viscous approach produces a more realistic damping behavior than Eq. 1a since it has greater effects when the R_s values diminish in the later phases of the impact event. The ultimate static resistance, R_{ud} , is the failure load determined by the dynamic test; it is also referred to as Nominal Resistance or Characteristic Resistance in LRFD terminology and considered comparable to the long term static ultimate resistance R_u .

In the 1960s Coyle and Gibson [8] performed experiments in support of the then increasing acceptance of the Smith wave equation analysis approach for driven piles. The authors impact-loaded triaxial samples and determined that the ratios of total dynamic resistance to statically determined resistance range between 1.4 and 2.6 for saturated sands and between 1.8 and 2.8 for remolded clays. The studies indicated a fairly linear increase of the R_t/R_u ratio between 0.3 and 3.7 m/s. These were impact velocities and, therefore, peak velocities. Variation of the velocities may have occurred while the samples failed, but could also be considered constant for all practical purposes. To explain what could be expected between static loading rates and those used in the tests the authors proposed the following expression.

$$R_t/R_u = 1 + j_{s-c}(v^N) \quad (2)$$

Coyle and Gibson [8] recommended $N=0.18$ for the clay samples and 0.20 for sand. Associated damping factors j_{s-c} should be determined based on the angle of internal friction for sands and based on the Liquidity Index for the clays. Figure 1 shows some of the Coyle-Gibson data, representing the soil samples with the highest and lowest relative resistance values for the sands and clays. Eq. 2 was then graphed for the recommended N -values and associated with j_{s-c} values. Eq. 2, obviously provides a reasonable, though not perfect, match of the Coyle-Gibson data. Considering the findings of other researchers, the data and N/j lines were then replotted on a semi-log scale in Figures 2a and 2b for clays and sands, respectively, again demonstrating a reasonable match of the range of velocities used in the tests. Figures 2a and 2b also include trend lines of the form

$$R_t/R_u = \alpha + \beta \log_{10}(v) \quad (3)$$

which appear as straight lines in the semi-log plots and which appear to provide an even better fit than Eq. 2. Figures 2a and 2b also include the Smith-viscous (Eq. 1b) R_t/R_s lines for j_s values of 0.5 and 0.65 s/m, values which are recommended for toe damping and clay shaft damping, respectively). Obviously, the Smith-viscous approach only works well in the typical range of pile driving velocities.

It is interesting to look at the intercepts of the straight trendlines with the $R_t/R_s = 1$ line. For example, for the CE35 and EA60 clays, the intercept, α , in Eq. 3 is 2.44 and 1.88 and the slope, β , is 0.438 and 0.176. Unfortunately, there is, to this paper's authors' knowledge, no known method to link these parameter to standard soil parameters. Note that these semi-log trend lines suggest static resistance below velocities less than $5(10^{-3})$ m/s for sands and below 10^{-5} m/s for clays. Figure 2b includes additional data points which will be discussed below.

Dayal and Allen [9] studied the resistance on cone penetrometers at speeds between 0.0013 and 0.81 m/s; the authors concluded that for sands no significant rate effects exist while, for clays, the penetration rate made a significant difference. They suggested the following equation to represent the effect of penetration rate on the soil cohesion C .

$$C/C_{\text{ref}} = 1 + K_L \log_{10}(v_r) \quad (4)$$

where C_{ref} is the apparent cohesion at a reference velocity and v_r is the penetration velocity divided by the reference velocity; K_L is a soil viscosity number which depends on the undrained shear strength. For the cone's sleeve the authors give a K_L range between 0.17 and 0.93 and for the cone tip 0.03 to 1.5 where the smaller numbers are for hard clays and typical numbers for the cone resistance are between 0.12 and 0.25. For a v_r of 10^6 these numbers would yield a cone resistance increase between 1.7 and 2.5. For two samples of medium stiff clay, Figure 2 includes a few results for sleeve (D+A-MS1-S, D+A-MS2-S) and cone (D+A-MS1-C, D+A-MS2-C). Unfortunately, not all results published by the Dayal and Allan match as nicely the trends exhibited by other data shown in Figure 2.

Krieg [10] investigated the effect of consolidation speed on undrained shear strength, S_u , as a function of Liquid Limit, L_l . Increasing the rate of consolidation 100 times increased the apparent S_u by about 15% to 16% for respective liquid limit values increasing from 50% to 200%. For lower L_l values the rate effect decreased to lower percentages. The author suggested a rate effect equation of the form:

$$S_u/S_{u,\text{ref}} = 1 + I_v \ln(v_r) \quad (5)$$

where I_v is a soil type specific viscosity index divided by $S_{u,\text{ref}}$ which is the undrained shear strength determined at the reference velocity, v_{ref} . According to [10], the viscosity index is approximately 0.02, 0.03, 0.04 and 0.05 for Liquid Limits of 31%, 42%, 64% and 130%. If, therefore, the velocity ratio is 6 OM then with $\ln(10^6) = 13.8$ the S_u/S_{ref} ratio (and thus possibly R_t/R_u) would be 1.28, 1.41, 1.55 and 1.69 (average of 1.48) and, therefore, somewhat lower than, for example, the Coyle-Gibson results. A case could be made that, should the liquid limit not be well known, R_{ud} should be reduced by a factor $1/1.48$ or about 0.7 to match the static nominal resistance, R_u .

Rodriguez, Alvarez and Velandia [11] compared dynamic and static loading test results and concluded that for high plasticity soils subject to high strain rates, Krieg's liquid limit based formula [10] should be used for an improved agreement between dynamic and static loading test results. The authors presented examples which indicated static load test results up to 30% less than

R_{ud} from CAPWAP® (see Rausche, Goble and Likins, [12]). These examples will be discussed further below.

Goble and Rausche [13] described the development of a dynamic testing method whose development began in the 1960s at Case Western Reserve University in Cleveland, Ohio. The so-called Case Method was based on force and velocity measurements taken at the pile top and a simple formula would allow for a dynamic load test that could be evaluated for every hammer blow in real time. The first phase formula considered the pile a rigid body and the static resistance was calculated from the force and acceleration times pile mass at the time of zero velocity. Today this approach is frequently called “unloading point method” in context with Force Pulse (Rapid) Testing (see for example Middendorp, Bermingham and Kuiper [14]). It defines the static soil resistance as the total resistance at the time of zero velocity and, therefore, the dynamic soil resistance as the total resistance minus the static resistance for all times during the dynamic event. The dynamic resistance is normally considered related to velocity only, although it may also contain inertial and thus acceleration dependent components. In this approach, the dynamic resistance component for the total dynamic loading event is, therefore, defined by the soil resistance at the time of zero velocity (i.e., the onset of pile rebound).

Subsequent developments of the formula considered the pile elasticity and searched for a point at which the static resistance, equal to total resistance minus damping resistance, would be a maximum. The damping force was calculated by multiplying the toe velocity, calculated from the top measurements, with a viscous damping factor. This still simple Case Method formula can be easily evaluated in a very short time, i.e., in the time between hammer blows during pile driving, however, it required assumption of a non-dimensionalized damping factor which typically ranged between 0.1 and 1.0 with the lower values appropriate for granular soils. A more rigorous signal matching analysis, matching computed values with the complete measured force and velocity signals, is known as CAPWAP® [12]; it will be referred to as CW in the following. The CW analysis also provided a simulated static load-displacement curve, a resistance distribution estimate and a calibration for the Case Method and its damping factor. The basic CW soil model, like the Smith-viscous formula (Eq. 1b), relates the dynamic resistance component to pile velocity.

As part of the project entitled “Dynamic Studies on the Bearing Capacity of Piles”, the Case researchers performed both static and dynamic tests on a 7 m long, 76 mm diameter model pipe pile driven into a deep deposit of clay and silt. Static testing was done at a Constant Rate of Penetration (CRP), typically at a rate of 0.25 mm/minute. In one case (see Goble et al. [15]) loads were also applied at a much faster rate of 38 mm/minute and then a dynamic test followed at 1.5 m/s. The maximum loads for the fast CRP and the dynamic test were 41% and 81% higher than at the standard test rate, respectively. Figure 2b depicts these three results; their tendency matches those of other tests quite well.

Figures 2a and 2b show that the Smith-viscous damping approach gives a rough approximation to the measured dynamic response, but only in the neighborhood of commonly encountered pile driving velocities. For much higher and lower velocities, the model would not

yield satisfactory results. That may be one reason why signal matching, based on Smith's soil model, can yield rather wide ranging damping factors which are difficult to relate to actual soil properties.

Taylor [5], Dayal and Allen [9], Krieg [10] and others assume a semi-logarithmic relationship between dynamic resistance and velocity. If their semi-log approach would be chosen for pile driving, so that it would be applicable to the varying velocities, $v(t)$, occurring during the dynamic loading event, then the relative total resistance should be written as follows:

$$R_t(t) = R_u [1 + j_{sl} \log_{10}(v(t)/v_{ref})] \quad (6)$$

With, perhaps, $v_{ref} = 10^{-6}$ m/s

Unfortunately, calculating $R_t(t)$ according to Eq. 6 in wave equation simulations and/or signal matching does not yield satisfactory agreement with the highly dampened force and velocity vs. time histories known from measurements. Instead this equation yields for the limited ranges of $v(t)$, observed in pile driving, a nearly constant dynamic resistance and undefined values at zero velocity. It may be possible to use the semi-log approach to determine the appropriate damping factor, j_{sl} , given R_t/R_u and the dynamic velocity, $v(t)$, but the calculated damping forces vs. time should then again be based on the linear viscous approach, e.g., Eq. 1b. This would make sense considering that experiments generally considered single force and velocity values and not a complete time history of an impact event to assess the R_t/R_u ratio.

FACTORS AFFECTING AGREEMENT BETWEEN STATIC AND DYNAMIC TESTING RESULTS

CW calculated capacity values have been compared with static load test results by Likins and Rausche [16] and good agreement has been found, particularly when comparing static and dynamic tests that were performed after similar waiting times to avoid having to deal with different soil setup or relaxation effects. In fact, statistically comparing CW with static loading test capacities did not on average indicate overestimations but, if anything, an underestimating bias. The good agreement of dynamic with static loading test results, even for cohesive soils may be attributed to the fact that the signal matching procedure considers the measured signal over the whole impact event which includes for cohesive soils often high initial soil inertia effects. That fact, in addition to the velocity dependence of damping factors mentioned earlier, would explain why Rausche, Nagy and Likins [17] found that the CW calculated damping factors vary broadly, often with little relationship to soil types, however, they do produce satisfactory static soil resistance agreement with static test results.

Relaxation, Soil Setup and Rate Effect

Due to remolding, pore water pressure changes or other effects, the dynamic movement of a pile through the soil, even at low speeds, tends to reduce the displacement dependent (static) component of the shaft resistance while occasionally increasing its toe resistance. In the latter case,

the loss of the temporary toe resistance gain, called relaxation, can cause serious overestimation of static soil resistance based on end-of-driving (EOD) measurements. Even static tests, performed without sufficient waiting times between pile installation and test, may indicate the only temporarily high resistance. Relaxation may occur on driven piles in very dense fine sands and silts due to dilatancy related negative pore water pressure; more frequently relaxation has been observed on piles driven into clay or silt shales, see for example Morgano and White [18]. In all of these situations, the toe resistance is high during driving but diminishes after driving. Hence, while the EOD resistance is high, a restrike test performed after a sufficiently long waiting time, will indicate the lower resistance values, and since the resistance is only temporarily higher and a restrike test would recognize it, relaxation is not a rate effect and will not be further discussed in this paper.

Soil setup is a reversal of a loss of resistance due to dynamic effects during pile driving or due to the drilling process; it generally occurs to a greater degree in cohesive than granular soil types. During the dynamic load application the shaft resistance loses strength, typically by factors between 1.5 and 10. The static long term resistance, R_u , is therefore the static resistance at the time of the test (for example, based on CW), R_{ud} , times a setup factor, $f_s \geq 1$, which depends on soil type as well as installation history of the pile. It is confusing, however, that in plastic soils which often are subject to soil setup, the rate effect generates an increased R_{ud} . This means that R_{ud} at the same time may be higher than R_u at the time of testing and this effect may be considered by a rate factor, $f_r \leq 1$. The long term static nominal resistance can, therefore, be calculated from the dynamically determined resistance, R_{ud} , using

$$R_u = f_s R_{ud} f_r \quad (7)$$

Test engineers may not be able to recognize the rate effect when a restrike test is done much earlier than the static test. In that situation, the additional setup effect more than compensates for the rate effect. If, however, the R_{ud} is determined from a test that occurred when practically all setup has occurred, then $f_r < 1$, i.e., an R_{ud} greater than R_u , would be recognized.

For Force Pulse (Rapid) Tests using the unloading point method of analysis to determine R_{ud} , the rate factor, f_r , ranges between 0.45 and 1.05 for highly cohesive and granular soil types, respectively, according to Weaver and Rollins [19]. If the signal matching approach is used, as described by Miyasaka [20], then the rate effect problem should be treated as for CW analyzed impact pile tests.

In the Case Method and in signal matching R_{ud} is determined by subtracting from R_t a damping resistance which is damping factor times pile toe velocity and f_r is then assumed to be 1.0. For signal matching with CW f_r is also normally assumed to be unity. However, occasionally signal matching does not automatically compensate for the rate effect, yielding results with $R_{ud} > R_u$ from the static loading test [11].

CW results sometimes suggest that total resistance, R_t , occurring during pile driving is of a magnitude equal to the long term static resistance, R_u , determined by a static test at a later time. This concept would imply that damping plus rate effects together match the soil setup effect.

Unfortunately, this simple idea does not yield sufficiently reliable results according to Rausche, Robinson and Likins [21].

Assessment of the rate factor from soil parameters is obviously severely hampered when, as is typical, only either SPT N-values plus general soil type (grain size) or the cone/sleeve resistance values are known. For routine projects the identification of the rate factor, f_r , must then be based on something as rudimentary as the distinction between plastic and non-plastic soils. The remedy has been, at least for driven piles, to perform statistical comparisons of dynamic with static test results (e.g., Paikowsky [22]) and calculate appropriate resistance factors or factors of safety. In such cases, recognizing the rate effect and reducing the R_{ud} values further would lead to potentially excessive and hence uneconomical factors of safety. When combining static with dynamic loading test results procedures such as described by Rausche [23] may be considered.

Recommended Sets per Blow and Partial Resistance Activation

Insufficient test energy and related underestimation of soil resistance occurs relatively frequently during a dynamic test and complicates the search for the correct rate factor, f_r . It happens when the pile has not moved enough under the impact load, to reach the nominal resistance at all points along the pile. Hannigan et al. [1] recommend enough energy so that sets per blow are at least 2.5 mm per blow for sufficient resistance activation, but also limiting the hammer energy sets per blow not exceeding 12 mm. These limits are reasonable for soils of low plasticity. It appears, however, as shown below, that for plastic soils, the hammer energy should be limited so as not to generate sets per blow exceeding 2 or maybe 2.5 mm to limit rate effects. This limit is reasonable for plastic soils where plunging failure occurs which requires little pile displacement for full resistance activation. Thus underestimation of resistance due to low sets per blow is unlikely while at the same time the moderate set per blow keeps velocities, damping and rate effects at levels that avoid overestimations of static soil resistance.

It also has to be recognized that sensitive soils lose strength after having reached a peak resistance. This implies that during the early blows of a restrike test on a pile in sensitive soils, the resistance along the pile may be in the pre-peak or at the maximum peak state of resistance activation and not in the residual strength state of the soil. Applying additional hammer blows, all with limited sets per blow, helps determination of peak and residual strength in a sensitive soil by the dynamic testing. It is, therefore, always wise to apply more than a single hammer blow to the pile during dynamic load testing.

EXAMPLES

The following examples show static resistance results obtained by standard CW analyses for construction piles drilled or driven in plastic soils. The examples were taken from actual pile installation projects where shaft resistance was predominant and only limited site information was available. The calculated dynamically determined static load vs. displacement curves are also shown after multiplying both load and displacement values by an adjustment factor of 0.70. By reducing both load and displacement, the calculated equivalent stiffness is maintained and since

much of the apparent pile stiffness is governed by the pile properties, this is reasonable for demonstration purposes. Alternatively, the static analysis may be repeated with the reduced resistance values for a more accurate displacement assessment. Admittedly though, it is not known how the soil stiffness values (i.e., static soil resistance divided by soil quakes) should change when adjusting for rate effects.

Auger-Cast-in-Place (ACIP) pile in plastic, soft lake deposits

Rodriguez, Alvarez and Velandia [11], have applied the Krieg [10] approach to improve agreement between dynamic and loading test results for two cases of 500 and 600 mm diameter and 26 and 35 m length ACIP, also called Continuous Flight Auger (CFA), piles. Static loads were applied in the beginning in 490 kN and at higher loads at 294 kN increments; each load increment was held for 5 minutes. The soil profile indicated deep lacustrine clay deposits with water content, liquid limit and plasticity index ranging between, respectively, 50 and 200%, 100 and 280% and 25 and 80. Comparing maximum load values, in the first case, the dynamic test, evaluated by CW, indicated an overestimation of about 18%. In the second case the overestimation was 29%. Based on Eq. 5 and the liquid limit values available for each layer, the CW calculated shear resistance values were reduced by 26 and 29%, respectively (Figure 3a and 3b) resulting in respective differences with the static test of -12 and +2%. The 0.7 multiplier yielded in these cases under-predictions of 17 and 10%. Since underestimations are less of a concern than overestimations, either the liquid limit based adjustment or the 0.7 multiplier appear to have merit for the soft, highly plastic soil types in this study.

Square concrete pile in soft, highly plastic clay

A sacrificial 12.5 m long 235 mm square concrete pile was driven on a construction site in Sweden into a highly plastic soft clay (S_u range between 13 and 20 kPa, I_p range 40 to 50) to a depth of 12 m without reaching a competent bearing layer. Normally in that geology piles are driven to rock. Dynamic tests were conducted during driving and 51 and 189 days later with CW calculated capacities of 255, 347 and 352 kN indicating a soil setup factor of $352/255 = 1.38$. A static test was conducted 183 days after pile installation with 30 kN load increments applied every 15 minutes. The pile reached plunging failure at a load of 260 kN which was 74% of the last restrike CW capacity. Reducing the CW restrike result by multiplication with 0.7 would yield 246 kN and would, therefore, be roughly 5% lower than the static plunging load. It is a co-incidence that in this case the end-of-driving R_{ud} matched the much later performed static test, R_u , quite well. Load-displacement curves for the static, EOD and restrike dynamic results are presented together with the reduced restrike curve in Figure 4.

Open ended pipe pile in high plasticity clay.

A 610 mm diameter pipe pile of 12.7 mm wall thickness and 37 m length, was driven open ended into highly plastic soils (I_p 20 to 80). The final set per blow was 19 mm. According to Hannigan, et al. [1] such a large set per blow would make capacity assessment inaccurate, however, this was of minor concern since high setup effects were expected in these soils. Indeed, the end of driving CW resistance was only 530 kN and, therefore, much less than the expected long term capacity. Three restrike tests, after 15 minutes, 24 hours and 49 days, were performed yielding

respective CW capacities of 740, 2290 and 3410 kN. The apparent setup factor was $3410/530 = 6.4$. The last restrrike test caused pile displacements of 3.8 mm per blow and, therefore, full resistance activation.

A maintained static loading test, performed 37 days after pile installation, applied three cycles of increasing maximum loads (660, 1320, 1985 kN) before reaching 2320 kN with 15.5 mm displacement in a fourth cycle after 27 hours of testing. The pile top then settled an additional 28 mm during a 1 hour time period at loads decreasing to 1720 kN. The maximum static load of 2320 kN was 68% of the 49-day CW result (3410 kN) and, therefore, close to the 70% of CW value on average suggested by the Krieg [10] study. Figure 5 presents load-displacement curves for the static restrrike dynamic test result both unreduced and multiplied by 0.7.

It should be added that the static load decreased to lower values with additional top displacement reaching a residual strength of 1720 kN at 76 mm top displacement. However, comparing the early restrrike result with the large displacement static resistance is not reasonable; instead comparison should have been made with later blows of the restrrike when the cumulative set of the dynamic test has reached an equivalent (in this case 76 mm) top displacement value and when some of the soil setup has been lost.

H-piles in high plasticity shale

Two 16 m long steel HP 254x85 H-piles (HP 10x57) were driven to a depth of 14.3 and 11.6 m and both were dynamically tested during a 10-day restrrike and statically after 4 weeks (Figures 6a and 6b). The static test followed the Quick Test [2] protocol, applying 222 kN loads and holding them for 4 minutes. The soils were soft silt and clay over high plasticity clay and silt shale. In these soils one has to be concerned about relaxation of the end bearing. However, the instrumented second test pile indicated approximately 470 kN end bearing which was in reasonable agreement with the CW calculated end bearing for both piles of 530 kN. Had relaxation been a problem, then the static end bearing should be much lower than the dynamic one. Also the dynamic records should have indicated a resistance increase during the restrrike which they did not. For Piles 1 and 2, respectively, penetrations per blow were 5 and 7 mm at the end of driving and 2.5 and 3.2 mm during the restrrike tests, indicating full resistance activation. The CW capacities were 2340 and 1640 kN while the static load-displacement curves reached peak loads of, respectively, 2000 and 1560 kN reducing to residual resistances of 1780 and 1370 kN; the residual values occurred at pile top displacements similar to those of the static tests and, therefore, the dynamic test results should in this case be compared with the residual values. Figures 6a and 6b show the static, dynamic and 70% of dynamic load-displacement curves. It is unusual that in such soil types the static test displays a higher stiffness than the dynamic test. It is attributed to the much longer waiting time for the static test and associated reduced dynamic softening of the soils.

Compared to the residual capacities, the CW restrrike results in over-predictions of 24 and 20% while the 0.7 reduction factor yields significantly lower results than the static values. However, the piles were driven into a silt shale (unit toe resistances for the 254 mm or 10 inch square plugged pile toe are in the neighborhood of 8 MPa) and the end bearing, therefore, should not be subjected to the 0.7 factor. Subtracting 500 kN end bearing and adjusting the remaining

shaft resistance by the 0.7 factor leads to adjusted CW capacities of $(2340 - 500) \cdot 0.7 + 500 = 1788$ and $(1640 - 500) \cdot 0.7 + 500 = 1298$ kN; the ratios of these reduced capacities vs. the unreduced CW results are 76 and 79%, respectively. Although not totally correct, these two factors were again applied to the respective loads and displacements. The resulting load-displacement curves were included in Figures 6a and 6b demonstrating reasonable agreement with the static curve.

CONCLUSIONS AND RECOMMENDATIONS

Rate effects making the dynamic strength of plastic soils almost 3 times higher than the static strength have been described in the general geotechnical literature. In general, however, dynamic testing, both of the High Strain Test or Force Pulse Test type, exhibit lower rate effects.

Signal matching using the Smith-viscous damping approach is effective in identifying most of these effects, because the whole dynamic loading event is used to identify velocity dependent resistance components. Loading velocities occurring during pile driving and dynamic pile testing occur within narrow bounds; this is another reason why relatively consistent static soil resistance results are achieved by signal matching analyses. However, Smith-viscous damping factors, determined by signal matching, may be highly variable, because they are as much dependent on loading velocity and inertia effects as they are a function of soil grain size.

Damping models based on either Coyle and Gibson [8] or other semi-logarithmic methods are not successful for signal matching analyses. However, they indicate that damping factors of the linear approach are highly dependent on penetration velocity.

Subtracting from the total measured resistance all velocity and acceleration dependent components identifies a static resistance which includes low or no rate effects in most soil types. An exception are plastic soils where the apparent resistance can be more significant. However, signal matching overestimations of static soil resistance due to rate effects can be expected to be within the range of overestimations that traditional factors of safety or statistical analysis based resistance factors cover. In any event, it is always wise to back-up with static loading tests the dynamic testing results on sites where no prior experience exists. Alternatively, for piles in highly plastic soils an adjustment as proposed by Rodriguez, Alvarez and Velandia [11] promises improved results. If the Liquid Limit which this method requires is not known then a 0.70 rate factor may be applied to the shaft resistance results determined by signal matching for the highly plastic soil layers. This adjustment of signal matching results may lead to moderate underestimations of the static soil resistance and it is therefore prudent to review whether or not the required factor of safety already covers the uncertainty of the dynamic test or an uneconomical foundation may result. The adjustment should only be done to the plastic soil resistance component. For example, a pile extending to a non-plastic bearing layer would only require an adjustment of the resistance contribution along the shaft. On the other hand, for a pile totally embedded in plastic soils, the adjustment should be made to both shaft resistance and end bearing.

Unusually high toe resistance in cohesive soils may be an indication of either a relaxation issue or a rate effect. If the dynamic test static toe resistance does not increase during the restrike test then it may be assumed that no relaxation exists. If in these cases the soil type at the pile toe is well known and its strength is well defined, then it may be appropriate to apply either a rate reduction factor or use a statically calculated value instead of the unusually high, dynamically determined toe resistance.

For High Strain testing of piles in highly plastic soils it is recommended to limit the set per blow to at most 2.5 mm, but also to generate enough penetration, by applying sufficiently many impacts, for an assessment of residual strength. It is always recommended to test with more than one hammer blow so that residual strength effects, loss of soil setup, relaxation and full soil resistance activation can be recognized.

Rate effects in Force Pulse (Rapid) Tests, analyzed using the unloading point method, may be addressed by, for example, recommendations of Weaver and Rollins [19] or other relevant publications. Alternatively, given a sufficiently short rise time (typically less than 5 ms) a signal matching analysis can be successful.

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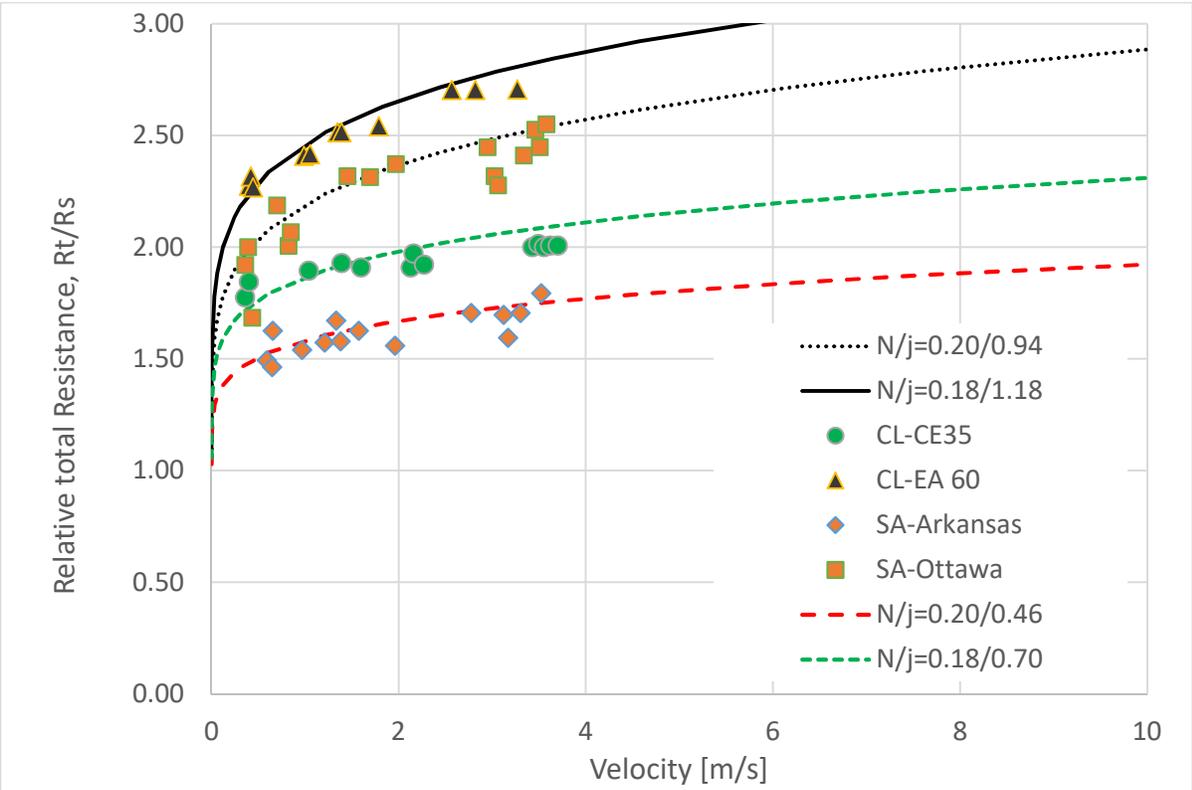


Figure 1. Coyle-Gibson: Highest and lowest sand (SA-Arkansas, SA-Ottawa) and clay (CL-EA 60, CL-CE 35) results and Eq. 2 trendlines for recommended exponents, N , and sample specific damping factors, j_{s-c} , on a linear velocity scale

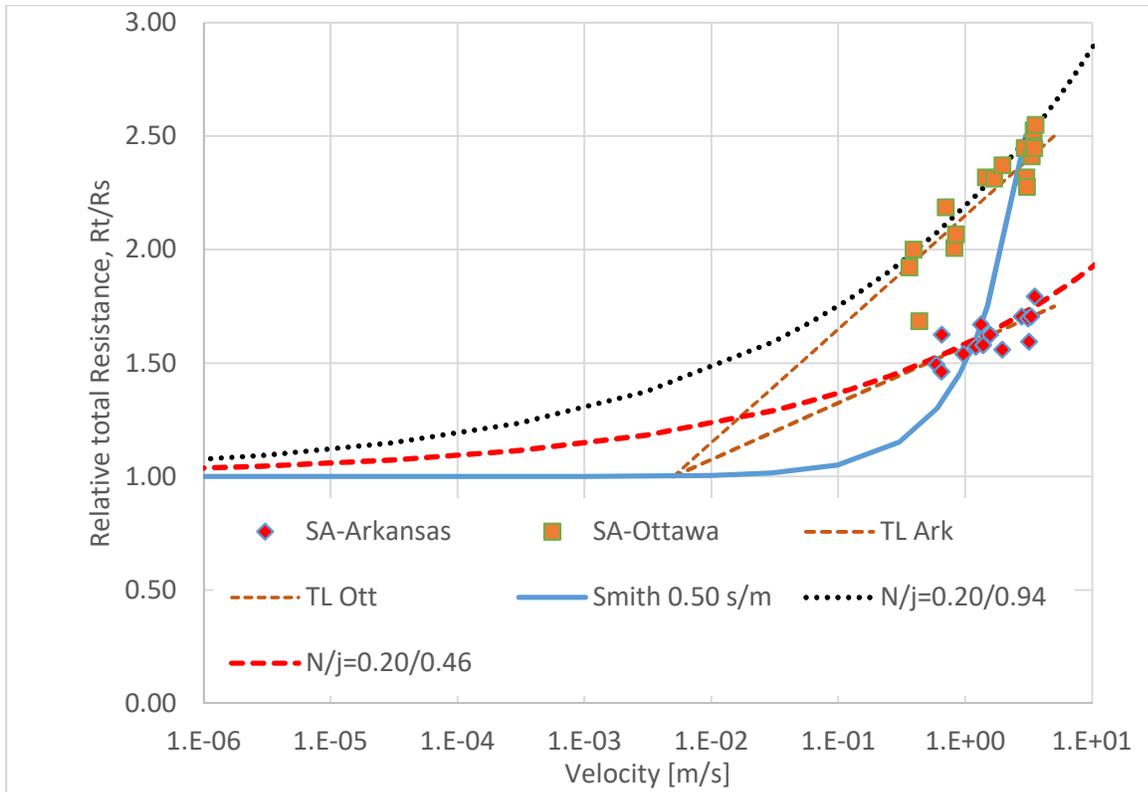


Figure 2a. Coyle-Gibson results for the sands of Fig. 1 on a semi-log scale together with linear trendlines in the semi-log scale

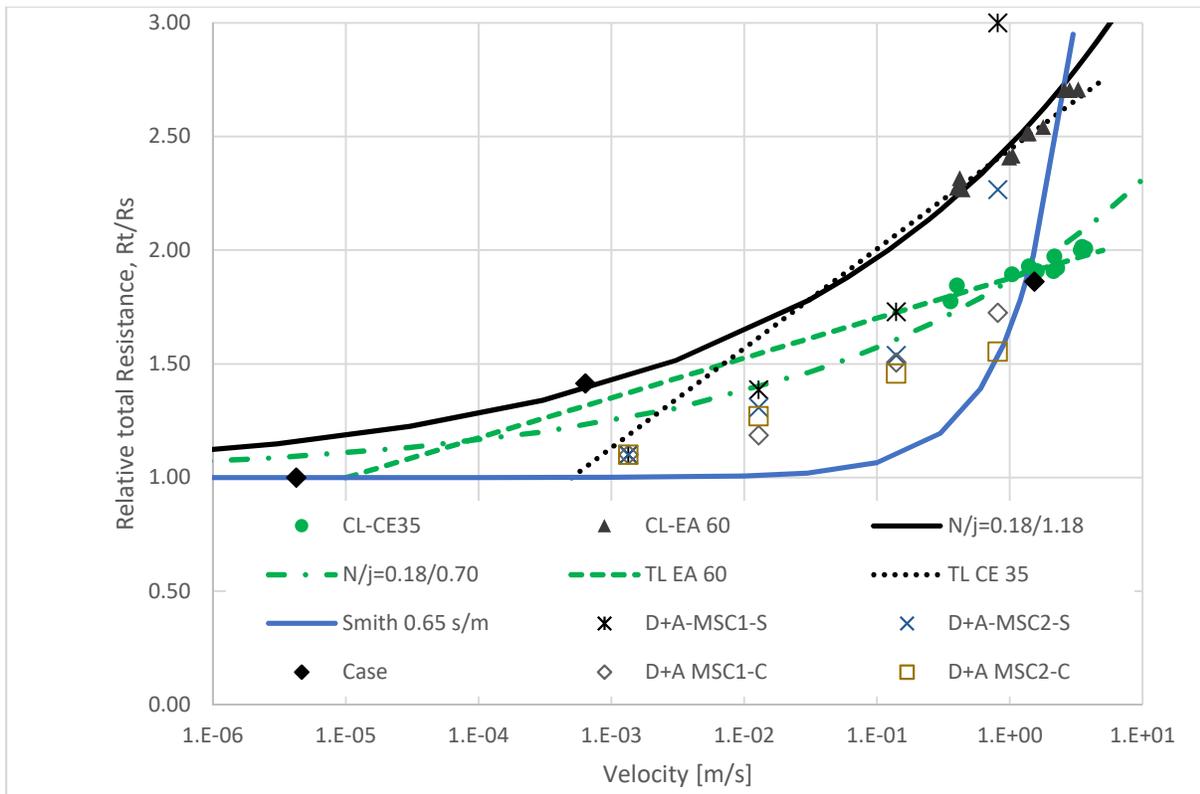


Figure 2b. Coyle-Gibson results for the clays of Fig. 1 on a semi-log scale together with linear trendlines in the semi-log scale; also shown is the Smith shaft damping relationship ($\gamma_s=0.65$ s/m) together with selected Dayal & Allen and Case results.

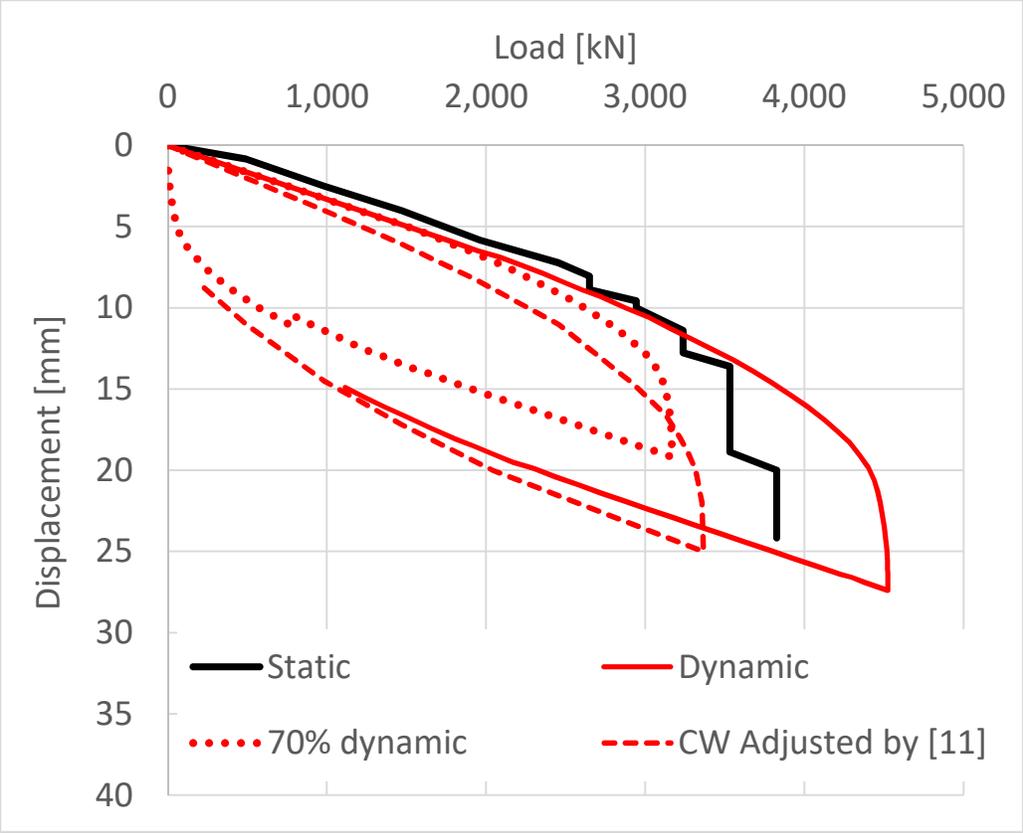


Figure 3a. Static, CW calculated and adjusted load vs. displacement curves for 500 mm diameter CFA Piles after [11]

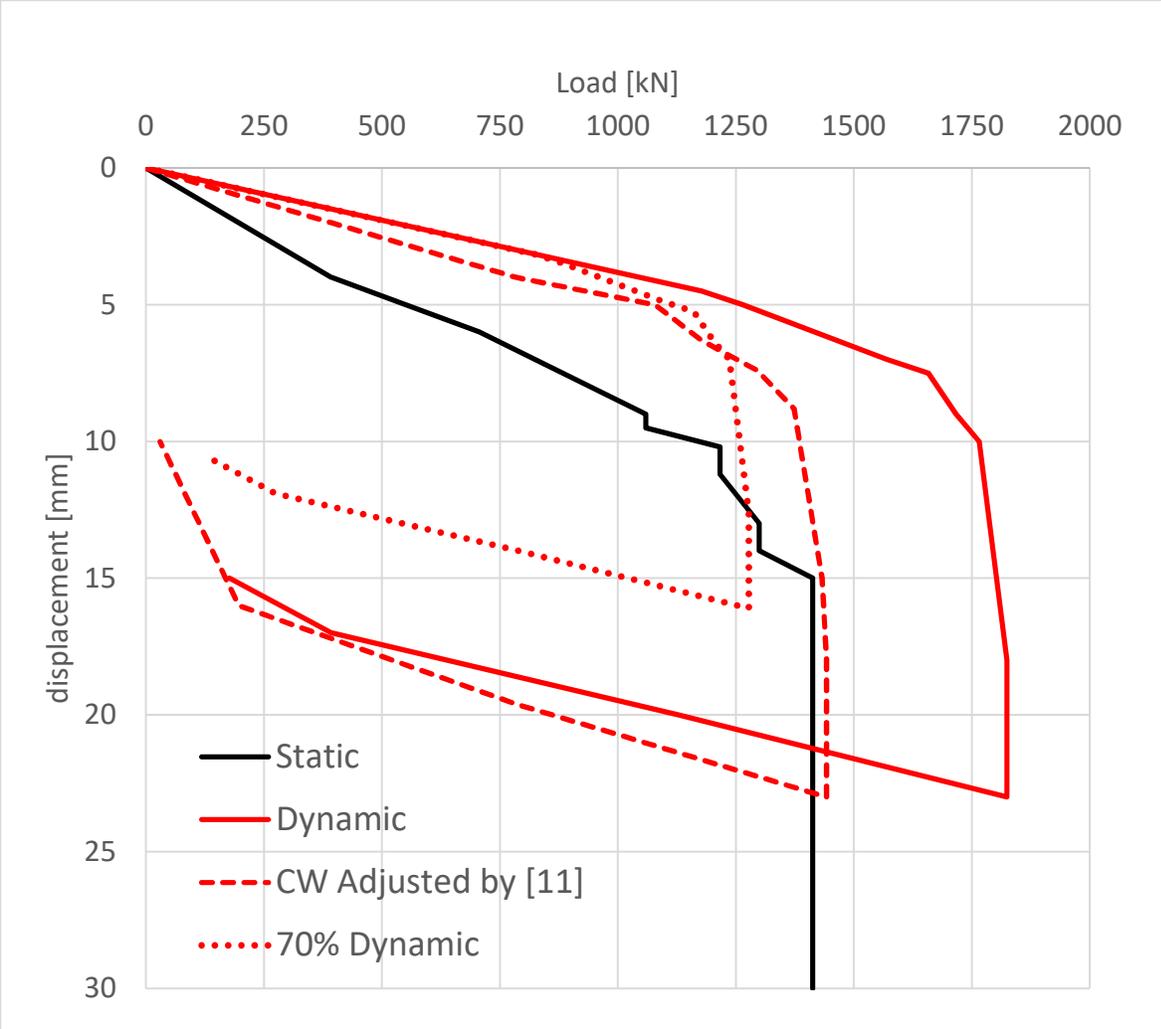


Figure 3b. Static, CW calculated and adjusted load vs. displacement curves for 600 mm diameter CFA Piles after [11].

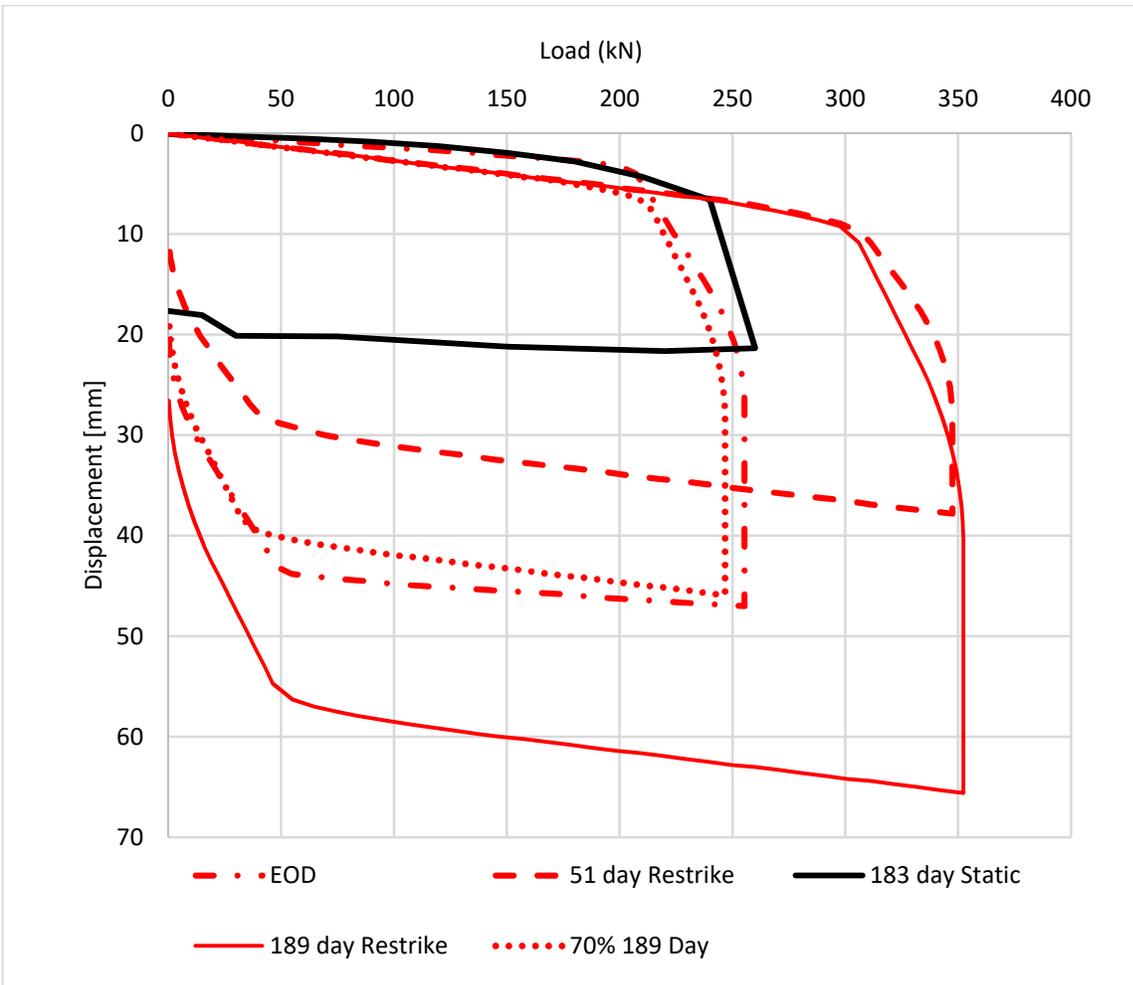


Figure 4. Static, dynamic and adjusted dynamic load-displacement curves for concrete pile

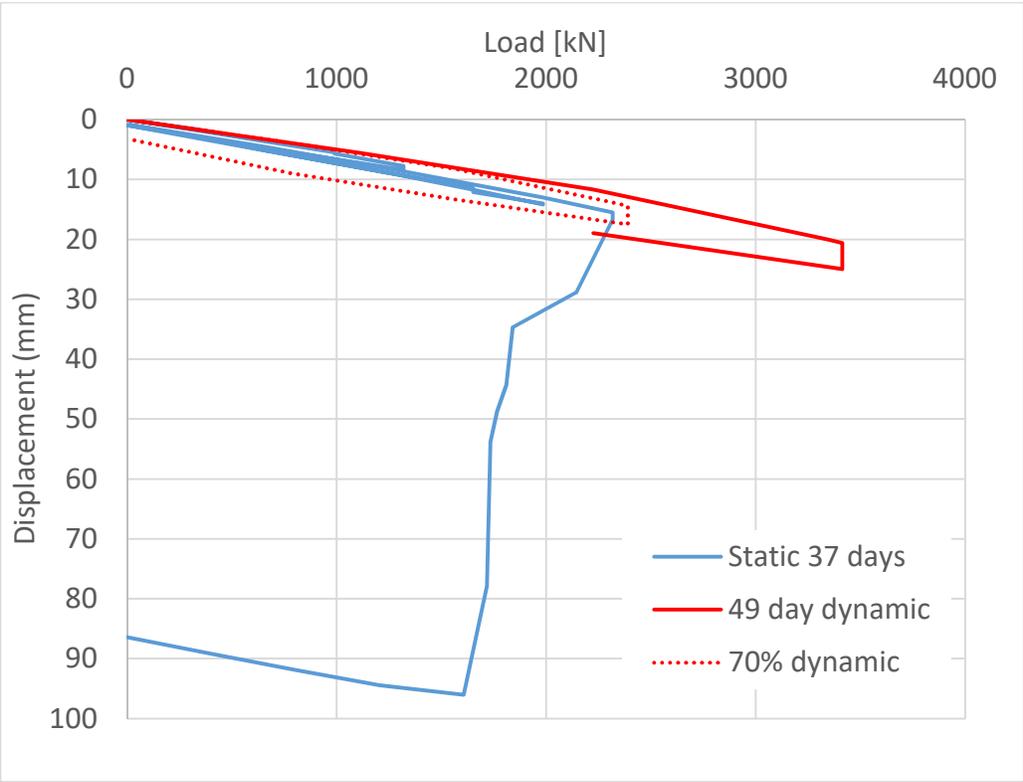


Figure 5. Static, dynamic and adjusted dynamic load-displacement curves for open ended pipe pile

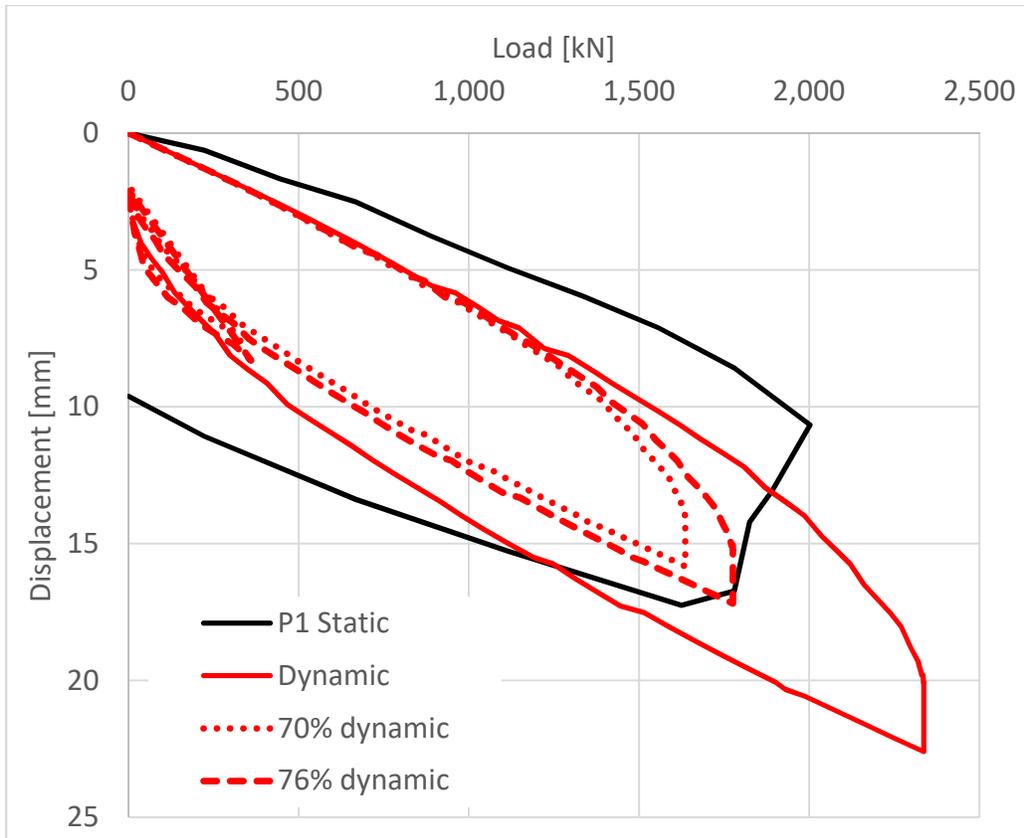


Figure 6a. Load-displacement curves for H-pile P1 driven into highly plastic clay and silt shale to 14.3 m depth

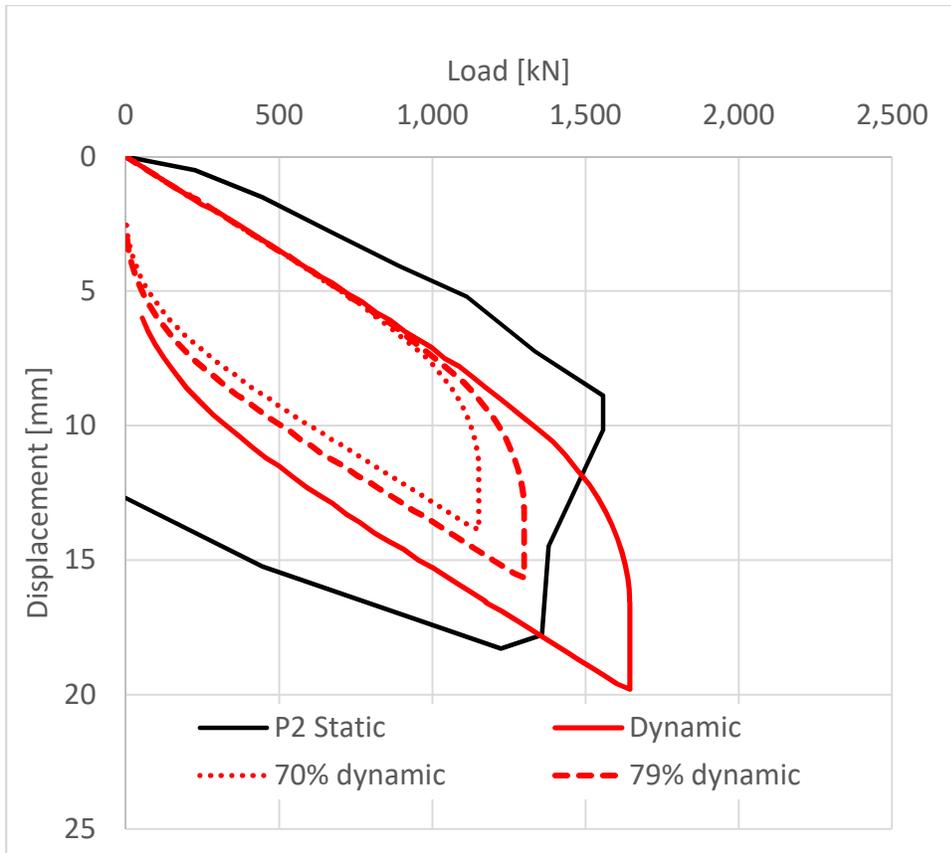


Figure 6b. Load-displacement curves for H-pile P2 driven into highly plastic clay and silt shale to 11.6 m depth.