

Analysis of Post-Installation Dynamic Load Test Data for Capacity Evaluation of Deep Foundations

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ABSTRACT: Construction of deep foundations causes changes to the original geotechnical condition of the surrounding soils and rock. The nature, extent, and effects of these alterations, and subsequent time-dependent modifications, on the long-term load bearing capacity of the foundation elements are mainly functions of the characteristics of the supporting geo-materials and type of installation method. Post-installation Dynamic Load Testing (DLT) is commonly used for capacity evaluation of driven and drilled deep foundation elements and is performed under impacts of the pile driving hammer or a special large drop weight loading device. Use of a large drop weight loading device is usually appropriate for cast-in-place shafts and driven piles in high-setup soils to activate the full bearing capacity. The DLT procedure consists of applying a specific number of blows (typically 2 to 10 blows) and measuring pile/shaft strain and motion time-records, and the resulting displacement. This paper presents discussions on the analysis of post-installation DLT data for capacity evaluation of driven and drilled deep foundations. Technical aspects related to the proper test record selection for analysis considering pile/shaft set per blow, changes of capacity from blow to blow with subsequent impacts, energy levels, and total penetration during the test are delineated. Analytical procedure and engineering evaluation of results are discussed with recommendations for practical use. Test records from actual case histories are utilized to demonstrate data characteristics and illustrate proposed numerical methods and recommended analytical procedures for rational evaluation of DLT results for engineering use.

KEY WORDS: Bearing capacity , CAPWAP, drop weight, dynamic testing, end bearing, hammer, Pile Driving Analyzer, piles, residual stresses , setup, shafts, skin friction, soil resistance.

INTRODUCTION

Deep foundations are generally categorized into driven piles, augercast piles, and drilled shafts (also called bored piles, or caissons), each with distinctive features and characteristics. Driven piles are manufactured elements usually made of wood, concrete, or steel, uniform or tapered profile, uniform or composite material, low or high soil displacement, and are installed with either impact or vibratory hammers. Augercast piles are constructed in-place by pressure grouting utilizing a hollow stem continuous-flight auger or by a displacement method whereby no soil is removed from the ground. Drilled shafts may be constructed in a dry hole or by using slurry, may utilize temporary or permanent steel casing, or no casing at all. Both augered and drilled foundations usually incorporate a steel reinforcement cage or individual bars.

Construction of deep foundations alters the original geotechnical condition of the surrounding soils and rock. The nature, extent, and effects of these (temporary, or permanent) alterations and subsequent time-dependent modifications on the “long-term” load bearing capacity of the foundation elements are mainly functions of the geotechnical properties of the supporting geo-materials, foundation type’s physical characteristics and installation mechanism (Coduto, 2001). Soil aging also has an effect (Schmertmann, 1991).

Construction of cast-in-place piles and drilled shafts may in some cases produce “soft-toe” effects due to soil loosening or unclean bottom resulting from sedimentation of suspended materials. Post-installation grouting may be used in special cases to enhance the end bearing resistance. The development of shaft resistance in skin friction along the length of augercast piles is influenced by soil type, the method of installation (i.e., soil displacement or non-displacement method, degree of over-rotation which can “mine” soils and reduce effective stresses, etc.), grout pressure and actual grout volume used for each depth increment.

Impact pile driving generally has desirable densification effects in non-cohesive soils resulting in enhanced pile capacity. Pile driving effects on cohesive soils are mainly remolding and development of excess pore water pressure, which result in a temporary change in the state of stress around the pile. Vibratory hammers can generate sometimes unpredicted reduced or increased soil density and strength.

Time-dependent changes of geotechnical conditions often have beneficial effects by increasing pile capacity over time; this phenomenon is commonly called “soil setup” or “pile freeze”. Modern progressive engineering practice takes setup into account to optimize pile design and minimize foundation costs (Komurka, 2004). Pile driving into calcareous soils or stiff clays may produce permanent negative effects on the soils and pile-soil interaction that may not be recovered with time. Piles driven into weathered shale or saturated dense fine-grained soils may lose end-bearing capacity over time; this unfavorable effect is commonly called “relaxation”.

The performance of a deep foundation is a function of its structural strength and integrity, geotechnical strength and deformation properties of the supporting soil or rock, pile/shaft-soil/rock interaction characteristics, and nature and magnitude of loads. Field assessment of a deep foundation element’s response to applied load and evaluation of its ultimate load bearing capacity is typically made with static or dynamic loading tests. Static load testing is performed using conventional

cumbersome loading methods, or by utilizing modern innovative procedures (Schmertmann, 1993). Dynamic Load Testing (DLT) is conveniently performed under the impacts of the pile driving hammer or a special-purpose large drop weight system. Static or dynamic load testing of cast in-situ shafts require that the grout or concrete has gained sufficient strength (typically at least five days after casting) to allow the geotechnical capacity of the foundation element to be evaluated. Dynamic load testing of driven piles may be performed during the pile driving process, and/or during restrike some time following initial installation (possibly ranging from a few hours for piles driven into sands to days or weeks for piles driven into clays or shale) for assessment of “long-term” pile load bearing capacity. However, project construction schedule requirements may restrain the maximum amount of waiting time allowed for post-installation testing. In all cases, both static and dynamic testing procedures provide information regarding the pile or shaft load bearing capacity at the time of testing.

The post-installation dynamic load testing procedure is typically performed under impacts of the pile driving hammer or a special-purpose large drop weight for the dynamic loading device. The latter is always needed for cast in-situ shafts or for driven piles in high-setup soils which sometimes require more energy to activate the full bearing capacity than available from the original pile driver. Testing typically consists of applying a specific number of blows (typically 2 to 10 blows) and measuring pile/shaft strain and motion time-records, and the resulting permanent set.

DYNAMIC LOAD TESTING

Modern dynamic testing methods of deep foundations are based on one-dimensional stresswave propagation theory and the measurement of force and velocity records at the pile/shaft top under impacts of the pile driving hammer, or a special-purpose drop weight.

Testing is routinely performed on driven piles and cast-in-place shafts (Hussein and Likins, 1995). For post-construction testing, the installation pile-driving hammer may be used during the restrike, provided it can mobilize sufficient capacity at the time of restrike. If not, such as in high setup situations which then result in “refusal” conditions, a larger hammer should be used to activate the added resistance. For testing of cast-in-place shafts, a drop hammer having a minimum weight equal to 1 to 2% of the ultimate pile capacity (or the desired proof load) will be needed for the test. Typically, two to four each reusable strain transducers and accelerometers are affixed around the perimeter of the pile or shaft at a location of approximately one to two diameters below its top. Field data acquisition and processing are made with specialty equipment such as the Pile Driving Analyzer[®] (PDA) system according to the Case Method (Rausche et al., 1985). Post-installation restrike testing consists of dynamic measurements under typically 2 to 10 blows. The pile or shaft net permanent set under each blow is also independently measured. Dynamic test records are analyzed according to the CAPWAP[®] Method which is a computerized system identification numerical process employing signal matching based on stresswave propagation principles for determining soil/rock resistance parameters (magnitude and stiffness) acting along the shaft length in skin friction and under the toe in end

bearing (Rausche et al., 1972). The analysis results include a load-movement graph simulating a static loading test. Comparisons between CAPWAP analyses and full-scale static load test results have shown excellent correlations (Likins and Rausche, 2004).

Since testing is done under a number of hammer blows, the data consists of as many test records. In many cases, the data include consistent signal features and one CAPWAP analysis is performed with records obtained under a typical blow representative of the whole test sequence (Seidel and Rausche, 1984). However, there are situations where analyzing only one test record would not be sufficient to fully represent the entire testing process, or the whole geotechnical circumstances related to the pile or shaft's ultimate load bearing capacity. For example, restrike testing using a diesel hammer may produce inconsistent records due to the variability of hammer stroke heights, and therefore energy levels, from blow to blow at the beginning of the restrike process. Sometimes DLT is performed by purposely varying the hammer energy by systematically increasing the drop height of the hammer to obtain dynamic data and set under successive test blows (Beim and Aoki, 1996).

The level of energy delivered to the pile or shaft top, along with set per blow, are important parameters that must be considered for the proper evaluation of the DLT results. A commonly quoted rule of thumb states that a test blow, producing a permanent pile set less than 2.5 mm may not measure the ultimate capacity, but only a maximum mobilized resistance or a lower bound pile bearing capacity. Such a test would still have value as a proof test if the mobilized capacity exceeds the required value. The 2.5 mm set value corresponds to a blow count of 400 blows/m (or 120 blows/ft) which is frequently quoted as a refusal blow count condition. Thus, at that driving resistance level uncertainty increases about complete resistance activation.

Incomplete resistance activation during a test blow with insufficient energy creates a problem when several low energy impacts cause soil setup resistance to be partially lost. In this case no single record indicates the total pile capacity. Superposition of results obtained from analyzing data under individual blows of a restrike test, or from an end of driving and restrike tests combination, attempts to correct this problem (Stevens, 2000; Hussein et al., 2002). Combining the results from multiple tests and/or blows has also been used to predict the long-term pile capacity in soils with time-dependent characteristics (Preim et al., 1989; Hussein et al., 2004; Rausche et al., 1996; Hussein et al. 2006). Superposition, however, is not a simple matter and, in some cases, the potential for overprediction exists. The present paper suggests guidance on test interpretation that may reduce the need for superposition.

THEORY OF RESISTANCE ACTIVATION

Static loading of a pile first causes relatively high displacements and full resistance activation near the pile top. With continued loading, resistance progressively activates along the pile. Once all shaft resistance and end bearing has been activated, both top and toe experience the same incremental displacements. In sensitive soils, after peak resistance has been reached, large pile displacements reduce resistance values along the pile so that the total peak resistance from all points along the pile is never simultaneously activated. This effect is sometimes called a progressive failure.

Dynamic loading first causes high downward motions and resistance development near the pile top, and short time periods later downward motions at lower and lower pile sections. The time between impact and the beginning of the pile toe movement can be determined by the wave travel; it depends on the speed of the stresswave which is a function of only the pile's elastic modulus and specific density. The downward motions generate pile displacement that activates static resistance forces. The soil also reacts with dynamic resistance forces which depend on the velocity of the pile particles and soil type.

Full static resistance activation requires that the pile displaces a distance necessary to reach soil plastification. Under ideal conditions this is an elastic deformation which is often called the "quake" in the pile dynamic literature (Smith, 1960). Figure 1 shows a typical two-load-cycle static test result together with the simulated load-movement plot from a dynamic test. This example was taken from a test series conducted at Apalachicola River and Bay Bridge in Florida in 1986. The dynamic restrike test on this voided 760 mm square precast, prestressed concrete pile followed the static test after a 5 day waiting period. The restrike permanent set per blow was 1.25 mm. Using the radiation damping soil model option, the CAPWAP program produced a good capacity match although it indicated a somewhat higher stiffness which could, at least in part, be due to the difference between the static and dynamic concrete elastic modulus.

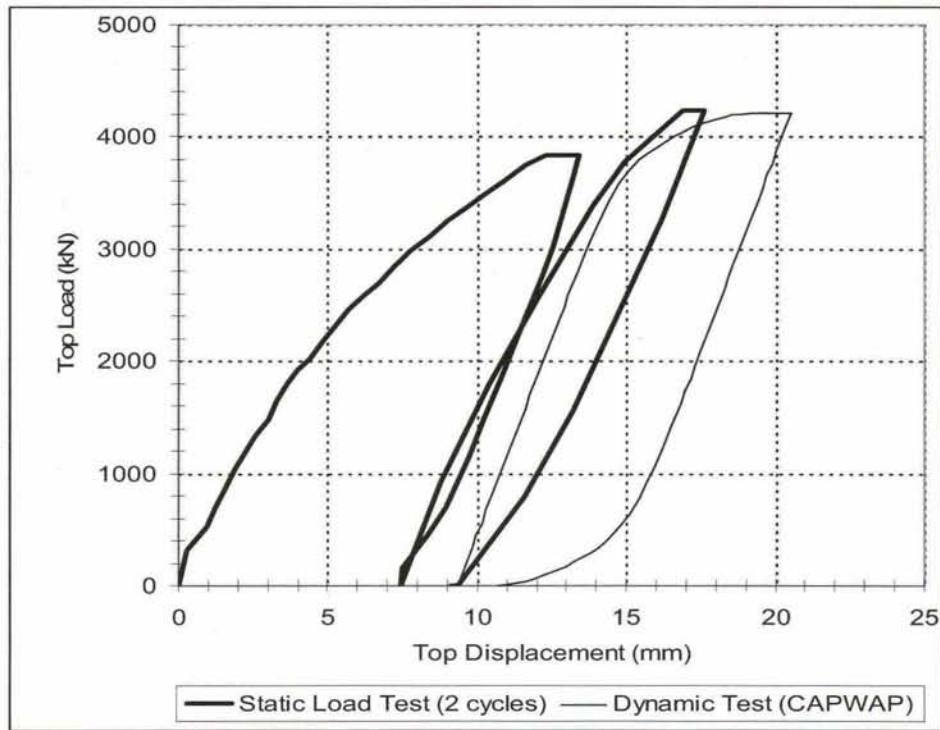


FIG. 1: Example of a two-cycle static test with subsequent dynamic

As is frequently observed, Figure 1 shows a first load cycle stiffness which is lower than that of the second load cycle. A similar behavior can be observed when applying several dynamic loads with successive hammer impacts. In either the static or dynamic case, a stiffened response can normally be observed under subsequent load cycles. Possible explanations include (a) residual stresses, (b) soil densification, (c) the gain of strength associated with increased effective stresses, and (d) possibly others, some of which are subsequently discussed.

Residual Stress Effects

Consider for the purpose of illustration the following case, observed by the authors. The first hammer blow of a dynamic restrike test series on long, flexible piles with high shaft resistance produced a 5 mm set while subsequent impacts only produced 1 or 2 mm sets. In this case soil setup generated most of the shaft resistance with time after pile installation. Thus, the piles experienced only negligible residual stresses during the installation process and practically were at a near zero residual stress state in the beginning of the restrike test. The first impact of the restrike test compressed the pile elastically; during rebound, the shaft resistance prevented the pile from fully expanding to a zero stress condition which created the high permanent top set. In this case it is possible and likely that the first impact did not move the pile toe, because the energy was used for pile compression. Later load cycles expended less energy on pile compression and therefore produced more permanent pile sets at both top and bottom. Clearly, residual stresses of a pile that was not subjected to high residual stresses during installation achieves a much larger top set under the first load cycle or restrike blows than later load cycles or impacts. The permanent pile set of the first blow is then not an indication that failure has been achieved in the test. Piles with high shaft resistance present already during installation would have a stiffer behavior during the load test because the pile would be precompressed by the residual stress effects. Residual stresses in the pile complicate the determination of the resistance distribution in either static or dynamic test; however, the total capacity activation is achieved under similar displacement conditions in either case.

Soil Densification

Once the pile toe begins to move, depending on the soil type and the type of construction, increasing toe displacements cause densification and thus increased capacities. In a driven pile this densification often has already taken place due to the pile installation and possibly due to the subsequent installation of neighboring piles. In a drilled shaft the soil at the bottom may be in a relaxed state during and after construction and large displacements may be required before the full capacity of the shaft can be activated. The allowable pile toe penetration, and therefore densification, under a static loading is a matter of judgment. Clearly, there is a vast difference between a driven displacement pile and a drilled shaft with the former displaying a much stiffer behavior. Both static and dynamic tests exhibit the same effects. Densification together with residual stress effects often makes the comparison between tests difficult: regardless of the order of static and dynamic tests for

correlation; the earlier test densifies and prestresses the soil yielding a stiffer response for the later test (or later impact response).

Pile Toe Resistance

In granular soils, the pile displacements during static loading increase the effective stresses between the soil particles and thus allow for increased toe resistance forces. As a result, failure at the toe may not be clearly pronounced and, in fact, the toe resistance may increase significantly with increasingly large toe displacements during a static load test. During Dynamic Load Testing the toe displacements for each impact are generally relatively limited and the cumulative effect of several blows is necessary to duplicate this effect of increased toe pressures. However, repeated dynamic loadings can only produce these similar toe resistance gain effects if sufficient shaft resistance maintains high toe pressures between hammer blows thereby allowing for a build up of a high residual toe resistance. If the pile toe is allowed to unload (rebound) to low resistance values between hammer blows, pressures at the pile toe will not show the same increase that either static tests or dynamic tests with residual stresses can generate.

Time Effects

It is well known that soil setup (Komurka, 2004) and relaxation (Morgano and White, 2004; Thompson and Thompson, 1983) effects cause time dependent changes of static soil resistance for driven piles. Load testing, either dynamic or static, therefore has to be performed after a sufficiently long waiting period following pile installation. Multiple tests with various wait times can establish the capacity change trend with time; dynamic tests on production piles after only short waits can be used to confirm pile acceptance if they follow similar trends. Load testing can reverse some of the time dependent capacity gains or losses, particularly if the tests cause large deformations and associated pore water changes. In relatively sensitive soils, setup gains may be lost and cause progressive failure (as discussed above) such that the full setup capacity is never apparent in either the static or dynamic test. On the other hand, pile toe relaxation may be reversed (resistance increases again) if testing produces sufficiently large sets.

Creep

Under sustained loads near failure, pile settlements often increase because of creep effects in the soil. Dynamic testing obviously cannot determine creep and even most static load tests are similarly limited. In special cases, the authors have observed a constant rate of creep under maintained static loading for an entire month. Historically, creep effects occurring during a pile's service life have been limited by maintaining working loads at levels below one half of the failure load.

FAILURE CRITERIA

Static load test capacity can be calculated from pile top load-movement curves according to a variety of criteria (Fellenius, 1975; Fellenius, 2001) to arrive at a clear interpretation of the “ultimate capacity” of the deep foundation element. One generally accepted method is referred to as the “Offset” or “Davisson” criterion. It suggests that failure of a pile occurs when the pile toe has achieved a permanent set which is 4 mm plus the pile toe diameter or width, D , divided by 120. The corresponding pile top set is calculated as though the pile would support all load in end bearing. The Federal Highway Administration recommends application of the Offset Criterion (which is among the more conservative definitions) for driven piles of less than 900 mm diameter and an offset of $D/30$ for larger sizes (Hannigan et al., 2006). For drilled shafts and augercast piles, the $D/30$ offset criterion is also reasonable. Other criteria merely consider the elastic plus plastic pile top settlement. Duzceer and Saglamer (2002) demonstrated that the definition of failure by various methods can result in different failure loads for the same static test by a factor of two.

Proposed Failure Criterion for Dynamic Tests

The foregoing remarks show that failure is not easily defined (neither for the static nor the dynamic tests cases) because of the variety of possible pile stress states and soil failure modes. In particular, possible underprediction due to both lack of capacity activation and loss of soil setup, and potential overprediction due to relaxation, have to be considered. The following failure criterion for dynamic tests is recommended to address these concerns:

1. Most importantly, the pile should be tested after a sufficient waiting time after construction for time-dependent soil strength changes to take effect.
2. During the dynamic test, the pile toe should achieve a total (elastic plus cumulative permanent) displacement corresponding to a reasonable failure criterion. The set should not be too large or soil setup would be lost and it should not be too small for eliminating sensitive capacity gains that would be quickly lost by progressive failure. Also the toe set should be large enough for soil densification and increased stress effects to take place. The authors recommend a total toe displacement of $D/60$ for failure which is consistent with the toe quake recommendation of the GRLWEAP wave equation analysis program (PDI, 2005). Further discussions will follow in the case studies below.
3. To be comparable to service piles, a similar “virgin” pile (e.g., “sister pile”) should be tested and evaluated so that capacities should not be affected by some preloading by either a previous static or dynamic load test that would generate residual stresses or densification and stiffen the pile response, particularly for drilled or augered piles.
4. For reasons of conservatism and because of the generally unknown residual stress state, the pile bearing capacity usually should not be based on the first

restrike test record. The first initial loading may include resistance components which could easily be lost when high static or dynamic deflections are applied, or cause changes in the residual stress state. Instead, we recommend applying a first blow with a relatively low energy which in all likelihood will not activate the full resistance. This is very important for piles that have an unexpectedly low capacity; the initial low impact will then result in a large permanent set and already reveal that low capacity with greater reliability than the higher energy impacts (which then need not be applied).

5. The results from the first dynamic test impact should be included in the calculated load-set curve, particularly for drilled shaft tests.
6. Where the geotechnical capacity exceeds the structural strength of the pile (e.g., piles founded in rock), the pile should only be loaded to a level sufficient to demonstrate that the required capacity has been activated to protect the pile integrity

When calculating the maximum toe movement in a restrike test, the permanent set of preceding impact loadings should be added. Dynamic test records of pile top force, F , and velocity, v , provide the necessary information for calculating the pile toe displacement, u_{toe} , and thus its maximum value from top force and velocity measurements. This can be done in closed form as follows:

$$u_{toe} = \int \{(W_{d1} - W_{u2}) / Z\} dt \quad (1)$$

where $W_{d1} = \frac{1}{2} (F(t_1) + v(t_1)Z)$ and $W_{u2} = \frac{1}{2} (F(t_2) - v(t_2)Z)$ are the downward wave at time 1 and upward wave at time 2, respectively. Time t_2 occurs $2L/c$ after t_1 with $2L/c$ being equal to twice the pile length divided by the pile material's stresswave speed. The pile impedance, Z , is the product of pile cross sectional area, wave speed and material density. Alternatively, instead of calculating in closed form, CAPWAP analysis may be used to determine the maximum pile toe displacement (see Figure 2 in Case 1 below).

The dynamic pile toe displacement may or may not be associated with a particular permanent set. Thus a soil that allows for a large elastic set, i.e., quake, at the bottom could reach the failure displacement prior to achieving a permanent set. When considering the pile top deflection, the elastic pile compression would be additional to the pile toe displacement. Considering the completely different construction and loading history of driven piles and drilled shafts (the former with residual stresses and the latter with a relaxed soil at the toe), it would be reasonable to apply different failure criteria to both pile types. However, the proposed criterion and test evaluation procedure (i.e., items 1 to 6 above) appear to be reasonable for both types of pile as demonstrated by several case studies discussed below.

CASE STUDIES

Case 1: Steel H-Pile A

An H-pile 356x109 (14x73) was driven into silts and clays with N values typically less than 15 at depths less than 30 m, underlain by clays and silts with N values of 40 to 100 to the pile termination depth of 45 m. The end of initial driving blow count was 125 blows per meter. Following a waiting period of 21 days, the pile was tested during restrike using a Delmag D30-32 impact hammer with strokes as high as 2.8 m for an early blow at 5 mm set per blow, decreasing to 2.2 m stroke for later blows with up to 15 mm set per blow. A static load test (SLT) was performed 48 days after pile installation. CAPWAP analyses were performed with dynamic records obtained under several blows (i.e., blows 1 through 4, and representative later blows 8 and 25). The static load test results predicted by CAPWAP, including both top and bottom loads vs. set, are summarized sequentially in Figure 2. Blows 1 through 4 are shown cyclically, offset by the preceding cumulative sets, while blows 8 and 25 follow with gaps representing the intervening blows' corresponding sets. The CAPWAP produced pile-top load-movement corresponds very well to the measured static load test curve. The toe loading was calculated as a standard part of the CAPWAP analysis. Obviously the toe loading for this H pile is relatively low and the pile is basically a friction pile. The failure toe displacement for a 356 mm pile would be 6 mm which is exceeded by the toe maximum displacement of the second impact. The second impact therefore yields the dynamically calculated capacity (2051 kN) which correlates well with the static test result.

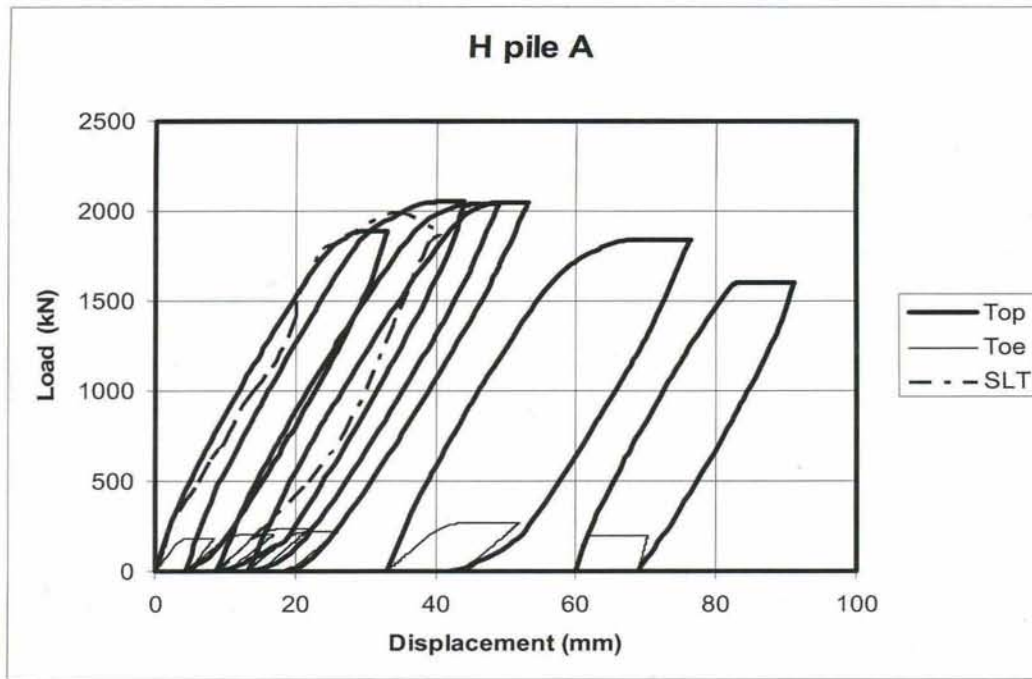


FIG. 2: Static load test results for H-pile A from CAPWAP and SLT

The shaft resistance distributions determined by CAPWAP analyses for several blows are shown in Figure 3. They clearly indicate the higher resistance in the lower, stiffer materials below 30 m. and the reductions of shaft resistance for later blows. Clearly, analysis of one of the later blows would under predict the shaft resistance. It is reasonable to expect that, after completion of the restrike, the capacity will again increase as soil setup regenerates the long-term service strength of the soil.

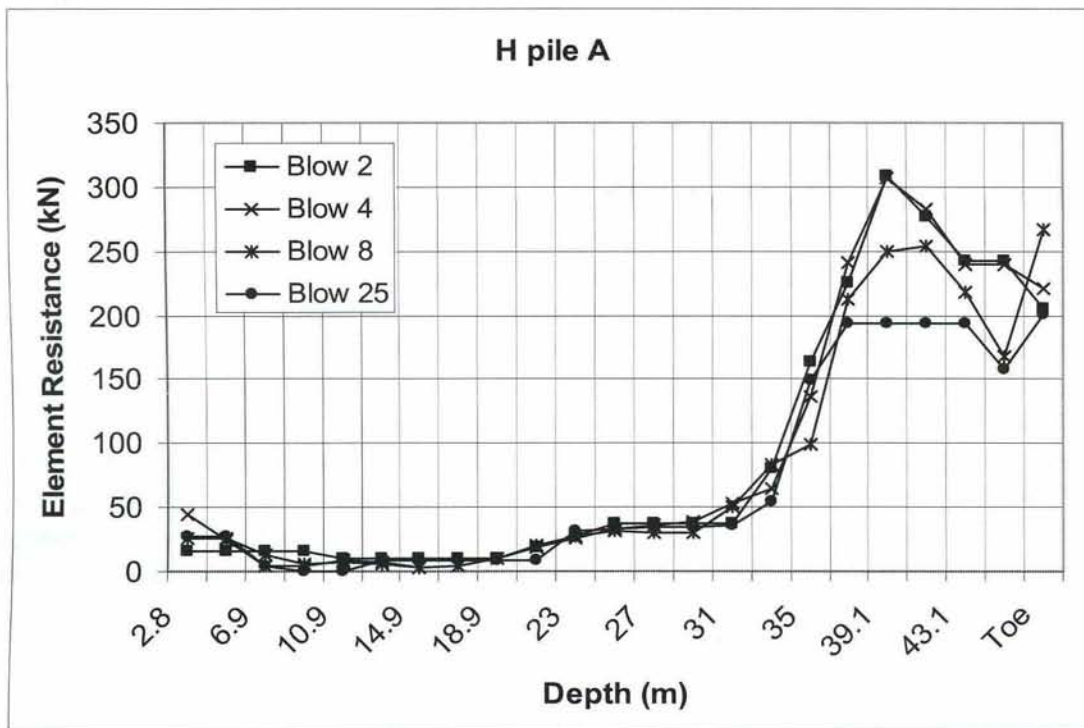


FIG. 3: Calculated resistance distributions for H- pile A

Case 2: H-Pile B

An H-pile 309 x 79 (12 x 53) was restruck with an APE D19-42 hammer for 10 blows. The first 5 blows resulted in 25 mm set. The next 3 blows produced an additional 25 mm set, and the final 2 blows also had 25 mm set. Soils were generally clayey fine sands (SC) or clay (CH) with SPT N values of 10 or less along the shaft. The toe was driven into Limestone (N-values of 50/5"). Figure 4 shows the CAPWAP results cyclically presented for the first 5 blows plus later blows 7 and 10. The shaft resistance decreases significantly between blow 1 and 2, with continued further decreases with subsequent blows. The end bearing gradually increased during the restrike. The resistance distribution curves calculated from several blows are shown in Figure 5. The results of the first blow include a very high shaft resistance and an almost completely activated toe resistance. The D/60 failure toe displacement, D/60, would be 5 mm and that has been achieved with the first restrike blow. According to rule 4 of the proposed failure criteria above, the second blow would therefore present the, admittedly somewhat conservative, bearing capacity result. This

result is reasonable because it would not include soil setup effects that could be lost under a sudden (dynamic) or excessive load due to progressive failure.

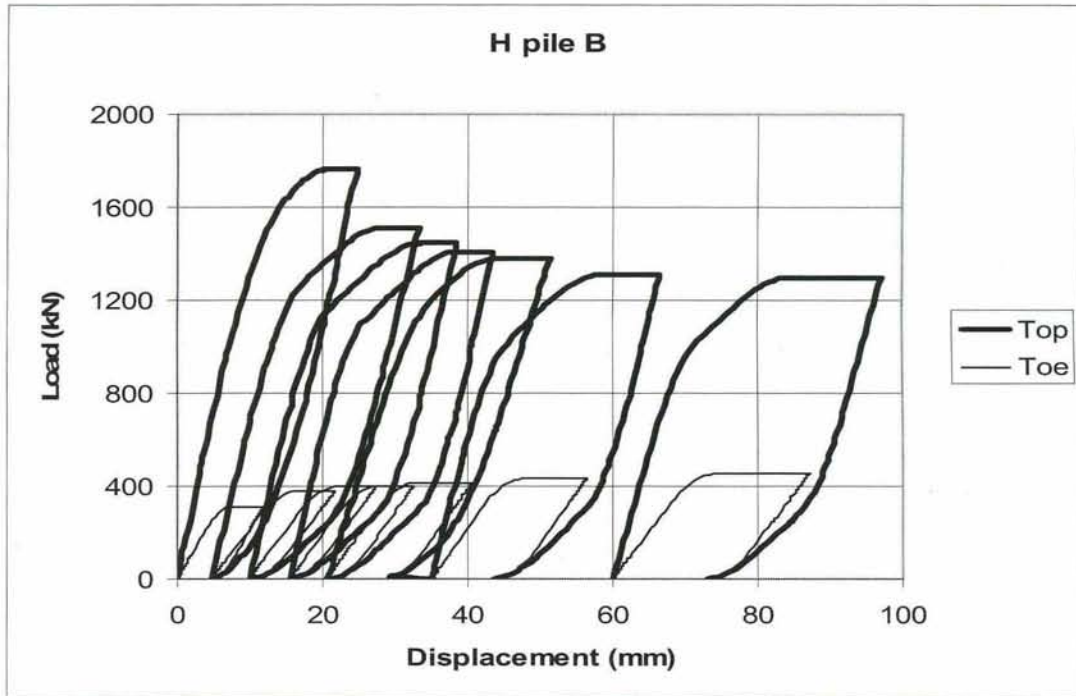


FIG. 4: H- pile B

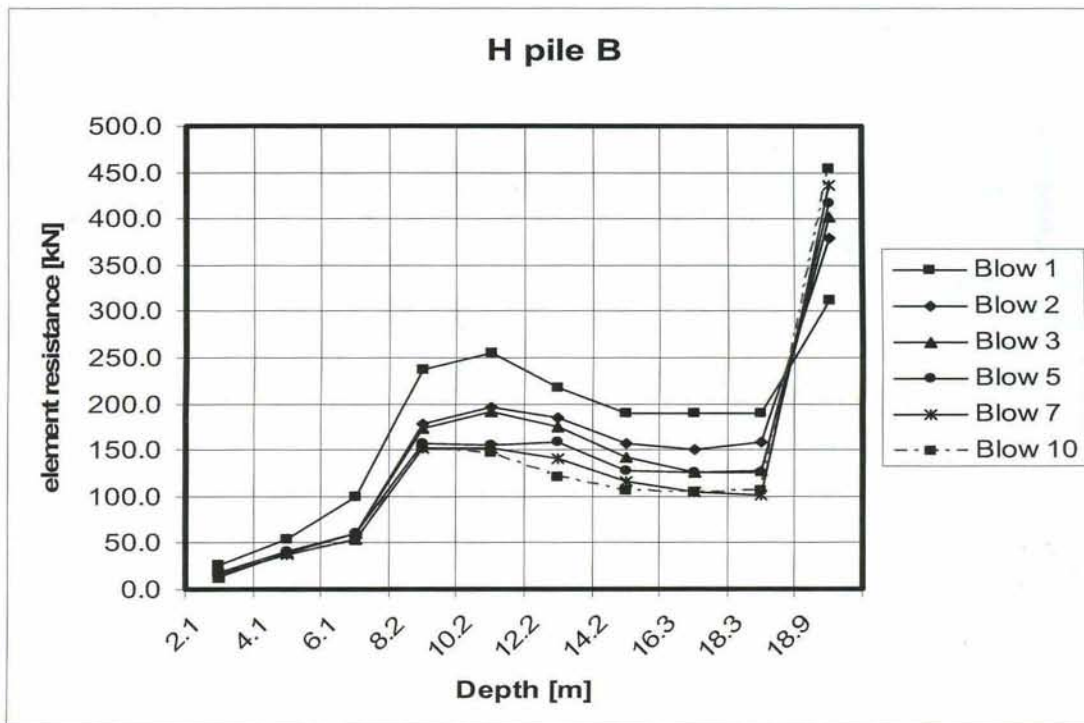


FIG. 5: H-pile B resistance distribution

Case 3: Prestressed Concrete Pier A

A 610 mm (24-inch) square prestressed concrete (PSC) pile was driven at a pier location for a large bridge to a depth of 25 m into saturated sand (fine and dense) with silts and clays by an APE D62-22 diesel hammer to a blow count of 270 blows per meter (3.7 mm set per blow) at a stroke of 2.7 m. After a waiting period of one week, the restrike of 10 blows resulted in a total net displacement of 30 mm (average 3.0 mm set per blow) at a hammer stroke of 2.7 m.

Figure 6 shows the CAPWAP load-set curves calculated from the records of the first 5 blows and blow 10. While there is relatively little change in the total capacity from blow to blow, the end bearing significantly increases and the shaft resistance significantly decreases from blow to blow as seen in Figure 6. This behavior is consistent with the soil type. During initial driving, and again upon re-driving like in this sequence of blows, negative pore pressures increase producing dilation in the silt or fine sand and, therefore, high negative pore water pressures. These pressures, in turn, cause temporarily larger effective stresses and thus more end bearing resistance.

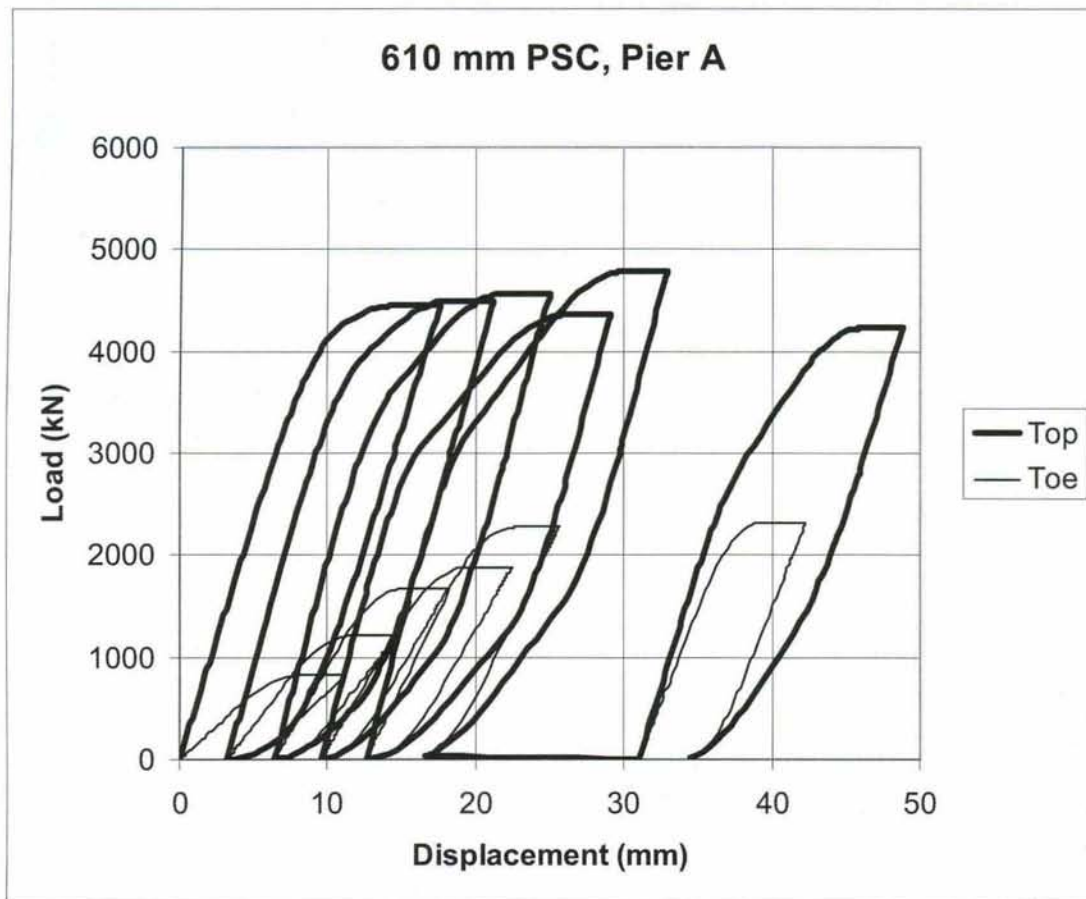


FIG. 6: Prestressed concrete pile, Pier A - CAPWAP results

With time, hydrostatic pore pressures are re-established and the toe then experiences relaxation effects. It is perhaps coincidental that the shaft reduction (e.g., loss of skin

friction setup) approximately balanced the toe increase during re-driving. Other observed cases with toe in dense saturated fine sands and silts have shown an increase in total capacity from blow to blow during restrike, reflecting the temporary change in pore pressure condition during driving. Although the total capacity was relatively insensitive during this test series, it would be important in most cases to carefully choose an early blow in restrike to properly account for the shaft setup and toe relaxation effects.

In this case, the failure toe total displacement would be 10.2 mm (610/60) and this displacement has been reached or exceeded already during the first restrike blow. Again from rule 4, the result of the second blow should be used for capacity determination and it would yield approximately 4500 kN capacity like the first restrike hammer blow.

Case 4: Prestressed Concrete Pile at Pier B

At Pier B of the same project as Case 3, the same D62-22 hammer drove and tested another 610 mm PSC pile, this time to only 19 m penetration depth. Restrike testing resulted in 45 mm set for 10 blows (4.5 mm set per blow, on average). With less penetration, and considering that soil characteristics vary across larger sites such as this one, the apparent behavior has both similarities and differences.

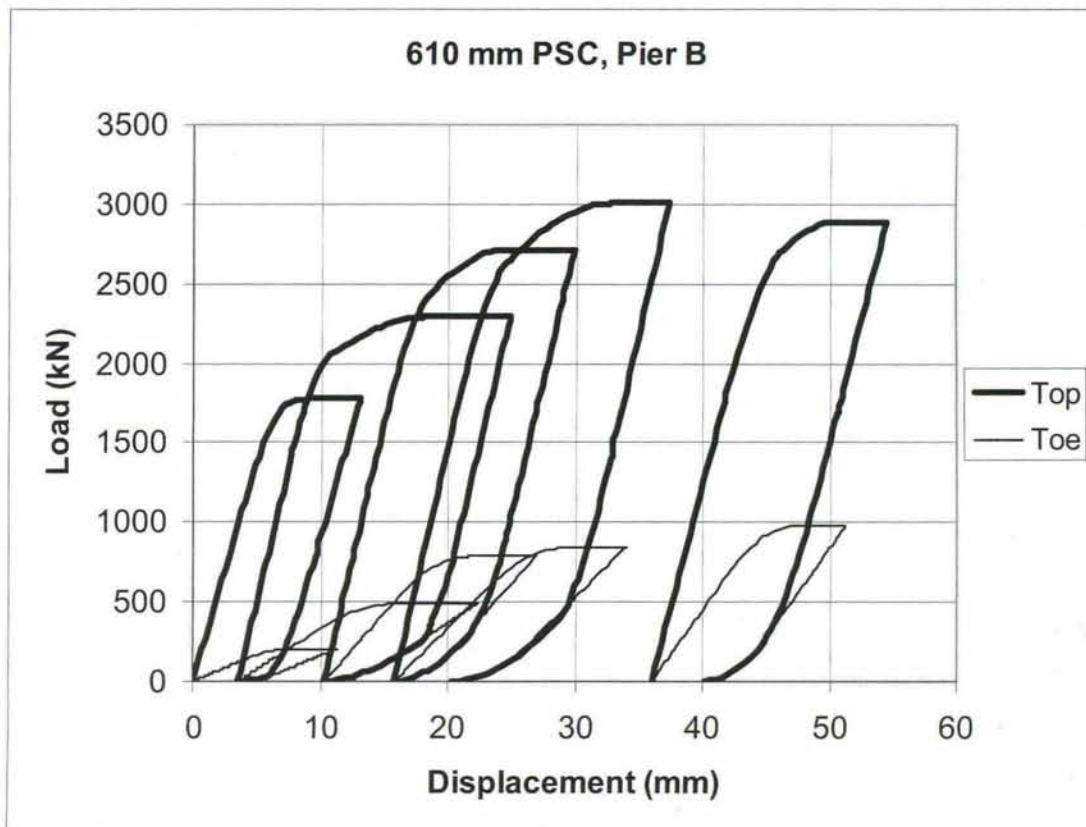


FIG. 7: Prestressed concrete pile, Pier B – CAPWAP results

It is clear from Figures 6 and 7 that the end bearing increase is somewhat similar to that observed at pier A. However, the shaft resistance remains fairly constant through this restrike. Thus the total capacity increases rather substantially during this 10 blow restrike, and selecting an early blow in the restrike is critical so that the in-place pile capacity is not overpredicted. Blow 1 transferred to the pile only 1/3 the energy of blow 2, and thus blow 1 may have not fully mobilized the capacity. Again, the record of the first blow should not be used for capacity assessment although it is valuable to judge the activation of both shaft resistance and end bearing. Also, since the first impact generated just barely the required failure toe displacement of 10.2 mm, the second hammer blow would be acceptable as the indicator of full capacity. The ultimate capacity of this pile is therefore likely to be in the range of 2200 kN, or less than half that at pier A owing to a lower end bearing, and with less depth of penetration, a lower shaft resistance.

Case 5: Drilled Shaft

This case study presents results obtained from dynamically testing an 1.8 m diameter drilled shaft with an embedment depth of approximately 20 m. Dynamic Load Testing consisted of applying four hammer impacts with a large drop weight, which produced a total cumulative shaft net penetration of about 14 mm. The CAPWAP calculated load-movement curves shown in Figure 8 indicate that the maximum top and toe displacements reached 31 and 27 mm, respectively.

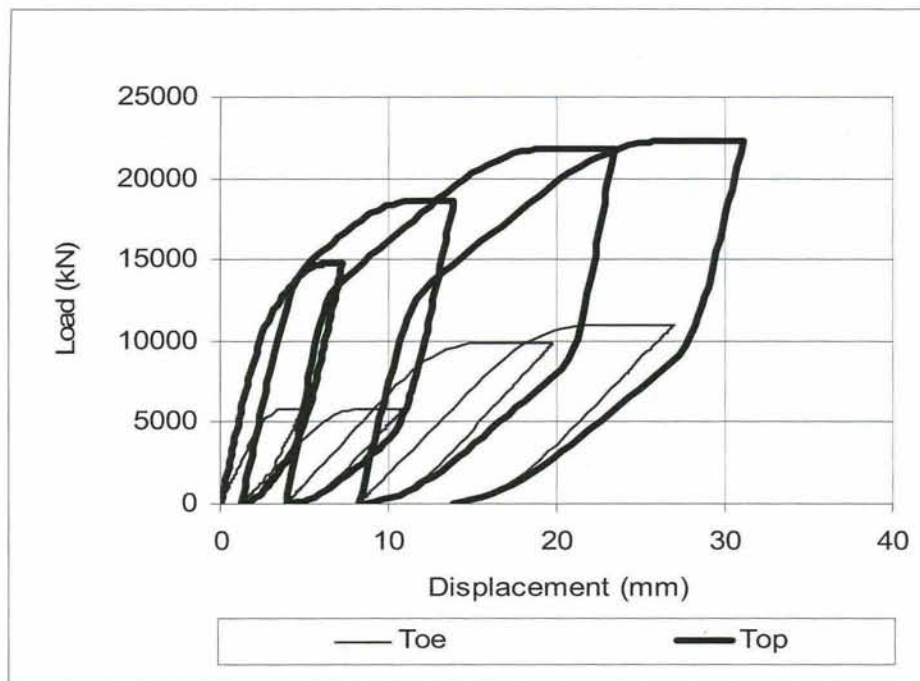


FIG. 8: CAPWAP calculated load cycles for 1.8 m diameter drilled shaft

The first two blows partially activated the available end bearing while the shaft resistance was activated under the second impact. The results of the third blow shows

a higher capacity (as compared to the second impact) primarily from the increase in end bearing; the rate of end bearing increase diminished with the fourth blow, and comes close to achieving the D/60 toe displacement criteria; it is likely that the next impact would have achieved the D/60 criteria at a load not much different than the result of this fourth blow. Note that the presentation of successive load-set curves allows for a reasonable means to evaluate the in-place activated capacity and the increase in resistance due to the development of additional end bearing resistance with further penetration. It should be added that for a drilled shaft, relaxation effects at the toe are typically not of concern since the geomaterial below the pile toe is in a disturbed state due to the mechanics of the installation method.

Case 6: Augercast Pile

An augercast (also referred to as augered cast-in-place or ACIP) pile of diameter 510 mm and length 21.2 m was installed into soils consisting of very soft clay for 11 m underlain by very dense sand. After allowing the pile to harden for a few weeks, a 60 kN drop weight applied 3 blows with drop heights 0.25, 0.50 and 0.75 m respectively, resulting in permanent set per blow of 0.9, 2.1 and 3.3 mm, respectively. The dynamic test results are presented in Figure 9. The failure toe displacement required for D/60 is 9 mm, and the third blow satisfies the criteria. The predicted capacity for the third blow, which is 2500 kN, was submitted to the client before the static test was performed, and it correlates almost exactly with the static load test performed about two weeks later.

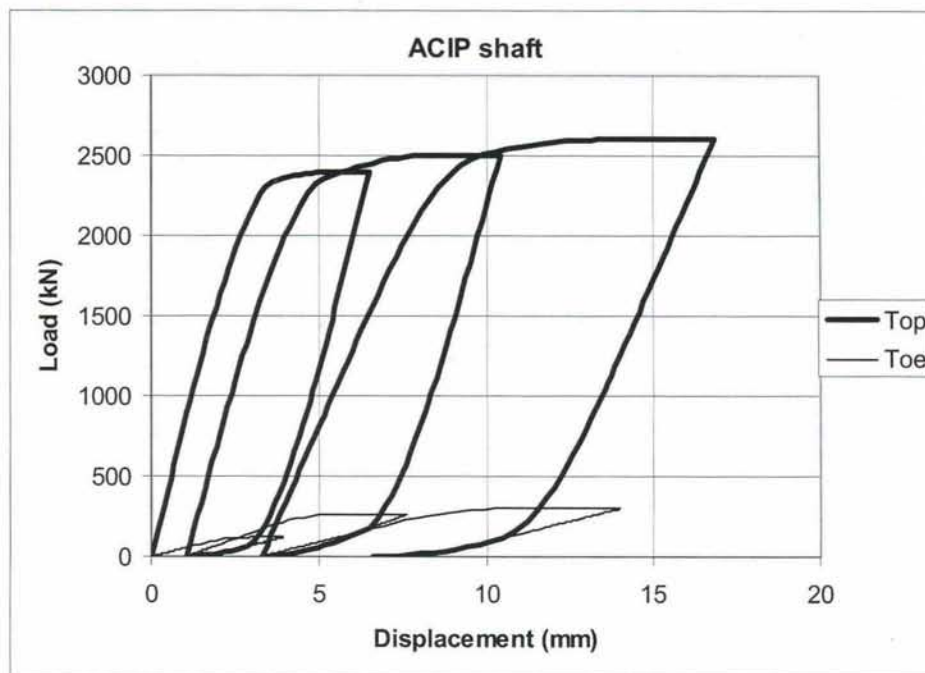


FIG. 9: ACIP CAPWAP results

SUMMARY AND CONCLUSIONS

This paper proposes a standardized procedure in which Dynamic Load Test records are selected and evaluated when assessing the post-installation load bearing capacity of driven piles, augercast piles, and drilled shafts. The method is closely related to the Offset or Davisson Failure Criterion that is frequently used for static load test evaluations. Different from current practice, the new criterion does not require that a specific minimum permanent set per blow be achieved but rather that the pile or shaft achieves a cumulative toe displacement of $D/60$ or more, where D is the pile or shaft diameter. In addition, a first load cycle (impact) of a lesser energy should be applied but its results should not be used to establish pile or drilled shaft capacity although it may be considered for the evaluation of resistance development trends. CAPWAP analysis of the dynamic records obtained under the first blow can also serve to complement the calculated load-set relationships of the second and later impacts. It is also sometimes helpful to perform CAPWAP analyses with data obtained under several blows as necessary and plot their load-set curves in a sequential presentation for assessment of setup and relaxation effects. The example records discussed in this paper demonstrated a variety of situations which can be clearly identified by the proposed analysis method. The method incorporates aspects that warrant further study and consideration for practical application and engineering use.

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