Pile Dynamics in Geotechnical Practice
Six Case Histories

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Pile Dynamics in Geotechnical Practice — Six Case Histories

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Abstract

Six cases are presented showing the benefit of dynamic testing and analysis of driven piles, comprising (1) a comparison between two types of piles and a correlation between CAPWAP determined capacity and load-movement response to that determined in a static loading test, (2) the benefit of time-dependent increase of pile capacity, (3) a study how a soil plug can be shown to replace a pile shoe, (4 & 5) the effect on pile capacity from removing soil inside the pile, (6) set-up and effect of excavation. Together the cases show that the dynamic testing can be used to investigate problems in design and construction of piles much beyond a simple capacity determination.

Introduction

Dynamic testing and analysis is the generic short-term for WEAP analysis, Pile Driving Analyzer (PDA) measurements, and CAPWAP analysis of measured data. Since its becoming available to the industry in the early 70's, dynamic testing and analysis has become an indispensable tool for the foundation engineer charged with designing piles, pile installation, and pile foundations, as well as verifying a design, and/or solving questions arising during construction. The following six case histories illustrate how dynamic testing and analysis was applied to resolve problems. In the first case, the PDA enabled existing experience with one pile type to be compared to a new pile type. It also demonstrates that dynamic and static testing methods agree well. The second case illustrates the specific advantage of PDA testing over conventional static testing in that many PDA tests can be performed quickly and economically—the alternative of carrying out a dozen or more static tests at the particular site would not have been feasible. In the third case, PDA was used to bear out that elements in an original design could be removed resulting in significant savings on the project. In the fourth and fifth cases, the PDA was used to pinpoint the adverse consequence of a faulty construction procedure. And, finally, the sixth case illustrates how a 30-day wait after EOID was rewarded with a set-up of capacity that almost doubled the pile capacity available for design enabling the designer to use shorter piles for the foundation. Ordinarily, the design would have had to be based on the much lower capacity determined in a restrike the day after the initial driving.

Case 1. Dynamic and Static Testing at JFK International Terminal, Jamaica, NY

Static and dynamic tests were performed for two types of 450-mm diameter piles tapering to 200-mm diameter over the lower 7.6 m. Two piles, Piles 1 and 2, were Monotube piles and two piles, Piles 3 and 4, were Steel-Taper-Tube piles. The piles were driven as two pairs of "identical twins" with a Juntan HHK-7 hammer to a depth of 18.0 m through a thick deposit of fine to coarse, medium dense to dense glacial sand at the JFK International Arrival Terminal, at the south shore of Long Island, New York. One purpose of the tests was to compare the two types of piles in terms of response to the driving and capacity at the

End-of-Initial-Driving (EOID). For one pile of each type, dynamic testing was also performed at Beginning-of-Restrike (BOR) three weeks after the driving. Capacity was determined both by dynamic testing (CAPWAP analysis) and static loading test.

Dynamic testing and CAPWAP analyses were performed at EOID for all four piles and at BOR for Piles 1 and 3 after a set-up wait of 19 days and 35 days, respectively. Piles 2 and 4 were not restruck, but were filled with concrete and subjected to static loading tests two days after the restrike testing of their "twins", Piles 1 and 3. Fig. 1.1 shows Wave Traces and results of CAPWAP analysis from Pile 3 at EOID.

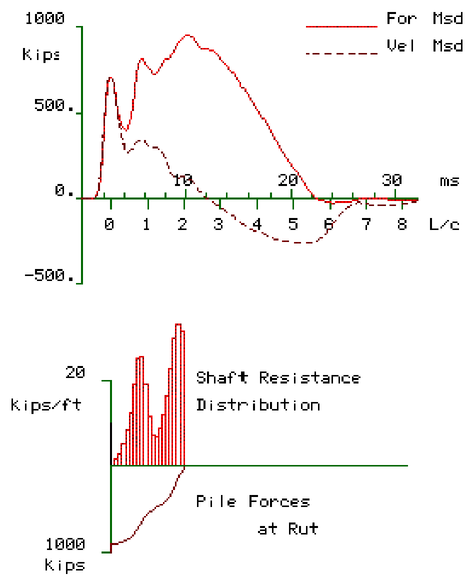

Fig. 1.1 Pile 3. EOID Wave traces and CAPWAP determined distribution of static resistance

For both Piles 2 and 4, and immediately prior to the start of each of the static loading tests, the authors presented a prediction of the capacity and pile head movement to be found in the tests. The predictions were based on the similarity between the two pairs of test piles and the assumption that the relative increase of capacity from EOID to BOR due to set-up found for Piles 1 and 3 would be the same for Piles 2 and 4. The prediction of movement considered the stiffness change due to the concreting of Piles 2 and 4, respectively. The increase of capacity of Piles 1 and 3 between EOID and BOR was 60 % for both piles. Applying the same proportional increase to the EOID of Piles 2 and 4 gave predicted capacities of 3,600 KN and 3,780 KN, respectively.

Tables 1.1 and 1.2 compile the results of the Dynamic Monitoring, CAPWAP Analyses, and Predictions. The capacity predictions were within 4 % and 1 % of the results found in the static loading tests (also included in the tables).

Because the Monotube has a 6.0 mm wall as opposed to the 9.5 mm wall of the Steel Taper Tube, its maximum driving stress is significantly larger than that of the Steel Taper Tube Pile. (The nominal steel yield values are 380 MPa and 310 MPa, respectively). Because of its heavier cross section, the Steel-Taper-Tube required fewer blows to drive and the penetration resistance (PRES) at EOID was smaller than that observed for the Monotube.

TABLE 1.1 MONOTUBE — PILES 1 AND 2

Test	PRES (bl/25mm)	EMX (KJ)	FMX (KN)	CSX (KPa)	R_{ULT} (KN)	Movement at R_{ULT} (mm)
1-EOID	3	53	2,730	310	1,910	
2-EOID	3	53	2,620	300	2,250	
1- BOR	10	59	3,210	370	3,020	
2-**Predicted Capacity and Movement**					3,600	18
2-**Static Loading Test**					3,730	21

TABLE 1.2 STEEL TAPER TUBE — Piles 3 and 4

Test	PRES (bl/25mm)	EMX (KJ)	FMX (KN)	CSX (KPa)	R_{ULT} (KN)	Movement at R_{ULT} (mm)
3-EOID	2	63	2,740	204	2,530	
4-EOID	2	72	2,920	220	2,360	
3- BOR	6	80	4,240	320	4,010	
4-**Predicted Capacity and Movement**					3,780	15
4-**Static Loading Test**					3,820	16

PRES = Penetration Resistance, EMX = Maximum Transferred Energy
FMX = Maximum Force CSX = Maximum Compressive Stress
R_{ULT} = Capacity Determined in Dynamic Test or Static Test

Fig. 1.2 shows the load-movement curves of the two tests. Both tests were carried to "failure". The CAPWAP based capacities and movements predicted for the two test piles are indicated in the diagram. The stiffness difference is mainly due to the thicker wall of the Steel-Taper-Tube pile.

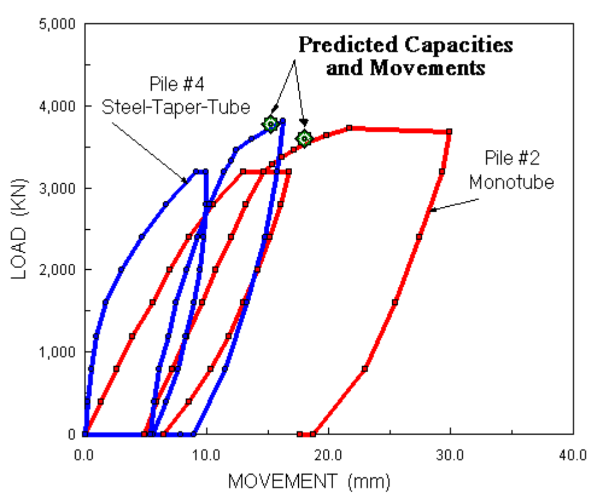

Fig. 1.2 Piles 2 and 4 Load-Movement Diagrams from the Static Loading Tests with CAPWAP Predicted Capacities and Movements

Case 2. Dynamic Pile Testing at Montreal River Bridge, Elk Lake, Ontario

Dynamic testing and analysis of pile foundations was performed for a four-span replacement bridge across Montreal River at HWY 65, Elk Lake, Ontario. The bridge is supported on two abutments and three piers, each having a single row of six piles.

The soil profile at the site consists of a heterogeneous material composed of sand, silt, and clay above a thick deposit of compact to dense glacial till starting at a depth of about 45 m. The groundwater table lies at the river surface level and the pore water pressure distribution is hydrostatic.

The piles are 310HP110 piles with a yield of 300 MPa and a cross sectional area of 141 cm^2 driven with a Delmag D30-32 single-acting diesel hammer. The required pile capacity is 2,660 KN. The piles were driven into the till layer, reaching embedment depths of 53 m through 62 m. One pile in the east abutment was redriven about one week after the EOID to a depth of 70 m. The penetration resistance (PRES) at end-of-initial-driving (EOID) was no more than about 1 blow/25 mm for the piles and about 2 blows/25 mm for the 70-m pile at end of second drive (EOD2). Fig. 2.1 presents the pile driving diagrams from all six piles in the east bridge abutment, and, for reference, also the N-values (shown as bars) established in the soils investigation. The six piles drove quite similarly. It is also clear that there is little correlation between the N-values and the PRES-values. At beginning of restrike, BOR, the equivalent PRES values were about 9 blows/25 mm.

Piles 1 through 6, East Abutment

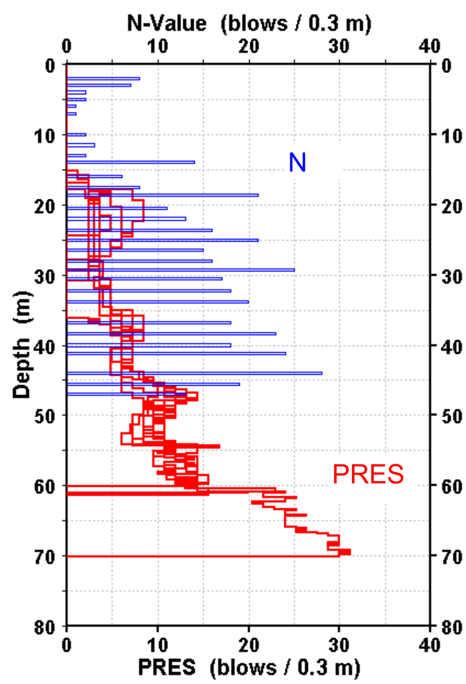

Fig. 2.1 Pile Driving Diagram and SPT N-Values

The design had been based on the piles being toe bearing with the pile structural strength rather than the bearing capacity governing the design. Evidently, the structural strength does not govern.

As a bit of an understatement, the pile capacity now became an issue and it was decided to verify the pile capacities by means of restriking with dynamic testing and analysis. Test piles were selected from within both abutments and from all three piers. The initial driving had taken place over some length of time, which means that the restrike tests were carried out at different times after EOID.

CAPWAP analysis on the restrike blows was performed to determine the capacity of each tested pile. Fig. 2.2 shows the pile capacities as a function of the number of days after the "initial driving". The solid symbols represent piles that were tested on two separate occasions. As evident from the curves, the pile capacities increased by about 25 % between one week and one month after EOID and about 50 % during the first month after EOID. To confirm the set-up gain, three piles having low capacity when tested at restrike about one week after EOID were again tested at a second site visit (these values are plotted with solid symbols in Fig. 2.2). The capacity was shown to have been increased considerably due to the set-up. Thus, as the testing proved that the long-term capacity of the piles was adequate, the piles were accepted as driven.

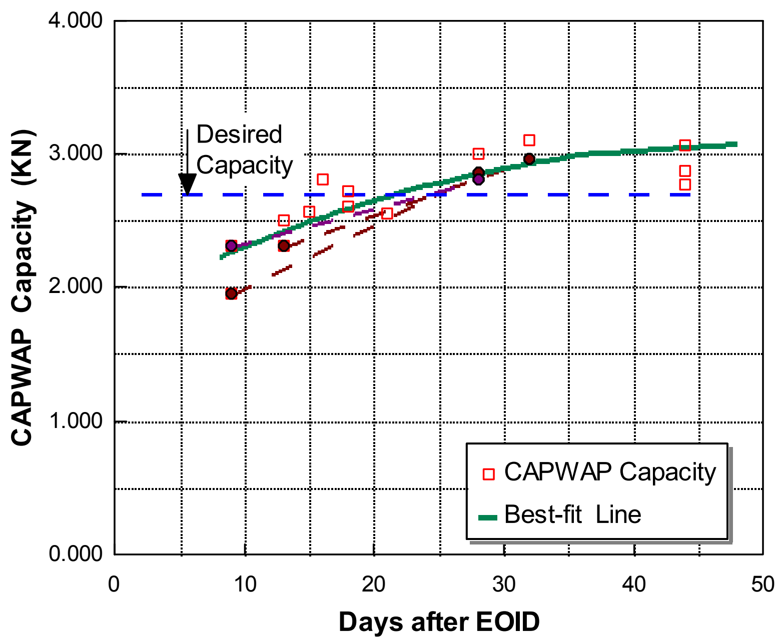

Fig. 2.2 CAPWAP Capacities vs. Set-up Days

Incidentally, had the set-up time not been considered, but the test results been looked at per pile location, or if only a few piles had been tested, the results would have been much less obvious, as suggested in Fig. 2.3.

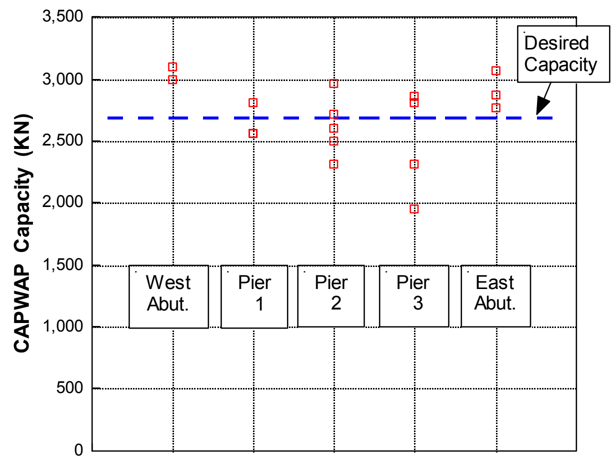

Fig. 2.3 CAPWAP Capacities per Pile Location

Case 3. Plugging of Open-Toe Piles and Set-up of Capacity

Pipe piles are often driven open-toe, because open-toe piles are usually easier and faster to drive than closed-toe piles — as long as the pile does not plug. Indeed, when the pile does plug, the pile termination criterion is rapidly met; "refusal" occurs. Often, partial plugging occurs resulting in increased penetration resistance, but the pile may still be advanced until reaching and forming a "solid" plug in the bearing layer. When that happens, there is very little difference in driving characteristics between the plugged pile and a pile driven closed-toe (although the mass of the soil plug may have some effect, usually causing a small reduction of penetration resistance).

Concreting the piles after the driving may be necessary to increase axial structural strength so that the design can make full use of the set-up. (Set-up is increased capacity due to reconsolidation after the driving and other reasons). Also, concreting the pipe is often necessary to enable the pile to resist horizontal loads. At the completion of driving, open-toe piles usually need to be cleaned out as they will have filled up with soil to a greater or lesser degree. Therefore, considering that cleaning out a pile is a cost factor, why are piles not always driven closed-toe? Well, it is a trade-off. As mentioned, open-toe piles are faster and easier, i. e., cheaper, to drive. Moreover, the pile shoe, or toe plate, is also a cost factor. Cleaning out a pile could disturb the soil at the pile toe and has to be done carefully, of course (as is shown in the fourth and fifth case histories).

As with everything else in deep foundation design and construction, Nature has a say, too. The open-toe pile may plug early during the driving, and, apart from the fact that this may appreciably slow down the work, what about the capacity of a plugged open-toe pile as compared to a closed-toe pile?

The opportunity to make that comparison arose on a recent project near Edmonton, Alberta. Two 24 m long, 273-mm diameter pipe piles were driven through a silt and clay deposit to bearing in a clay till having water content of about 25 % and SPT N-indices of about 30± bl/0.3 m. The piles were identical, but for one pile being driven open-toe and the other closed-toe by means of a plate welded flush with the pile outside diameter. The open-toe pile was easier to drive until the last few feet before the end of driving. At end-of-initial-driving (EOID), the open-toe pile had an about 14 m long soil plug. Concreting the remaining (upper) 10 m length provided the desired stiffness to resist horizontal loading.

The observed penetration resistances at EOID were the same for the two piles: 3 bl/25 mm. In restriking the next day (15 hours later), the PRES value had increased to 8 bl/25 mm, again the same for both piles. The CAPWAP results are presented in Table 3.1.

The Pile Driving Analyzer (PDA) wave traces presented in Figs. 3.1 and 3.2 are almost identical for the two piles at both EOID and restriking (BOR). CAPWAP analyses showed that they also had the same total capacity, and shaft and toe resistances, as well as the same quake and damping values. For both piles, the capacity of the piles almost doubled between EOID and BOR due to set-up.

The similarity between the pile responses makes it obvious that open-toe or closed-toe made no difference with regard to the pile capacity. Driving the piles open-toe and discontinuing the use of the pile toe plate saved time and reduced costs.

Table 3.1 CAPWAP Results

	CAPACITY			QUAKE		PRES
	R_{TOTAL} KN	R_{SHAFT} KN	R_{TOE} KN	SHAFT mm	TOE mm	Bl/25 mm
EOID						
open	850	550	300	2.5	7.0	3
closed	950	500	450	2.5	7.0	3
RSTR						
open	1,500	800	700	2.5	4.0	8
closed	1,550	850	700	2.5	5.0	8

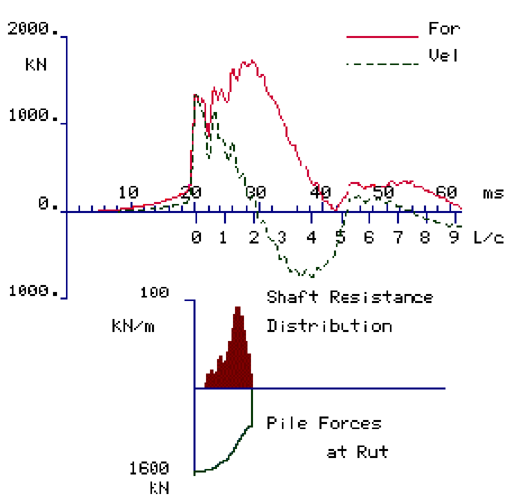

Fig. 3.1 BOR **Open**-Toe Pile

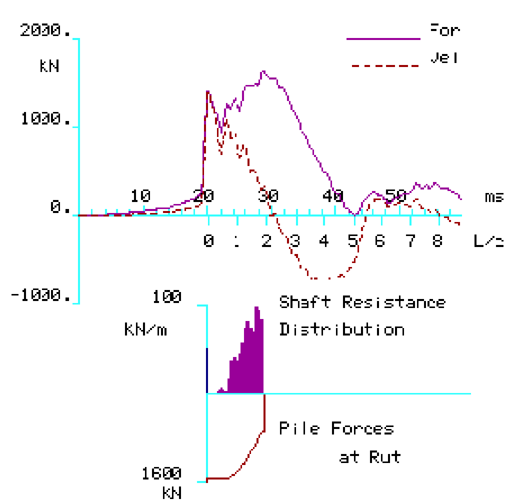

Fig. 3.2 BOR **Closed**-Toe Pile

Case 4. Removing Soil inside an Open-Toe Pile

The method of removing soil inside an open-toe pile is usually by means of an “air lift”, if necessary combined with jetting and/or first loosening the plug using a chopping bit. An “air lift” is created by pumping—injecting—water and air into the pile immediately above or into the surface of the plug, discharging the spoils up the inside of the pile (small diameter piles) or in a separate discharge pipe (large diameter piles). The flow of the water and air up the pile (or in the discharge pipe) pulls the spoils along. The process creates a suction in or immediately above the surface of the plug in relation to the hydrostatic conditions without the airlift. It is rarely recognized that a suction has also been created in relation to the soil conditions at the toe of the pile.

Normally, the bearing layer at the pile toe consists of fine-grained glacial till, mudstone, or other low-permeability soils which prevent the air-lift suction from interfering with the soil at and outside the pile toe. This is because these soils can sustain a pore pressure gradient. In such soils, therefore, even when the air lift is brought right down to the toe of the pile, the bearing layer and pile toe resistance are not affected. However, when the bearing layer consists of pervious soils that cannot sustain a pore pressure gradient, an upward flow develops in the pile: water flows from the pile toe through the soil column inside the pile. The process loosens the soils at and below the pile toe. In the extreme, fines outside the pile toe will be sucked into the pile to be discharged by the air lift, which all but destroys the toe capacity.

Open-toe steel pipe piles with a diameter of 1,067 mm (42 inches) were used for a bridge rehabilitation project in Terrace, in northern British Columbia. The piles were driven 10 m into a very dense silt, sand, and gravel deposit with N-indices in excess of 50 bl/0.3 m. In driving the piles, soil rose up inside the pile. The piles were tested in restrike and dynamic testing was performed at beginning-of-restrike, BOR. The CAPWAP determined capacity was 3,200 KN with shaft and toe resistances of 1,700 KN and 1,500 KN, respectively. The small toe resistance value indicates that the soil had not formed a plug, but was acting as a free column inside the pile. This means that a portion of the shaft resistance came from the inside surface of the pipe. In preparation for concreting, the soil column was removed. The removal of the soil column was by means of an air lift. To determine if the removal of the column had had any significant effect on the shaft resistance and to verify that the toe resistance had not become smaller, a second restrike test was performed. The BOR CAPWAP capacity was now 2,300 KN, that is, 900 KN smaller than before the airlift, with shaft and toe resistances of 1,000 KN and 1,300 KN, respectively. Figs. 4.1 and 4.2 show the shaft resistance distributions determined in the CAPWAP analyses of blows from the two BOR’s. A comparison between Figs. 4.3 and 4.4 shows that the shaft resistance along the lower portion of the pile is much smaller than before the airlift, inferring that the soil has indeed been affected by flow of water and soil from the soil and into the pile. The comparison results cannot conclusively show to what extent the reduced capacity is due to the removal of the soil column and to what extent it also is due to disturbance of the soil around and below the pile toe. However, the method of removing the soil in the piles was changed to a bailing procedure and maintaining a positive head of water inside the pile. The airlifted pile was then driven deeper to restore capacity. No further dynamic testing was performed.

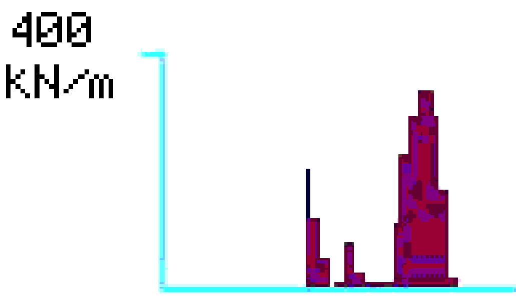

Fig. 4.1 CAPWAP Results immediately before Air Lift R_{ULT} = 3,200 KN

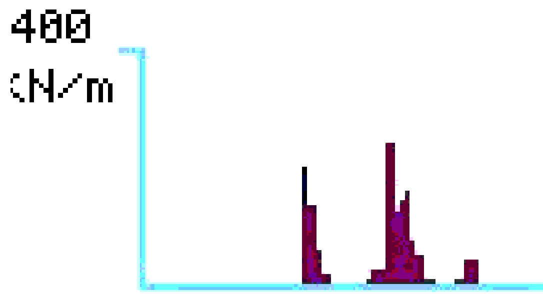

Fig. 4.2 CAPWAP Results immediately after Air Lift R_{ULT} = 2,300 KN

Case 5. Cleaning out the Jetting Tube

Dynamic measurements were performed on prestressed piles driven for the Port of Los Angeles at Long Beach, California. The piles were 33.5 m long, 600 mm diameter prestressed concrete piles with a 75 mm center hole going all the way through the pile (used for jetting the pile before seating it by driving). The soils at the pile toe consisted of dense fine sand.

Two weeks after a test pile intended for a static loading test had been jetted down and then seated by driving without jetting for an additional about 3.7 m, a restrike test was performed. The CAPWAP determined pile capacity at the end of restrike (EOR1) was 5,670 KN at a penetration resistance (PRES) of 13 bl/25mm and energy ratio of 45 %. After the restrike test, it was found that the lower portion of the center hole had become filled with sand. To get access to the pile toe in order to install a toe telltale, the center hole was cleaned using air lift. After a few minutes of cleaning, the center hole was plumbed**:** now, there was actually more sand in the hole than before the air lift! A new restrike test with dynamic monitoring was made and a CAPWAP analysis was performed on the first blow of restrike. This showed that the capacity of the pile determined for the first blow after air lift was no more than about 1,390 KN at a PRES of 6 bl/25mm and energy ratio of 28 %. The loss of capacity, about 4,300 KN, is mostly loss of toe resistance, R_t, which went from 4,010 KN down to 710 KN. In addition, the air lift destroyed about 450 KN of shaft resistance immediately above the pile toe. The analysis also shows that the shaft resistance some distance above the pile toe was unaffected by the air lift, in fact, it increased somewhat due to set-up.

The capacity improved somewhat by driving the pile about 3 m deeper. However, the air lift had adversely affected the soil conditions deep below the pile toe and the original capacity was not restored. At the end of the re-drive (EOD2), a CAPWAP analysis showed the capacity to had only increased to 3,470 KN. The final penetration resistance was 9 bl/in. The results of the CAPWAP analysis were confirmed by a static loading test. The Davisson Offset Limit Load was 3,340 KN agreeing well with the EOD2 CAPWAP result.

Although the airlift "mishap" severely affected the objective of the static test, which was to verify the capacity of the construction piles, the test established that the dynamic testing and analysis was a reliable tool for the site. Therefore, the first restrike result of 5,670 KN capacity was considered representative for the construction piles at the site. No more static testing was carried out and all verification and proof testing performed for the 2,000 construction piles driven subsequently for the project was based on dynamic testing and analysis.

The results of CAPWAP analysis at the beginning of restrike (BOR2) after the air lift and immediately before the static loading test (EOD2) are presented in Figs. 5.1 and 5.2, showing the distribution of shaft and total resistances.

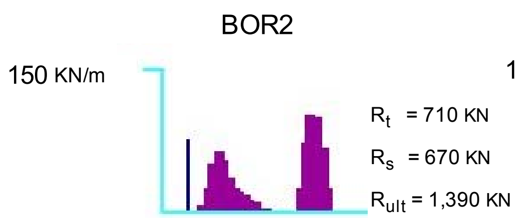

Fig. 5.1 CAPWAP Results immediately **after** Air Lift

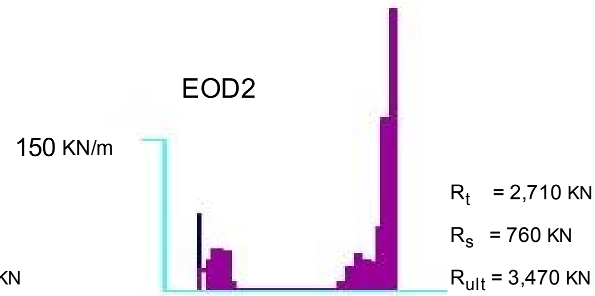

Fig. 5.2 CAPWAP Results Immediately **before** the Static Test

The case history illustrates that when pervious soils exist at the pile toe, using air lift to clean out a soil column can adversely affect the pile bearing capacity.

Case 6. Set-up and Effect of Excavation on Capacity

Dynamic testing and analysis was performed for an apartment complex at False Creek, North Shore, Vancouver, BC. The soil profile at the site consists of about 2 m of sand and gravel fill placed on an about 10 m thick layer of silty sand, and sandy silt, followed, at a depth of about 12 m, by dense "till like" silt and sand until bedrock (sandstone) at a depth of 25 m. The groundwater table lies at a depth of 3.5 m. The pore water pressure is probably somewhat artesian in the sandstone, which results in an upward pore pressure gradient at the site.

The building was to be founded on two sizes of pipe pile, 325 mm with 12.5 mm wall and 457 mm with 9 mm wall both driven by a 3,860 kg drop hammer using an about 1.5 m height-of-fall. The design called for a pile capacity of 1,800 KN and 2,300 KN, respectively.

Six test piles (two of the 324-mm piles and four of the 457-mm) were driven to depths of about 16.5 m across the site and dynamic testing was carried out at Initial Driving, at Early Restrike (within 24 hours after End-of-Initial-Driving, EOID), and at Beginning of Second Restrike blow (BOR2) 27 days after EOID. One 324-mm pile was restruck also 71 days after EOID. Fig. 6.1 shows the penetration resistance at initial driving for the piles.

In restriking the piles after 27 days using a height-of-fall of 2.4 m, the penetration resistance had increased to about 100 bl/0.3 m (equivalent PRES) due to set-up developing since the EOID.

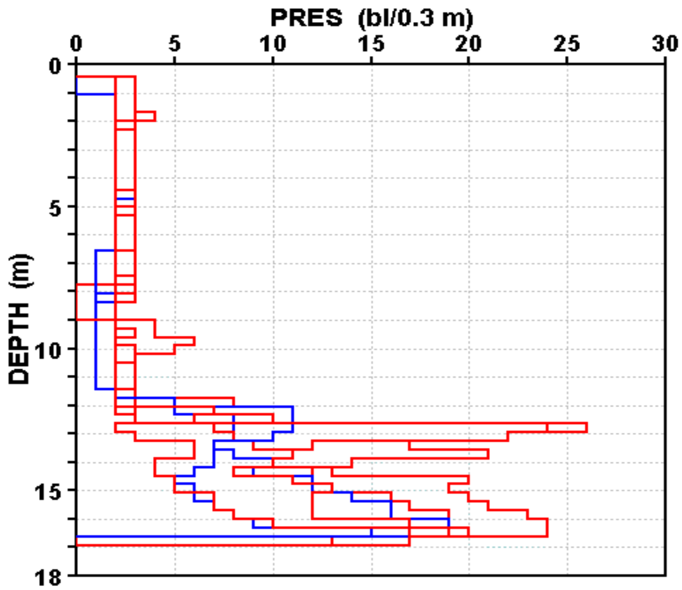

Fig. 6.1 Pile Driving Diagram of Six Test Piles

CAPWAP analysis of the dynamic test data indicated that the piles had been driven to a capacity at EOID of approximately 900 KN with a variation of about 200± KN. CAPWAP analysis on the first blow of the Early Restrike Records indicated a moderate capacity increase of about 300 KN. The new values were clearly smaller than the desired pile capacities.

Based on experience from previous projects in the Greater Vancouver area, it was expected that the increase of capacity after EOID would require more than a day to develop. Therefore, instead of lengthening the piles to obtain more capacity, the test piles were left alone until restruck 27 days after EOID. CAPWAP analysis on the first blow of the 27-day records (BOR3) indicated that the capacities had increased by an additional set-up amount of about 1,200 KN. (Obtaining good records to analyze from the first restrike blow is necessary, because the set-up deteriorates after only a few blows). The 27-day capacity of the 457-mm pile was not proportional to the pile size, as it was only one-third larger than that of the 324-mm pile. (It is probable that the set-up for the smaller diameter pile took shorter time to complete). The 27-day capacities were about 1,800 KN and 2,400 KN with most of the capacity obtained in the till. The corresponding shaft and toe resistances were 1,200 KN and 1,500 KN, and 600 KN and 900 KN, respectively, for the 324-mm and 457-mm piles. The single 71-day restrike indicated a small continued set-up.

To clearly see the set-up trend at the site, the capacities of the six test piles were normalized to the 27-day value. The results are plotted in Fig. 5.2 showing the normalized capacity in percent of the 30 day capacity at EOID, BOR1, BOR2, and BOR3 (one pile).

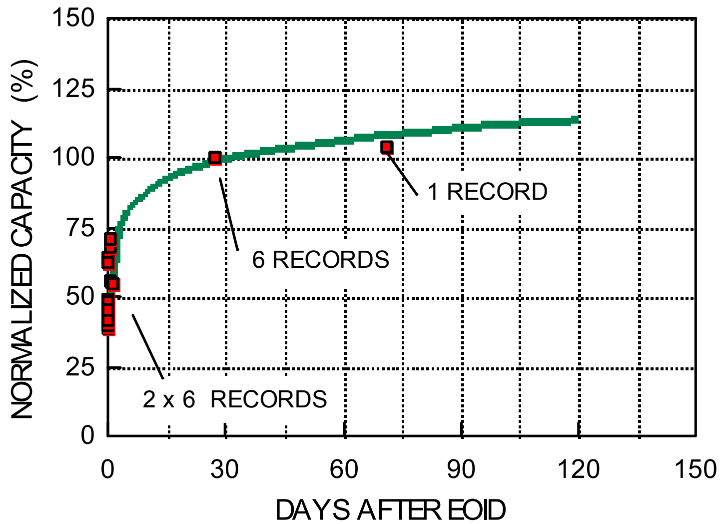

Fig. 6.2 Normalized Capacities of Six Test Piles at EOID and BOR

The results indicate that after a month of set-up, the capacities of piles driven similarly to the test piles will be equal or larger than the desired values. Static analysis simulating one-month conditions indicated beta-coefficients of 0.3 through 0.6 above a depth of 12 m and 0.8 to 1.0 in the till below 12 m combined with an N_t-coefficient of approximately 60 to 80.

The foundation conditions at the site were complicated because portions of the site were to be excavated by as much as seven metre after the driving of the construction piles. As a consequence of the excavation, the groundwater table will be lowered. Shaft and toe resistances will therefore reduce proportionally to the changes of effective stress in the soil (the toe resistance to a lesser degree) necessitating that the construction piles be driven deeper into the till to achieve a higher initial-driving capacity that could provide an allowance for the reduction due to the excavation.

A static loading test was performed on one pile 125 days after EOID after the excavation had been completed. Before the excavation, CAPWAP analysis on a 70-day restrike blow had shown a capacity of 1,550 KN. A static analysis of the capacity after excavation indicated that the reduced effective overburden stress would have lowered the capacity by about 250 KN (to 1,300 KN). In the static loading test, the pile failed in plunging at 1,335 KN confirming the results of the CAPWAP and the static analyses. The test results and the static analysis were then used to determine the final foundation depths of the construction piles.

Conclusions

The first case history illustrates how results of dynamic monitoring at end of initial driving and at restrike were used to determine capacities for two types of tapered steel piles and indicates the correlation to penetration resistance and driving stresses. The results of the static loading tests confirmed the relevance of CAPWAP determined pile capacities.

The second case history demonstrates that, to make sense of the 16 different values of pile capacity, the different lengths of set-up time had to be considered. The third case indicates how results of dynamic testing and analysis served to resolve whether or not a reliable soil plug had formed in the piles. The fourth and fifth cases demonstrate that cleaning out a pipe by means of airlifting is hardly a simple and risk-free operation. The final case indicates again the benefit of considering the time-dependent increase of capacity when designing pile foundations. It also shows the merit of combining dynamic testing and analysis with static analysis of resistance distribution.