Using dynamic pile testing to overcome surprising soil variations

Salem, H.S, Lavergne, H.R., and Fellenius, B.H., 2008. Using dynamic pile testing to overcome surprising soil variations. Deep Foundations Institute, 33rd Annual Conference on Deep Foundations and 11th International Conference on Piling and Deep Foundations, October 15-17, 2008, New York, 7 p..

USING DYNAMIC PILE TESTING TO OVERCOME SURPRISING SOIL VARIATIONS

H. (Sam) Salem*), Hector Lavergne Ramirez#), Bengt H. Fellenius*)
*)AATech Scientific Inc. Ottawa, Ontario #)GMTS Corp., San Juan, Puerto Rico

A site investigation for a wide and tall hotel building, in the San Juan Bay area, Puerto Rico, indicated a soil profile of silty clay on sand, soft and loose enough to require piled foundations. The apparent uniformity of the soil profile across the site indicated that a single pile driving and termination criterion could be applied to the installation of the piles — driven precast concrete piles. Because of the size of the site and importance of the project, the project also included a static and dynamic testing programme. The dynamic monitoring using the Pile Driving Analyzer (PDA) verified a consistent hammer operation. However, the results of the static loading tests showed significant and inconsistent variation in pile capacity between locations and little correlation between end-of-driving and restrike penetration resistances (blow-counts) and pile capacity. Therefore, it was not possible to rely on a simple depth and blow-count termination criterion for the site. In contrast, pile driving response in terms of measured reflected force and mobilized resistance coupled with CAPWAP analysis gave a reliable correlation to the results of the static loading tests. As presented in the paper, the dynamic testing programme was enlarged and used to develop and verify a complex set of termination criteria tailored for the multitude of conditions encountered across the site to achieve and ensure economical and safe pile installation.

Background

The subject of this case history is the construction of the Sheraton Convention Center in San Juan, Puerto Rico. The proposed foundation, designed by GMTS Corp., consisted of driven 12-inch diameter precast concrete piles, driven in segments of 10, 15, 20, and 25 feet.

Static pile capacity analyses resulted in estimated pile lengths ranging from 70 to 80 feet below existing grade, for allowable (working) loads of 220 kips. An extensive index pile testing programme was conducted by GMTS Corp. to verify the design and to establish termination criteria.

A total of nine index piles were driven across the site, ranging in embedment depths from 65 through 107 feet below existing grade. Seven of the piles were intended for axial compression testing and two for pullout resistance testing.

A Linkbelt 520 double-acting diesel hammer (rated energy of 26.3 k-ft) was used to drive the index piles and subsequently the production piles. The piles were stopped at a penetration resistance, exceeding 100 blows per foot of penetration in the last 2 to 8 feet. Table 1, below, summarizes the pile driving records near the end of driving.

Table 1 – Summary of Test Pile Driving Records

Pile No.	Cumulative feet with an average blow count of				Pile Length (ft)
	31-45	46-60	61-100	>100	
TP-1	7	2	7	8	65
TP-2	8	2	9	3	64
TP-3	7	7	8	---	71
TP-4	14	2	5	5	73
TP-5	6	3	8	3	65
TP-6	7	2	4	3	75
TP-7	7	3	1	2	107
TTP-4	1	1	2	---	53
TTP-7	---	---	---	---	70

Based on the termination records listed in Table 1, Piles TP3, TP5 and TP7 were selected for axial static compressive testing. For clarity, only Piles TP3 and TP5 will be discussed in this manuscript since TP7 showed a similar behavior to that of TP3.

Site history and geology

The site of the subject project is located within the PR Convention Center District (PRCCD), in San Juan, Puerto Rico, in an area known as Isla Grande (Big Island). Originally, this area consisted of natural mangroves and lagoons, which were filled in by the end of the 1930's and early 1940's by the US Navy for the former San Juan Naval Air Station.

The materials used for fill consisted of dredged material from the Bahía de San Juan, with the uppermost 6 to 8 feet consisting of select fill obtained from limestone hills surrounding the bay area. The thickness of the fill deposit, including the dredged material and select fill, is variable, as the general area consisted of sand mounds, and lagoon and swamp deposits incised by multiple channels with depths ranging from a few feet to tens of feet. The general elevation of former San Juan Naval Air Station site is approximately 3 meters above mean Sea Level.

As indicated above, the area of the project was, originally, a lagoon, with deposits consisting of sand, silt and organic material. The topography of the bottom of the area was variable.

The borings drilled at the subject site disclosed an uppermost fill deposit extending from ground surface to depths ranging from 6 to 10 feet in depth. Underlying the above, the borings disclosed soft compressible deposits consisting of mixtures of sand, silt, and clay, with variable amounts of organic material (peat). The organic materials were mostly observed in layers ranging in thickness from a few inches to as much as 15 feet in places. This deposit extends to depths ranging from 25 to 45 feet in depth. The maximum thickness of this deposit was generally encountered in the central and northern portion of the site, although a specific pattern could not be established.

Underlying the above-described deposits, the soil borings disclosed interbedded strata of silty clays with variable amounts of sands and clayey sands. The consistency of the cohesive samples is generally very stiff to hard, while the relative density of the granular samples (silty and clayey sands) is generally dense to very dense, with N-indices ranging from the low 30's to in excess of 150 blows per foot of penetration.

The above strata are underlain by the natural limestone formation of the area.

Since the construction of the former San Juan Naval Air Station sixty years ago, no additional fill has been deposited at the site, and although settlement continues to occur throughout the site, the magnitude has been gradually diminishing. The site design for the PRCCD required that no additional fill be placed at the site, to minimize the possibility of triggering a new consolidation process. Additionally, all the settlement sensitive structures at the PRCCD were to be supported on stone columns or piles.

The surprise

A graphical representation of the driving logs of Piles TP3 and TP5, are presented in Figures 1 and 2, respectively, along with corresponding values of N-indices from nearby borehole.

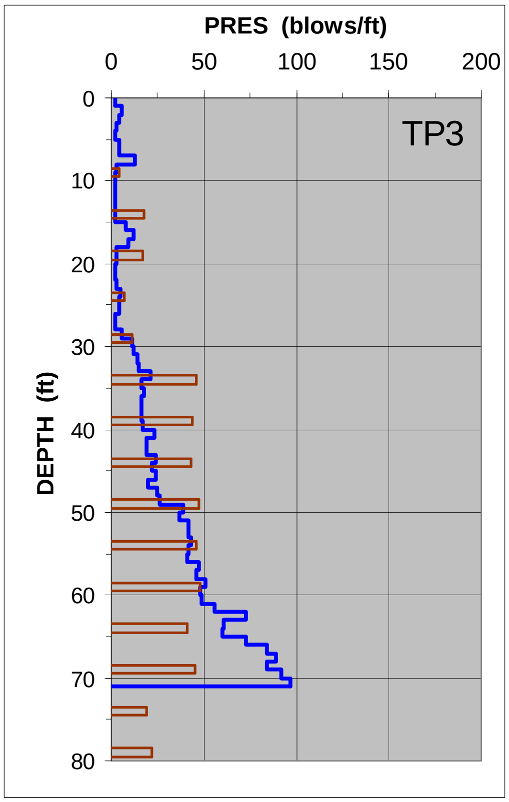

Figure 1: Driving log – Pile TP3

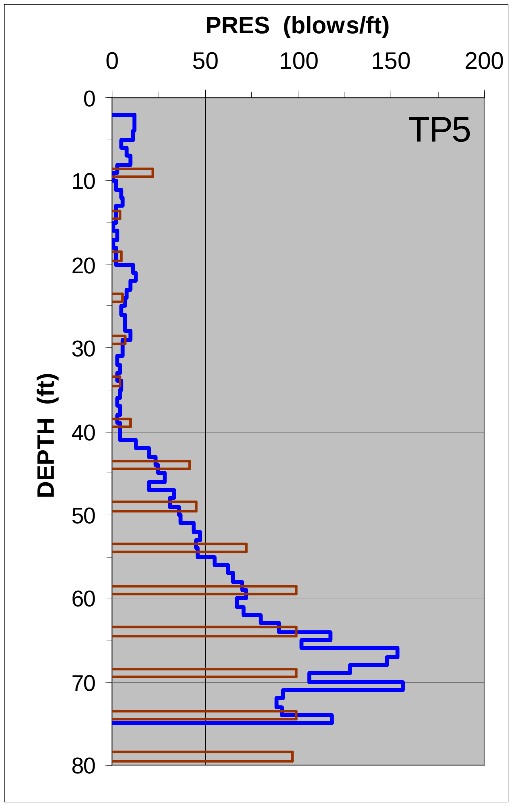

Figure 2: Driving log – Pile TP5

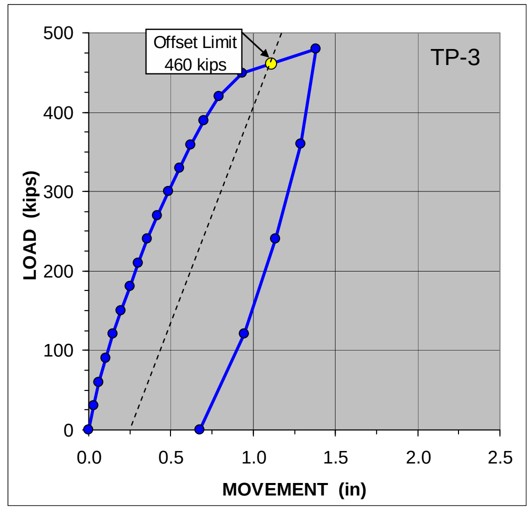

Figure 3: Static loading test – Pile TP3

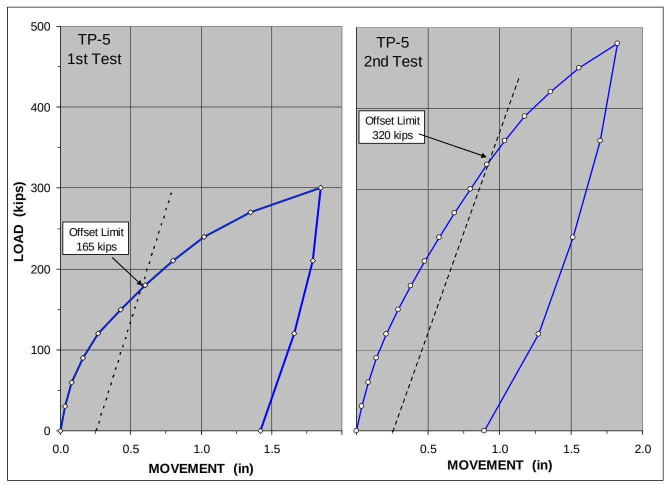

Figure 4: Static loading tests – Pile TP5

The spaced red bars in Figures 1 and 2 are the recorded N-indices from boreholes near the corresponding pile location, while the continuous blue line represents the recorded number of blows per foot of pile penetration.

The two piles are driven at the same site about 100 feet from each other on the same day, and with the same hammer. Of the two, one would expect Pile TP5 to show a significantly higher capacity in the static loading test two weeks after the end-of-driving.

What started as a routine sequence of static loading tests to verify the capacity of index piles, turned into a reality check. Based on the Davisson Offset Limit interpretation, Piles TP3 and TP7 tested at 460 kips and 496 kips, respectively, while Pile TP5 showed an Offset Limit of a mere 160 kips.

The actual test results for Piles TP3 and TP5 are shown in Figures 3 and 4, respectively. Following the first static loading test, Pile TP5 was driven an additional 10 feet to a depth of 75 feet and then re-tested about two weeks later. The second loading test of TP5 showed an Offset Limit of 330 kips, still short of the required capacity of 440 kips for the project.

Due to the erratic pile behavior between pile penetration, blow count and pile capacity, it was decided to include dynamic pile testing into the continued pile-testing programme for the project.

As driving of the production piles progressed, the erratic behavior of the piles became evident. Piles as close as 20 feet from each other disclosed significant variations in driving conditions, attaining the original termination criterion at depths of 40 to 50 feet, while nearby piles were driven to depths of 90 to 100 feet below existing grade at penetration resistances as low as 5 blows per foot.

In another localized area of the site, pile heads started shattering when the pile toe reached a depth of about 35 to 38 feet. A subsequent borehole drilled at this location revealed the presence of a very dense layer of coarse sand. The greatly increased force reflection from the pile toe in this layer, together with the impact force still entering the pile, exceeded the pile's structural strength.

It should be noted that the hammer performance was consistent throughout the driving of index piles and production piles. Furthermore, another hammer of similar make and model was brought to the site later during production piling to speed up the construction. The same trend of erratic pile behavior was encountered while using the second hammer. Comparative dynamic testing showed no difference in performance between the two hammers.

Dynamic (PDA) testing

In light of the unexpected results of the index pile testing programme, GMTS Corp. retained AATech Scientific Inc. (ASI) to perform dynamic (PDA) testing on index piles and production piles at the start of piling. The purpose of the PDA testing was two-fold:

1. Investigate the discrepancy between the behavior of the index piles during driving and their performance under applied loads

2. Establish reliable termination criteria for production piling which take into account the erratic pile behavior across the site.

During the first phase of PDA testing, all seven index piles were tested at restrike, plus an additional pile replacing Pile TP5. While the hammer was suitable for driving the piles, in the restrike tests after several weeks of soil setup, the hammer could not fully mobilizing the pile capacity, where the penetration resistance exceeded 20 blows per inch. The hammer did however mobilize enough resistance (in excess of 440 kips), in all but Piles TP5 and TP7, verifying that the capacity for the project was adequate. No further investigation was deemed necessary for Pile TP7 since it had already passed the static loading test (The Offset Limit was 395 kips in the test).

The initial restrike test on Pile TP5 (after the static test) showed a mobilized resistance of 335 kips (CAPWAP) with no measurable displacement of the pile head beyond elastic compression. The pile was then driven two additional feet (redrive) while instrumented and monitored, to a final depth of 73 feet. During the redrive, the measured resistance first decayed to 310 kips then rose to 360 kips by the end of redrive.

The dynamic testing of Pile TP5 provided a wealth of information about the site characteristics and pile behavior. The pile response at the beginning and the end of the restrike test was modeled by CASE Pile Wave Analysis Program (CAPWAP). CAPWAP is a one-dimensional numerical modeling software that assists in determining the soil model parameters and pile resistance distribution. This is done by matching the recorded pile response to the stress wave generated by the hammer impact with a calculated response using the wave equation formulation and iterating on the pile-soil model parameters. A similar analysis was performed on blows from the beginning of restrike testing of each index pile.

The soil models determined by CAPWAP analysis provided an explanation for the erratic correlation between the observed penetration resistance and the pile capacity. For example, a comparison of the profile of soil parameters determined for Piles TP3 and TP5, showed that Pile TP5 had been driven into soil which required a larger movement (i.e., larger quake) before the soil resistance was mobilized, especially in the upper 40 feet of softer compressible deposits. This meant that a larger number of blows was required to advance the pile as opposed to Pile TP3 which showed a more usual distribution of quake values. The profiles of computed quake values along the shaft and at the toe of both piles are presented in Figure 5 (toe quake is represented by a red bar).

The higher quake values also explain why the same hammer could not mobilize as much resistance in Pile TP5 at the very large penetration resistance at restrike (no net penetration) as opposed to the case for the other test piles.

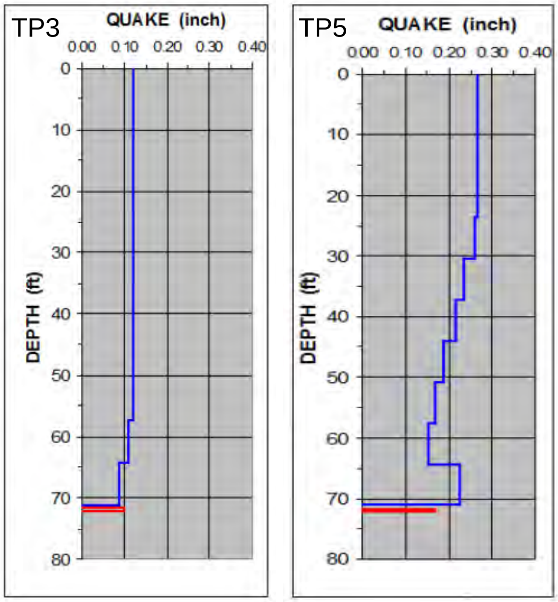

Figure 5: Quake values from CAPWAP analyses

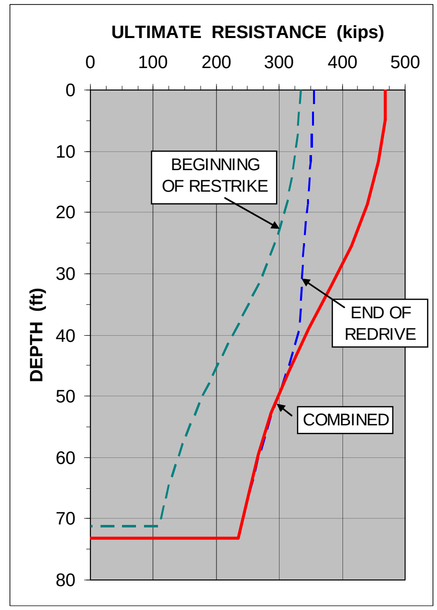

Figure 6: Pile TP5 resistance profile – CAPWAP

The dynamic testing of Pile TP5 was also instrumental in ensuring that the pile was driven to an adequate capacity. Although the resistance measured at the end of redrive was not sufficient, a minimum long-term capacity was predicted based on combined results of CAPWAP analyses at the beginning and end of the final redrive (last 2 feet of penetration). As the redrive progressed, the shaft resistance degraded within the upper 40 feet of the softer compressible soils and organic fill. The breakdown of upper shaft resistance allowed more driving energy and force to reach the pile toe, which facilitated its penetration into more competent bearing material, and at the same time allowed the mobilization (and measurement) of higher resistance through larger toe movement. By combining the resistance profiles from the beginning and end of the redrive (i.e. accounting for the recovery of the decayed shaft resistance), a long-term pile capacity in excess of 450 kips can be reasonably predicted. This analysis is illustrated in Figure 6 which shows the cumulative pile resistance profiles as computed by CAPWAP analyses.

Following the index pile testing, more cases of erratic pile behavior were encountered. Additional PDA testing was performed at various occasions to develop termination criteria that would address the encountered variation in pile behavior across the site, as well as to verify the capacity of selected production piles. As a result, a complex set of criteria and guidelines was developed for monitoring the driving of remaining production piles. Periodic checks on the hammer in addition to the frequent verification of hammer performance through dynamic testing.

Discussion

Although the larger soil quake can explain the erratic pile behavior at this site, there is nothing in the geotechnical data that can explain the large quake difference between soils at two adjacent site locations. Neither the descriptions in the borehole logs nor the measured characteristics of the soils suggest such discrepancy in pile behavior. The compilation shown in Figure 7 shows reported N-indices from Boreholes 3 and 5 (near corresponding Piles TP3 and TP5, respectively), and natural water content at different depths from boreholes across the site. Profiles of water content values suggest consistent soils across the board, even though soil description varies somewhat between the different locations. More intriguing is the fact that the N-indices suggest, without a doubt, that Pile TP5 would show a significantly higher resistance. Instead, the measured resistance of the index piles seems to be, in this case, inversely proportional to the N-indices.

These results, to say the least, put a significant damper upon relying on crude tools such as SPT N-indices to design piles in such inherently unpredictable soils.

One can speculate as to which materials exhibited the high quake characteristics. There is no shortage of suspects at this site. It could be the bed of the old mangrove ponds and swamps, or the hydraulic silt and sand dredged from the channel and used as a backfill some sixty years ago.

If anything, this case history highlights the vital role of dynamic testing, and more generally, the role of a properly designed index testing programme, in building deep foundations. In other words, the problem at the Sheraton site could have been easily missed, even with the most commonly applied practices in foundation engineering. It is in the opinion of the authors that the main credit in this case is due to GMTS Corp. for due diligence and thorough verification applied at the onset and throughout this project.

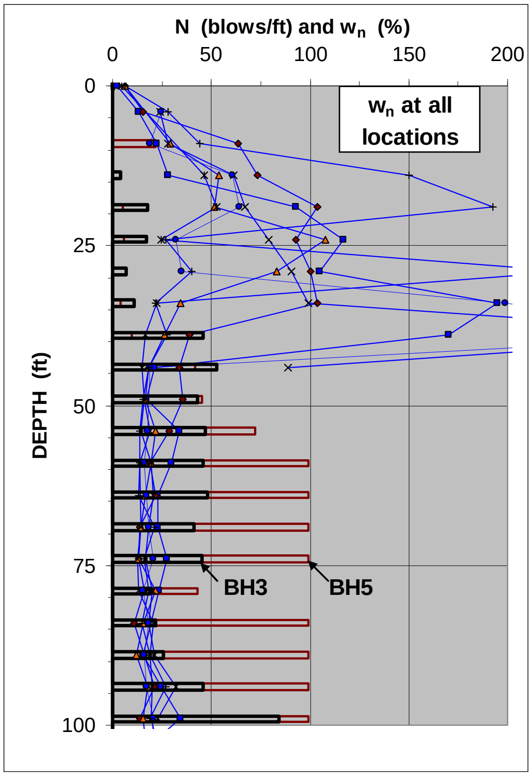

Figure 7: compilation of soil properties