PERFORMANCE EVALUATION OF CFA VS. BENTONITE SLURRY DRILLED SHAFTS UTILIZING DROP WEIGHT TESTING

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ABSTRACT: The performance of drilled shafts is known to be controlled by the type and quality of the construction methods. A comparison between different methods or different contractors is typically not available under the prevailing bidding and testing procedures.

Six drilled shafts 0.70 m in diameter and 25.0 m long were installed and tested to structural failure at a site in Haifa, Israel. Three of the shafts were constructed using the Continuous Flight Auger (CFA) construction method and three of the drilled shafts were constructed using bentonite slurry.

Site and shafts details are provided. The drop weight testing system is reviewed. Test results comparing the load carrying capacity of the shafts, load distribution and structural outlines are presented and discussed. While the bentonite slurry construction resulted in a more uniform constructed foundation capable of carrying higher load, the explanation to the variation at the given site conditions may be a result of the contractor's quality rather than advantages of one technology over another.

BACKGROUND

Overview

Drilled shafts are the prevailing deep foundation solutions in some parts of the world. Subsurface conditions consisting of hard clay, cemented sands, karstic limestone, low

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groundwater table, combined with high price of steel and fuel (transportation) in contrast with low price of concrete, make drilled shafts the preferable solution of deep foundations in Israel. As a result, various construction and quality control technologies are developed there and explored. Bentonite slurry drilled shaft (wet) construction is most commonly used while Continuous Flight Auger (CFA) is a relatively newcomer.

The bentonite method refers to the "wet" construction method (slurry-displaced method) in which bentonite slurry is used to keep the borehole stable during excavation. The construction process consists of the following stages:

(i) Drilling equipment is used to drill to the groundwater surface.
(ii) Bentonite slurry is introduced into the hole and the drilling is continued to the full depth of the hole. The slurry elevation is kept continuously above the groundwater surface elevation during construction and its quality is assured at the end of the construction.
(iii) A reinforcement steel cage is placed in the slurry.
(iv) Concrete is placed in the excavation using tremie with the bottom of the tremie remaining below the surface of the concrete. While the column of the concrete rises in the excavation, the slurry is displaced and pumped back into storage.

The Continuous Flight Auger (CFA) construction sequence is comprised of five stages:

(i) The digging head of the auger is fitted with an expendable cap.
(ii) The auger is screwed into the ground to the required depth.
(iii) Concrete is pumped through the hollow stem, blowing off the expendable cap under pressure.
(iv) Maintaining positive concrete pressure, the auger is withdrawn all the way to the surface.
(v) Reinforcement is placed into the pile up to the required depth.

The prices of both construction technologies are comparable (in Israel) with advocates to each of the techniques. Bentonite slurry construction is assumed to allow for better quality control (concrete volume per length, tubes for CSL etc.) while suspected of having potentially reduced interaction and strength at the interface with the subsurface soils, in particular at the tip. CFA method leaves more questions as to the quality of the construction (in spite of instrumentation monitoring auger and concrete pressure, and concrete volume), but is assumed to guarantee better interaction with the soil, i.e. higher friction and end bearing free of a possible slurry "cake" at the concrete-soil interface.

A large multi-story expansion of a pharmaceutical manufacturing complex called for the use of deep foundations. With similar cost estimation of bentonite and CFA construction and the need for performance verification, the owner had decided to conduct pre-bid testing of six deep foundation elements, constructed at the same location, three using bentonite slurry and three using CFA technique. Impact tests utilizing a drop weight system were used to examine the shafts.

Site and Subsurface Conditions

The site is located in Haifa Bay, west of Kiryat Ata intersection. The subsurface profile at the site is described in Figure 1. The upper 0.25 m consisted of an existing concrete slab. The subsurface itself consists of a yellow to gray uniform fine sand to a depth of 20 m; from 0 to 4 m the sand is medium dense and it becomes dense to very dense from

4 to 20 m. The sand layer is underlain by a gray to black, stiff, high plasticity clay from 20 to 22.7 m. A very dense yellow sand and calcareous sandstone underlies the clay layer to a depth of 30 m (which corresponds to the end of the borings). Groundwater was encountered during drilling at a depth of 5.8 m, which may not accurately represent stable groundwater level. Six separate test piles designated as B1 through B3 and C1 through C3 (see Figure 2 – Site Layout) were tested. The piles were located within the footprint of the proposed structure but were not planned to become a part of the permanent foundations.

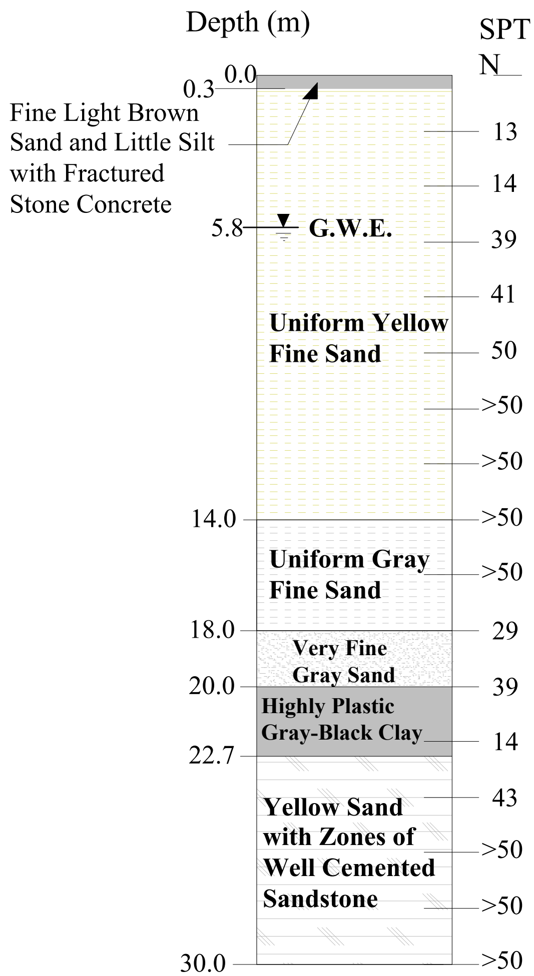

FIG. 1. Subsurface Conditions at the Test Site.

Pile Details

Details about the piles construction are provided in Table1. Three of the piles were constructed using the Continuous Flight Auger (CFA) technique between August 8 to 11, 2002 and built with type B-30 concrete (nominal strength of 30MPa). The CFA pile extensions above ground level (see details in Table 1) were built on August 21,

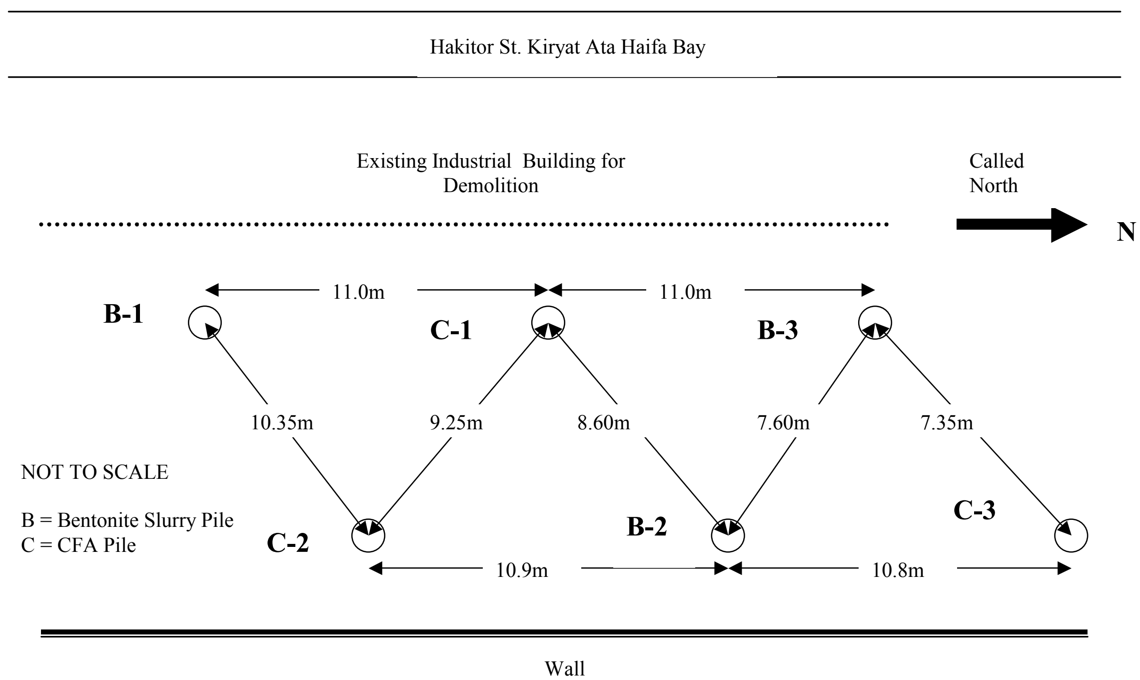

FIG. 2. Site Layout

TABLE 1. Summary of Drilled Shaft Construction Details

Shaft No.	Shaft Type	Date of Construction	Date of Extension Construction	Height of Extension (m)	Comments
C-1	CFA	8/11 to 8/13	8/21	1.40	Concrete mushroom around pile top was broken prior to extension construction, all steel sleeves removed prior to testing.
C-2	CFA	8/11 to 8/13	8/21	1.45	
C-3	CFA	8/11 to 8/13	8/21	1.34	
B-1	Bentonite Slurry	8/18	8/18	1.38 (1.12) [3]	No separation between pile and slab, poor quality concrete on top and circumference
B-2	Bentonite Slurry	8/19	8/19	1.50	Non-round extension
B-3	Bentonite Slurry	8/19	8/19	1.15	Non-round extension

Notes
1. All nominal pile sizes are 70 cm in diameter and 25 meters in length
2. All steel sleeve extensions of the bentonite piles are 1.7 meters long.
3. Number in parentheses is the height of the extension after the top was cut off to enable placement of the guide system

2002 using B-50 concrete. The remaining three piles were constructed using an auger bucket under wet drilling technique of bentonite slurry on August 18 and 19, 2002. The bentonite slurry pile extensions were built on the same day along with the pile construction. All piles were designed to have a nominal size of 70 cm in diameter and 25 m in length. The nominal cross-sectional area is 3847 square centimeters. The maximum allowable compressive stress limit is around 26 MPa, based on 85% of the 28-day compressive strength (0.85 f′c). The maximum allowable tensile impact stress is 1.47 MPa, based on three times the square root of the 28-day compressive strength (3 $f'c^{1/2}$ in units of pound per square inch). The 28-day concrete compressive strength (fc) was reported to be around 30 MPa. The concrete extensions cast on top of the piles were approximately 1.5 meters in length.

IMPACT TESTING OF DRILLED SHAFTS USING DROP WEIGHT SYSTEMS

Background and Use

Increasingly, drop weight systems are being used to dynamically test cast-in-place deep foundations. Conventional pile driving hammers are often inadequate to test these deep foundation types since (i) they typically cannot deliver enough energy to mobilize the ultimate bearing capacity, and (ii) the size and location of the foundation member can present problems in adequately delivering the energy from the ram to the pile. Simple drop weight systems have therefore been developed to overcome the limitations of the conventional hammers and allow for dynamic testing of in place constructed deep foundations.

An in depth review of various available drop weight systems, and evaluation of the method is presented by Paikowsky et al. (2003). A typical drop weight system consists of four components: a frame or guide for the drop weight (ram), ram, a trip mechanism to release the weight, and a striker plate/cushion. Strain gages and accelerometers are placed at the pile top to obtain stress wave measurements utilizing available PDA's (Pile Driving Analyzers). Figure 3 shows the setup of an Israeli Drop Weight Impact Device, developed and used by GeoDynamica and GTR to test drilled shafts in Israel. This device is similar in principle to other drop weight systems presently in use with the distinction of modularity in ram weight as well as uniqueness in trip mechanism. The Israeli Drop Weight Impact Device uses modular weights that can be arranged into ram weights of 2, 4, 5, 7, or 9 tons and has an adjustable drop height of up to 4 meters thereby allowing for potential energy of up to 36 t·m (260 kip·ft.). Typically, a pile cushion is used to even pile stresses occurring during impact.

(a) schematic (b) photograph

FIG. 3. Israeli Drop Weight System (after GTR, 1997).

Advantages

Drop weight systems have several advantages when compared to traditional static load testing methods. Most of these advantages are similar to those presented by standard dynamic pile testing, namely:

- Rapid testing time allowing to carry out tests on several shafts in a single day.
- The ability to deliver high force and energy to mobilize the capacity, and hence test large deep foundations.
- The ability to use available transducers and data acquisition systems (such as the Pile Driving Analyzer of Pile Dynamics and the TNO Foundation Pile Diagnostic System).
- The test provides a means to conduct high strain integrity testing concurrent with capacity determination.
- Low test cost relative to standard static load test.
- Current analysis techniques include field methods (such as the Case Method and the Energy Approach) and signal matching (e.g. CAPWAP).

Disadvantages

There are several disadvantages and limitations relating to the use drop weight systems:

- The selected mass and drop height must be of sufficient magnitude to mobilize the resistance of the deep foundation shaft in order to obtain adequate capacity measurements.
- The installation process of in-situ deep foundations (drilled shafts, cast-in-place piles, etc) can cause irregularities in pile shape and homogeneity that can affect current analysis methods.
- For increased quality of the obtained measurements the gauges need to be away from the impact and as high as possible above the ground. This results in the need to create "extensions" of cast in place shafts and the use of multiple gauge systems.
- Although several studies have already been conducted comparing dynamic and static measurements of cast-in-place deep foundations (Rausche and Seidel, 1984, Jianren and Shihong, 1992, Townsend et al., 1991), a comprehensive comparison study has only recently been completed and presented by Paikowsky et al. (2003). Only a few studies are known to compare static and dynamic testing of CFA piles, one of which was presented by Cannon (2000).

FIELD TESTING

Impact Device

The aforementioned drop weight device was used to provide high force and energy impacts necessary to mobilize the soil resistance acting along the side (friction) and tip (end bearing) of the shafts. The device is typically adjusted to test shaft heads ranging from 60 to 80 cm in diameter. Due to non-round, non-uniform pile head extensions, on site modifications had to be performed specifically for the presented testing.

Ram weights of 7 tons (74 kN including attachment elements) were used during testing for this project. The strokes were typically varied between 0.25 and 2 meters, resulting in rated energies between 18.5 and 148.0 kN-m. Plywood sheets varying in total thickness between 40 to 100 millimeters in thickness were used for the pile cushion along with a 25 mm thick steel plate (for details see Table 2).

Instrumentation

The instrumentation consists of four strain gage and four accelerometer transducers attached approximately 1m below the top of the pile extension. A strain gage and accelerometer pair were bolted 90 degrees apart on the circumference of the pile to minimize the effects of uneven impact and pile bending. This instrumentation provides information about driving stresses (compressive and tensile), driving system performance (alignment of ram and transferred energy), and pile capacity. To further enhance the ability to monitor data quality, one accelerometer was attached to the ram itself, allowing measurement of the ram acceleration, and hence, the force developed at the top of the extension. This enabled independent measurement of the impact forces evaluated via the strain gages.

TABLE 2. Summary of Dynamic Load Test Results Taro Corporation, Haifa

Shaft No.	Depth (m)	Diam. (cm)	Blow No.	Ram Weight (kN)	Cushion Thick [1] (mm)	Stroke[2] (m)	Max Transferred Energy[3] (kN*m)	Max Displacement[3] (mm)	Pile Set[4] (mm)	Max Compressive Stress[3] (MPa)	Max Tensile Stress[5] (MPa)	Max Pile Top Force[3] (kN)	Case Method Capacity[6] (kN)	Energy Approach Capacity[7] (kN)	CAPWAP Capacity[8] (kN)	Predicted Ultimate Capacity[9] (kN)
B-1	25	70	2	74	3P+S+2P	1.01	38.2	8	2/3.0	15.8	2.5	6015	4005	6950	5600	5500
B-2	25	70	4	74	2P+S+2P	1.51	67.5	7	2/2.0	36.2	0.7	13749	5358	15000	5792	6000
B-3	25	70	6	74	2P+S+2P	2.01	114.0	15	3/5.0	30.9	5.5	11753	5778	11400	5695	6000
C-1	25	70	3	74	3P	0.95	31.9	6	2/3.0	24.8	0	9432	4220	7090	5450	5500
C-2	25	70	4	74	3P	1.33	40.9	6	1/1.0	22.6	0.8	8578	5325	11690	6145	6000
C-3	25	70	9	74	3P+S	2.00	77.0	13	7/na	29.3	2.6	11150	6553	7700	5050	5000

Notes

1. The striking plate for all tests was 46.5 cm diameter and 10 cm in thickness. The cushion system consisted of a combination of plywood and steel plates of the following thickness and diameter: Plywood (P) = diam = 70 cm and thick = 2 cm, Steel (S) = diam = 65 cm and thick = 2.5 cm
2. The stroke was typically increased from 0.25 m to 2.5 m during testing of each pile.
3. The maximum transferred energy; displacement, compressive stress, and force are determined by the PDA at the gage locations.
4. The pile set is presented as two values (1/1.0). The value on the left was determined from the PDA by integrating the acceleration measurements twice and the value on the right was measured independently using a level (accurate to 0.1 mm).
5. The maximum tensile stresses were calculated by the PDA and can be located anywhere along the pile shaft.
6. The Case Method was determined using the RMX method and a damping coefficient of 0.5 (RX5).
7. The Energy Approach Procedure was developed by GTR personnel and was proven to provide high accuracy long-term driven pile capacity at end of driving.
8. The CAPWAP capacity was determined using a computer program, which is capable of providing an estimate of the soil distribution.
9. The predicted pile capacity was determined based on the three methods described above.

The PDA is a computer fitted with a data acquisition and a signal conditioning system. The transducers are connected to the PDA via cables. During impact, the strain and acceleration signals are recorded and processed for each blow. The strain signal is converted to a force record and the acceleration signal is integrated to a velocity record. The PDA saves selected blows containing this information to disk and determines the compressive stresses, displacement, and energy at the point of measurement (pile top). In addition, the tensile stresses can be calculated and the pile bearing capacity determined using a procedure known as the Case Method. This information can be viewed on the computer screen during driving. Selected blows can be further processed to predict the static pile capacity using the Energy Approach method and CAPWAP analyses.

Testing Procedure

Shafts C1 to C3 were tested on August 28, 2002. Shafts B1 to B3 were tested the next day on August 29, 2002. A 7-ton ram (74 kN including attachments) was used to apply the impact force for all test piles. Between 5 and 13 blows were applied to each pile. The stroke was gradually increased from 0.25 meters to 2.5 meters in order to ensure that: (1) the impact stresses were as evenly distributed as possible, (2) the pile cushion was properly compressed prior to the last few blows, (3) allowable stress limits could be closely monitored, and (4) full mobilization of capacity could be observed prior to damaging the piles.

TESTING RESULTS

General

The results of the dynamic testing program are summarized in Table 2. Table 2 includes the pile depth below ground, shaft diameter, stroke, maximum transferred energy, maximum displacement, pile set, maximum compressive stress, maximum tensile stress, and maximum force for one selected blow on each pile. The maximum transferred energy, displacement, pile set, compressive stress, and force are determined by the PDA at the gage locations and are representative for the blow indicated. The ram stroke was measured in the field. The maximum tensile stress was estimated by the PDA and can occur at any location along the shaft. Also included in Table 2, are the pile bearing capacities as predicted by the Case and Energy Approach methods in the field and CAPWAP analyses in the office. Table 3 summarizes the CAPWAP results in more details separating the friction and end-bearing contributions as well as the contribution of the upper section of the shafts.

Field Observations and Driving System Performance

The pile set (permanent displacement) varied between 0 and 5 mm under each blow. The set was determined based on two procedures (1) double integration of the acceleration record (from the PDA) and (2) independent measurement using a level (accurate to 0.1 mm). The total set for all piles was relatively low, due to the high frictional resistance along the pile. For the 74-kN ram and a stroke height between approximately 1 and 2 m, the transferred energy ranged from 32 to 114 kN-m for the various analyzed blows. The overall driving system efficiency varied, therefore, between 42 and 77%, which is higher than that typically observed in drop weight systems (Paikowsky et al. 2003). The overall high driving system efficiency was a result of changes made to the system during this project.

TABLE 3. Summary of CAPWAP Results Taro Corporation, Haifa

Shaft No.	Depth (m)	Diam (cm)	Blow No.	Capacity (kN)				Skin Friction Total %	Quake (mm)		Damping (sec/m)	
				Upper 5m Friction	Lower 20m Friction	Tip	Total		Side	Tip	Side	Tip
B-1	25	70	2	1100	3000	1500	5600	73	3.0	6.0	0.460	0.126
B-2	25	70	4	1797	2996	999	5792	83	3.0	2.0	0.630	0.472
B-3	25	70	6	1000	3696	999	5695	82	3.0	13.0	0.442	0.151
C-1	25	70	3	2305	2545	600	5450	89	1.0	6.0	0.545	0.314
C-2	25	70	4	2198	2448	1499	6145	76	1.3	5.0	0.528	0.755
C-3	25	70	9	2200	2250	600	5050	88	1.0	10.0	0.593	0.629

Pile Integrity and Stresses

The maximum compressive stresses ranged between 16 and 25 MPa for strokes between approximately 1 and 1.5 meters and between 28 and 36 MPa using the higher strokes (approximately 1.5 to 2 m). The maximum tensile stresses ranged between approximately 1 and 8 MPa. All shafts were tested until signs of damage (crumbling) of the pile extensions were detected. Typical shafts' top conditions following the tests are shown in Figure 4.

Pile Construction

Evaluation of the shafts' construction can be obtained via the assessment of the variations (profile) of the impedance along the shaft used in the signal matching analysis (CAPWAP). The pile impedance is a measure of the concrete quality (through modulus and density) and cross sectional area in the following way:

$$I = EA/c \tag{1}$$

$$c = \sqrt{E/\rho} \tag{2}$$

for which: I = pile impedance
E = modulus of elasticity
A = cross-sectional area
c = speed of one-dimensional wave propagation

ρ = unit density

(a) (b)

FIG. 4. Shaft's Top Extension Condition Following Drop Weight Tests (a) Shaft B3 Bentonite Slurry Cast in Place Shaft with a Steel Sleeve, (b) Shaft C3, CFA Constructed Shaft with Steel Sleeve Removed from the Above Ground Extension.

An expected impedance for a 70 cm diameter shaft ($A = 3848\ cm^2$) of concrete with $c = 4000$ m/s and $\rho = 2400\ kg/m^3$ is 3765 kN/m/s. Figure 5 describes the variation of the impedance along the shaft used in CAPWAP analyses for each of the shafts tested. Three distinct observations can be made in relation to the profiles presented in Figure 5; (i) the shafts constructed with bentonite slurry seem to be relatively uniform and present overall impedance equal or better than the one expected (excluding the lower part of B2), (ii) all shafts seem to have some type of increase in the impedance at the upper section, and (iii) all the CFA constructed shafts show a distinct decrease in the impedance, to a level lower than the expected value, below the depth of about 20 m. As the impedance profile is based on a combination of cross-sectional area and quality of concrete, a reduction below the expected value from any of the reasons is of concern. More so, the shaft model in the CAPWAP analyses was based on its anticipated (constructed) length. When the applied impact results with a high stress short duration stress wave, the stress reflection from the tip is clear and the shaft's length can be determined. Often in drop weight testing the produced stress wave is either not sharp enough (as in the presented tests), or the energy is not high enough to mobilize the shaft's tip and hence its clear detection. As such, the analysis utilized the designed length of 25 m. The above observation regarding the CFA shaft can in essence be interpreted that either the quality of the shaft in the lower 5 m was compromised, or the shaft was not constructed to the planned depth of 25 m.

To elucidate this situation, further analyses have been carried out on shafts B1, C1, C2, and C3. The dashed lines related to shafts C1, C2, and C3 in Figure 5 are the

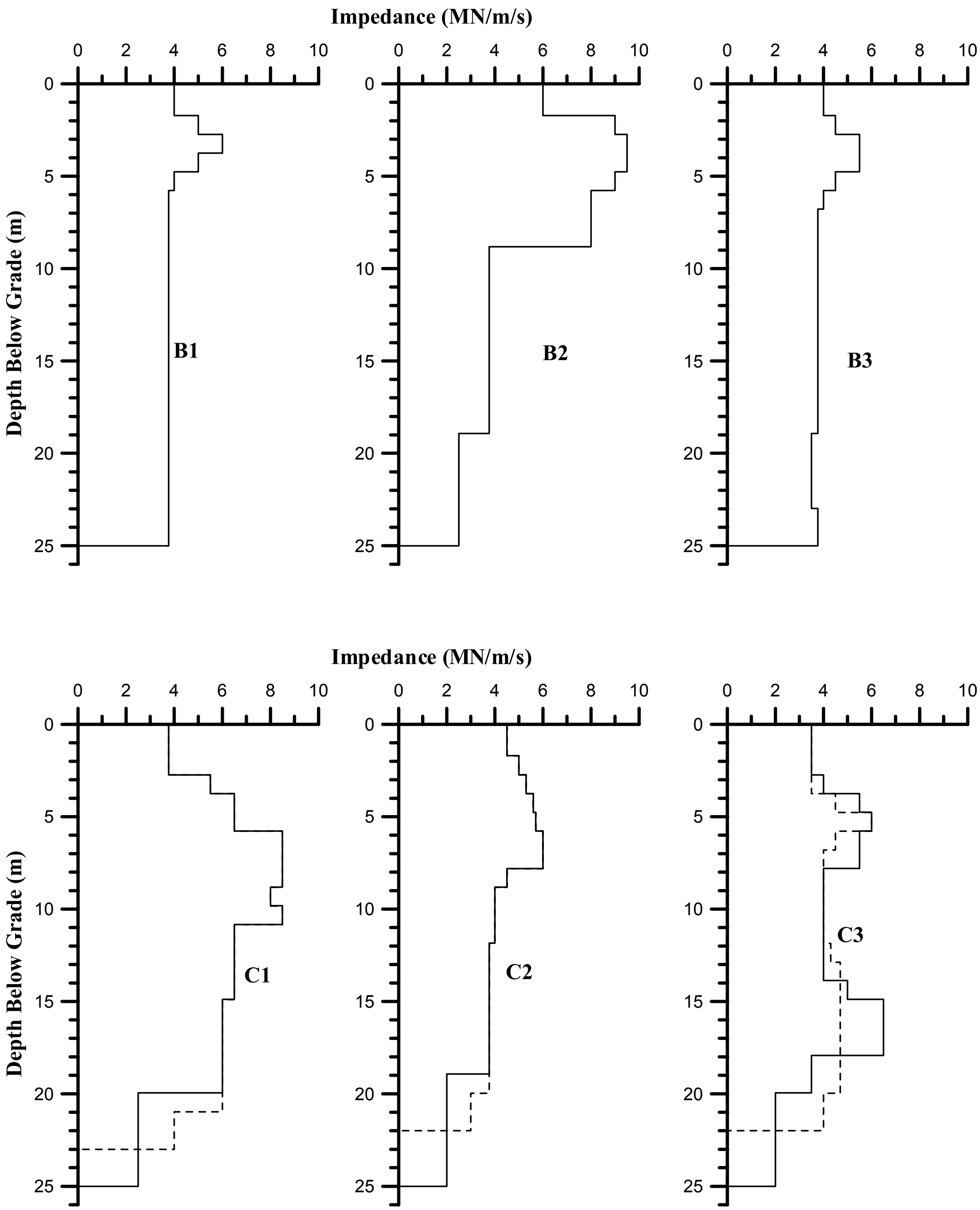

FIG. 5. Variation of Shaft Impedance with Depth for the Bentonite (B1 – B3) and CFA (C1 – C3) Constructed Shafts.

impedance used when assuming a shaft length below ground surface of only 22.0 m to 23.0 m. The revised analyses required small change in the capacity distribution and the use of the same soil parameters. The total resistance remained about the same (5,450, 6,045 and 5,030 kN vs. 5,450, 6,145, and 5,050 kN for C1, C2 and C3 in the new analysis vs. the original analysis, respectively), while the quality of the match obtained in the analyses increased. It was, therefore, concluded that the CFA shafts were constructed 22 – 23 m long, shorter than the planned length. To further examine the validity of this analysis, shaft B1 was reanalyzed assuming a shaft length of 22.0 m. In spite of many trials, the analysis resulted with a match of much lower quality affirming the inability to arbitrarily "shorten" a good quality shaft.

Pile Bearing Capacity

Pile bearing capacity was determined using the Case Method, Energy Approach, and CAPWAP procedures. Table 2 presents the capacities for the tested piles. The Case Method capacities (using the RMX procedure and a damping factor of 0.5), ranged between 4,005 kN and 5,563 kN. The use of the damping factor of Jc=0.5 for the predominantly granular subsurface provides value slightly conservative. The CAPWAP capacities varied between 5,050 and 6,145 kN. Table 3 present the results of the CAPWAP analyses in more detail. The total capacity, frictional capacity in the upper 5 meters, frictional capacity below a depth of 5 meters, end bearing (tip) capacity, and percentage of skin friction are included. The percent skin friction was consistently between 75 and 90%. Comparison of the dynamic prediction values of impacted drilled shafts to actual static load test results was presented by Paikowsky et al. (2003), and for driven piles under restrike conditions by Paikowsky and Stenerson (2000) and Paikowsky (2002). Both studies suggested high accuracy of this analysis for both cast in place piles (bias = 1.05, COV = 0.12, n = 39) and driven piles under restrike conditions (bias=1.16, COV = 0.34, n = 162). The bias in both cases represents the static capacity over the dynamic predicted value. Cannon (2000) presented a comparison between dynamic and static load tests on CFA shafts. His results suggested very good correlation between the two, including high accuracy of the modeled pile in the dynamic analysis, which matched well the recorded volume increase of 205% of the nominal design.

Figure 6 presents the distribution of the friction and accumulated load along the shaft including the tip resistance. The dashed lines in the distribution of shafts C1, C2, and C3 represent the revised length analysis discussed in the previous section. It can be concluded that within the accuracy of the differentiation (between the bearing components), a similar soil-pile interaction was observed for all piles regardless of the construction method. Inspecting the friction distribution along the shaft and the build up of the resistance (load) along it suggests, however, that the piles constructed with bentonite exhibit overall a much better distribution all along it, while the CFA constructed shafts exhibit high friction in the upper 12 to 14 m but very low frictional resistance below this depth. For one, this lack of friction and lower end bearing in the lower part affirms the aforementioned conclusions that the CFA shafts were constructed shorter than designed. Referring to Figure 1, it is possible that the shorter CFA shafts either ended in the clay layer or just about penetrated through it.

The results also suggests high soil mobilization around the areas of larger impedance, which can be associated with "bulging" out zones and/or some interaction between the shaft and the slab (see comments in Table 1).

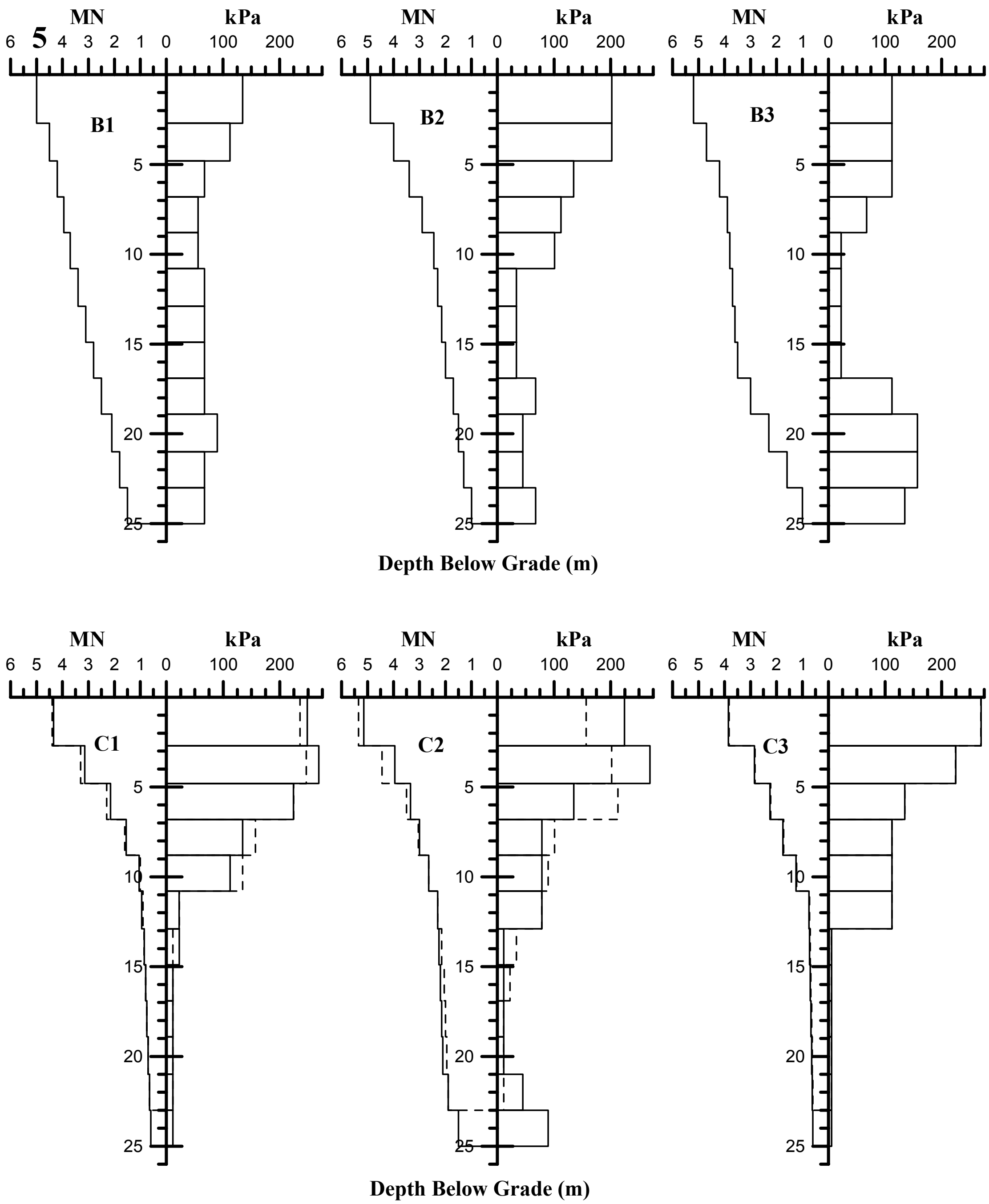

FIG. 6. Distribution of Friction and Accumulated Load Along the Shaft (Including Tip Resistance) for the Bentonite (B1 – B2) and CFA (C1 – C3) Constructed Shafts.

The Energy Approach method was developed by GTR personnel (Paikowsky, 1982, Paikowsky and Chernauskas, 1992, and Paikowsky and Stenerson, 2000) and was proven to provide high accuracy long-term pile capacity when applied to driven piles at the end of driving. This method relies on the relationship between the transferred energy and the work done by the pile during penetration. Measured values are used to determine the capacity (transferred energy, maximum pile displacement, and set). The method though not ideal for cast in place piles provides an indication for the upper end of the pile capacity in such cases. The Energy Approach capacities ranged between approximately 7,000 and 15,000 kN. In general, they were consistently higher than the Case method and CAPWAP capacities.

CONCLUSIONS

The presented data from the drop weight testing and the analyses leads to the following conclusions:

i. The predicted shaft capacities range between 5,000 and 6,000 kN (500 and 600 tons), thereby did not appear to be significantly different for either shaft type.
ii. The CFA shafts apparently developed more frictional resistance in the upper portions of the pile, while the bentonite slurry piles developed more evenly distributed friction. The percentage of tip resistance to the total capacity was lower for the CFA piles, but within a similar range for either pile type.
iii. Based on the CAWAP analyses, the unit skin friction over the lower 20 meters of the bentonite slurry shafts is around 75 kN/m^2 as compared to 55 kN/m^2 for the CFA piles. The unit end bearing resistance averaged around 2,500 kN/m^2.
iv. The shaft profile (based on CAPWAP) was more uniform for the bentonite slurry piles than for the CFA piles. The profile of the shaft impedance is based on material stiffness hence comprised of the combination of cross sectional area and quality of concrete. The CFA shafts were most likely constructed to a depth of 22 to 23 m only. In general, differences between the two pile types are expected, due to the different construction and concrete placement procedures.
v. This study appears to show that the tested bentonite slurry shaft construction resulted with a higher quality deep foundation element than that constructed by the CFA method. Considering, however, that the CFA shafts were shorter than designed, one may conclude that the importance of the quality of the construction and its monitoring are more significant than the specific construction technology. In other words, the lower performance of the CFA shafts may not be a testimony for a lower quality product, but rather to a lower quality craftsmanship.

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