Results of Dynamic and Static Load Tests on Helical Piles in the varved clay of Massachusetts

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ABSTRACT

Helical Piles are increasingly being used to support and rehabilitate structures subjected to compressive axial loads. Traditionally, empirical correlations between installation torque and load capacity of helical screw piles are used as the field quality control. Load tests, however, are always recommended to validate the installation process and design methodology.

This paper describes a test program performed on helical piles installed at the National Geotechnical Experimentation Site of the University of Massachusetts - Amherst campus (UMass-Amherst), well known for its deep varved clay deposit. Seven of these piles were submitted to Dynamic Load Tests using the Pile Driving Analyzer® (PDA), and also to Static Load Tests. Good agreements were obtained between the results of the dynamic and static load tests, encouraging the use of the former testing method as a viable alternative to the latter for determining the compressive load capacity of helical piles installed in cohesive soils. The results of the load tests were also compared to the predictions based on torque correlations, resulting in correlation coefficients which were higher than the ones traditionally used.

INTRODUCTION

Helical piles were invented by Alexander Mitchell in 1836 and have been used mainly to resist tension as anchor foundation elements. In the last decade, however, they have increasingly been used to support and rehabilitate structures subjected to compressive axial loads.

Helical piles consist of one or more circular helical plates affixed to a central shaft of smaller diameter. They are fabricated from steel and the helices are generally attached to the shaft by welding and then galvanized for extra protection against corrosion.

There are several advantages associated with the use of helical piles. For example, they can be installed using lightweight, sometimes hand-operated equipment, in small and limited spaces; also, the installation does not produce residues, excessive vibrations, or disruptive noise, and is unaffected by the groundwater table. The ease and flexibility of their installation and the cost effectiveness of the solution spurred the growth of the helical pile industry.

The works of Wilson (1950), Robison & Taylor (1969), Hoyt & Clemence (1989), Ghaly & Hanna (1991) and Perko (2000), among others, established the theoretical background for the design of Helical Piles. To verify the bearing capacity of those piles the industry relies mainly on torque correlations. The use of empirical torque correlations, however, is viewed with reservation by some engineers, who see a dependency of the procedures adopted by the installer on the results. Also, some engineers have misgivings about determining the capacity of helical piles using only torque measurements, without taking into consideration geotechnical parameters (Cannon, 2000).

It is recognized that the use of traditional geotechnical design methods, such as individual bearing and cylindrical shear methods, can predict the helical pile capacity (Perko, 2009). Load tests, however, are necessary to establish the pile capacity to torque ratio (Kt), used to control the capacity of the production piles.

This requirement motivated the present research project, which submitted Helical Piles to both Dynamic Load Tests (DLT) and to Static Load Tests (SLT). The results of the two kinds of load tests were compared, and the correlation of the load tests with torque measurements was also investigated.

DESCRIPTION OF THE SITE

The National Geotechnical Experimentation Site (NGES) of the University of Massachusetts - Amherst campus (UMass-Amherst) is situated in the Connecticut River Valley and within the region of the former glacial Lake Hitchcock. The lacustrine sediment deposits originated as a result of an ice-wall dam, which formed across the valley in northern Connecticut creating seasonal deposition and settling of fine-grained particles over the coarser glacial till for a period of approximately 4000 years. This soil is locally known as Connecticut Valley Varved Clay (CVVC) and extends from Northern Vermont to Central Connecticut in the present Connecticut River Valley. The individual varves of the CVVC are on the order of 2 to 8 mm (0.08 to 0.31 in) in thickness and are generally horizontally layered.

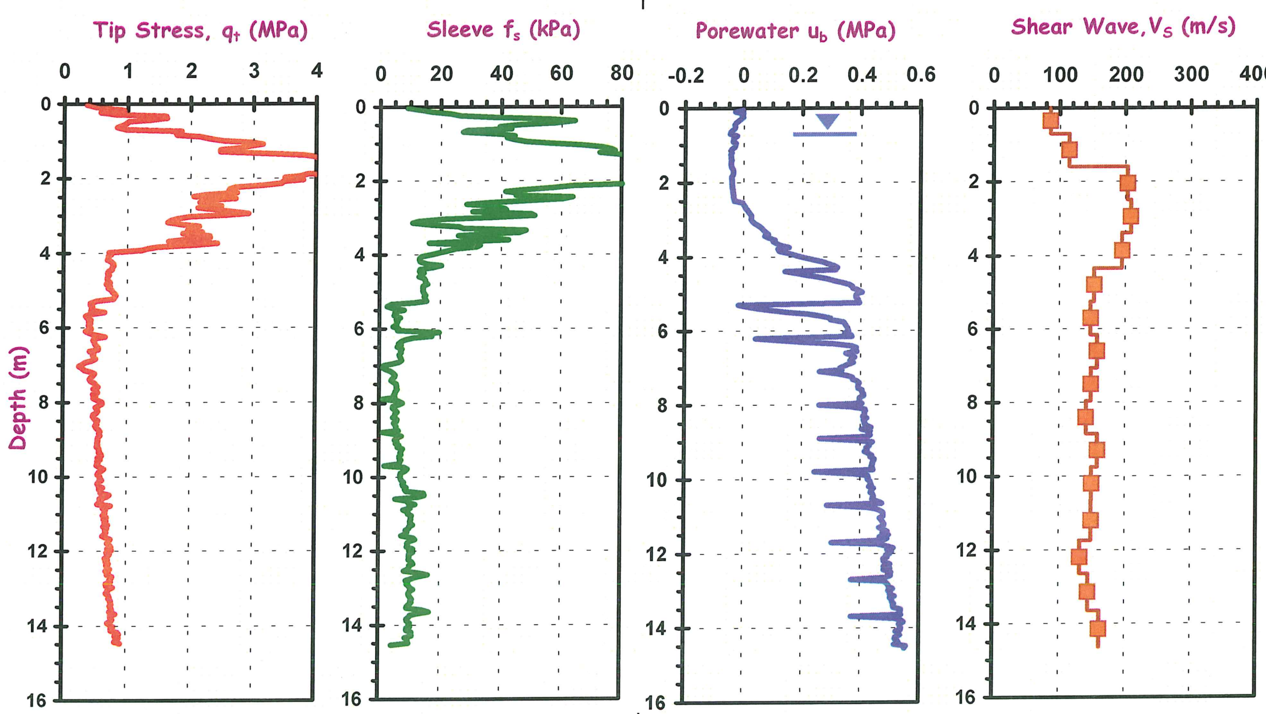
One of the reasons for choosing this particular site was that its stratigraphy and soil properties have been extensively studied and are very well documented (Lutenegger, 2000). The tests were performed in Area B, which consists of approximately 1.5 m (5 ft) of stiff silty-clay fill overlaying a thick 30-m (98 ft) deposit of late Pleistocene lacustrine varved clay. The fill consists of CVVC placed about 30 years ago after excavations at the Town of Amherst Wastewater Treatment plant, which is adjacent to the site. Below the fill, the CVVC has a well-developed stiff over-consolidated crust that was formed as a result of surface erosion, desiccation, ground water fluctuations and other physical and chemical processes (Lutenegger and Dearth, 2004).

Fig. 1 shows the results of seismic piezocone tests with dissipations, termed SCPTù (NCHRP, 2007), performed in the upper 15 m at the NGES of the University of Massachusetts – Amherst campus.

The CPTU profile in Fig. 1 clearly shows the upper silty-clay fill and the underlying stiff clay crust to approximately 4 m (13 ft). A sharp decrease in soil strength can be seen below this depth, which marks the boundary with the underlying soft lacustrine varved clay. Typical index properties of the natural soft clay are: liquid limit ≈ 58, plasticity index ≈ 32, natural water content ≈ 65, clay fraction (CF < 2μ) ≈ 50 and OCR ≈ 2 (Mayne et al 1999). The mean annual ground water level in the NGES site is about 1.3 m (4.3 ft) (Lutenegger and Dearth 2004). The exact water level depth at the time of the tests was not measured.

DESCRIPTION OF THE PILES

All piles have the same helices and shaft configuration. The shaft is a circular steel tube with 73.0 mm (2-7/8 in) outside diameter and 5.5 mm (0.217 in) wall thickness. The 2.13 m (7 ft) long bottom sections have three circular pitched bearing plates (helices), the first one

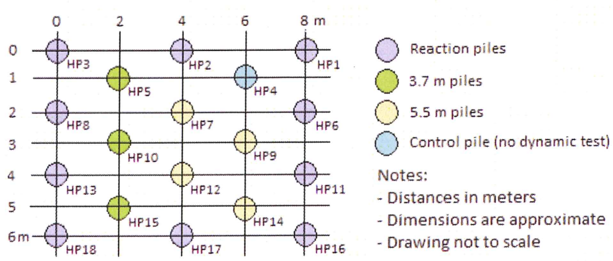


[FIG 1] Seismic piezocone results at NGES in Amherst, Massachusetts (after DeGroot and Lutenegger 1994) with a diameter of 203.2 mm (8 in) welded 152 mm (6 in) above the tip, the second with a diameter of 254 mm (10 in) welded 610 mm (24 in) above the first, and the third one with a diameter of 305 mm (12 in) welded 762 mm (30 in) above the second. The space between the helices is therefore three times their diameter, as is usual practice. The pitch distance of the helices is 76.2 mm (3 in), and the thickness of the helix plates is 12.7 mm (1/2 in). The extensions are 2.13 m (7 ft) long, with 0.15 m (6 in) overlap connections, resulting in 1.98 m (6.5 ft) effective extension length. Because of the temporary function of the piles the steel was not galvanized, as would be the usual practice.

Three piles (HP-5, HP-10 and HP-15) were installed to a depth of 3.7 m (12 ft) below grade, and consisted of one bottom section and one extension. Five piles (HP-4, HP-7, HP-9, HP-12 and HP-14) were installed to a depth of 5.5 m (18 ft) and consisted of one bottom section and two extensions. It was expected that the shorter piles would show more resistance than the longer ones, due to the drop in soil resistance at 4 m (13 ft).

INSTALLATION

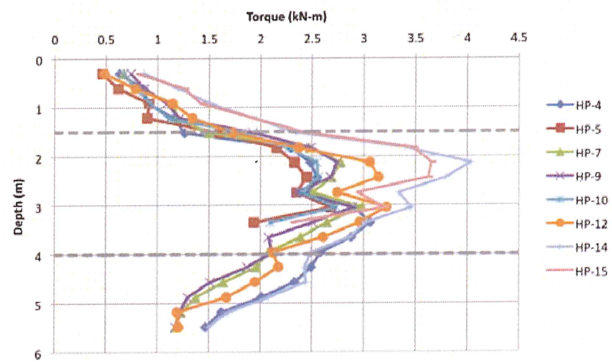
A total of 18 piles were installed at the NGES of UMass-Amherst on the 2nd and 3rd of July of 2010. Ten piles were used to verify different procedures of installation and to serve as reaction piles. Eight piles were designated as test piles: one of them was used as a control pile with only a SLT performed, and the other 7 helical piles were subjected to both DLT and SLT. The relative location of the different piles is shown in the pile installation plan of Fig. 2.



[FIG. 2] Pile installation plan

The installation of helical piles consists of embedding them into the soil by applying a torque to the head of the central shaft, which causes the helix or helices to penetrate the ground in a "screwing" motion. A downward force may also be applied to the helical pile during installation to facilitate the helices in "biting" into the soil and advancing the downward movement of the pile. To minimize the disturbance to the soil during installation, the helical pile should be advanced into the ground at a rate of one pitch distance per revolution, and multiple helices should be spaced along the shaft in multiples of the pitch, such that subsequent helices follow the same path as the initial helix when penetrating the ground (Ghaly et al., 1991).

A field control installation log was recorded relating the pile depth to the torque and the number of revolutions for every 0.3 m (1 ft) penetration; this is the typical inspection procedure for helical pile projects. Fig. 3 shows the graphs of the torque measured during installation of the test piles (including that of control pile HP-4), versus depth.



[FIG. 3] Torque measured during installation versus depth

TORQUE CORRELATIONS

The concept of correlating installation torque to axial capacity for helical piles is analogous to the relationship between the pile driving effort and pile capacity. Indeed, several authors have attempted to express such an empirical torque to capacity relationship (Hoyt and Clemence, 1989; Narasimha Rao et al., 1989; Ghaly and Hanna, 1991; Perko, 2000; 2009). Those torque formulas have been used in the helical pile industry for many years; however, they do not explicitly consider soil profile or soil parameters and therefore are too crude for advanced design requirements. Once calibrated by load testing, however, the torque formulas are a convenient and accepted Quality Control procedure for production piles.

The widely used acceptance criteria for helical piles AC-358 (ICC Evaluation Service, 2007), includes a formula, initially proposed by Hoyt and Clemence (1989), relating the final installation torque (T) to the ultimate axial capacity

(Q), and defines a torque correlation factor (Kt) such that:

$Q = K_t T$ [1]

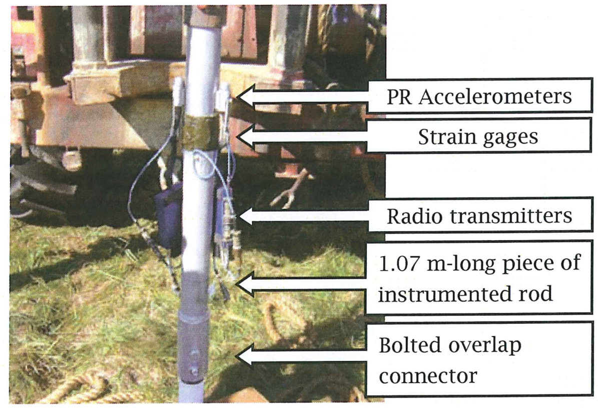
The K_t factor is an empirically developed quantity which typically ranges from 23 m^{-1} to 32.8 m^{-1} (7 ft^{-1} to 10 ft^{-1}). For the 73.0 mm (2-7/8 in) outside diameter round shaft (considered a "conforming" system) AC-358 suggests a K_t of 29.5 m^{-1} (9 ft^{-1}); however, AC-358 also states that the parameter K_t shall be verified by full-scale field installation and load tests.

It should also be noted that only tension tests were analyzed by Hoyt and Clemence (1989), as a basis for the torque correlation currently in use. More compressive tests on piles installed in various types of soil are necessary so that statistical analyses can be performed to obtain correlations between the tension and compression capacity of helical piles.

DYNAMIC LOAD TESTS

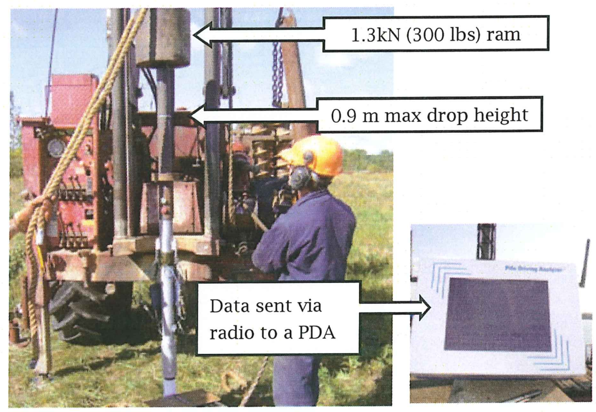
DLTs are performed by installing strain gages and accelerometers close to the top of the pile, and recording the resulting force and velocity signals while the pile is impacted by a pile driving hammer or similar device (Rausche et at., 1972). It is now extensively used with most kinds of foundation piles, and has the advantage over SLTs of being much faster and of lower cost. Although numerous good agreements between the results of SLTs and DLTs on other kinds of piles have been published (Likins, Rausche, 2004), the use of DLTs with helical piles had little application to date. One of the objectives of this work was therefore to verify the viability of using DLTs as a routine way of determining the bearing capacity of helical piles.

The DLTs were performed on September 15, 2010, about 2 ½ months after installation of the piles. Because of the small diameter of the shaft, the conventional bolted strain transducers normally used for dynamic pile testing could not be employed in this case. A calibrated 1.07 m (3.5 ft)-long piece of extension rod with bonded strain-gages was used instead. Two conventional piezo-resistive (PR) accelerometers were bolted to the rod, and the strain and acceleration signals were sent to a Pile Driving Analyzer (PDA) by means of radio transmitters. Fig. 4 shows the instrumented rod with sensors and radio transmitters attached to the top of the pile. It also shows the type of bolted overlap connector used to assemble the rod string.



[FIG. 4] Dynamic Load Test sensors arrangement

The tests consisted of a few blows applied by a truck-mounted cathead SPT rig fitted with a 1336 N (300 lbs) weight, dropped from a maximum height of 0.9 m (3 ft). Fig. 5 shows the setup used. The ideal ram weight and drop height is a function of the test load, shaft size and failure criterion, among other parameters, and can be determined for example by wave equation analyses (Rausche, 2000).



[FIG. 5] Dynamic Load Test setup

A CAPWAP® analysis (Rausche et al., 2010) was performed for each pile, using the data from the blow with the highest energy. CAPWAP uses signal matching procedure, based on a modified and extended Smith soil model, to determine the static and dynamic soil resistance parameters; it also provides a simulated static pile-top load versus displacement curve which can be directly compared with the corresponding curve from a SLT.

The pile was modeled as a uniform rod, with a cross-sectional area corresponding to that of the 73.0 mm (2-7/8 in) shaft. The impedance increases caused by the helices were neglected, since they would be very small for the 1 m (3.3 ft) long pile segments of the numerical model. Small reductions in impedance or the use of "slacks" were necessary to model the joints along the shaft. No additional end bearing was modeled to account for the upper two helices. The good signal matching and good agreements of DLT and SLT results obtained using this model suggest that, at least for this type of soil, there was minimal soil resistance developed at the bottom of the two upper helices. Practically all soil resistance above the lower helix apparently originated from shearing of the soil plug trapped between the helices. This is in agreement with the method for predicting the axial compressive capacity of helical piles proposed by Livneh and El Naggar (2008).

A shaft radiation damping model (Likins et al., 1996) was used in the CAPWAP analyses. In order to obtain good agreement with the static load test results, however, lower values for the damper had to be used, compared to those recommended in the CAPWAP documentation. This was attributed to the fact that the effective circumference of the skin friction behavior, controlled by the helix diameters, was larger than the value entered of the outside perimeter of the 73.0 mm (2-7/8 in) pipe.

Fig. 6 shows the summary table of CAPWAP results for pile HP-7, taken as an example, and Fig. 7 shows the measured force and velocity records, together with a diagram of the pile model showing the impedance decreases at the joint locations.

CAPWAP SUMMARY RESULTS

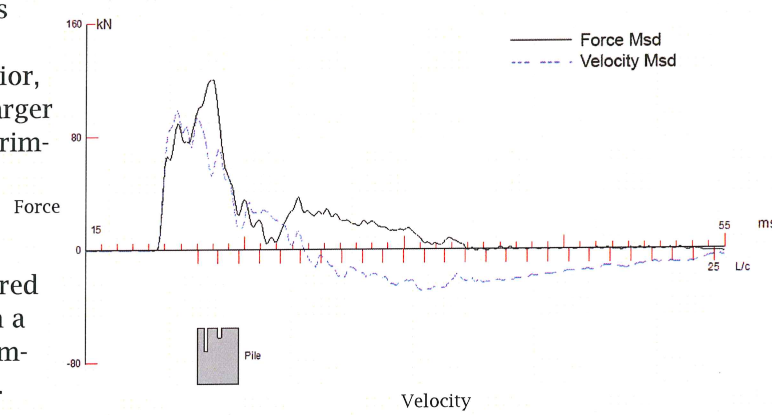
Total CAPWAP Capacity: 44.0; along Shaft 16.9; at Toe 27.1 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa	Smith Damping Factor s/m	Quake mm
				44.0					
1	2.2	1.1	0.0	44.0	0.0	0.00	0.00	0.000	3.432
2	3.3	2.2	0.9	43.1	0.9	0.82	3.56	1.098	3.437
3	4.4	3.3	3.6	39.5	4.5	3.27	14.24	1.098	3.437
4	5.5	4.4	5.3	34.2	9.8	4.81	20.96	1.098	3.437
5	6.6	5.5	7.1	27.1	16.9	6.44	28.07	1.098	3.437
Avg. Shaft			3.4			3.08	13.43	1.098	3.437
Toe			27.1				6470.48	0.459	8.860

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		0.447	0.300
Unloading Quake	(% of loading quake)	297	103
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	50	
Resistance Gap (included in Toe Quake) (mm)			0.283
Soil Plug Weight	(kN)		0.10
Soil Support Dashpot		0.310	0.000
Soil Support Weight	(kN)	1.26	0.00

max. Top Comp. Stress = 126.0 MPa (T= 22.7 ms, max= 1.000 x Top)
max. Comp. Stress = 126.0 MPa (Z= 0.6 m, T= 22.7 ms)
max. Tens. Stress = -3.51 MPa (Z= 4.4 m, T= 53.7 ms)
max. Energy (EMX) = 0.71 kJ; max. Measured Top Displ. (DMX)=10.01 mm

[FIG. 6] CAPWAP Summary Table for HP-7

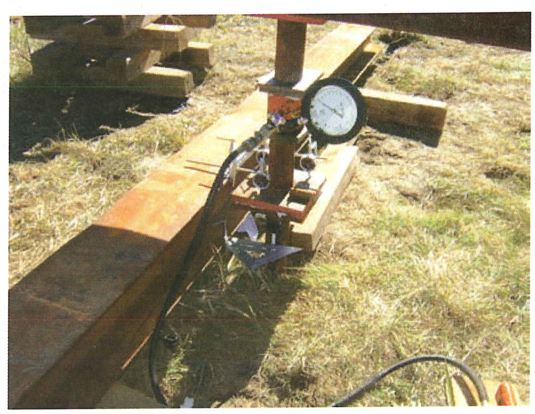


[FIG. 7] Force-Velocity record for pile HP-7, and pile model showing impedance decreases at joint locations

STATIC LOAD TESTS

The SLT is the traditional way of verifying the bearing capacity and displacement behavior of deep foundations; however, it requires a relatively large reaction system and long preparation time, which may have unfavorable economical impact on the construction. On the other hand, SLTs are still the standard with which other test methods are compared, so they were performed in this case for comparison purposes.

The SLTs were performed between October 10 and 15, 2010, i.e., 25 to 30 days after the DLTs. They were carried out according to Procedure A (Quick Test) specified by ASTM D1143/D1143M - 07, with measurement of displacements for load increments of 5% of the anticipated failure load. Each increment was kept constant for 15 minutes, until what was perceived as plunging failure was achieved in all cases. Fig. 8 shows the typical SLT setup, including reference beam, dial gages and hydraulic pressure gage as employed for all piles.



[FIG. 8] Static Load Test setup

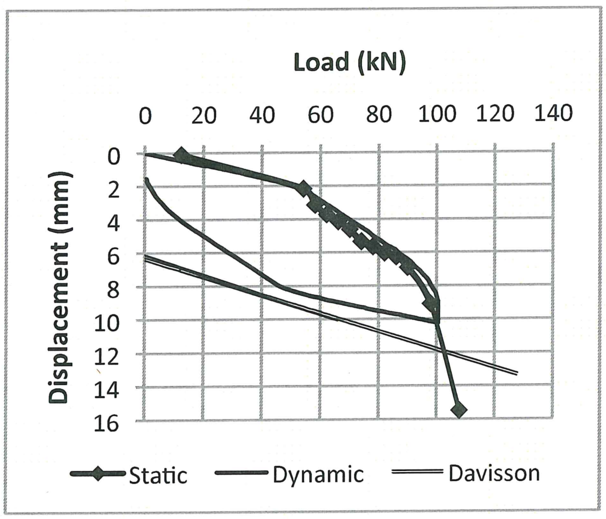
The results of the SLTs are discussed below, together with those of the DLTs.

RESULTS

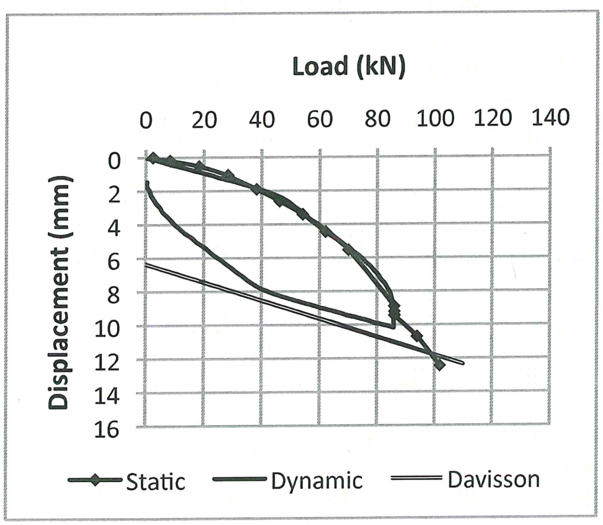
Figs. 9 to 15 show the comparison for each pile between the load-displacement curve of the SLT and the corresponding curve simulated by the CAPWAP program. The figures also show the Davisson offset limit (Davisson, 1972). This method defines the failure load as that corresponding to the movement that exceeds the elastic deformation of the pile by a value of 3.8 mm (0.15 inch) plus the diameter of the pile (in the present case taken as that of the largest helix, that is, 305 mm or 12 inches) divided by 120. The elastic deformation of the pile is calculated using the expression:

$D = PL / AE$ [2]

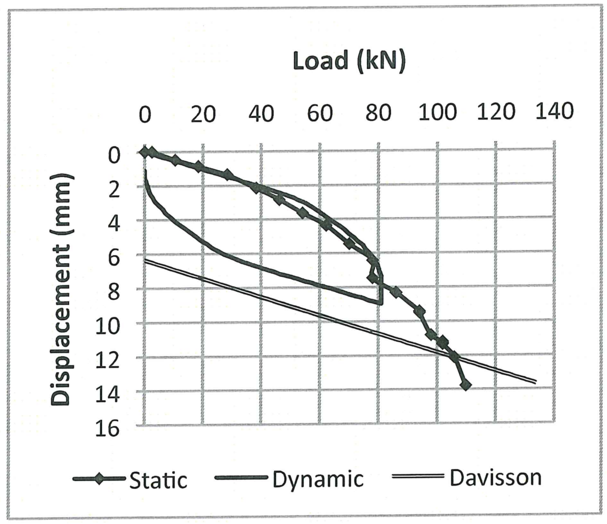
Where D is the deformation for the applied load P, L is the length, A is the cross-sectional area of the shaft and E is the elastic modulus of the pile material. Table 1 summarizes all results in numerical form.



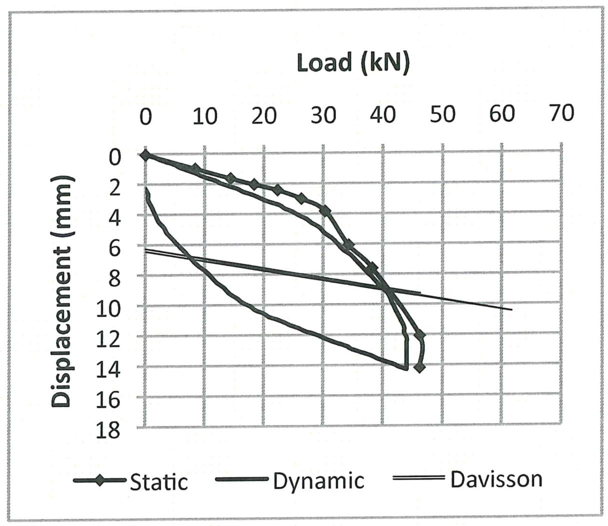
[FIG. 9] Results for HP5 (3.7 m)



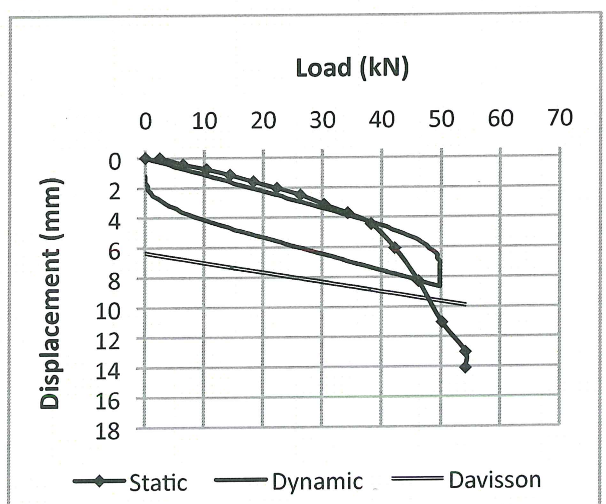
[FIG. 10] Results for HP10 (3.7 m)



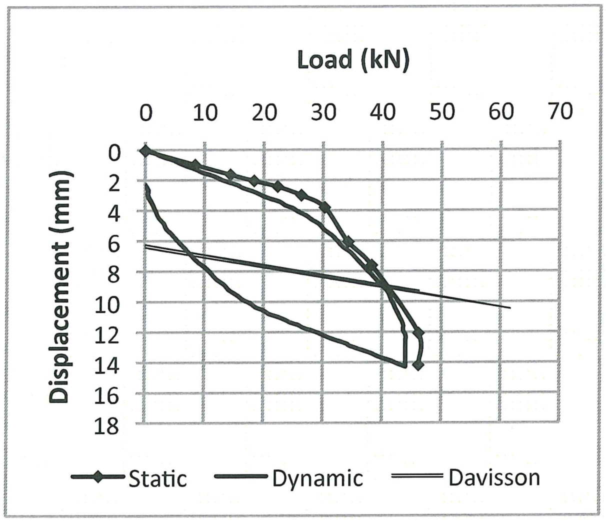
[FIG. 11] Results for HP15 (3.7 m)



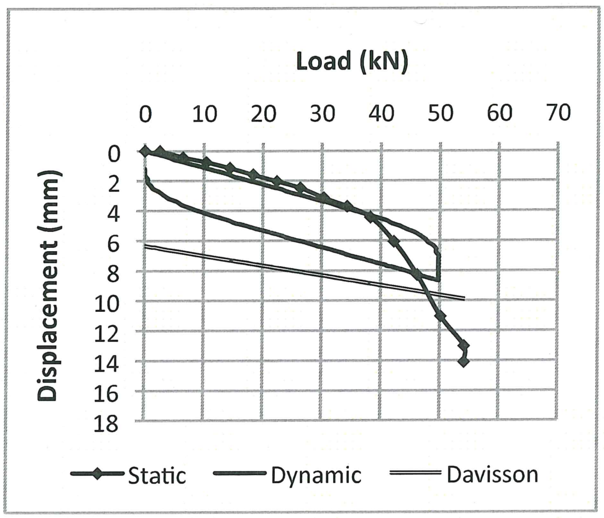
[FIG. 12] Results for HP7 (5.5 m)



[FIG. 13] Results for HP9 (5.5 m)



[FIG. 14] Results for HP12 (5.5 m)



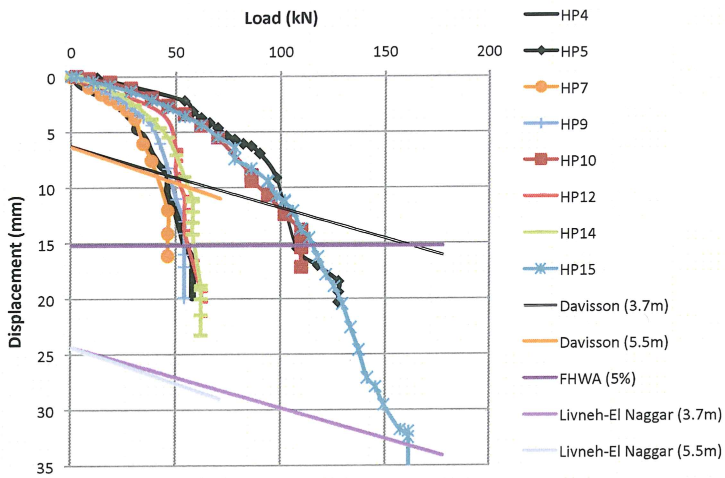
[FIG. 15] Results for HP14 (5.5 m)

The Davisson criterion was chosen since most comparisons of DLT and SLT results available in the literature were obtained using this method (Likins and Rausche 2004), and it is also one of the most widely used methods in North America (Fellenius, 2001). In order to guarantee the best comparisons between DLT and SLT results interpreted by the Davisson criterion, the usual DLT practice requires that the energy applied is sufficiently large to produce a set per blow of 2.5 mm (0.1 in) or more (Likins et al, 2000). For the shorter higher capacity piles, however, the sets were of the order of only about 2 mm (0.08 in) , and the results shown in Figs. 9 to 11 are clearly indicating that the energy applied was not sufficient to mobilize the full static capacity. This explains the somewhat conservative DLT results for piles HP10 and HP15. The SLT simulation curves predicted by the CAPWAP analysis of the DLT data, however, are in good agreement with the actual SLT, up to the point of the maximum DLT displacement achieved. A higher applied energy would have been recommended for the 3.7 m (12 ft) piles, by using a heavier weight and/or increasing the drop height.

Besides the Davisson criterion, several other definitions of pile capacity evaluated from load-movement records of SLTs can be found in the literature (Fellenius, 2001). A criterion specifically for helical piles has been proposed by Livneh and El Naggar (2008), defining the failure load as that corresponding to the movement that exceeds the elastic deformation of the pile (calculated using expression 2 above) by 8% of the largest helical diameter. For drilled shafts, the Federal Highway Administration (FHWA) for example defines the failure load as that corresponding to a gross settlement of 5% of the diameter (O'Neill and Reese, 1999). Fig. 16 below shows the Load-Displacement curves

[TABLE 1] Summary of results

Pile	HP5	HP10	HP15	HP4	HP7	HP9	HP12	HP14
Depth (m)	3.7	3.7	3.7	5.5	5.5	5.5	5.5	5.5
Torque (kN-m)	1.9	2.1	2.3	1.5	1.2	1.2	1.2	1.5
SLT - Davisson (kN)	102.8	97.9	101.9	50.0	38.3	48.5	52.0	56.5
DLT - Davisson (kN)	100.5	85.9	81.0	-	38.3	49.8	49.4	56.5
SLT/DLT	1.02	1.14	1.26	-	1.00	0.97	1.05	1.00
K_t static (m^{-1})	53.1	46.8	44.2	33.3	32.5	41.2	43.1	37.9
K_t dynamic (m^{-1})	51.9	41.0	35.2	-	32.5	42.3	40.9	37.9
Average K_t static (m^{-1})		48.0				37.6		



[FIG. 16] Results of the Static Load Tests

of the SLTs, and the thresholds corresponding to three failure criteria: Davisson, FHWA (5% of largest helical diameter) and Livneh-El Naggar.

It can be seen that in this case the Davisson criterion produced more conservative results than the two other criteria. For longer piles and/or smaller diameter helices, it is possible that the FHWA criterion produces similar or even more conservative results than the Davisson criterion. The Livneh-El Naggar criterion, on the other hand, calls for extending the load application to much larger displacements. In any event, if the results of the SLTs are to be analyzed using a less conservative criterion, like the Livneh-El Naggar, the energies applied in the DLT would have to be increased, so that permanent sets larger than 2.5 mm (0.1 in) are produced.

The question of the failure criterion that should be adopted for DLTs on helical piles is one that requires further investigation. For drilled shafts and augercast piles, for example, an alternative criterion to the 2.5 mm (0.1 in) minimum set has been proposed based on the total accumulated toe displacement caused by successive blows with increasing energy (Rausche et al., 2008). A suitable criterion will have to be developed for helical piles, considering their particular characteristics and installation method.

Additional research is also necessary to verify the agreement of SLT and DLT results for piles in non-cohesive soils. At least one case study presented in the literature (Cannon, 2000) suggests that for moderately dense to dense medium sands the skin friction shows resistance concentrations at the helix locations, whereas the model used in the analyses presented herein only shows a resistance concentration at the location of the leading helix (included in the end bearing as, for example, shown in the CAPWAP summary table of Fig. 6).

CONCLUSIONS

Dynamic and Static Load Tests were performed on seven helical piles installed in the NGES at Amherst, Massachusetts. Good agreements were obtained between the results of the SLTs interpreted by the Davisson criterion and the results of the DLTs, and the general shapes of the load-displacement curves are similar for both tests. This shows that DLT is a viable alternative to determine the compressive load capacity of Helical Piles, at least in cohesive soils, with added advantages of lower cost and execution time.

Although AC-358 suggests a torque to capacity correlation factor K_t of 29.5 m^{-1} (9.0 ft^{-1}) for the size of piles tested, the results obtained in this research show that for compressive loads on CVVC this factor proved to be too low, leading to overly-conservative design. This confirms the necessity of executing one or more load tests on each site for determining the best correlating value of K_t. It should also be noted that torque correlations depend on the particular pile size and configuration and also on the type and condition of the soil where the piles are installed. This was also shown in the present study, where substantially different values of K_t were obtained for piles installed in the different bearing strata.

ACKNOWLEDGMENTS

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REFERENCES

1. ASTM D1143/D1143M – 07. Standard Test Methods for Deep Foundations Under Static Axial Compressive Load. ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428-2959, United States

2. ASTM D4945-08. Standard Test Method for High-Strain Dynamic Testing of Deep Foundations. ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428-2959, United States
3. Cannon, J.G., 2000. The application of high strain dynamic pile testing to screwed steel piles. *Proceedings of the Sixth International Conference on the Application of Stress-wave Theory to Piles*: São Paulo, Brazil; pp. 393-398.
4. Davisson, M. T., 1972. High capacity piles. *Proceedings of Lecture Series on Innovations in Foundation Construction*, American Society of Civil Engineers, ASCE, Illinois Section, Chicago, March 22, pp. 81 - 112.
5. DeGroot, D.J. and Lutenegger, A.J., 1994. A Comparison Between Field and Laboratory Measurements of Hydraulic Conductivity in a Varved Clay. Hydraulic Conductivity and Waste Contaminant Transport in Soil. *ASTM STP 1142*, American Society for Testing and Materials, West Conshohocken, Pa., pp. 300-317
6. Fellenius, B.H., 2001. What Capacity Value to Choose from the Results of Static Loading Test. *Fulcrum*, Deep Foundations Institute, New Jersey.
7. Ghaly, A. M. and Hanna, A. M., 1991. Experimental and Theoretical Studies on Installation Torque of Screw Anchors. Canadian Geotechnical Journal, Vol. 28, no. 3, pp. 353-364.
8. Hoyt, R. M. and Clemence, S. P. 1989. Uplift Capacity of helical Anchors in Soil. *Proceedings of the 12th International Conference on Soil Mechanics and Foundation Engineering*, Rio de Janeiro, Brazil, Vol. 2, pp. 1019-1022.
9. ICC Evaluation Service, Inc., 2007. Acceptance Criteria for Helical Foundation Systems and Devices. 5360 Workman Mill Road, Whittier, California 90601 (www.icc-es.org).
10. Likins, G. E., Rausche, F., Thendean, G., Svinkin, M., September, 1996. CAPWAP Correlation Studies. *Proceedings of the Fifth International Conference on the Application of Stress-wave Theory to Piles 1996*: Orlando, FL, pp. 447-464.
11. Likins, G. E., Rausche, F., August, 2004. Correlation of CAPWAP with Static Load Tests. *Proceedings of the Seventh International Conference on the Application of Stresswave Theory to Piles 2004*: Petaling Jaya, Selangor, Malaysia, pp. 153-165.
12. Likins, G., Rausche, F., Goble, G.G., 2000. High strain dynamic pile testing, equipment and practice. *Proceedings of the Sixth International Conference on the Application of Stresswave Theory to Piles*, Balkema, Roterdam
13. Livneh, B., El Naggar, M.H., 2008. Axial testing and numerical modeling of square shaft helical piles under compressive and tensile loading. *Canadian Geotechnical Journal*, 45(8): 1142-1155.
14. Lutenegger, A. J., 2000. National Geotechnical Experimentation Site - University of Massachusetts. National Geotechnical Experimentation Sites, ASCE, pp. 102-129.
15. Lutenegger, A. J. and Dearth, A., 2004. Influence of Ground Water Table Fluctuation on Lateral Load Behavior of Rigid Drilled Shafts in Clay Crust. *GeoSupport 2004*, Orlando, pp. 395-404.
16. Mayne, P.W., Schneider, J.A and Martin, G.K., 1999. Small and Large-strain soil properties from seismic flat dilatometer tests. Pre-failure Deformation Characteristics of Geomaterials, Jamiolkowski, Lancellotta & Lo Presti (eds), Balkema, Rotterdam, Holland.
17. Narasimha Rao, S., Prasad, M. D., Shetty, M. D. and Joshi, V. V., 1989. Uplift Capacity of Screw Pile Anchors. *Geotechnical Engineering*, vol. 20, n.2, pp. 139-159.
18. NCHRP – National Cooperative Highway Research Program, 2007. *Synthesis 368 – Cone Penetration Testing*. Transportation Research Board, Washington, DC
19. Perko, H. A., 2000. Energy Method for Predicting the Installation Torque of Helical Foundation and Anchors. New Technological and Design Developments in Deep Foundation Technologies, ASCE, pp. 342-352.
20. Perko, H. A., 2009. Helical Piles: A Practical Guide to Design and Installation. 1st Edition by John Wiley & Sons Ed., 512 p.

21. Rausche, F., Moses, F., Goble, G. G., 1972. Soil Resistance Predictions From Pile Dynamics. Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers. Reprinted in Current Practices and Future Trends in Deep Foundations, *Geotechnical Special Publication No. 125*, DiMaggio, J. A., and Hussein, M. H., Eds, August, 2004. American Society of Civil Engineers: Reston, VA, pp. 418-440.

22. Rausche, F., 2000. Wave Mechanics and the Wave Equation. The Design and Installation of Higher Capacity Driven Piles, Pile Driving Contractors Association, Annapolis, MD.

23. Raushe, F., Likins, G.E., Hussein, M.H., 2008. Analysis of Post-Installation Dynamic Load Test Data for Capacity Evaluation of Deep Foundations. From Research to Practice in Geotechnical Engineering

24. Rausche, F., Likins, G. E., Liang, L., Hussein, M.H., 2010. Static and Dynamic Models for CAPWAP Signal Matching. The Art of Foundation Engineering Practice, *Geotechnical Special Publication No. 198*, Hussein, M. H., J. B. Anderson, W. M. Camp Eds, American Society of Civil Engineers: Reston, VA, pp. 534-553.

25. O'Neill, M.W., Reese, L.C., August 1999. Drilled shafts: construction procedures and design methods, *FHWA-IF-99-025*. Federal Highway Administration, McLean, Va.

26. Robison, K. E. and Taylor, H., 1969. Selection and Performance of Anchors for Guyed Transmission Towers. *Canadian Geotechnical Journal*, Vol. 6, pp. 119-135.

27. Wilson, G., 1950. The bearing capacity of screw piles and screwcrete cylinders. *Journal of Institution of Civil Engineers*. Vol. 34, pp. 4-73.