High Strain Dynamic Load Testing on Helical Piles – Case Study

by

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ABSTRACT: Helical piles are progressively evolving as a viable and cost effective deep foundation solution over traditional deep foundation systems. Historically, helical piles were generally of a small size mainly used to resist light loads, whereas today helical piles are becoming larger in size and used over a diverse range of industries and geological settings, involving higher applied loads in more complex applications. Quality control of the helical pile typically consists of measuring torque during installation, and performing a static load tests as a way to correlate the torque to capacity relationship. High-Strain Dynamic Load Testing (HSDLT) of helical piles has become an alternate method to the often costly and time consuming traditional "top-down" static load testing. HSDLT has been performed for decades on driven and drilled deep foundations, and recently the technology has been applied to helical piles as well. Dynamic tests can be performed relatively quickly to develop the load versus displacement curves and to evaluate the ultimate helical pile capacity. Several HSDLT's were performed on helical pile foundations for The Dow Chemical Company as part of the building 949 addition located in Midland Michigan, and the results of the testing are presented herein as a case study, which includes a discussion on the analysis methods.

INTRODUCTION

Helical piles, also known as screw piles or helical piers, consist of one or more circular helical plates welded to either a steel cylindrical pipe or square steel shaft (Figure 1). The piles are advanced into the ground using a hydraulic torque motor, often mounted to an excavator type of mobile construction machinery. Helical piles can be installed quickly, precisely, and with minimal impact to the environment in any climate. The operation results in low installation noise, no vibrations, and no cuttings to excavate or remove. In addition to being simple to install, helical piles can be removed just as easily.

Standard round shaft diameters of helical piles range from 2 7/8 inch to 16 inch in diameter with helix diameter sizes up to 48 inches. Single or multiple helix piles can be custom designed to any length within the safe and practical limits of the material. The simple design and installation makes helical piles versatile and economical for many different types of structures, both permanent and temporary. Due to the minimal invasiveness of the installation procedure, helical piles are also uniquely suited for underpinning and other remedial foundation support.

As with all deep foundation types, the helical pile design process addresses various sources of uncertainty. These uncertainties include estimation of loads, variability of ground conditions and geotechnical material properties, and the prediction of the behavior of the superstructure, substructure and the soil that supports it (CFEM, 2006). Closed form design philosophies based on limit state design or allowable (working) stress design incorporate safety factors that are intended to account for uncertainties and minimize risk. However, an acceptable safety factor applied to estimated resistances hardly ensures a safe design. Full-scale load testing is the only accurate method of verifying design assumptions, determining ultimate load carrying capacity and predicting the load-settlement behavior of a pile, thus enabling a true assessment of "safety".

HELICAL PILES THEORETICAL DESIGN MODEL

The theoretical design model used in this design was the individual plate-bearing method for designing helical piles, which describes the helical pile as a series of independent plate anchors embedded at different depths. Bearing failure is assumed to occur above or below each individual helix when the pile is loaded in tension or compression, respectively. The applicability of the individual plate bearing model is determined by the inter-helix spacing ratio ($^S/_D$) of the pile, where experimental data has shown that $^S/_D \geq 3$ insures that the helices act independently without influencing one another during bearing failure. The pile's ultimate capacity is considered to be the sum of the bearing capacity of the soil above each helix (in uplift) or below each helix (in compression), plus the adhesion (cohesive soils) or friction (cohesion less soils) acting along an effective shaft length (Tappenden, 2007). The theoretical compressive capacity prediction utilizing the closed form solution (equation 1) for helical piles in cohesive soil is provided below:

$$Qu = \frac{\pi}{4} D_{helix}^2 (S_u N_C + \gamma' H) + \pi D_{shaft} f_s L_f \qquad (1)$$

where

Q_u	= Theoretical Ultimate Pile Capacity	D_{helix}	= [1]Bearing area of helix
S_u	= Undrained Shear Strength	N_c	= Bearing Capacity Factor
γ'	= Effective Unit Weight of Soil	H	= Embedment Depth of Helix
D_{Shaft}	= Nominal Shaft Diameter	L_f	= Pile Shaft Length
f_s	= Adhesion between soil and pile		

[1]It should be noted, during installation a soil plug fills the bottom of the shaft, and the convention is to use the helix area plus the area of the soil plug when determining the bearing area under compressive load. In the case of tensile loading only the helix diameter (minus the shaft diameter) is used.

Figure 1. Typical Helical Piles.

HIGH STRAIN DYNAMIC TESTING

The basis for all modern dynamic testing techniques is the research performed at the Case Institute of Technology, later Case Western Reserve University, from 1964 to 1976 by Goble, Rausche, and Likins. The research yielded the Case Method analysis, which evaluates the response of a uniform slender rod (or pile) under a high strain impact using one dimensional wave theory. By measuring the force and velocity of a pile during an impact with the Pile Driving Analyzer® (PDA), the induced stresses, uniformity (integrity), and static soil resistance (capacity) can be evaluated. The Case Pile Wave Analysis Program, or CAPWAP®, was developed to further analyze the collected force and velocity data. The CAPWAP program is a signal matching program that has the capability to evaluate complex situations including non-uniform piles, and yields a more refined static soil resistance estimate as well as an estimate of the distribution of the static soil resistance. The analysis also generates a simulated static load vs. settlement curve based on the input resistance distribution and dynamic soil parameters. Figure 2 shows a typical simulated load vs. settlement curve from CAPWAP analysis.

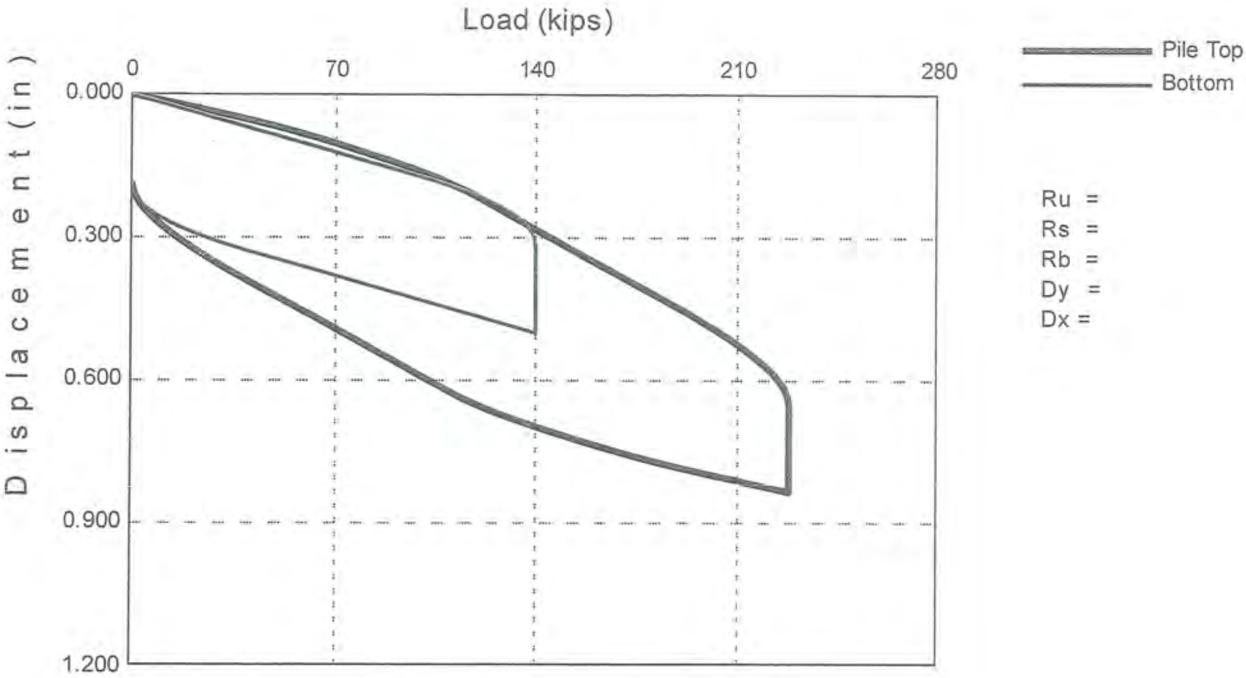

Figure 2. Example of Simulated Static Load vs. Settlement Curve from CAPWAP Analysis.

Application of High Strain Dynamic Testing to Helical Piles

In the application of dynamic testing methods to helical piles, all of the principles remain the same as for driven or drilled piles. Wave mechanics and stress wave theory are directly applicable to the measured force and velocity of helical piles under impact. CAPWAP analysis of the collected data is also very similar, however, in a previous study; Beim (et al. 2012) neglected the impedance increase caused by the helices in the pile model. The piles were modeled as a uniform section, since the impedance increases caused by the helices would be very small. Beim (et al. 2012) also concluded that a shaft radiation damping model (Likins et al. 1996) may be necessary to obtain good agreement with static load test results. Any differential movement above and below section connections can be modeled using small impedance reductions or "slacks" (Rausche et al., 2010) where necessary.

Limitations

The same limitations that apply to high strain dynamic testing of driven or drilled piles also apply when testing helical piles. Results from dynamic testing provide the mobilized capacity. If the permanent set under impact is zero or very small (roughly < D/120), it is likely that all of the soil resistance has not been mobilized and the resulting capacity estimate should be considered lower bound. Dynamic testing evaluates soil resistance at the time of testing. Time dependent changes cannot be quantified without testing at least two different times. For helical piles, it is likely that additional shaft and possibly end bearing will develop with time as the disturbed soil achieves its long term condition around that pile. The open end profile of these relatively large helical piles also may behave differently under dynamic and static loading. For example, a soil plug may develop under a static load and increase the effective end bearing area while this soil plug may shear and provide little resistance under the high accelerations generated from dynamic loading.

TEST DESCRIPTION

Our client for this project, The Dow Chemical Company (Dow), is one of the world's largest companies specializing in chemical, agrosciences, plastics, and advanced materials covering a broad range of technology-based products to customers in high growth sectors such as electronics, water, energy, coatings and agriculture. Dow employs approximately 54,000 people worldwide, and provides more than 5,000 products to approximately 160 countries, and which are manufactured at 188 sites in 36 countries across the globe.

The Midland, Michigan branch of Dow is where the HSDLT on the helical piles was carried out. The actual test site was located at the southeast quadrant of the intersection of G Street and 17th Street on the plant site, in between buildings 948 and 949. The work area between the two buildings was approximately 29 ft. wide by 102 ft. long. Building 948 is located adjacent and north of Building 949, and the proposed work required that these buildings be joined by process equipment and piping to increase the overall efficiency of the plant. The foundation scope of work required the installation of 45 helical piles to support various equipment and parts of the two buildings super structures, with the requirement that the work be completed in a very condensed timeframe. The applied loads to be resisted by the helical pile foundations ranged from 87 kips to 111 kips, with the performance criteria that the maximum pile head would not exceed ½ inch in vertical displacement. The three helical pile types designed to resist the various loads are outlined in Table 1 below. Also, as part of the scope of work, an approved load test methodology was required to verify the performance of the helical pile design.

The helical pile was chosen because it was consider as a viable foundation solution to overcome tight access and confined installation parameters. Another consideration in choosing helical piles was the fact that there were several duct banks underneath the worksite that could not be subjected to heavy traffic and any vibration from drilled cast-in-place operations and driven piers, respectively.

All helical piles were constructed using ASTM A252, grade 3 pipe and ASTM A36, 44W plate for the helices. The equipment used to install the helical piles was a large hydraulic excavator outfitted with a drive head motor capable of delivering a maximum 115,000 ft-lbs of torque. As height restrictions onsite posed limitations on the pile installation lengths, the lead and extension sections were manufactured in 26 ft segments, and then coupled together in the field. The couple section allowed for three 1- ¼ inch diameter high strength SAE Grade 8 bolts to bolt the extension and lead sections together.

Dow representatives specified which production pile locations were to be tested and identified the production locations as numbers 52-14, 52-18 and 62-02 for the three different helical pile types P1, P2, and P3, respectively. The stipulation given by Dow was that satisfactory testing results of all three helical pile types was required before any of the remaining production piles could be installed.

Table 1. Summary of Helical Pile Types.

Production Pile No.	Pile Type	Test No.	Shaft Dia. O.D. (in)	Pipe Wall Thickness w.t. (in)	Helix Dia. (D1) (in)	Helix Dia. (D2) (in)	Pile Length (ft)	Design Pile Capacity (kips)
52-14a	P1	P1-TP1	9 5/8	0.395	20	20	50	87
52-14b	P1	P1-TP1	9 5/8	0.395	20	20	50	87
52-10	P2	P2-TP	9 5/8	0.395	22	22	50	96
62-02	P3	P3-TP	9 5/8	0.395	24	24	50	111

Installation of the first helical pile began on December 5, 2012, and helical pile P1 at location 52-14 was the first to be installed. Upon installing this pile to 44 ft below grade, a hard layer of soil was encountered at the toe causing the helical pile to spin without further advancement (spin-out). The spin-out resulted in a reduction in the installation torque reading to 25,000lb-ft below the expected minimum of 53,000lb-ft, therefore another production helical pile to be tested was added 46 inches, center to center, to west of 52-14 position and designated 52-14B. 52-18 and 62-02 were installed to 45 ft. depths and with final torque readings above the minimum values prescribed.

SOIL CONDITIONS AND GEOLOGY

The geologic history of Midland, Michigan is dominated by the influence of glacial action and ancient seas. In Midland, the soils are typically loamy and sandy, with various soil types formed in loamy glacial till, till plains, moraines, lake outwash, and lake bottom lands. These glacial deposits account for the uniformly level land in Midland where major soil types are typically found on 0% to 6% slopes. Most of the sandy soil formations, such as the Cohoctah, Kingsville, Oakville, Pipestone, and Wixom, are the predominant soil types found in Midland, and are generally poorly drained, and have a high potential for frost heave (1979 Midland County Soil Survey).

Site Specific geotechnical boring information was carried out by McDowell & Associates in 2010, and supplied in the request for design package by Dow for the production area test site. The most representative boreholes nearest to the test site, adjacent to buildings 948 and 949, were identified as logs No.8332 and 8333, which were auger drilled to depths of 45.5 ft. and 20.5 ft., respectively. The soils described were very consistent between the two logs: a thin layer of asphalt pavement overlying compact loamy sand to 9 ft depth, stiff clay extending to 45 ft depth, over an extremely hard layer of clay below 45 ft depth. Both test holes were wet upon completion of the drilling, with the groundwater level measured approximately 4ft below ground surface. Standard penetration test (SPT N) readings showed values between 7 to 28 blows per foot in the sand layer, 6 to 10 in the stiff clay, and 61 in the deep hard clay layer.

TEST PROCEDURE

Application of Impact

A drop hammer was utilized to impact the helical piles. The drop hammer chosen was an APPLE VI dynamic load testing system. This system consists of a guided 4.5 ton drop weight which is supported by a frame. The ram is lifted to the desired height and hydraulically clamped in place. The hydraulic clamp is then released and the ram drops in freefall. The drop heights ranged from 2 inches to 18 inches during testing at this site. Figure 3 shows a stock picture of the APPLE VI dynamic load testing system.

Figure 3. Stock photo of the APPLE VI Dynamic Load Testing System.

Pile Preparation for High Strain Dynamic Testing

For testing, the helical piles were cut off 4 ft. above existing grade. Holes were drilled and tapped 24 inches below the cut off elevation for strain transducer and accelerometer attachment to the shaft of the helical pile. Four reusable strain transducers were attached to the helical pile shaft near its top at 90 degrees from each other for collecting force data. Two piezo-resistive (PR) accelerometers were placed at 180 degrees from each other and were used to collect velocity data. In addition, a "follower" was constructed and instrumented in a similar manner. The follower consisted of a 2 foot long section of pipe of the same diameter and thickness as the helical pile. A steel pipe sleeve was welded to the outside of the follower and fit over the helical pile to hold the follower in position. A 12 in. x 12 in. x 0.25 in. thick steel plate was welded to the top of the follower to be used as an impact surface. Two 0.5 in. thick pieces of plywood were placed on top of the follower as an impact cushion. Force and velocity measurements were collected on the follower during testing for comparison with data collected directly on the pile. The comparison of data is outside the scope of this paper and, therefore, will not be discussed. All results discussed are from evaluation of the data collected from measurements taken directly from the helical pile testing.

Testing Sequence

After the APPLE IV load testing system was leveled and centered over the test pile, 3 or 4 impacts were applied to the piles, ranging from 2 inches to 18 inches in height. The initial impacts with very small drop heights were to evaluate the hammer pile alignment and verify all instrumentation was in working condition and properly attached. After each impact, the permanent set of the pile was measured using a sight level. The specific drop heights for each pile and the corresponding measured sets are shown in Table 2.

ANALYSIS METHOD AND RESULTS

Because permanent settlement was observed under each impact, it was important to include each impact in the generation of a load vs. settlement curve for each pile. To generate a simulated load vs. displacement curve using force and velocity data from several impacts, superposition of simulated load vs. displacement curves from CAPWAP was used (Rausche et al. 2008). Theoretically, a single impact could be applied and a simulated load vs. displacement curve generated, however, general practice is to apply multiple impacts with increasing force and measure the corresponding set of each. The maxima of each superimposed load vs. settlement curve can then be traced to generate an overall load vs. settlement curve for the pile. This curve can be evaluated in the same manner as a load-settlement curve from a standard static load test. Figure 4 shows an example of the method used to generate the estimated load vs. settlement curves for each tested pile.

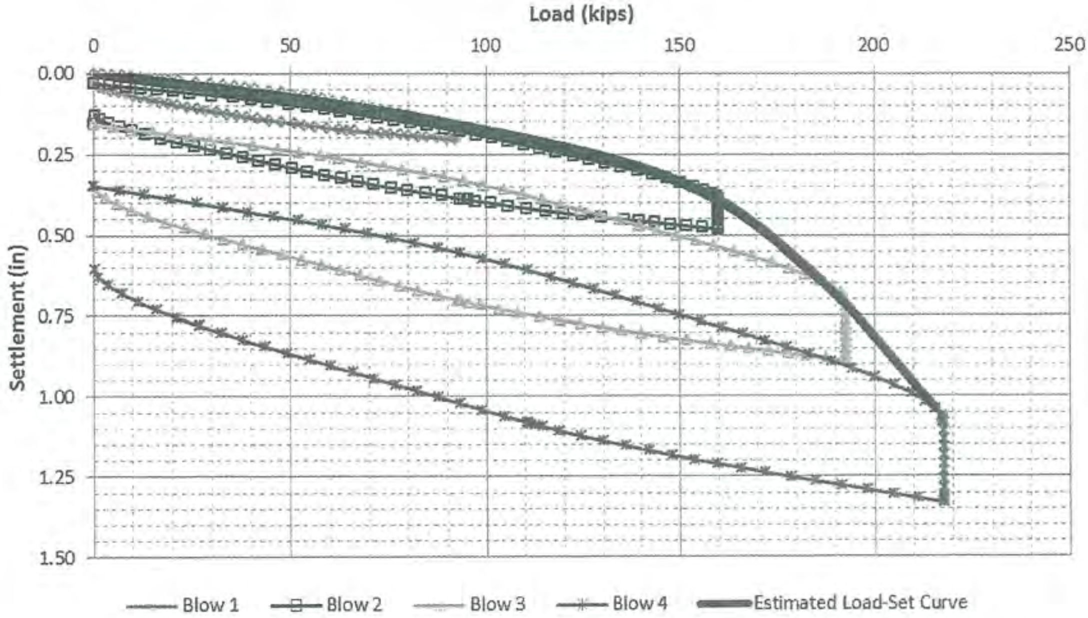

Figure 4. Superimposed Simulated Static Load vs. Settlement Curves for 52-10.

Case Method and CAPWAP Results

For each hammer impact, the PDA records the force and velocity as well as several other calculated quantities. Table 2 summarizes the observed and calculated information from the Case Method analysis. CAPWAP analysis was performed on each impact, and the results indicated that the majority of the resistance, roughly 60% to 75% under the highest energy impact, was from end bearing on the bottom helices. This end bearing condition was expected based on the soil profile indicating extremely hard clay layer at the tip elevation. The majority of the shaft resistance, roughly 60% to 70% was indicated to occur near the bottom of the pile, likely the result of the shearing of the soil plug between the helices or end bearing effect of the upper helical. Table 3 shows the results of the CAPWAP analyses.

Table 2. Summary of Case Method Results.

Pile No.	Pile Type	Penetration [1] Depth (ft)	Final [2] Installation Pressure	Drop Height (in)	Pile [3] Set	Transf'd Energy (kip-ft)	Max. Comp[4]	
							Force (kips)	Stress (ksi)
52-14a	P1	44' - 11"	1000 psi	2	1/16"	0.8	66	5.8
				6	3/16"	3.4	116	10.2
				9	1/4"	5.7	142	12.4
52-10	P2	45' - 3"	3500 psi	2	1/32"	0.7	89	7.7
				6	3/32"	2.9	154	13.4
				12	3/16"	6.2	195	17.0
				18	1/4"	10.1	224	19.5
52-14b	P1	46' - 5"	3000 psi	2	1/32"	0.7	71	6.2
				6	1/8"	2.8	101	8.8
				12	1/4"	6.4	140	12.2
				18	5/16"	9.9	183	16.0
62-02	P3	45' - 0"	3200 psi	6	1/8"	2.4	138	12.1
				12	1/16"	5.0	199	17.3
				18	3/16"	8.8	226	19.7

Table 3. Summary of CAPWAP Results.

Pile No.	Pile Type	Penetration Depth (ft)	Drop Height (in)	Pile Set (in)	Max Mobilized Capacity		
					Total (kips)	Shaft (kips)	Toe (kips)
52-14a	P1	44' - 11"	2	1/16"	78	46	32
			6	3/16"	125	52	74
			9	1/4"	143	45	98
52-10	P2	45' - 3"	2	1/32"	92	52	40
			6	3/32"	160	74	86
			12	3/16"	193	70	123
			18	1/4"	218	57	161
52-14b	P1	46' - 5"	2	1/32"	79	55	24
			6	1/8"	115	52	63
			12	1/4"	150	43	107
			18	5/16"	188	42	146
62-02	P3	45' - 0"	6	1/8"	139	71	68
			12	1/16"	195	73	122
			18	3/16"	228	88	140

CONCLUSION AND DISCUSSION

Based on the estimated load vs. settlement curves shown in Figure 5, the estimated resistance at ½ inch of vertical settlement ranged from 120 to 185 kips for the helical piles tested. In all cases, the test results support that the helical pile resistances exceeded the design loads, which is also summarized in Table 4 below.

Table 4. Summary of Capacity Results.

Pile Designation	Pile Type	Design Capacity (kips)	Pile capacity (kips)	
			at settlement of 0.25 inch	at settlement of 0.5 inch
P1-TP1	P1	87	88	123
P2-TP1	P2	96	135	178
P1-TP2	P1	87	95	120
P3-TP1	P3	111	135	185

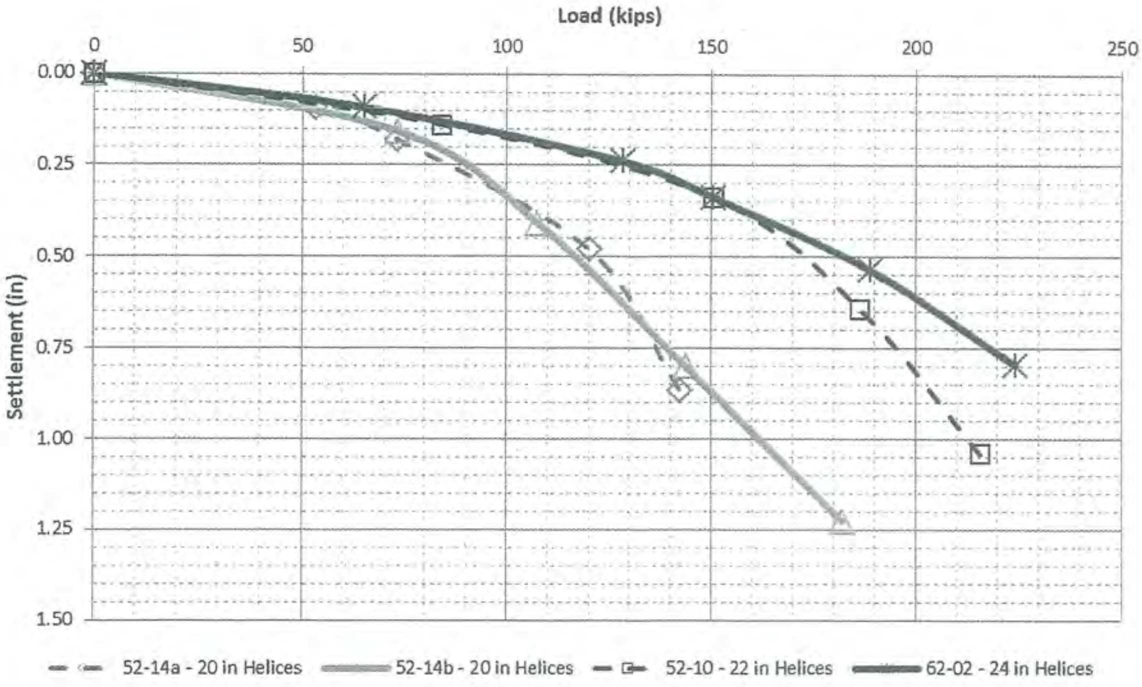

Figure 5. Estimated Load vs. Settlement Curves using High Strain Dynamic Testing and CAPWAP Analysis.

It is interesting to note that for pile tests TP1 and TP2 on helical pile type P1, the test results were very similar even though 52-14a had spun-out near the end of installation and did not achieve the anticipated torque. From this observation, it can be concluded that it is highly likely that both helical piles had the bottom helix bearing on the hard pan layer. It was noticed that helical pile types P2 and P3 had identical estimated load vs. settlement curves up to an applied load of approximately 150 kips, and that additional end bearing resistance was provided by the larger 24 inch diameter helices on pile type P3 at higher loads, when compared to the type P2 helical pile with the slightly smaller 22 inch diameter helices.

The estimated load vs. settlement curves produced from HSDLT, and the associated analysis, produced reasonable results that concurred with the theoretically anticipated results based on the design criteria for helical piles. HSDLT showed that for helical piles the testing is a viable, cost effective alternative to the traditional static load test for helical piles. All of the tests discussed above, were performed in a single day, whereas traditional static load testing of the same piles would likely have taken a week to complete or longer.

A recent surge in awareness by engineers of helical pile technology has this foundation type becoming selected more often than traditional concrete and driven steel piles, over a diverse range of industries and geological settings, involving higher applied loads in more complex applications. The cost savings alone resulting from using helical piles in most cases can be significant, and having an inexpensive, quick, and reliable means of verifying pile performance through HSDLT only adds to provide assurances to clients about the benefits of using a helical pile foundation system.

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