Design of Dynamically Wind-Loaded Helical Piers for Small Wind Turbines

Russell Buhler[1] and Amy B. Cerato, P.E., M.ASCE[2]

Abstract: The expansion of alternative energy has created a demand for sustainable alternatives for wind turbine foundation design. This study investigates the proposed application of helical piers as foundations for guyed cables of small (1–10-kW) wind towers. Before the foundation system can be implemented, pier response to typical working loads and extreme environmental conditions must be determined. Field conditions were determined by equipping an existing wind tower with accelerometers and load cells while monitoring wind speed to determine tower response. A full-scale testing program was conducted, which simulated dynamic loading conditions based on the tower response. The testing program varied between typical working conditions and extreme load events to determine the critical loading case and creep potential from long-term loading. This paper discusses the effects of dynamic loading on helical pier performance and compares the results to that of uplift prediction methods typically used in helical pier design.

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Introduction

Motivated by economics and/or environmental concerns, many homeowners are interested in alternative energy sources and small wind systems (<10 kW), which may be used to reduce electricity costs. However, the purchase and installation of a 10-kW wind turbine would realistically take approximately 20 years to repay the investment by the owner. Since today's homeowners are more mobile than past generations, wind energy with portable "green" foundation systems would increase the salability of alternative energy.

A helical pier is defined as a foundation element consisting of a central shaft with at least one helix plate located on the shaft with its axis positioned parallel to the shaft's length (Davis 2009). The piers are installed by applying compressive force and torque to the shaft causing the helices to screw through the soil. Helical foundations can be designed to resist both uplift and compressive forces. The focus of this study is on uplift capacity due to the application as guyed cable anchors. The traditional method for constructing the foundations for wind towers involves excavation and construction of subsurface reinforced concrete anchors, which requires the anchoring to be completed several weeks before the wind system is to be erected so that the concrete has time to cure. Not only is construction time an issue but also site accessibility is an issue: many times, wind towers are erected in remote areas without access for concrete placement. Helical piers provide the benefit of decreased construction time and accessibility and can be placed year round since there are no concrete curing requirements. The equipment used to install the piers is smaller and more mobile which creates the ability to place the turbines in remote areas and the piers can be removed and reinstalled at another site which would allow for owner mobility.

Designing and constructing foundations using helical piers require greater knowledge of site conditions than a typical gravity concrete anchor. Traditional methods for predicting capacity of helical piers use the site information and soil strength parameters to create theoretical predictions of pier capacities based on soil mechanics. These values are then verified using installation torque correlations during pier installation. However, there is limited knowledge on sustained dynamic loading effects on helical pier pull-out capacity for use in the small wind power industry. It is possible that changes in environmental conditions, such as a rising water table, could reduce effective stress above the helices creating a susceptibility to creep. The sustained dynamic action could also create conditions that are conducive to pier creep. If either of these conditions creates excessive movement of the pier, it may be necessary to maintain the system by tensioning the cables periodically to remove the slack. It is the purpose of this study to test full-scale helical piers under sustained cyclic load and determine the amount and rate of creep occurring as a result of the tower and cable vibrations and changing environmental conditions.

Uplift Capacity Prediction Methods

Three methods are commonly used in the uplift capacity prediction of helical piers. Two methods are based on soil mechanics with a third based on empirical observations. The soil mechanics prediction methods are the cylindrical shear method and individual bearing method. Several different equations have been pro-

[1]Graduate Research Assistant, Dept. of Civil Engineering and Environmental Science, Univ. of Oklahoma, 202 W. Boyd St., Room 334, Norman, OK 73019. E-mail: russellbuhler@ou.edu

[2]Assistant Professor, Dept. of Civil Engineering and Environmental Science, Univ. of Oklahoma, 202 W. Boyd St., Room 334, Norman, OK 73019 (corresponding author). E-mail: acerato@ou.edu

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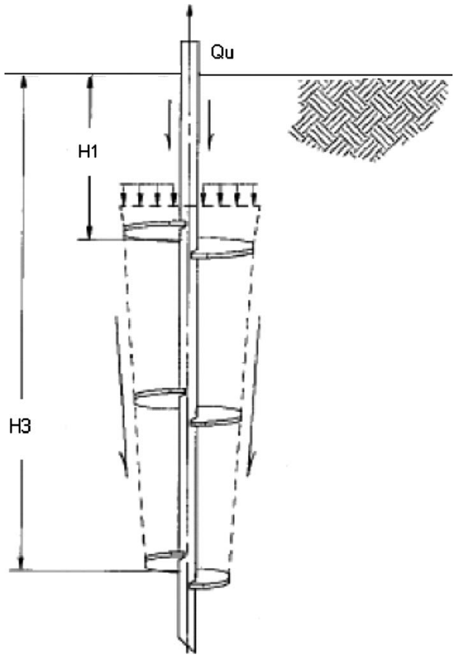

Fig. 1. Cylindrical shear surface (Lutenegger 2009)

posed based on the soil mechanics approaches to estimate the uplift capacity including Mooney et al. (1985), Narasimha Rao et al. (1990, 1993), Ghaly et al. (1991), and others. Fig. 1 depicts the cylindrical shear method failure surface. It assumes that the top and bottom helical plates are connected by a cylindrical surface. The capacity is determined by calculating shear resistance of the cylindrical failure surface and the bearing of the top helix on the above soil. Fig. 2 shows the individual bearing surface method failure surfaces. This method assumes that failure occurs above each individual helix and the uplift capacity is the sum of the bearing capacity of each individual helix.

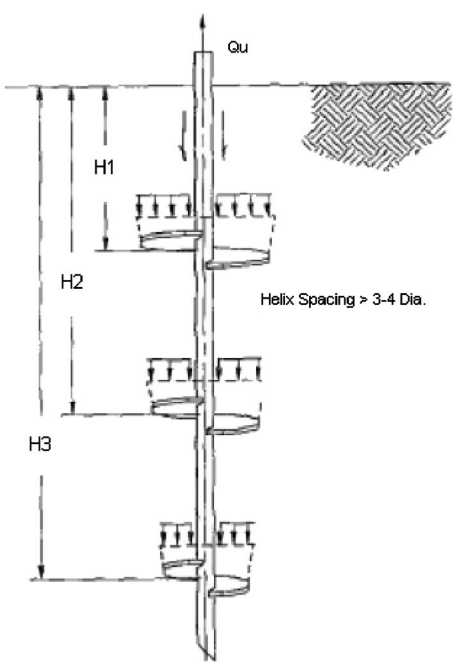

Fig. 2. Individual bearing failure (Lutenegger 2009)

The two soil mechanics methods require detailed knowledge of the soil properties at the site, which necessitates a full spectrum of soil testing, to determine the input parameters for the uplift equations. This process is expensive and time consuming and soil testing cannot be done for each pier. Therefore, general site profiles are created that may produce inaccuracies in input parameters for individual piers in varying soil conditions.

An empirical method first published by Hoyt and Clemence (1989) was based on installation torque of the pier. The method related installation torque to pullout capacity using an empirical factor as seen below

$$Q_{\mathrm{ult}} = K_t T$$

where Q_{ult}=ultimate uplift capacity; K_t=empirical torque factor; and T=average installation torque.

Hoyt and Clemence (1989) collected results from 91 multihelix load tests from published literature and private files for analysis. It was determined that the installation torque should be averaged over the final distance of penetration equal to three times the largest helix diameter. An empirical factor (K_t) of 33 m^{-1} (10 ft^{-1}) is to be used for all square shaft piers used in this study. Theoretically, the K_t value should vary according to pier geometry and soil properties; however, the value of 33 m^{-1} (10 ft^{-1}) is the default value used for square shafted anchors regardless of size or soil conditions. They concluded that the torque correlation method more accurately predicted the uplift capacity of the piers than either of the soil mechanics methods. This may be due to the natural variation in soil borings across a test site, which are used for predictions using the soil mechanics methods. Even though Hoyt and Clemence found better results from the torque correlation method, this method should be used only as verification for the soil mechanics methods. The torque correlation method can also be invalidated when refusal conditions are encountered during installation or when water table increases after installation (Cerato and Victor 2009). The method relies on averaging the final distance of penetration equal to three times the largest helix diameter, so an excessive final value would create an unrepresentative average value for uplift prediction. The torque correlation method also is generally only accepted when the pier's top helix is at least five times its diameter below the ground surface (Pack 2000).

Knowledge about the behavior of helical piers subjected to cyclic loading is limited because little research has been conducted examining the effects. There are some disagreements about the effect dynamic loading has on the static capacity of helical piers. Previous research has found that preloading and cyclic loading can decrease the strength of cohesive and cohesionless soil but in some cases increase the postcyclic static uplift capacity (Hanna et al. 1978; Hanna and Al-Mosawe 1981; Clemence and Smithling 1984; Cerato and Victor 2009).

Research by Clemence and Smithling (1984) found that there was significant reduction in static capacity of piers after cyclic loading. Hanna and Al-Mosawe (1981), however, reported a significant increase in capacity after prestressing. Strengthening after dynamic loading was also seen during an independent study done several years later by Cerato and Victor (2009). Clemence and Smithling (1984) expected that a difference in behavior of the piers could be a result of the installation. They expected that if soil is loosened during installation then the cyclic loading will increase the capacity of the pier during static loading, while if the soil is densified during installation the pier will lose static capacity if subjected to cyclic loading. The cyclic loading would densify loose soil but loosen dense soil.

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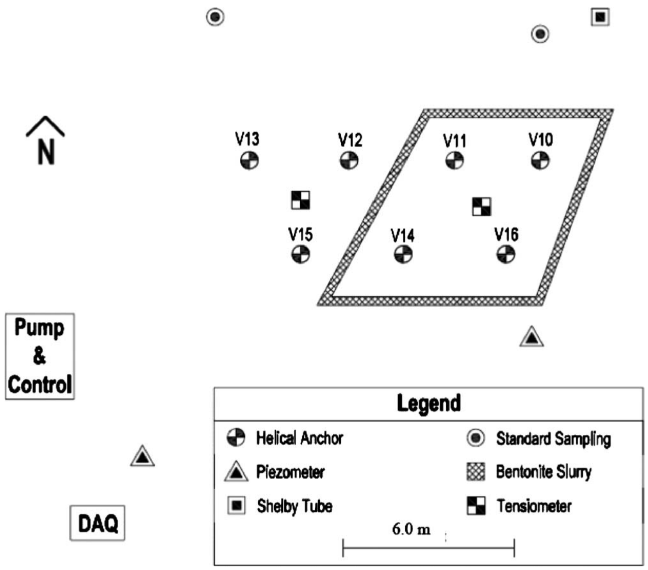

Fig. 3. Site plan

Cyclic loading of helical piers also poses the threat of progressive creep over time, which would be of great importance when applied to wind towers with a 30-year design life. Ghaly et al. (1993) discussed unpublished results that upward cyclic creep for one cycle is almost 100% recoverable if the ratio of cyclic loading to ultimate static uplift capacity is less than 25%. A later independent paper from Cerato and Victor (2009) found that cyclic loads at 25–40% of the static uplift capacity minimize the long-term creep. No research in clays has been conducted discussing the effect of cyclic loading span (i.e., difference between minimum and maximum loads in a cycle) on the capacity of helical piers. Hanna and Al-Mosawe (1981) found that anchors in dry sand showed a higher creep rate at lower loads with greater span due to loss of any preloading. They found that "the sand near the anchor changed gradation during repeated loading due to grain grinding and crushing." This increased the density and decreased the effective normal stress allowing the anchor to slowly pull through the sand. In this study, the anchors are embedded in a clayey soil, so there will be no change in gradation that decreases normal stress and allows the anchor to pull though the soil.

Site Information

Testing was conducted at the Univ. of Oklahoma (OU) south campus on the east side of the Fears Structural Engineering Laboratory. The site plan is presented in Fig. 3 showing the relationship of the helical piers to the test borings and piezometers. Hand auger borings were conducted within a half meter of each pier to a depth of 3.3 m (11 ft) to determine any variation between individual piers.

The subsurface profile was developed using the results of hand auger borings, the split spoon sampling, and the Shelby tubes. The results of the exploration are presented in Fig. 4 with an illustration of an installed helical pier at the site to show the relationship between soil layers and helical plates. The testing area consists of a lean clayey soil that increases in sand content with depth. There was only slight variation in the hand auger borings at each individual pier.

Soil testing conducted includes Atterberg limits, hydrometers, unconfined compression, and sieve analysis. The soil properties from all testing were averaged for each layer to create the results shown in Table 1. These values were used in uplift capacity prediction calculations.

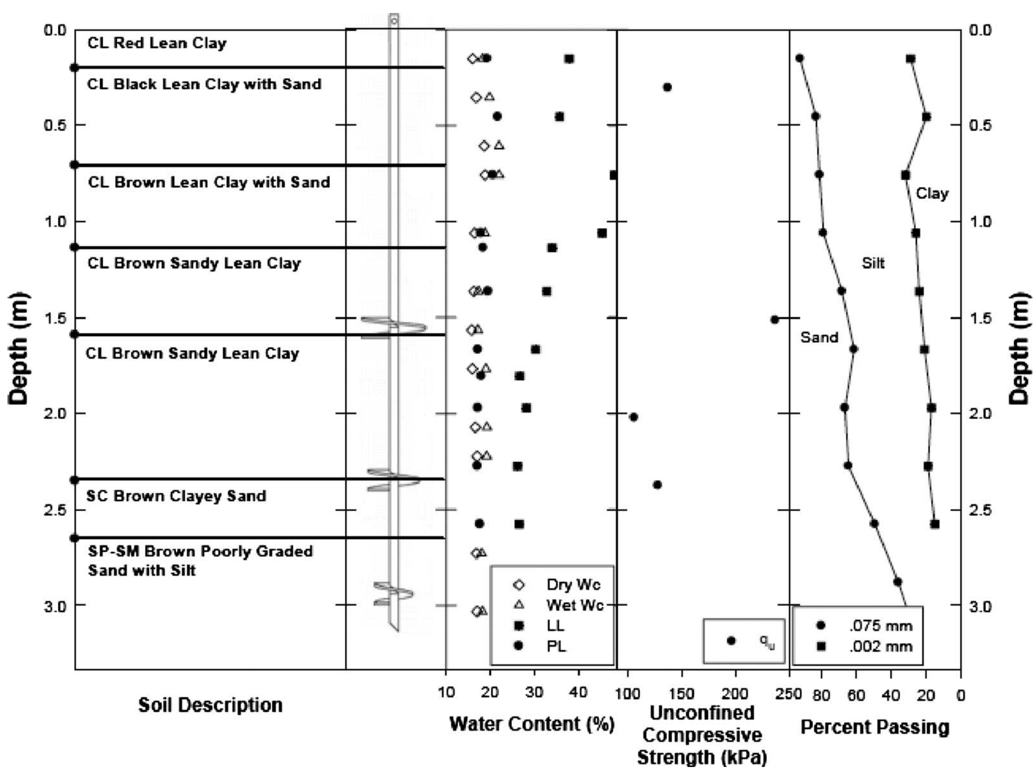

Fig. 4. Soil profile

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Table 1. Soil Properties

Depth (m)	W_c (%)	PL	LL	Activity	Φ'/Q_u (kPa)
0–0.2	16	19.3	37.9	0.64	138
0.20–0.71	17.7	21.7	35.7	0.7	138
0.71–1.14	17.7	19.3	46.7	0.96	237
1.14–1.60	16.1	19	33.4	0.55	237
1.60–2.36	16.5	17.4	27.9	0.58	106
2.36–2.67	16.9	17.7	26.6	0.59	129
2.67–3.05	17	NP	NP	N/A	29°

The water table was monitored in two locations to determine any gradient across the site. Past testing had shown that a rise in water table could significantly reduce uplift capacity of helical piers (Cerato and Victor 2009). Using rainfall data from the Oklahoma Mesonet and the results of the water table monitoring, Fig. 5 was created to show the relationship between rainfall and water table rise before, during, and after testing. The Oklahoma Mesonet is a network of over 110 automated environmental monitoring stations covering Oklahoma designed and implemented by scientists at the OU and at Oklahoma State Univ. There is at least one Mesonet station in each of Oklahoma's 77 counties and this study used rainfall data from station No. 121 located 2.1-mi northwest of Norman, Oklahoma in Cleveland County. As can be seen, the water table was at 2.5 m at the time of installation. During the testing period the water table receded as low as 3.3 m on January 30, which is about the time of piers V12 and V15 testing, and rose as high as 2 m on May 2 near the time of piers V10 and V16 testing.

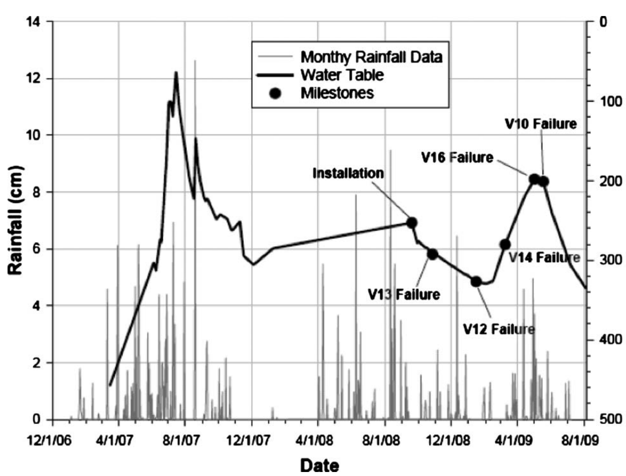

Fig. 5. Site water table profile

Helical Pier Installation

Seven piers were installed at the site on September 19, 2008. The piers were the Hubbell Power Systems SS 175 model, which have a square shaft with a width of 5.7 cm (1.75 in.). Six piers had three helices measuring 20.32, 25.4, and 30.48 cm (8, 10, and 12 in.) in diameter with one pier having two helices measuring 20.32 and 25.4 cm (8 and 10 in.). All piers were installed to a depth of 3 m (10 ft) to satisfy the design criteria of the top helix being five helix diameters below the ground surface. The target installation torque was 3 kN m (2,200 ft lb), but this criterion was satisfied before the minimum depth of 3 m (10 ft).

The torque rating for this particular series of helical piers is 14.9 kN m (11,000 ft lb) and the ultimate uplift capacity limit is 489.5 kN (110 kip); however, the ultimate mechanical tensile strength is limited by the coupling to 445 kN (100 kip). Each pier was screwed into the earth with a high-torque hydraulic drive mounted on a rotary auger drill rig equipped with an in-line torque-monitoring device. Torque readings were taken every 0.305 m (1 ft) with the final three readings being used for uplift capacity prediction. Table 2 shows the installation torques at each reading for all piers.

Table 2. Installation Torques

Anchor V-10		Anchor V-11		Anchor V-12		Anchor V-13		Anchor V-14		Anchor V-15		Anchor V-16	
Depth (m)	Torque (kN m)	Depth (m)	Torque (kN m)	Depth (m)	Torque (kN m)	Depth (m)	Torque (kN m)	Depth (m)	Torque (kN m)	Depth (m)	Torque (kN m)	Depth (m)	Torque (kN m)
0.30	1.36	0.30	1.08	0.30	1.36	0.30	1.08	0.30	1.08	0.30	1.08	0.30	1.08
0.61	1.36	0.61	1.36	0.61	1.63	0.61	1.49	0.61	1.08	0.61	1.36	0.61	1.36
0.91	1.63	0.91	1.63	0.91	2.58	0.91	2.03	0.91	1.90	0.91	2.44	0.91	1.63
1.21	2.44	1.21	2.71	1.21	3.25	1.21	2.71	1.21	2.58	1.21	3.25	1.21	2.03
1.52	2.71	1.52	2.71	1.52	3.66	1.52	2.71	1.52	2.71	1.52	3.66	1.52	2.58
1.82	3.39	1.82	4.07	1.82	5.15	1.82	3.80	1.82	4.07	1.82	5.15	1.82	2.98
2.12	3.80	2.12	3.53	2.12	4.61	2.12	3.80	2.12	3.80	2.12	4.20	2.12	3.66
2.42	3.93	2.42	4.20	2.42	5.69	2.42	3.80	2.42	4.34	2.42	6.51	2.42	3.93
2.73	4.20	2.73	5.15	2.73	5.83	2.73	4.34	2.73	6.10	2.73	6.78	2.73	3.93
3.03	4.34	3.03	4.47	3.03	4.07	3.03	4.34	3.03	6.51	3.03	5.02	3.03	4.47
Average of last 3	4.16	Average of last 3	4.61	Average of last 3	5.20	Average of last 3	4.16	Average of last 3	5.65	Average of last 3	6.10	Average of last 3	4.11

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Fig. 6. Test setup

After installation, a trench was dug that surrounded four of the piers and was filled with bentonite slurry. The trench served as an impermeable barrier that allowed for manipulation of soil saturation during the testing of the four piers. Water conditions were the same for all piers at the time of installation with a water table of 2.75 m (9.1 ft).

Testing Assembly

The testing frame consisted of two W18×55 beams spanning 4.2 m (14 ft) between two railroad tie cribbing stacks. The railroad ties allowed for height to be added when more actuator stroke was needed. A 222.5-kN (50-kip) hydraulic actuator was placed between the two beams and centered. The beams and actuator were secured to reduce movement during loading. A picture of the testing setup is presented in Fig. 6. The actuator was run by a hydraulic pump and control box. A load cell was located between the hydraulic actuator and the pier and connected to the control box. Load control was used during testing, so the control box and pump would automatically adjust to maintain the same load until manually changed. The control box was able to produce cyclic loads at a range of frequencies (1–5 Hz).

The lead from the load cell, after connection to the control box, was routed to a data acquisition unit powered with solar energy. The unit took readings at a rate of 50 Hz and had enough memory to collect data for the entire length of the test (approximately two to three weeks). Displacement of the pier was measured using a wire pot connected to a reference beam. The reference beam was a piece of angle iron, 1.8 m (6 ft) long, and anchored into the ground with rebar. The length of the bar was necessary to make sure that there was no disturbance of the reference beam from the soil above the pier helices. The wire pot was connected to the same data acquisition unit as the load cell and data collected at the same rate.

Testing Program

All tests were long term (approximately two to three weeks) with the intent to simulate loading that would be felt over the lifetime of a tower foundation. Testing began with the application of the working load. The working load for all tests was 8.9–17.8 kN (2–4 kip) at a rate of 5 Hz. This was the default load that the piers were subjected to a majority of the time. Only when "shock" wind loads were applied was the loading different from the working load. This load case was based on two factors: the results of the instrumentation of an existing 10-kW tower and the recommended helical anchor seating load of 20% of its design load.

Two piers (V12 and V13) had different loading regimes. Loading of these piers was more similar to past research at OU (Cerato and Victor 2009). The working loads were the same for these two piers but shock loads did not increase the load span. Instead, the maximum load was raised but the difference between the minimum and maximum dynamic loads was generally kept at 8.9 kN (2 kip). Pier V13 was loaded to a maximum of 44.5–89 kN (10–20 kip), then was failed statically. Pier V12 was loaded at 8.9 kN (2 kip) spans up to 80.1–89 kN (18–20 kip). Testing then transitioned to maximizing load span to show the effect of span on displacement. The 80.1–89-kN (18–20-kip) span was followed by 44.5–89, 8.9–44.5, 8.9–66.8, and 8.9–89 kN (10–20, 2–10, 2–15, and 2–20 kip) at a rate of 1 Hz each.

The shock loads for the rest of the piers (V10, V11, V14, V15, and V16) were applied at a rate of 1 Hz. The rate was reduced because the high wind loads in the field have longer wave periods than the typical working load. Also the testing equipment was unlikely to be able to achieve enough fluid movement to produce the highest wind loads at a faster rate. The load was applied for approximately 1 h and monitored for movement. If it was determined that the displacement had plateaued, then the test resumed under normal working conditions. However, if it was determined that the pier was continually displacing without an increased load, the pier was considered failed. Typically, a displacement of 5.1 cm (2 in.) is considered failure in helical pier applications; however, to get a better idea of soil behavior, the piers were loaded until there was constant displacement at constant load even after 5.1 cm (2 in.) was reached. This could be justified because in wind tower applications, if a pier was to move 5.1 cm (2 in.) or more at one time and stop, the guyed cable could be tightened on the turnbuckle. The problem arises when the pier continuously creeps under a sustained load.

Shown in Table 3 is the wind load schedule for piers V10, V11, V14, V15, and V16. These loads took place once a day for approximately 1 h until failure was achieved. Between each 1-h shock load, the piers were pulled with a 8.9–17.8 kN (2–4 kip), 5-Hz seating load to simulate normal working conditions. The same loading schedule was applied to the five piers listed above. If the pier failed prior to the maximum value, loading was stopped.

Table 3. Wind Loads Tested

Minimum load (kN)	8.9	8.9	8.9	8.9	8.9	8.9	8.9	8.9	8.9	8.9	8.9	8.9	8.9
Maximum load (kN)	26.7	35.6	44.5	53.4	62.3	71.2	80.1	89	97.9	106.8	115.7	124.6	133.5

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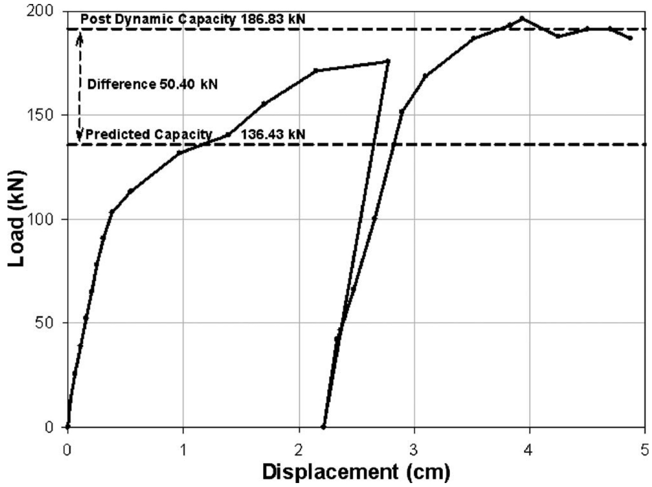

Fig. 7. Pier V13 static test

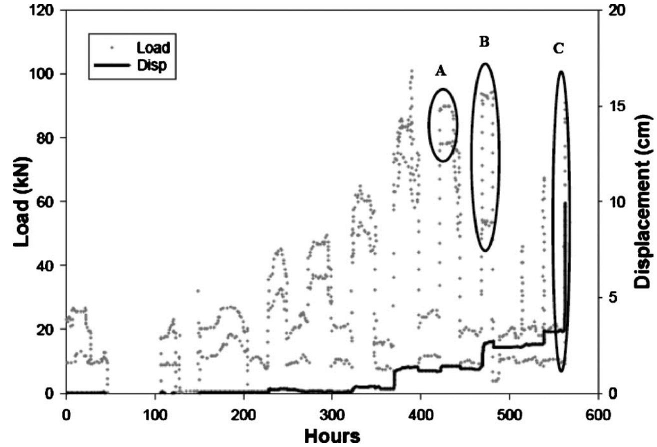

Fig. 8. Pier V12 dynamic results

Effect of Dynamic Loading on Static Capacity

Pier V13 was dynamically loaded then statically failed to determine the effect of dynamic loading on static uplift capacity. The predicted uplift capacity of the pier was 136.5 kN (30.67 kip) based on the torque correlation method. The water table had receded from 2.5 m (8.3 ft) at installation to 2.7 m (9 ft) at the time of failure. Pier V13 was tested dynamically with a span of 8.9 kN (2 kip) to a load of 44.5–53.4 kN (10–12 kip); then, the span and load were increased to 44.5–80.1 kN (10–18 kip). After this load schedule, total displacement was less than 2.54 cm (1 in.).

After dynamic loading, a hand operated hydraulic jack was attached. Load was applied and then held constant at regular intervals described in the testing program until displacement of the pier stopped. Displacement was determined to have stopped if over a 30-s interval there was no change. This is less than the typical 20-min period used for incremental load testing. From previous experience, it was known that once displacement stopped, it would not continue until a higher load was applied. Failure was determined when, at a constant load, the pier continued to displace. The results of the test can be seen in Fig. 7 along with the relationship to the uplift capacity prediction at installation. The postdynamic failure load was 186.8 kN (42 kip) which was 50.4 kN (11.67 kip) higher than expected. The load returning back to zero was due to the hydraulic jack running out of stroke. The test was stopped and restarted with spacers added to provide the extra stroke needed to determine failure.

This increase in capacity is most likely due to the densification of the soil above the helices during the dynamic testing. It is possible that some increase in capacity is a result of the receded water table; however, it is unlikely that a difference of 0.21 m (0.7 ft) would cause the significant difference in capacity that was measured. During a typical helical pier installation, the helices of the pier loosen the soil as they are screwed into the soil. Dynamic loading at loads below that of failure would likely compact the loosened soil to values greater than that of the in situ soil. The static testing of pier V13 following dynamic loading provided similar conclusions to that of previous research at the same site (Cerato and Victor 2009).

Effect of Large-Span Dynamic Loading

To study the effect of increasing span on dynamic uplift capacity, pier V12 was tested under two different loading regimes. Constant span with increasing load was followed by increasing the span incrementally until the maximum loads were the same. Fig. 8 shows the results of this testing method.

Areas A, B, and C show the three areas where the maximum load was 89 kN (20 kip). Area A has a load of 80.1–89 kN (18–20 kip) and showed very little displacement. Area B shows an increase in span to 44.5–89 kN (10–20 kip) and displacement of about 1.5 cm (0.6 in.), while Area C shows an increase in span to 8.9–89 kN (2–20 kip) and anchor failure with testing stopped after displacement of about 7.5 cm. (3 in.). These three points show that it is the dynamic span rather than the maximum load, which has a greater effect on helical pier performance at loads significantly less (~50%) than predicted ultimate static uplift capacity. Similar results were found by Hanna et al. (1978) in dry sand conditions who stated "the displacement of an anchor depends primarily on the load change during a cycle of loading; the higher the load change, the smaller the number of cycles to cause failure." However, in the sandy conditions of the study of Hanna et al. (1978), all piers eventually failed even at very low spans, but it took many cycles. In this research, it was found that at low spans, there was a maximum amount of displacement that could be reached until a higher load was added. It is likely that the difference in soil type is the cause of this difference in behavior. In dry sand conditions, it is likely that the sand would shift around the pier and fill the void under the helices that is created during the peak load rather than being densified above the helices as would be seen in the unsaturated clay conditions found in this study.

Effect of Water Content Change on Dynamic Uplift Capacity

All piers were installed on the same day in the same moisture conditions. A bentonite slurry wall was installed around four piers and topped with a mound of dirt to create a levee. During testing of the piers outside the leveed area, the water used for cooling of the hydraulic pump was diverted into the enclosed area to create standing water conditions. The water was allowed to seep into the soil with the intent of raising the water table artificially. The method unfortunately did not raise the water table because of the permeable sand layer directly beneath the clay layer but did increase the water content of the area by an average of 2%. The

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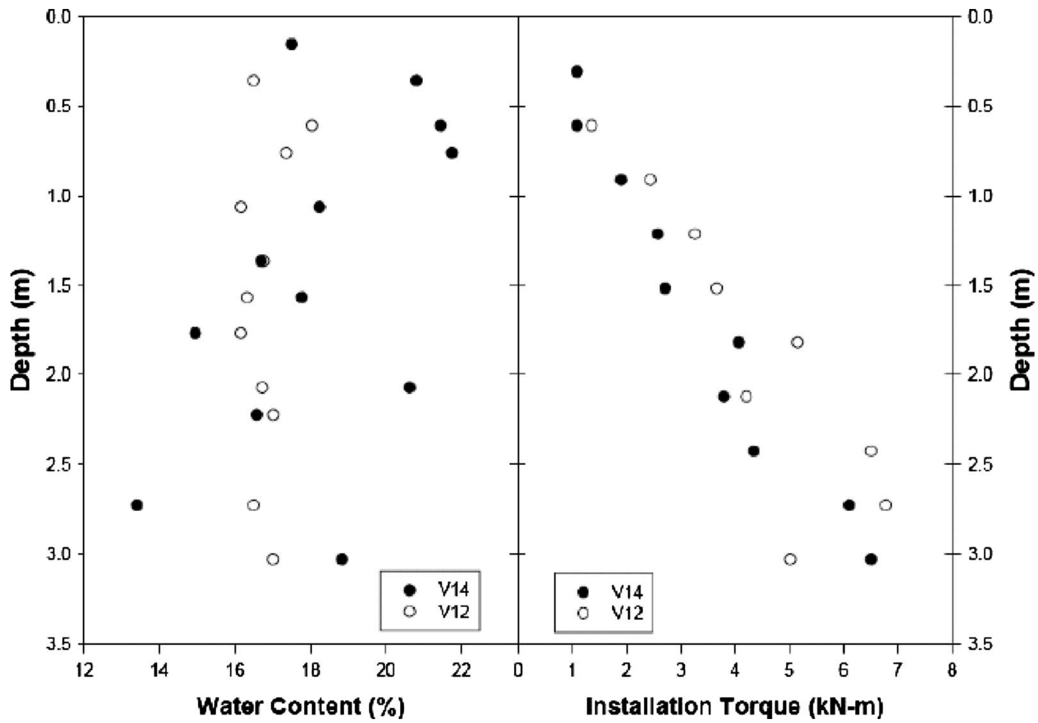

Fig. 9. Pier V12 versus Pier V14 properties comparison

following is a comparison of piers tested in both moisture conditions.

Piers V12 and V14 were installed to similar installation torques of 5.2 and 5.6 kN m, respectively (3,833 and 4,167 ft lb), with V12 being tested in natural conditions and V14 with raised moisture content. While most of the moisture content determinations for both conditions show that pier V14, located inside the moat, did indeed have higher moisture contents on average throughout the profile, there were two samples where the reverse was true. During the time that the moisture content samples for these two piers were taken (June 3, 2009) the natural water table outside of the moat was at 2.4 m (8 ft). Therefore, the water content results in the sandy soil below 2.4 m could be skewed depending on the sample taken. When the hand augers were pulled, the soil was in a slurry state below the water table, which resulted in water content samples that possibly had excessive water included. When the results below 2.4 m are neglected, the average moisture content is 1.8% higher for pier V14. Fig. 9 shows the installation torques and water content profiles. Pier V12 had higher installation torques at all depths until the last foot when V14 exceeded the torque of V12.

Both piers demonstrated the same capacity as seen in Fig. 10. Pier V14 did experience much more displacement at lower spans likely due to V12 being preloaded at low spans before increasing the span. Despite the higher displacement of V14, the loading was not considered critical (continuous displacement at constant load) until the determined failure load of 8.9–89 kN (2–20 kip).

Piers V12 and V14 had similar installation torques and the same maximum dynamic span failure even though the moisture content profile of Pier V14 was consistently 2% above that of Pier V12. These results show that the capacity of two anchors with similar torque installation values but differing water contents ($\Delta 2\%$) was not affected. It was expected that the suction in clayey soils would decrease with the increased water content and that this decrease in suction would decrease the capacity of the piers or simply cancel out the increase in effective stress. More testing would need to be conducted to identify a more consistent trend. A water content change of only 2% in the subsurface may not be enough to see a difference in uplift capacity in this type of soil.

Effect of Helical Pier Geometry on Dynamic Uplift Capacity

Helical piers come in a variety of different sizes and helix geometries. The majority of this study tested three helix piers with diameters of 20.32, 25.4, and 30.48 cm (8, 10, and 12 in.). Previous research has indicated that three helix piers perform better under dynamic conditions than two helix piers (Cerato and Victor 2008). One of the two helix piers (V16) was tested to compare with the three helix piers tested. Pier V10 was selected for comparison because it was the closest pier, had the same water content profile, and had the same uplift capacity prediction from the installation torque method. Fig. 11 shows the dynamic testing results for piers V10 and V16. Both piers were continued after failure was determined and, as expected, displacement continued.

These results show that the three helix pier (V10) is a better selection for dynamically loaded helical piers. The two helix pier (V16) had the lowest dynamic span capacity of 8.9–62.3 kN (2–14 kip) of all seven piers tested, including pier V10, of which the only difference was helix geometry. V16 also failed much sooner (78 h versus 170 h) than V10 due to inclement weather during the first 75 h of V10 testing making it difficult to apply any shock loads. During this 75-h period, the loads could not be altered and it is unlikely that these low loads affected the actual failure span. The reduction in capacity is likely due to the concentration of the cyclic load on two helices rather than the distribution of the impact load over three layers. This could cause local bearing capacity failure resulting in the displacement during the shock load period.

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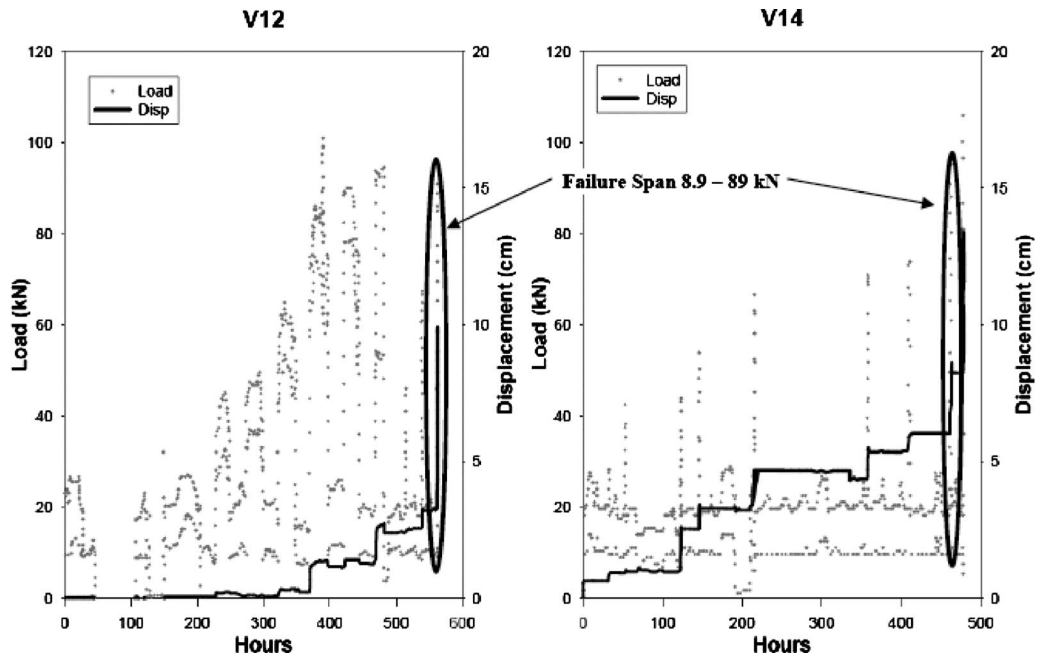

Fig. 10. Piers V12 and V14 results

Comparison to Uplift Prediction Methods

Uplift capacity predictions were made using four methods: cylindrical shear, bearing plate, Helicap, and torque correlation. Due to the high level of homogeneity across the site, average soil properties from Fig. 4 and Table 1 were used for the cylindrical shear, bearing plate, and Helicap methods.

Calculations for the cylindrical shear method were based on the formulas presented by Mooney et al. (1985). Their equation was modified to calculate a layered soil profile featuring sand and clay. The following equation was used:

$$Q_u = A_1 c N_{cu} + \sum \pi D_a c(H_3 - H_1) + \sum \frac{\pi}{2} D_a \gamma' (H_3^2 - H_1^2) K_u \tan \varphi'$$

where Q_u=uplift capacity; A_1=area of top helix; C=cohesion at helix or cylinder; N_{cu}=uplift capacity factor for cohesive soils; D_a=diameter of cylinder; H_3=depth to top of cylinder; H_1=depth to bottom of cylinder; γ'=effective unit weight; K_u=coefficient of lateral earth pressure; and φ'=undrained friction angle.

Since the soil above the top helix is clayey, the first term represents bearing capacity in clay soil. The second term is for

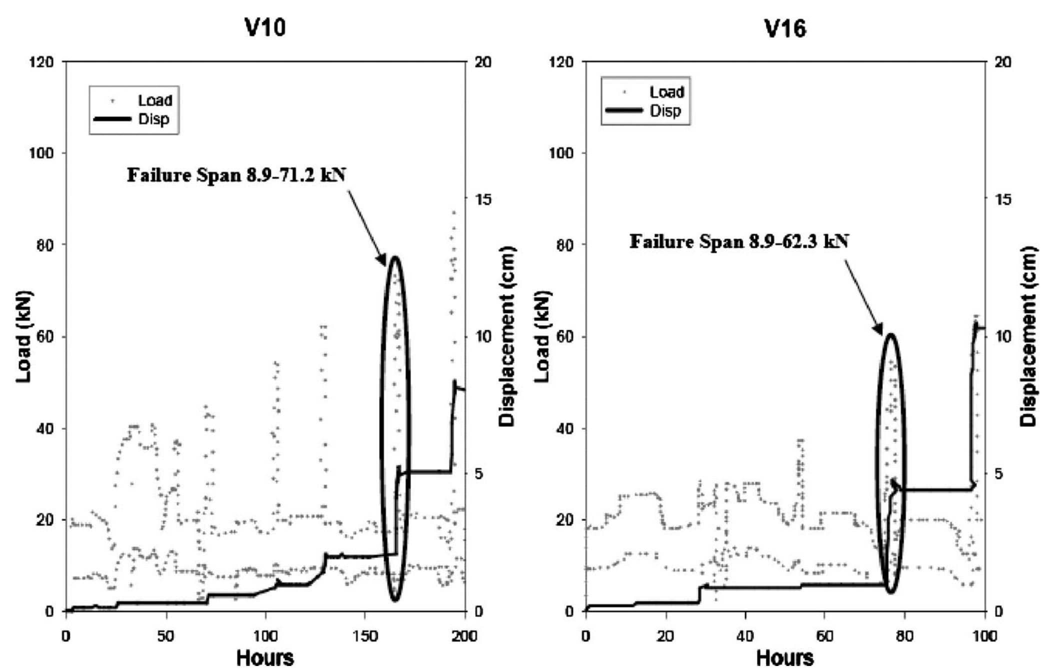

Fig. 11. V10 versus V16 comparison

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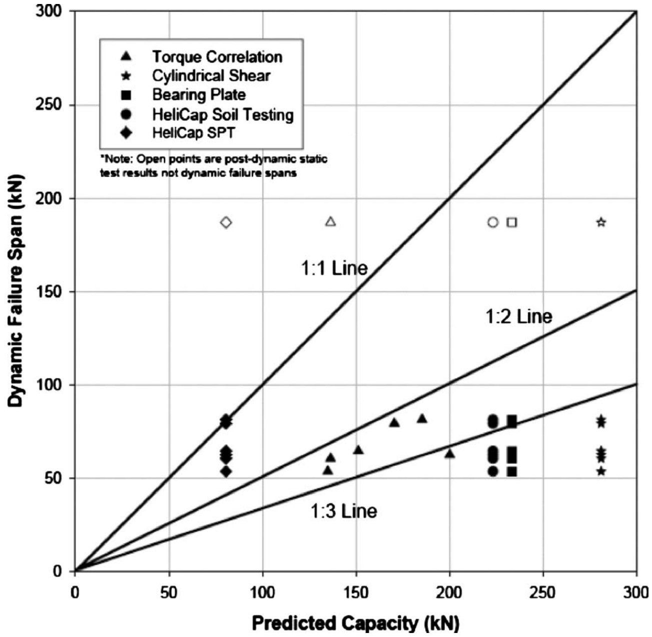

Fig. 12. Predicted capacity versus actual capacity

cohesion and the third term is for frictional resistance. The original equation was simplified by neglecting the effects of shaft friction, pier weight, weight of soil between helices, and suction on the bottom helix because they have minimal effect on results. For this study, rather than averaging the properties and creating one cylinder, each layer had a cylinder proportional to the size of the failure plane that would pass through it. Final calculations analyzed four different soil layers between the helices. An N_{cu} value of 9.4 was used as recommended by Mooney et al. (1985) for piers with an H/D ratio of 5 or greater. A K_u value of 0.5 was assumed. The result was a cylindrical shear uplift capacity prediction of 281.2 kN (63.2 kip).

The bearing plate method is based on the theory that all helices fail in bearing simultaneously with the uplift capacity being the sum of the bearing capacity of each individual helix. The equation used was again a modification of Mooney et al. (1985) and is as follows:

$$Q_u = \sum N_{cu} C A_e + \sum \varphi' H A_e N_{qu}$$

where A_e=area of each helix

Using a N_{cu} value of 9.4 and N_{qu} value of 10 (Mooney et al. 1985), this method produced an uplift capacity prediction of 233.6 kN (52.5 kip). The Helicap program uses equations similar to the bearing plate method for analysis. All relevant parameters were manually input into the program to produce an uplift capacity prediction of 224.7 kN (50.2 kip). When standard penetration N values are input into the program rather than laboratory testing results, the Helicap predicted uplift value is reduced to 80.6 kN (18.1 kip). This is due to the conservative values assumed for cohesion and friction angle based on the blow counts. The results of both soil mechanics analyses, both Helicap inputs, and torque correlation values based on Table 2 and the default empirical factor of 33 m^{-1} (10 ft^{-1}) are plotted in Fig. 12. These prediction methods have been found to be appropriate when predicting static uplift capacity, but as will be seen, an additional reduction term must be added when designing anchors anticipated to have large-span dynamic loads.

In Fig. 12 it can be seen that the data points for the soil mechanics prediction methods are aligned vertically for each method. This is due to the averaging of soil properties across the site to produce the most representative results. Since all six dynamically loaded piers were installed in the same soil with the same soil properties, each soil mechanics prediction method would only produce one prediction for all six piers. Each pier's actual failure load span results are then aligned vertically along this prediction. As has been noted, the predicted value is a static uplift capacity and the actual failure value is a dynamically loaded span. As can be seen there were essentially three distinct spans over which all six piers failed: 8.9–71.2, 8.9–80.1, and 8.9–89 kN (2–16, 2–18, and 2–20 kip). It should be anticipated that the predicted static uplift capacity will be significantly higher than that of the dynamically loaded failure span, and therefore, an additional reduction factor must be added to the prediction methods to account for anchors experiencing large-span dynamic loads. The three ratio lines show the comparison of using no dynamic load span reduction (1:1), a reduction factor of 2, which is the typical helical pier reduction factor in design (1:2), and a reduction factor that would provide adequate capacity for large-span dynamic loading (1:3).

It can be seen that the only prediction that results in a reduction factor lower than 2 is that of the Helicap calculation using conservative cohesion and friction angle values based on standard penetration results. This method comes the closest in predicting actual dynamic failure span magnitudes of dynamically loaded anchors; however, it does not have an adequate factor of safety (FS) embedded. The torque correlation method seems to consistently overpredict the failure span capacity of a dynamically loaded helical pier by a factor of 3; however, this is expected, as the torque correlation method is only valid in static uplift. This overprediction could be eliminated by reducing the predicted torque correlation uplift capacity by 3 to account for the large-span dynamic loads.

The high values of the soil mechanics prediction methods could be a result of the high cohesion value for the layer of soil above and around the top helix. Testing showed a cohesion value of greater than 4,500 psf for this layer. This layer had also a much higher N value of 15 as compared to 6 for the rest of the layers. The Helicap program uses a cohesion of 1,875 psf for a N value of 15. The difference in these two values creates the biggest difference in results. This discrepancy shows that the uplift capacity predictions are only as good as the soil properties that are found or assumed for the site. It is difficult to recommend a large-span dynamic loading reduction factor to these prediction methods because they are so dependent on the soil properties chosen. The torque correlation method works outside these parameters and appears to have more consistent results than the soil mechanics approaches and could be consistently reduced by 3 to account for the dynamic loading affects.

Conclusions

This paper discusses the results of the field testing of seven piers sharing the same installation depth and testing site. Effects of moisture content, dynamic loading type, and number of helices were investigated to address potential issues with using helical piers as wind tower guy cable foundations. All piers were tested to failure and the site underwent extensive soil testing to determine relevant soil properties. The findings of this full-scale field study are as follows:

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- Large span dynamic loading will significantly reduce uplift capacity of helical piers. Results showed the maximum span to be an average of only 40% of the predicted static uplift capacity. Therefore, if the torque correlation method is used, it should be reduced by a factor of 3 for helical pier applications subject to large-span dynamic wind loads. This conclusion should be limited to spans where the maximum load does not exceed 50% of the predicted static uplift capacity.
- Increasing soil moisture content slightly ($<\Delta 2\%$) without changing the water table had no discernible effect on the capacity of helical piers in this type of soil during this study. Piers in soils with increased water content showed equal capacity to comparable piers in drier soil. There is an indication of increased displacement, however, at lower loads in testing in increased water content conditions. More research would be needed to identify a trend in the results and make a conclusion.
- Dynamic span has a much greater effect on helical pier performance than maximum load applied. For the same maximum load, a lower minimum load will produce significantly more displacement.
- Three helix piers showed greater dynamic span capacity than a two helix pier that had the same installation torque and soil conditions. Fewer helices increase the possibility of local bearing capacity failure at each helix. These results, however, were based on a single two helix pier test.
- All wind loads applied during testing occurred for 1 h at a rate of 1 Hz (resulting in approximately 3,600 cycles). A single design wind load event would only cause a few of the maximum span wind loads. As a result it would be unlikely to see significant movement in a single event. The piers could be monitored for displacement and the guy wires tightened over the lifetime of the tower to compensate for any movement.

References

Cerato, A. B., and Victor, R. (2008). "Effects of helical anchor geometry on long-term performance for small wind turbine foundations subject to dynamic loads." *The Journal of the Deep Foundations Institute*, 2(1), 30–41.

Cerato, A. B., and Victor, R. T. (2009). "Effects of long-term dynamic loading and fluctuating water table on helical anchor performance for small wind tower foundations." *J. Perform. Constr. Facil.*, 23(4), 251–261.

Clemence, S. P., and Smithling, A. P. (1984). "Dynamic uplift capacity of helical anchors in sand." *Proc., 4th Australia–New Zealand Conf. on Geomechanics*, Vol. 1, Australian Geomechanics Society and New Zealand Geomechanics Society, Perth, Western Australia, 88–93.

Davis, R. (2009). "The helical pier defined." *Helical pier world*, ⟨http://www.helicalpierworld.com/archives/hpdefinition.aspx⟩ (July 13, 2009).

Ghaly, A., Hanna, A., and Hanna, M. (1991). "Installation torque of screw anchors in dry sand." *Soils Found.*, 31(2), 77–92.

Ghaly, A., Hanna, A., Ranjan, G., and Hanna, M. (1993). "Closure to 'Helical anchors in dry and submerged sand subjected to surcharge.'" *J. Geotech. Engrg.*, 119(2), 392–394.

Hanna, T. H., and Al-Mosawe, M. J. (1981). "Performance of prestressed anchors under slow repeated loadings." *Proc., 10th Int. Conf. of Soil Mechanics and Foundation Engineering*, Vol. 2, ICSMGE, Stockholm, 127–133.

Hanna, T. H., Sivapalan, E., and Senturk, A. (1978). "The behaviour of dead anchors subjected to repeated and alternating loads." *Ground Eng.*, 2(4), 28–34.

Hoyt, R. M., and Clemence, S. P. (1989). "Uplift capacity of helical anchors in soil." *Proc., 12th Int. Conf. on Soil Mechanics and Foundation Engineering*, Vol. 2, ICSMGE, Rio de Janeiro, 1019–1022.

Lutenegger, A. J. (2009). "Cylindrical shear or plate bearing?—Uplift behavior of multi-helix screw anchors in clay." *Geotechnical Special Publication No. 185*, IFCEE, Orlando, 456–463.

Mooney, J. S., Adamczak, S., Jr., and Clemence, S. P. (1985). "Uplift capacity of helix anchors in clay and silt." *Uplift behavior of anchor foundations in soil*, ASCE, Reston, Va., 48–72.

Narasimha Rao, S., and Prasad, Y. V. S. N. (1993). "Estimation of uplift capacity of helical anchors in clays." *J. Geotech. Engrg.*, 119(2), 352–357.

Narasimha Rao, S., Prasad, Y. V. S. N., and Prasad, C. V. (1990). "Experimental studies on model screw pile anchors." *Proc., Indian Geotechnical Conf.*, Indian Geotechnical Society, Bombay, 465–468.

Pack, J. S. (2000). "Design of helical piles for heavily loaded structures." *New technological and design developments in deep foundations*, ASCE, Reston, Va., 353–367.

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