Helical Piles Foundation for Wind Turbines: Full-Scale Testing of a Single Helical Pile in Sand

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ABSTRACT

Helical piles represent an attractive deep foundation system, which can be used to support wind turbines, solar farms, energy pipelines and telecommunication and transmission towers. This paper investigates the performance of full-scale single helical piles installed in sandy soil and subjected to lateral loads similar in characteristics to those induced by wind loads on wind turbine foundations. Three areas are investigated: the stiffness characteristics of the pile under low-strain wind loads; the effect of cyclic loading on the stiffness of the piles and its impact on serviceability requirements; and the effect of cyclic loading on the ultimate capacity of the pile foundation. The direct outcome of the testing program is to develop design tools, which will aid in the performance based design of helical piles supporting wind turbines.

1. INTRODUCTION

Helical piles represent an efficient deep foundation system for wind turbines, on-shore and off-shore. Helical piles are made of a central steel shaft, either a solid square shaft or circular pipe, with one or multiple helical plates affixed to it. They are installed into the ground by applying a torque at the pile head through a drive head. The piles could be installed to any depth and at any angle provided that the soil conditions are favorable.

Wind turbines are becoming an increasingly popular source of renewable energy in which the kinetic wind energy is converted into electrical power. Wind is the second most important renewable energy source in Canada which generates about 3.5 per cent of total electricity in Canada, according to Natural Resources Canada. Three-bladed horizontal axis wind turbines are the most common with current wind turbine heights reaching as high as 90 m.

Wind loads are cyclic in nature with predominant frequencies typically lower than 0.1Hz (Boggs & Dragovich, 2006). In addition, when wind blows against the blades the blades rotate. This rotational motion coupled with any imbalances in the central shaft, rotor, and blades yields an exciting dynamic force at the rotational speed of the blades. Moreover, as the blades pass through the central tower they generate a dynamic force at the blade pass frequency (BPF) which is equal to the number of blades x the shaft speed (Byrne and Houlsby, 2003).

The typical rotational speed for three-bladed wind turbines is 0.25Hz with a BPF equal to 0.75Hz (Haigh, 2014). Therefore, the helical piles foundation for a typical three-bladed horizontal axis on-shore wind turbine would be subjected to large lateral and rocking cyclic wind loads, and dynamic loads operating at the rotational speed of the blades as well as the three times the operational speed (i.e. BPF).

As such, a proper foundation design should ensure that the wind turbine's dominant natural frequencies are away from the exciting forcing frequencies while accounting for long term cyclic degradation of foundation stiffness.

The lateral cyclic and dynamic performance of helical piles is rarely addressed in the literature. This paper presents a detailed description of the test setup, experiment procedure, and test results of the full-scale single helical pile subjected to two-way cyclic lateral loads and dynamic loads.

2. BACKGROUND

2.1. Theory

The lateral performance of a pile can be characterized by its load-displacement curve, which displays the pile head displacement to an applied lateral load at its head. It can be obtained experimentally via full-scale testing or numerically.

The pile lateral stiffness can be evaluated employing a closed form solution that was developed based on the continuum approach and assuming plane strain conditions (e.g. Novak, 1974). However, the solution is limited to linear elastic pile and soil materials. This approximation is representative of cases where the applied loads on the pile result in shear strains in the soil less than 10^{-5}, which is typical for foundations supporting vibration machinery.

One of the most common numerical approaches for estimating the lateral response of piles is the p-y curves approach proposed by Reese (1984). In this method, the pile is simulated as a beam and the soil resistance is represented by a series of springs attached to the pile along its shaft as shown in Figure 1. The force-deformation relationship of these springs is represented by p, defined as the soil reaction per unit length of the pile at a given depth and y is the corresponding lateral deflection of the pile at that depth. The p-y curves could represent linear or non-linear force-deformation behaviour and are a function of the soil parameters.

El Naggar and Novak (1995,1996) developed a computationally efficient model for evaluating the lateral response of piles based on the Winkler hypothesis, similar to the p-y method. Their method accounts for soil nonlinearity, slippage and gapping at the pile–soil interface. El Naggar and Bentley (2000) further developed this model by employing dynamic p-y curves that account for the hysteretic behavior of the soil and energy dissipation during dynamic loading.

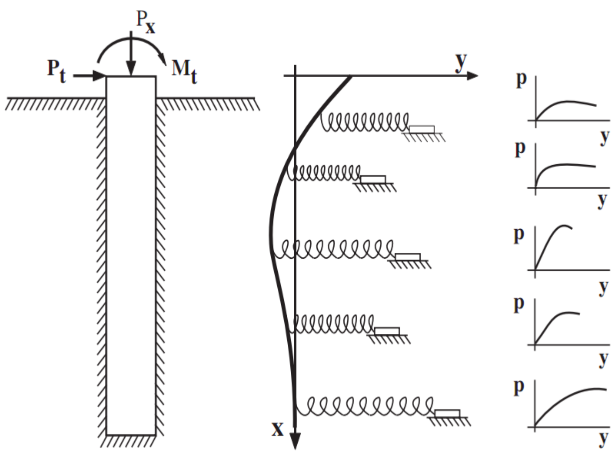

Figure 1. Discretization of a laterally loaded pile using p-y nonlinear springs (from Ensoft, Inc.)

2.2. Field testing

Prasad and Rao (1996) conducted one-way sustained cyclic load testing on model helical piles and straight shaft piles installed in clay. They found that cyclic performance of helical piles was better than straight shaft piles (i.e. without helical plates) with the same embedment and shaft diameter. On the other hand, Abdelghany and El Naggar (2010) conducted full-scale one-way sustained lateral cyclic tests on helical piles. They reported lateral capacity degradation due to the cyclic loading.

More recently, El Sharnouby (2012) investigated the cyclic two-way lateral performance of helical piles. He found that two-way cyclic loading resulted in overall degradation in pile stiffness and capacity. He also noted that the degradation attributed to the formation of gaps rather than reduction of soil strength parameters. He also found that one loading direction shows stiffer response than the opposite loading direction due to the formation of gaps.

Elkasabgy and El Naggar (2013) investigated the lateral low-strain dynamic performance of single helical piles installed in clay till. They reported a decrease in soil stiffness near the pile shaft because of the disturbance caused by the helices during pile installation.

3. FULL-SCALE TESTING

3.1. Pile geometry and layout

Two full-scale single helical piles (SH-1 and SH-2) were tested under lateral monotonic and cyclic static loads as well as dynamic loading at the Englekirk Structural Engineering Center facility at the University of California San Diego (see Figure 2 for pile layout).

Both piles were identical in geometry and were installed within 5m of each other such that the soil properties at each pile location are similar. The central pile shaft was made of a welded steel pipe (ASTM A500, Grade B) with diameter equal to 114.3mm (4.5in) and a wall thickness equal to 8.6mm (0.337in). Each pile had a single 254mm (10in) diameter helical plate with an overall pile length equal to 3.96m (156in).

Pile SH-1 was tested under static monotonic load only and pile SH-2 was subjected to low-strain dynamic loading followed by a static cyclic loading.

3.2. Site investigation

The site was a man-made soil pit that is 9m (30ft) deep with approximate dimensions of 20x20m (66x66ft) at the ground level and 9x9m (30x30ft) at the bottom of the pit. The pit was filled with an uncontrolled granular soil fill several years before the current testing program.

Two seismic cone penetration tests (SCPT) with low-strain shear wave velocity measurements were conducted at the site (see Figure 2 for SCPT locations) to aid in characterizing the soil stratigraphy and obtain soil stiffness and strength parameters. The SCPTs were advanced to 8m (27ft) below the ground surface.

The soil comprised a mix of sand, silt, and gravel with cobbles as large as 100mm (4”). Based on the cone tip resistance (q_t), sleeve friction (f_s), and pore water pressure (u) measurements from the SCPT sounding, the soil behavior type (SBT) can be interpreted (Robertson, 1990). While the SBT is not meant to substitute conventional soil classification and characterization laboratory techniques, it provides a good indication of the soil type based on a large database of different soils with correlated laboratory tests with field CPT tests.

Negligible pore water pressure measurements (almost zero) were observed from the SCPT soundings, which indicates a dry or moist, but not saturated, soil. It was also found that the values of sleeve friction, f_s, were very high relative to q_t (i.e. high R_f ratio) indicating either a high fines content or large horizontal stresses. The latter is reasonable since the site was originally a pit that was back filled with uncontrolled granular soil fill which would cause high horizontal stresses due to soil placement.

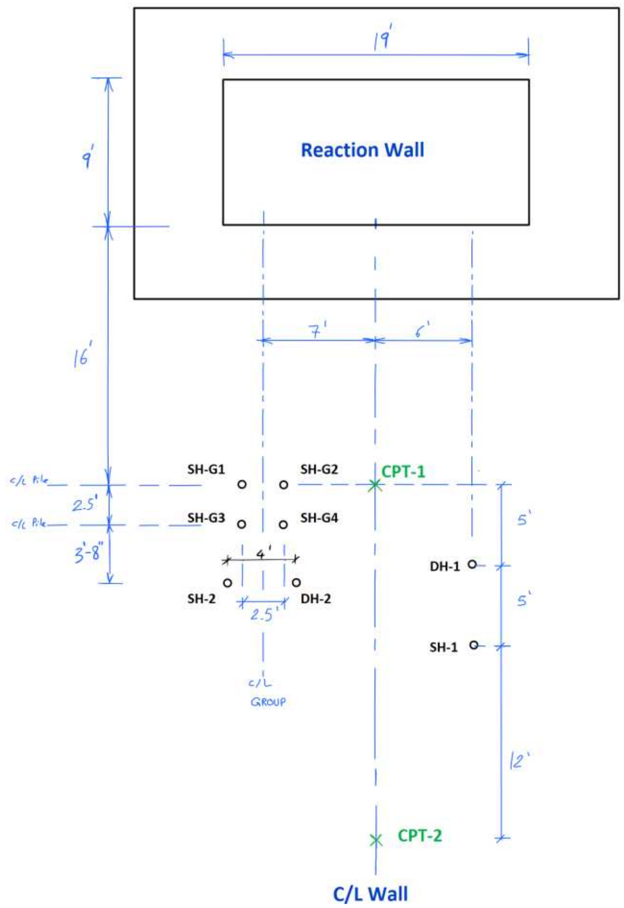

Figure 2. Pile layout and SCPT sounding locations (plan view)

The low-strain shear wave velocity (Vs) measured during the SCPT was found to be extremely high at the top 1.5m at SCPT-1 (i.e. around 900m/s on average), but the average shear wave velocity 1.5m below ground surface was 300m/s (1000ft/s). However, the accuracy of the measured Vs values is usually not reliable in the top 1 to 1.5m. On the other hand, the measured Vs profile at SCPT-2 displayed a gradual increase in with depth with an average of 300m/s (1000ft/s), as shown in

Figure .

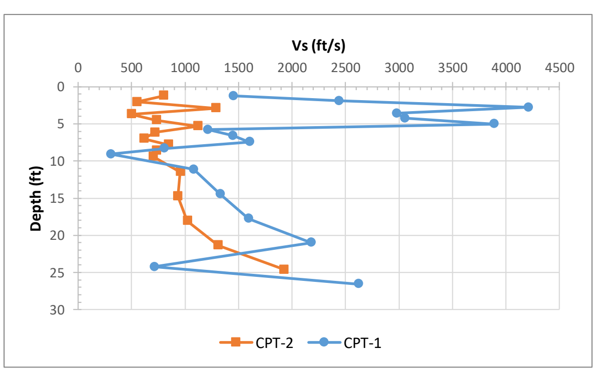

Figure 3. Shear wave velocity variation with depth

3.3. Pile installation torque

The installation torque of a helical pile provides a good indication of the soil strength and can be used to provide a good estimate of the pile axial capacity through empirical correlations.

The installation torque was recorded at 0.3m (1ft) increments. Both piles, SH-1 and SH-2, had similar installation torque profiles, which indicates similar soil strength properties at both locations.

3.4. Soil plug

Both piles, SH-1 and SH-2, exhibited a soil plug formation at their tips and was measured from the ground surface to be about 2.9m deep (9.5ft), which indicate a plug length of about 0.75m (2.5ft). This plug length is about 7 shaft diameters (i.e. 7d).

3.5. Instrumentation

Each pile was instrumented with two opposite quarter-bridge strain gages, one on each side of the pipe extremes, at different elevations. The strain gages were installed inside the pipe shaft after pile installation and were distributed along the top portion of the shaft. The strain gages are capable of measuring strains up to 5%.

The pile head displacement and acceleration, and corresponding rotation were measured using two linear string potentiometers, and two unidirectional accelerometers situated along the load application direction. The top string potentiometer and accelerometer are located at the elevation of point of load application, while the bottom one was located 76mm (3in) below that elevation, as shown in Figure 4. The bottom string potentiometer and accelerometer, along with the top ones, were used to approximate the pile head slope at each load increment.

The static monotonic and cyclic loads were applied using two single acting hollow hydraulic jacks, 60ton each and controlled by an electric hydraulic pump as shown in Figure 4. The load was measured using two 90ton (200kips) hollow load cells with full-bridge strain gages.

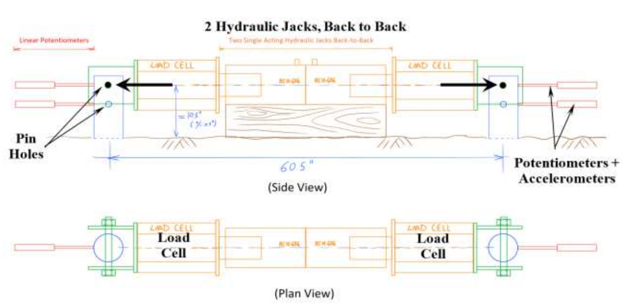

Figure 4. Static test setup for SH-1

The dynamic load was applied using a 320kg (710lb) eccentric mass shaker that is capable of rotating at speeds up to 8Hz. The shaker consisted of two counter rotating trays that can be filled with additional masses to provide a wide range of dynamic forces. The trays were configured to provide a unidirectional dynamic force. The shaker can provide a maximum dynamic load equal to 22.5kN (5000lbs) at 8Hz when the trays are empty. The shaker was controlled by a variable frequency drive (VFD). Figure 5 provides a schematic of the dynamic loading setup.

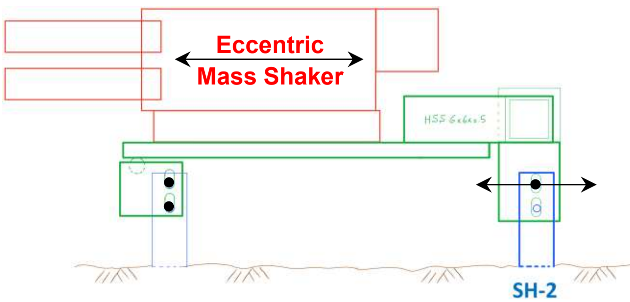

Figure 5. Dynamic test setup for SH-2

3.6. Test setup

As shown in Figure 4, the static monotonic loading was applied to SH-1 at its pile head in a free-head condition. The load was applied through a load jack while reacting on another pile situated at 15 shaft diameters (i.e. 15d) from SH-1. The load was applied at about 270mm (10.5in) above the ground surface with overall pile projection equal to about 300mm (12in). The test setup and loading procedure were in accordance with ASTM D3966-07 (ASTM, 2007).

To conduct the dynamic loading for SH-2, the eccentric mass shaker was attached to a 38mm thick (1.5in) steel plate. The plate was attached (pinned) to SH-2 and DH-2 at the front while supported vertically on a roller supports that were attached to piles SH-G3 and SH-G4 (see Figure 2) at the back such that any lateral loads would be transferred to SH-2 and DH-2 only.

The static cyclic test setup for SH-2 was identical to the monotonic test except that two hydraulic jacks situated against opposite sides of the steel plate were used to apply two-way cyclic loading such that when one jack was pressurized the other jack had zero pressure.

4. RESULTS

4.1. Static monotonic test

Pile SH-1 was loaded until the linear displacement potentiometers reached full extension. The load-displacement curve at the point of load application (i.e. 270mm above ground surface) is shown in Figure . The pile appears to behave in a linear manner up to displacement equal to 20mm (0.75in) with a corresponding lateral load equal to 37kN. The load-displacement relationship then becomes nonlinear with no apparent failure up displacement equal to 50mm (2in) and a corresponding load equal to 67kN. Along the nonlinear region, the pile shaft cross-section started yielding which was indicated by the strain gages readings. The pile appears to reach its ultimate lateral strength at a displacement equal to 75mm (3in), which is about 0.65 shaft diameter (0.65d), and a corresponding load equal to 78kN.

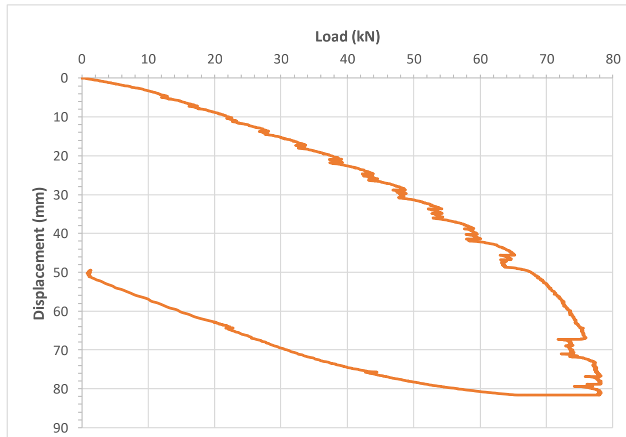

Figure 6. Static load-displacement curve for SH-1

4.2. Dynamic test

The amplitude of the dynamic force generated by the eccentric mass shaker is proportional to the rotating speed through the following relation:

$$F = m \times e \times \omega^2 \qquad [1]$$

where m is the mass of the empty rotating trays, e is the distance from the empty trays center of gravity to the center of rotation, and ω is the rotational speed in rad/s.

The maximum dynamic force (F) produced by the shaker at frequency equal to 8Hz (50.3rad/s) is equal to 22.5kN (5000lb). Accordingly, the mass unbalance of the shaker can be obtained by rearranging Eq. 1, i.e.:

$$m \times e = \frac{F}{\omega^2} = 8.905 \text{ kg.m}$$

The shaker was operated up to a speed of only 310 rpm (i.e. frequency of 5Hz) such that the generated force (F = 4.7kN per SH-2) was low enough to be considered low-strain and the soil around the pile was considered to behave in a linear fashion. This ensured that the following cyclic test results was not affected by any nonlinearity during the dynamic load testing. The displacement time history of SH-1 along with the mean displacement are shown in Figure .

As shown in Figure 7, the response along the initial loading direction was higher than the response along the opposite direction. Consequently, the mean displacement was not zero especially at speeds ≥ 2.5Hz. Moreover, it can be shown that with increasing number of cycles within the same frequency and as the load amplitude increased, especially at frequencies greater than 4.2Hz, the pile response and its stiffness changed.

It is believed that the changes in stiffness result from changes in the soil gapping depth around the pile rather than changes in the soil stiffness. The reason is that when the dynamic load was terminated, the pile returned to its original position with a negligible permanent deformation.

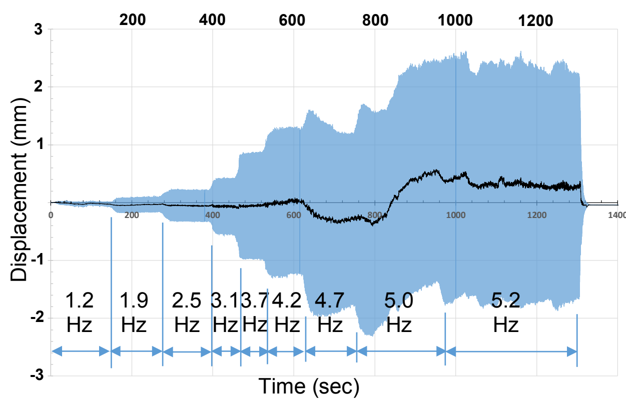

Figure 7. Displacement time history and mean displacement for SH-2

The average steady-state displacement response at different exciting frequency is shown in Figure . The average displacement gradually increased as the speed (and load amplitude) increased.

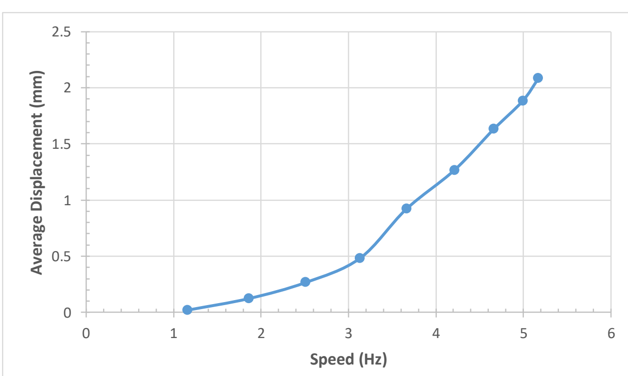

Figure 8. Average steady-state displacement response at corresponding speeds

4.3. Static cyclic test

Five cyclic load increments were applied as shown in Table 1. The test terminated when the maximum hydraulic jack stroke was reached. The cyclic load-displacement curve is shown in Figure .

Table 1. Cyclic load test increments and number of cycles

Load increment (kN)	Cycles
12.5	15
25	15
37.5	15
50	5
75	3

The cyclic loading resulted in an overall reduction in pile stiffness. It was also noted that the pile was stiffer along one loading direction than the opposite loading direction due to the formation of gaps as well as soil plasticity. The reason is that when the pile was unloaded, a significant permanent displacement existed. It was observed that the envelope of the cyclic load-displacement curves (i.e. backbone curve) had the same shape as the static response curve, but with an overall reduced stiffness, which is similar to the observations made by El Sharnouby (2012).

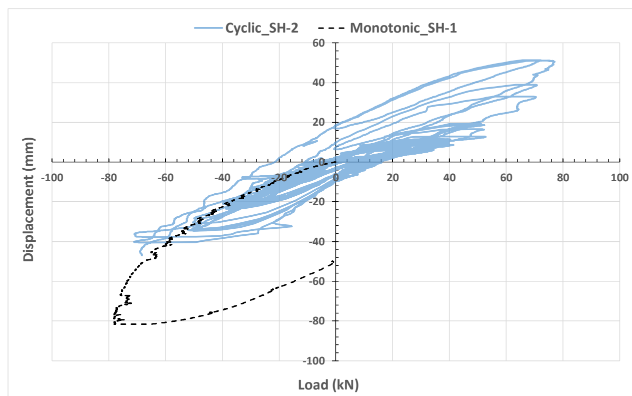

Figure 9. Cyclic load-displacement curve for SH-2 versus monotonic load-displacement curve for SH-1

5. CONCLUSIONS

Full-scale lateral monotonic and cyclic load tests as well as dynamic load test were carried out on two single 4m long helical piles with a single helix that were installed in a sandy soil profile. The load-displacement curves were measured for the static load tests and the steady state response curve was obtained under different dynamic excitation frequencies up to 5Hz. The cyclic load-displacement curves showed degradation in stiffness with increasing number of load cycles and increasing load amplitude compared to the monotonic static load-displacement curve.

The dynamic displacement response time history showed changes in stiffness as the number of cycles increased. In addition, the stiffness changed based on the load direction; the stiffness along the initial loading direction was less than the stiffness along the opposite direction due to the formation of different size gaps on both sides. However, the pile returned to its original position after the dynamic loads were terminated indicating an elastic behavior for the pile-soil system.

6. ACKNOWLEDGMENT

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