# Lateral Resistance of High Capacity Helical Piles – Case Study

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ABSTRACT
Although there are several methods available for estimating the axial capacities of helical piles, there is little literature available on their lateral performance. The new generation of high capacity helical piles necessitates exploring their lateral behaviour. The results of a lateral pile load test program and the field monitoring of helical piles installed in dense sand or very hard clay till are presented in this paper. Soil stratifications and ground water conditions are also summarized. The effect caused by different installation methods is also highlighted. The results of the load tests are compared to a theoretical model using L-Pile Plus 5. Based on the results of this study it was found that helical piles can develop considerable resistance to lateral loads and this resistance is almost exclusively controlled by the shaft diameter.

RÉSUMÉ
Bien qu'il y a plusieurs méthodes disponibles pour estimer les capacités axiales de tas hélicoïdaux, il y a de petite littérature disponible sur leur exécution latérale. Les nouvelles générations d'haute capacité tas hélicoïdaux nécessitent explorer leur comportement latéral. Les résultats d'un programme de test de chargement de tas latéral et de champ contrôlant de tas hélicoïdaux installés dans le sable dense ou l'argile très dure jusqu'à sont présentés dans ce papier. Salir des stratifications et des conditions d'eau souterraine sont aussi résumées. L'effet de méthode d'installation est aussi souligné. Les résultats des tests de chargement sont comparés à un L-TAS d'utilisation de modèle théorique 5. Fondé sur les résultats de cette étude il a été trouvé que les tas hélicoïdaux peuvent développer la résistance considérable aux chargements latéraux et cette résistance exclusivement est presque contrôlée par le diamètre d'arbre.

## 1 INTRODUCTION

There are several sources that contribute to horizontal (or lateral) loads and moments to piles, such as: wind loading, earthquakes, unbalanced earth pressures, axial thrust on pipelines, and load eccentricity. Therefore deep foundations are frequently designed to resist such loads.

Over the past few years significant advances have been made on the installation and increasing axial capacities of helical piles. Helical piles with axial capacities in excess of 3 MN are now in use. However, very little information is available on their lateral behaviour since helical piles were historically used either as anchors to resist uplift loads or as a foundation for residential housing to resist small compressive loads and their shafts were either square or rounded with small diameters between 45 mm and 114 mm. The availability of high torque rotary heads has facilitated the installation of large diameter helical piles into competent soils such as very dense sand or very hard clay till. Helical piles with shaft diameters up to 508 mm have been successfully installed into hard soils. With these relatively large diameter helical piles, their lateral resistances have become a considerable component of their overall capacity.

The objectives of the present study were to evaluate the lateral resistance of high capacity helical piles installed into either dense to very dense sand, or very hard clay till soils as well as compare between the measured and estimated lateral resistances of helical piles using p-y curves. In order to achieve these objectives, seven full- scale lateral load tests were carried out using helical piles with different shaft diameters that varied between 324 mm and 508 mm. Details of pile configuration, testing set up and load test results are provided in the following sections.

## 2 SITE DESCRIPTION

The testing site is located at about 70 km north of Fort McKay, in northern Alberta, Canada. Four different locations across the site were selected for testing and they are referred to as Sites 1 to 4. Sites 1 and 2 represent sandy soils (i.e. cohesionless soils) while Sites 3 and 4 represent clay till soil (i.e. cohesive soils). Soil stratification and parameters at each site are summarized in Table 1.

### 2.1 Subsurface Soil Conditions

Soil stratigraphy at Site 1 consists of sand layers that extend to the end of test hole at depth of about 18 m. The sand layer extended between ground surface and 2 m below existing ground was poorly graded, gravelly, brown, moist and medium dense. The sand between the depths of 2.5 m and 10 m was fine grained to silty, well graded and dense to very dense. The sand layer that was encountered below a depth of 10 m was fine grained, well graded, wet and very dense. The upper sand layer that extended to depth of about 10 m was compact to dense while the lower sand layer was very dense. Standard Penetration Test (SPT) blow counts varied between 23 to 37 blows per 300 mm of penetration for the upper sand zone while SPT blow counts for the lower sand layer varied between 41 and 63 blows per 300 mm of penetration. Ground water level at the test hole location measured upon completion of the test hole, was relatively shallow and was about 5.4 m below existing ground surface.

The soil stratigraphy at Site 2 consisted of surfacial sandy silt to a depth of about 2.5 m over glacial till to a depth of about 3.7 m over dense to very dense sand that extended to a depth of about 13.1 m below existing grade. The sand was cemented between the depths of 4.5 m and 7 m. Ground water level at Site 2 was estimated from pore pressure dissipation tests at a depth of about 7.1 m below existing ground surface.

The soil stratigraphy at Site 3 consisted of clay till that extended to a depth of about 6.4 m underlain by very dense sand that extended to the end of the test hole at a depth of about 30.6 m. The clay till was sandy with some silts and contained traces of fine to coarse subrounded gravel up to 50 mm in size. A fine to coarse subrounded gravel lense was encountered at a depth of about 2.2 m and extended to about 2.5 m. A seam of black woody debris was also encountered at a depth of about 2.5 m. SPT blow counts varied between 17 and 64 blows per 300 mm of penetration indicating very stiff to very hard consistency. It should be noted that a very hard soil layer was encountered during drilling at the interface between clay till and sand layers at depths of about 6.0 m to 6.4 m. Ground water level at the test hole location was relatively shallow and was measured upon completion at about 5.2 m below existing ground surface. It should be noted that groundwater levels were taken immediately upon completion of the test log, and therefore may not represent the equilibrium conditions.

The soil stratigraphy at Site 4 consists of surfacial sandy silt to a depth of about 1.7 m over glacial till layers to depth of about 13.9 m below existing ground level. Groundwater was estimated to be at level of about 14.5 m below existing ground surface.

Table 1. Summary of soil properties

| Depth m | Soil description | SPT blow count per 300 mm, | Total unit weight, kN/m$^3$ | Undrained Shear Strength, kPa | Frictional resistance angle, φ (°) |
|---|---|---|---|---|---|
| **Site 1** | | | | | |
| 0 – 10 | Sand, compact to dense | 23-37 | 18 | - | 36 |
| 10 – 18 | Sand, dense to very dense | 41-63 | 20 | - | 40 |
| **Site 2** | | | | | |
| 0 – 2.5 | Sand, dense | NA | 19 | - | 33 |
| 2.5 – 3.7 | Glacial Till, stiff | NA | 18 | 55 | - |
| 3.7 – 4.3 | Sand, dense | NA | 19 | - | 32 |
| 4.3 – 13.1 | Sand, very dense | NA | 19 | - | 36 |
| **Site 3** | | | | | |
| 0 - 2 | Clay Till, very stiff | 17 | 18 | 100 | 0 |
| 2 - 4 | Clay Till, hard | 32 | 20 | 200 | 0 |
| 4 – 6.5 | Clay till, very hard | 64 | 20 | 400 | 0 |
| > 6.5 | Sand, dense to very dense | 42-60 | 20 | 0 | 40 |
| **Site 4** | | | | | |
| 0 – 1.7 | Sand, compact | NA | 18.5 | - | 31 |
| 1.7 – 9.9 | Glacial Till, stiff | NA | 18 | 85 | 0 |
| 9.9 – 13.9 | Glacial till, very stiff | NA | 18 | 115 | 0 |

## 3 TEST PILE CONFIGURATION

The configurations of the different piles considered for the helical pile load test program are summarized in Table 2. Figure 1 provide a typical helical pile configuration. Helical piles types identified by even numbers were for piles with double helixes (i.e. type 6, 6A and 8) while piles identified with odd numbers were for piles with a single helix. All piles were rounded-shaft type with different shaft diameters that varied between 324 mm and 508 mm and helix diameters that varied between 762 mm and 1016 mm. Helixes for piles type 6 and 6A were spaced at 3 times their helix diameter, while helixes for piles type 8 were spaced at 2 times their helix diameter. Steel pipes were ASTM A232 Grade 3 steel with a yield strength of 310 MPa. The helical piles tested in this program were manufactured by ALMITA Manufacturing Ltd of Ponoka, Alberta.

Table 2. Summary of pile configurations

| Pile Type | Shaft | | Helixes | | |
|---|---|---|---|---|---|
| | Dia mm | Thickness mm | Dia mm | thickness mm | No of Helixes |
| 3 | 324 | 9.5 | 762 | 25.4 | 1 |
| 3A | 324 | 9.5 | 610 | 25.4 | 1 |
| 5 | 406 | 9.5 | 914 | 25.4 | 1 |
| 6 | 406 | 9.5 | 914 | 25.4 | 2 |
| 6A | 406 | 9.5 | 762 | 25.4 | 2 |
| 7 | 508 | 9.5 | 1016 | 25.4 | 1 |
| 8 | 406 | 12.7 | 813 | 25.4 | 2 |

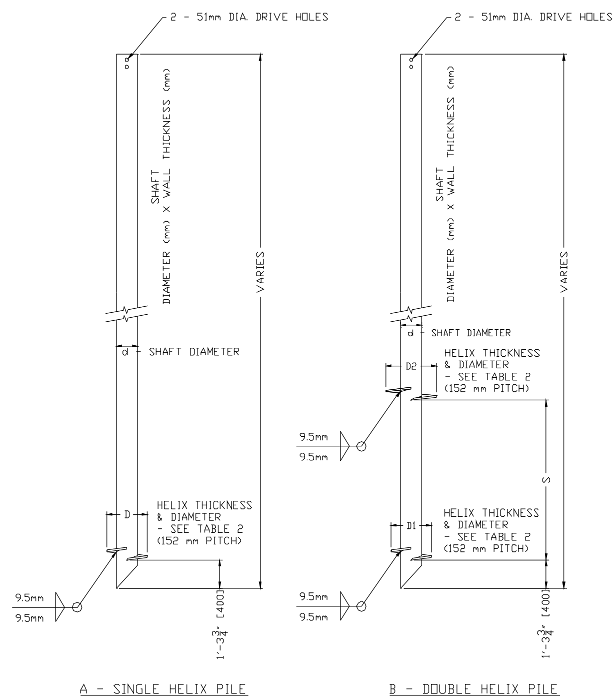


Figure 1. Typical helical pile configuration

## 4 INSTALLATION MONITORING

The pile installation equipment comprised a drive unit mounted on a tracked Excavator. The drive unit contained a hydraulic motor that provided the torque for rotation of the screw pile into the ground to a maximum torque of 250,000 ft.lbs (339 m.kN). Field monitoring of pile installations including the torque recorded at the end of pile installation and depth of embedment are summarized in Table 3. Figure 2 shows a typical helical pile installation.

Site 1 was heated prior to pile load tests since load tests were performed during winter and the upper soil zone was frozen. The ground surface was heated using glycol heating coils under insulated tarps and was then covered with about 0.6 m of sand fill to protect the native ground from freezing prior to testing. Glycol heating coils were placed on the top of the fill at testing area and were covered with insulated tarps and heating the ground continued. Site 1 was then stripped after installing the piles and the site was covered with insulated tarps. Two pile installations were carried out at Site 1 including piles ST16 and ST17. Both piles were drilled with an auger to depths of about 5.5 m to 5.4 m. It can be seen from Table 3 that the measured torque values at the end of installation were 338.3 kN.m for pile ST16 and 273.9 kN.m for pile ST17. Total embedment depth for pile ST16 was 6.5 m. Installation was terminated for pile ST17 with shaft diameter of 324 mm at total embedment depth of 5.6 m due to reaching its maximum rated torque capacity.

At Site 2, pile ST23 was installed without predrilling since the pile was installed during the fall (October 2009). The measured torque values during installation for pile ST23 was increased with depth till the end of pile installation. The measured torque at end of installation for pile ST23 was about 306 kN.m and the corresponding embedment depth was 5.3 m.


Figure 2. Typical helical pile installation

**Table 3** Summary of pile installation

| Site Location | Test ID | Pile Type | Shaft Diameter mm | Installation Torque at end of installation kN.m | Embedment Depth m | Soil Plug Thickness m | Predrill Depth m |
|---|---|---|---|---|---|---|---|
| Site1 | ST16 | 6 | 406 | 338.3 | 6.5 | 4.7 | 5.5 |
| | ST17 | 3 | 324 | 273.9 | 5.6 | - | 5.4 |
| Site 2 | ST23 | 8 | 406 | 306.0 | 5.3 | 2.8 | NA |
| Site 3 | ST10 | 3 | 324 | 211.5 | 5.1 | 2.8 | 3.1 |
| | ST18 | 5 | 406 | 338.3 | 5.4 | 1.9 | No |
| | ST19 | 7 | 508 | 338.3 | 5.1 | 2.8 | 4.0 |
| Site 4 | ST43 | 8 | 406 | 338.0 | 13.6 | 2.6 | NA |

At Site 3, a total of three installations were performed for piles ST10, ST18 and ST19. Measured torque at the end of installation for pile ST10 (Type 3) with a shaft diameter of 324 mm was 211 kN.m and the corresponding embedment depth was about 5.1 m. Pile ST18 was advanced into the subsurface to a total embedment depth of about 5.4 m. Pile ST19 was advanced into a predrilled hole about 4 m deep installed using an augur 406 mm in diameter. The total embedment depth for pile ST19 was 5.1 m.

At Site 4, pile ST43 was installed without predrilling. The measured torque at the end of installation for pile ST43 was 338 kN.m and the corresponding embedment depth was 13.6 m.

Figure 3. Typical lateral load test setup

## 5 TEST SETUP

An oblique view of a typical lateral load test setup is shown in Figure 3. The lateral load test setup consisted of a test pile that was installed at about 3 m away from a reaction pile. The reaction pile, typically a helical pile with shaft diameter similar to the test pile was installed to a minimum depth of about 5.2 m below ground surface. Loads were applied at a distance of about 200 mm above ground level using a 200 ton hydraulic jack. The hydraulic ram acted directly against a steel strut placed between the base of hydraulic cylinder and the reaction pile. Lateral movements were monitored at three points along the pile's free length (at distances of 200 mm and 500 mm above ground surface) to measure lateral deflections at point of load application and to assess the rotation at the pile head. The lateral movement were measured using two LDTs with 0.01 mm accuracy and 150 mm travel and a dial gauge with 0.025 mm subdivisions, and 50 mm travel.

The lateral pile load tests were conducted in general accordance with ASTM D3966-07, Standard Method of Testing Deep Foundations under Lateral Loads. All LDT readings were recorded automatically at the same time increments (30 seconds) throughout the test duration.

## 6 TEST RESULTS

### 6.1 Site 1 – Cohesionless Soil

The results of lateral load tests for piles ST16 and ST17 carried out at Site 1 are presented in the form of load deflection curves in Figure 4. It can be seen from Figure 4 that the lateral responses of both piles were nonlinear. Gaps were formed behind the piles during testing indicating a plastic deformation of the soil in front of the pile within the upper soil layers. Pile ST16 (Type 6A) with a shaft diameter of 406 mm and total embedment depth of 6.5 m, and pile ST17 (Type 3A) with shaft diameter of 324 mm and total embedment depth of 5.6 m, were loaded to maximum loads of about 240 and 225 kN, respectively which corresponded to maximum deflections of about 78 mm and 80 mm, respectively. When piles rebounded to zero load, the net or permanent displacements were about 7 mm and 14 mm. Comparing between the response of both piles indicate that pile ST16 with a shaft diameter of 406 mm showed slightly higher lateral resistance than pile ST17 with shaft diameter of 324 mm. The lateral resistance of pile ST16 was about 4% to 19% higher than that of pile ST17. The slight increase of the lateral resistance of ST16 at low deflection levels despite its larger diameter, is likely due to effect of soil disturbance that was caused by the predrilling process.

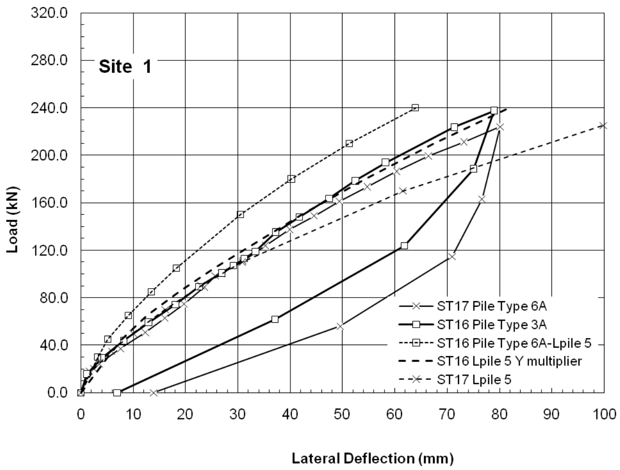


Figure 4. Lateral loads versus deflections for piles ST16 and ST17 at Site 1.

## 6.2 Site 2 – Cohesionless Soil

The results of lateral load test for pile ST23 carried out at Site 2 is presented in the form of a load deflection curve in Figure 5. It should be noted that pile ST23 was Type 8 with a shaft diameter of 406 mm and embedment depth of 5.3 m and was installed without predrlling. It can be seen from Figure 5 that the lateral response of pile ST23 was also nonlinear. An abrupt increase of the lateral resistance was observed at a deflection level of about 9 mm. This behavior suggests that a soft zone around the pile may be present as a result of installation process. Gaps were also formed behind the piles during testing indicating a plastic deformation of the soil in front of the pile within the upper soil layers. Pile ST23 was loaded to maximum load of about 300 kN which corresponded to maximum deflection of about 48 mm. When the load rebounded to zero, a net or permanent deflection of about 6 mm was observed.

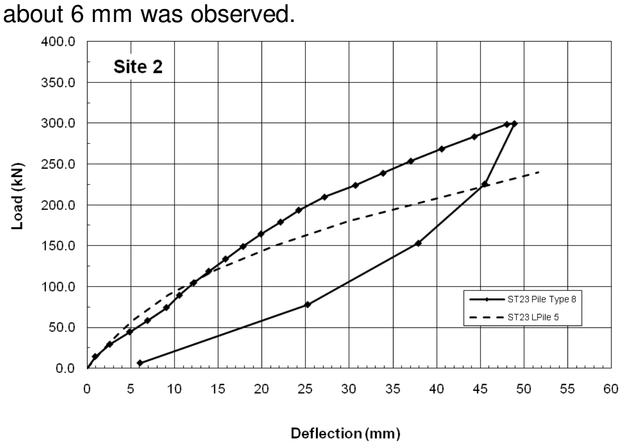


Figure 5. Lateral loads versus deflections for pile ST23 at Site 2.

## 6.3 Site 3 – Cohesive Soil

The results of lateral load tests for piles ST10, ST18 and ST19 carried out at Site 3 are presented in the form of load deflection curves in Figure 6. Piles ST10 (Type 3), ST18 (Type 5) and ST19 (Type 7) were loaded to maximum loads of about 101 kN, 389 kN and 409 kN, respectively which corresponded to maximum deflections of about 39 mm, 78 mm and 79 mm, respectively. When piles rebounded to zero load, the net or permanent displacement was about 8 mm and 24 mm, for piles ST10 and ST18 respectively. It should be noted that the predrilling process was used for installing piles ST10 and ST19. However, pile ST18 was installed without predrilling. Comparing between the response of piles ST10, ST18 and ST19 indicate that pile ST18 with a shaft diameter of 406 mm showed significantly higher lateral resistance than pile ST10 with a shaft diameter of 324 mm. The lateral resistance of pile ST18 was about 300% higher than that of pile ST10. The significant decrease of the lateral resistance of ST10 is likely due to effect of soil disturbance caused by the predrilling process during pile installation.

Comparing between piles ST18, (Type 5) with shaft diameter of 406 mm and ST19, with shaft diameter of 508 mm, indicate that both piles showed similar lateral response despite the larger shaft diameter of pile ST19. The lower resistance of pile ST19 compared to ST18 is likely due to the use of the predrilling process for pile ST19, since installation was with an augur size of 406 mm (smaller than the size of the shaft). Comparing between lateral resistance of piles ST10 and ST19 indicate that using a smaller augur size had reduced the effect of soil disturbance on the lateral resistance. It can be also noted for both piles ST10 and ST19 that the early portion of the load deflection curves were softer. This observation suggests that during the early stages of lateral loading the disturbed soil zone around the pile had considerably reduced the lateral resistance. However at higher deflection levels, the lateral loads were resisted by larger portions of the ground which include both disturbed and native soils and therefore the overall performance was improved at higher loads. The soil disturbance can be reduced by using an augur with smaller size than the shaft.

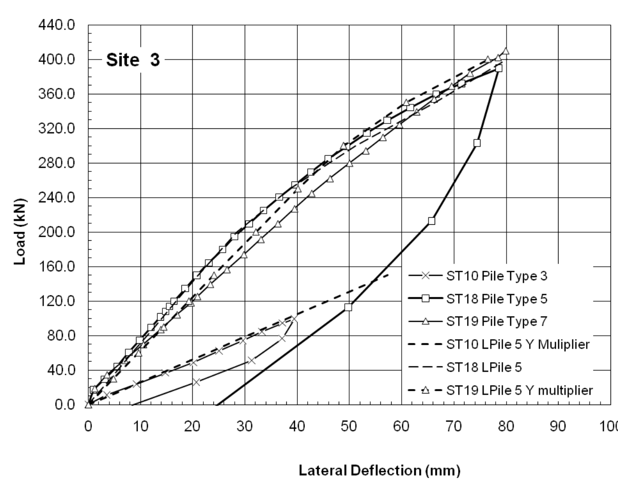


Figure 6. Lateral loads versus deflections for piles ST10, ST18 and ST19 at Site 3.

6.4 Site 4 – Cohesive Soil

The results of the lateral load test for pile ST43, carried out at Site 4, are presented in Figure 7. It should be noted that pile ST43 was installed without the predrilling process. It can be seen from Figure 7 that the lateral response of pile ST43 was nonlinear. The lateral loads at deflection levels of 6 mm, 12 mm, were 88 kN and 135 kN, respectively. Pile ST43 was loaded to maximum load of about 300 kN which corresponded to a maximum deflection of about 55 mm. When pile ST43 rebounded to zero load, the net or permanent displacement was about 18 mm. It should be mentioned that a permanent displacement of 18 mm is an indication of reaching the upper zone of the soil's plastic zone since pile was laterally loaded to excessive loads (about 3.4 times load at displacement level of 6 mm).

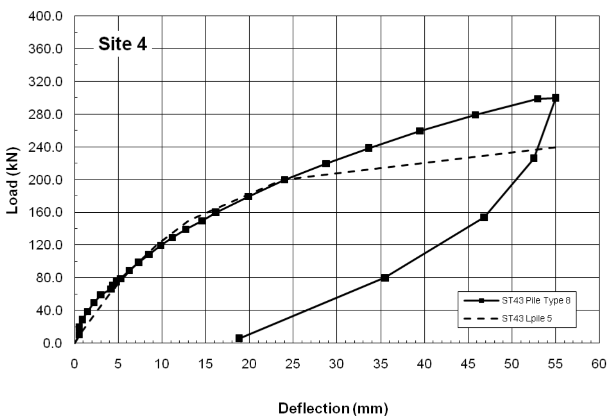


Figure 7. Lateral loads versus deflections for pile ST43 at Site 4.

6.5 Evaluating Installation Effects

In order to evaluate the effects of predrilling a pilot hole prior to pile installation, the lateral load test results of pile ST17, with shaft diameter of 406 mm and embedment depth of 5.6 m, was compared to pile ST23, with similar shaft size and embedment depth of 5.3 m, installed using standard helical pile installation and the results are plotted in Figure 8. It can be seen from Figure 8, that pile ST23, installed without predrilling resisted at all load levels almost twice the loads of pile ST17 where the predrilling process was used. Therefore the effect of predrilling is unfavourable on the lateral capacity of piles and therefore, it is suggested to avoid the predrilling process, whenever it is possible, to obtain the full lateral resistance of helical piles.

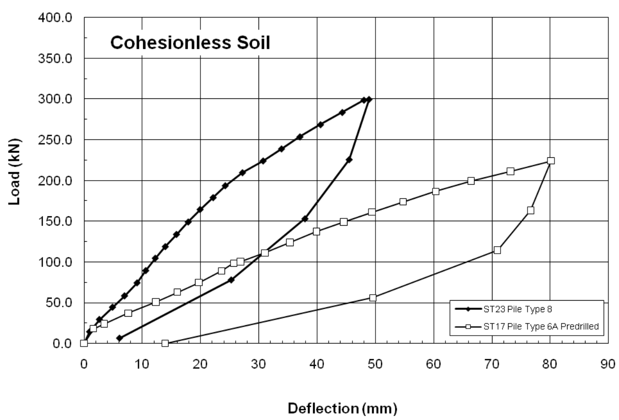


Figure 8. Effect of installation process on the lateral resistance - cohesionless soils.

6.6 Evaluating Creep Effects

In order to evaluate the creep effect on the pile load test results, secondary deflection versus time at load increments of 35 kN (low load level) and 75 kN (high load) were plotted for pile ST10 (tested at Site 1 in cohesionless soils) and the results are shown in Figure 9. It can be seen from Figure 8 that at both load levels, about 90% of the secondary deflection was obtained within 5 minutes. After five minutes the creep rate was almost steady at a rate of about 3 mm/hour. The finial secondary deflection at the end of the load increments of 35 kN and 75 kN was about 4.5 mm and 5.3 mm, respectively. This observation supports that most of the deflection at any load increment for dense to very dense sand soils is likely to occur in the first five-minute period and therefore the creep effect is minor. Therefore, the period of sustained loads after five minutes has a very small effect on the load-deflection characteristics.

Secondary deflection versus time at load increments of 35 kN (low load level) and 225 kN (high load) were plotted for pile ST17 (tested at Site 3 in cohesive soils) and the results are shown in Figure 10. It can be seen from Figure 9 that at both load levels, about 80% of the secondary deflection was obtained within 5 minutes at a load level of 35 kN. After five minutes, pile ST17 continued to creep at a rate of about 2 mm/hour. The final secondary deflection at the end of the load increments of 35 kN was about 2.8 mm. At the high load level of 225 kN, about 62% of the secondary deflection was obtained within 5 minutes and pile continued to creep at an average rate of about 20 mm/hour. Therefore for cohesive soils, the creep effect is considerable and therefore full adherence to ASTM D3966-07, Standard Method of Testing Deep Foundations under Lateral Loads standard is imperative to improve the quality of lateral load test data.

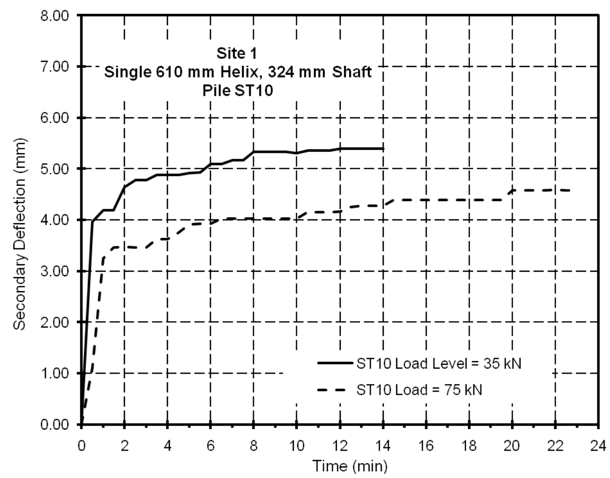


Figure 9. Incremental deflection versus time for pile ST10 at Site 1 – cohesionless soil.

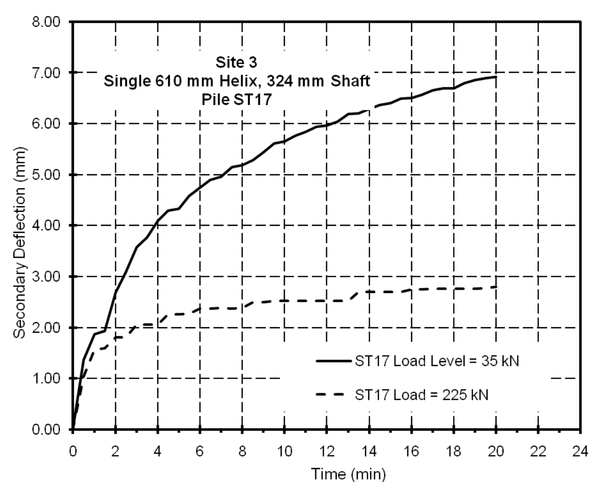


Figure 10. Incremental deflection versus time for pile ST17 at Site 3 - cohesive soil.

6.7 Lateral Pile Capacities

As identified in the Canadian Foundation Engineering Manual (2006), the lateral capacities of piles may be limited by the following factors: the capacity of soils, excessive bending that exceeds the structural capacity of the pile, or deflection of pile heads. For dense sand soil and hard clay till soil load test results are presented in this study, failure usually occurs by rotation of the pile within the upper soil zone. In this case, a large deflection is required to mobilize the passive resistance of the soil near the pile head and at the pile toe. It can be seen from Figures 4 and 6 that piles ST16, ST17, ST18, and ST19, were loaded to relatively high lateral loads and the corresponding lateral deflections were about 78 mm to 80 mm without sign of distinct failure. Therefore, the ultimate lateral load of piles may be specified to satisfy a limiting lateral deflection criterion that meets the structural requirements for the superstructure. In most cases of sensitive structures like bridges and oil facilities, lateral deflection is limited to value of 6 mm. However, higher lateral deflections may be considered depending on the sensitivity of the structure to lateral movements and type of loading. The lateral loads at deflection levels of 6 mm, 12 mm and 25 mm are presented in Table 4. It can be seen from Table 4 that all piles continued to resist higher loads at high deflection levels. The lateral resistance of piles increased with increasing the diameter of the pile shaft. As mentioned earlier, piles installed using a predrilling process offered considerably lower lateral resistance compared to piles installed without predrilling. For example, for piles ST16 (installed using a predrilling process) and ST23 (without predrilling) with the same shaft diameter of 406 mm and similar embedment depths into comparable soils, pile ST16 offered lateral resistance of about 61 kN while pile ST23 offered lateral resistance of 104 kN at same deflection level of 12 mm.

**Table 4** Summary of lateral load results

| Site ID | Pile ID | Shaft Diameter mm | Lateral load @ deflection | | |
|---|---|---|---|---|---|
| | | | 6 mm | 12 mm | 25 mm |
| Site 1 | ST16 | 406 | 43.5 | 61.1 | 103.0 |
| | ST17 | 324 | 36.7 | 51.7 | 99.1 |
| Site 2 | ST23 | 406 | 50 | 104 | 195 |
| | ST10 | 324 | 22 | 30 | 63 |
| Site 3 | ST18 | 406 | 60 | 89 | 180 |
| | ST19 | 508 | 51 | 86 | 155 |
| Site 4 | ST43 | 406 | 88 | 136 | 205 |

## 7 COMPARISON BETWEEN MEASURED AND ESTIMATED LATERAL RESISTANCES

As the lateral load capacities of piles are critical, the analyses of laterally loaded piles should be carried out using the p-y curves, whereby the nonlinear strength-deformation characteristics are modeled by load-displacement curves developed for the various soil layers. The problem is a soil-structure interaction since soil reaction is dependent upon pile movement and the pile movement is also dependent upon soil response. The problem is also highly nonlinear since the soil response is nonlinear especially at large deflection levels. With the method of p-y curves, the solution is obtained through iterative procedure performed using LPILE Plus 5 (ENSOFT Inc., 2005) software.

Lateral loading analyses for single piles considered in the present study have been completed using LPILE Plus 5 for static, sustained loading using the method of nonlinear p-y curves. Loads were applied at about 0.2 m above ground surface to simulate lateral load point of application for free head condition. The geometry and structural properties of two-section piles were modeled using the software. The load-displacement curves (i.e. p-y curves) are established using Reese's model for sand and stiff clay without free water with initial soil modulus, k for clay till soil.

### 7.1 Results of LPile Plus 5 Analyses at Sites 1 and 2 (Cohesionless Soils)

The results of LPILE Plus 5 analyses for piles ST16 and ST17 with shaft diameters of 406 mm and 324 mm tested at Site 1 are also presented in Figure 4. It should be noted that an attempt was made to estimate the lateral resistance of pile ST16 assuming no soil disturbance, however, the estimated values were about 15% higher than the measured lateral resistance. Therefore, y multiplier of 2.5 was used to calibrate the lateral resistance of pile ST16 against the measured values. The use of p-y multiplier allowed modelling the soil disturbance that occurred during pile installation due to the predrilling process. The drill augur that was used for predrilling had a diameter similar to the pile shaft, however the predrilled hole was about 50 mm larger than the size of the augur and therefore soil disturbance had occurred which affected the lateral resistance of helical piles tested at Site 1. The estimated lateral resistance of pile ST17 agreed well with the measured values especially at low deflection levels. It should be also mentioned that a slightly different installation procedure was used for pile ST17 where the predrilled hole was filled with sand tailings prior to pile installation. Therefore the soil disturbance due to predrilling process was minimized and y multiplier was not used for pile ST17. However at high deflection levels, LPile Plus 5 software underestimated the lateral resistance of different piles by about 10 to 12%.

The estimated lateral resistance of pile ST23 at Site 2 is also presented in Figure 5. It can be seen from Figure 5 that a reasonable estimate was obtained for the lateral resistance of pile ST23 especially at low displacement levels. It should be mentioned that pile ST23 was installed using a standard screw pile installation method without predrilling and therefore its lateral resistance were in reasonable agreement with the measured values.

### 7.2 Results of LPile Plus 5 Analyses at Sites 3 and 4 (Cohesive Soils)

The results of LPILE Plus 5 analyses for piles ST10, ST18 and ST19 with shaft diameters of 324 mm, 406 mm and 508 mm, tested at Site 3 are presented in Figure 6. It should be noted that an attempt was made to estimate the lateral resistance of piles ST10 and ST19 assuming no soil disturbance, however, the estimated values were about 80% to 100% higher than the measured lateral resistance for piles ST10 with relatively small shaft size and the estimated lateral resistance for pile ST19 with 508 mm shaft diameter was about 50% higher than the measured values. To quantify the effect of soil disturbance y multiplier factors were used to account for the soil disturbance by increasing the lateral deflections at the same load levels. A y multiplier of 3.5 was found to provide a reasonable estimate for lateral resistance for piles ST10 and y multiplier of 2.5 provided a reasonable estimate for pile ST19. It should be noted that the larger y multiplier is used for ST10 where the augur diameter was the same size as the size of the shaft and smaller y multiplier was used for ST19 where augur with smaller augur size than the shaft was used to install ST19. The estimated lateral resistances for piles ST10 and ST19 using y multiplier agreed reasonably with measured values.

An estimate for the lateral resistance of pile ST18 was performed using soil parameters from Table 1 and without using a y multiplier since ST18 was installed without predrilling. The estimated and measured lateral loads for pile ST18 agreed closely which confirm that the soil disturbance has a major effect on the installation of piles. It should be also mentioned that the effect of soil disturbance due to predilling process is more severe for cohesive soils encountered at Site 3 than that that for cohesionelss soils. In cohesionless soils, sand tends to slough into the predrilled hole and therefore reducing the size of the hole, however in cohesive materials, the predrilled holes typically stay open and therefore a possible gap around the pile shaft may form which will decrease the their lateral resistance especially at low deflection levels.

The estimated lateral resistance of pile ST43 tested at Site 4 is also presented in Figure 7. It can be seen from Figure 7 that a reasonable estimate was obtained for the lateral resistance of pile ST43 especially at low displacement levels.

## 8 CONCLUSIONS

The results of seven lateral pile load tests carried out using different pile configurations in either dense to very dense sands or very stiff to hard clay till are presented in this paper. The test results are summarized as follows:

1. Piles were installed successfully with different degrees of complexity depending on soil conditions and time of the year.

2. If predrilling process is used to install piles due to presence of very hard soil layers or frozen soils, caution should be taken to avoid using auger with larger diameter than shaft size. Auger size should be limited to at least 50 mm less than the size of pile shaft.

3. All piles offered a considerable lateral resistances despite of installation technique. Therefore helical piles can resist considerable lateral loads. Even larger loads can be resisted using battered piles. Therefore it is the author's opinion that helical piles are a viable option for deep foundations supporting lateral loads.

4. The predicted lateral resistance of different piles using LPILE Plus 5, agreed well with measured resistance especially at low deflection levels. Y multiplier can be used to assess the effect of poor installation or soil disturbance. Therefore LPILE may be used to estimate the lateral resistance of helical piles in absence of load test data with reasonable accuracy. However selection of soil parameters for LPile analysis is quite critical and therefore accurate estimate of soil parameters such as undrained shear strength for cohesive soils and frictional resistance angle for cohesionless soils is required.

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