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HIGH CAPACITY HELICAL PILES – A NEW DIMENSION FOR BRIDGE FOUNDATIONS

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ABSTRACT

Helical piles provide several construction and performance advantages over conventional concrete and steel piles such as: high compressive and uplift capacities, the speed of their installation, suitability for construction in limited access conditions, installation in frozen or swampy soil conditions and their cost effectiveness. In addition to that, helical pile installation is a free-vibration process. Therefore installation of helical piles for bridge foundations in urban areas is very favourable in term of reducing the level of noise and minimizing vibration to nearby structures. For bridge foundations applications, it is necessary to qualify and quantify high capacity helical piles performance. This paper presents the first full-scale axial compression and tension (uplift) testing program executed on large capacity helical piles. A total of thirteen tests were carried out, including nine axial compression tests and four tension (uplift) tests. The test setup and procedures are described. The results of the axial compressive and tensile pile load tests and field monitoring of helical piles with either single helix or double helixes installed in either dense sand or very dense to very hard clay till soils are presented in this paper. The data presented is therefore considered valuable to other researchers and engineers considering the use of helical piles to support short to mid span bridges. Based on the results of this study it was found that helical piles can develop significant resistance to axial compressive loads up to 2500 kN and tensile loads up to 2000 kN.

1. INTRODUCTION

A helical (screw) pile, installed by applying torque at the pile head, is an old type of foundation that was widely used in many countries before the advent of reinforced concrete piles (Kurian and Shah 2009). However, its application was limited to soft soil conditions since its installation was mainly dependant on human power. The increasing popularity of the helical piling system in the recent years is attributed to a combination of development of powerful hydraulic heads and viable advantages of helical piles compared to conventional deep foundation systems. Helical piles are easily installed using relatively small equipment, can be removed and reused and they allow immediate loading upon installation. Likewise, in the case of high ground water level, helical piles save dewatering and/or pumping of the construction site (Bobbitt and Clemence 1987).

Short and medium span bridges are frequently supported by group of steel driven piles. The main advantage of using helical piles to support bridge structures is the speed of installation, cost effectiveness, and their high resistance to compression and uplift loads. In order to exploit the viable advantages of helical piles in bridge projects, the paper summarizes the results of helical piles installed in both dense sand and very stiff to very hard clay till soils. The current paper summarizes geotechnical conditions, testing details, and results. Full scale helical pile load testing program included both axial compression and tension (uplift) tests.

2. SUBSURFACE CONDITIONS

The testing site is located in northern Alberta, Canada. The testing site is a large site with variable soil conditions. Two test locations were selected to represent the sand material (i.e. cohesionless material) and clay till (i.e. cohesive material). The ground surface at the test locations were flat lying. The pile load test location where subsurface soils are mainly sandy is referred to as Site 1, while the location where soil layers are clay till materials is referred to as Site 2.

2.1 Site 1

Soil stratigraphy at test Site 1 consists of surfacial silt, sand and clay till layers that extended to a depth that varied between about 1.1 m and 1.4 m underlain by a medium dense to very dense sand layer that extended to a depth of about 33 m. Soil properties are summarized in Table 1. Sand layer extended to depth of about 11.4 m was medium dense to dense while sand below that depth was dense to very dense. Sand was medium grained and contained lenses of silt. Standard Penetration test (SPT) blow counts varied between 14 to 58 blows per 300 mm of penetration for the upper sand zone indicating a medium to very dense sand state while SPT blow counts for the lower portion of the sand layer varied between 45 and 92 blows per 300 mm of penetration indicating a dense to very dense state. Ground water level at the test hole location was about 3.6 m below existing ground surface.

2.2 Site 2

Soil stratigraphy at test Site 2 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 depth of about 30.6 m. Clay till was sandy with some silts and contained traces of fine to course subrounded gravel up to 50 mm in size. A fine to course 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 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. The undrained shear strength for each soil sublayer, C_u, was estimated assuming C_u = 6.25 N, where N is the SPT blow count per 300 mm of penetration and the results are summarized in Table 1. Ground water level at the test hole locations was relatively shallow and was measured upon completion of test holes to be about 5.2 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 – 11.4	Sand, medium dense to very dense	14-58	18	-	35
11.4 – 33	Sand, dense to very dense	45-92	20	-	40
Site 2					
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

3. PILE CONFIGURATION

The configurations for 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 4 and type 6) while piles identified with odd numbers were for piles with a single helix. All piles were rounded pipes 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 double helix piles were spaced at 3 times their helix diameter (i.e. S/D = 3, where S is the spacing between the helixes and D is the helix diameter). Steel pipes were Grade 3 steel with a yield strength of 310 MPa.

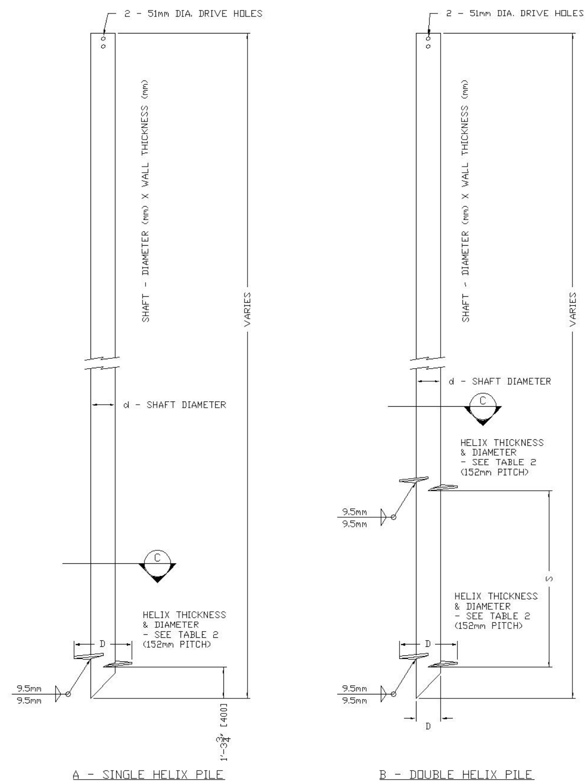

Fig. 1. Typical helical pile configurations

Table 2 Summary of pile configurations

Pile Type	Shaft		Helixes		
	Diameter mm	Thickness mm	Diameter mm	thickness mm	No of Helixes
3	324	9.5	762	25.4	1
4	324	9.5	762	25.4	2
5	406	9.5	914	25.4	1
6	406	9.5	914	25.4	2
7	508	9.5	1016	25.4	1

4. INSTALLATION MONITORING

Figure 2 shows a typical installation of a helical pile. Helical piles are typically installed through the use of mechanical torque applied at the pile head. Due to frozen soil conditions near ground surface and hard installation conditions between depths of about 1 and 3 m, pilot holes with size less than or equal to shaft size were predrilled to penetrate frozen and very hard soils. Field monitoring of helical pile installation and pile load testing was undertaken by Almita personnel. Table 3 provides a summary of the pile installations at test Sites 1 and 2, including the maximum torque recorded, predrill depth, thickness of soil plug and depth of embedment.

Fig. 2. Typical helical pile installation

Table 3 Summary of pile installation

	Test ID	Pile type	Shaft diameter mm	Installation torque at end of installation kN.m (ft.Ibs)	Embedment depth m	Soil Plug thickness m	Predrill depth m
Site1	ST1	4	324	211.5 (156,000)	9.0	5.1	7.6
	ST2	3	324	211.5 (156,000)	9.5	4.1	6.1
	ST3	4	324	211.5 (156,000)	9.5	4.4	6.1
	ST20	5	406	338.3 (249,500)	6.1	3.8	4.5
	ST21	5	406	338.3 (249,500)	5.7	1.9	4.0
	ST22	7	508	338.3 (249,500)	5.75	2.9	4.0
Site 2	ST5	4	324	211.5 (156,000)	5.9	1.7	5.2
	ST6	4	324	211.5 (156,000)	5.7	1.8	4.8
	ST7	3	324	211.5 (156,000)	5.7	2.0	5.0
	ST13	5	406	338.3 (249,500)	5.8	-	4.1
	ST14	5	406	338.3 (249,500)	5.6	-	3.9
	ST15	7	508	338.3 (249,500)	5.4	2.8	4.1

At test Site 1, all tested piles were predrilled using a drill auger to depths that varied between 4.0 m and 7.6 m below existing ground surface. Upon completion of the predrilling process, the depths of the open holes were measured and varied between 1.6 m to 3.5 m due to sloughing of sand material. It can be seen from Table 3 that the measured torque values at end of installation for piles ST1, ST2 and ST3 (types 3 and 4) were at 211 kN.m. The predrill depth for pile ST1 was 7.6 m and for piles ST2 and ST3 was 6.1 m and soil plug thickness varied between 4.1 m and 5.1 m. Pile ST1 was installed to an embedment depth of 9.0 m while piles ST2 and ST3 were installed to a depth of 9.5 m. It can be seen from Table 3 that the measured maximum torque at the end of installation for piles ST20, ST21 and ST22 (Type 5 and Type 7) was 338 kN.m. The predrilling process was also carried out to depths that varied between 4.0 m to 4.5 m and the size of the drill augur used for predrilling piles Type 5 and 7 were 406 mm and 508 mm, respectively which is similar to the size of the shaft of the helical piles. Pile ST20 had an embedment depth of about 6.1 m while piles ST21 and ST22 were embedded to depths of about 5.7 m.

Predrilling was also carried out at test Site 2 for all piles to facilitate their installation. Test piles ST5, ST6 and ST7, ST13, ST14 and ST15 were predrilled to depths that varied between 3.9 m and 5.2 m below existing ground surface. Upon completing the drilling process, the depths of the open holes were measured and all holes stayed open to their full drilling depth. Measured torque at the end of installation for piles ST5, ST6, and ST7 (Types 3 and 4) with shaft diameter of 324 mm was 211 kN.m and the corresponding embedment depths varied between about 5.7 m and 5.9

m. Piles were installed immediately above the very hard clay till layer that existed at a depth of 6 m. Measurements of depth to soil plug indicated that the soil plug inside the piles were formed and the depth of soil plugs varied between 1.7 m and 2.0 m which indicate that a full plug may not formed. Piles ST13 and ST14 (Type 5) with a shaft diameter of 406 mm were successfully installed using a 338 kN.m drive head. Torque at the end of installation for piles ST13 and ST14 was 338.3 kN.m (249,500 ft.ib) and embedment depths were 5.8 m and 5.6 m. Pile ST15 (Type 7) with shaft diameter of 508 mm was installed to a maximum depth of 5.4 m without installation difficulty.

5. LOAD TEST EQUIPMENT AND TEST PROCEDURES

The axial compression and tensile load tests were carried out in accordance with ASTM standards D 1143-07 and D 3689-07. Since the main objective of the load tests was to determine the ultimate bearing capacity of the pile, Procedure A (Quick Test) was adopted for nine of the tests wherein numerous small load increments were applied and maintained constant over short period of time intervals. Procedure B (Maintained Test) or slow test was used for three compression tests to provide a basis for comparison between both test methods and to study the creep effect on the axial load test results. Eight axial compression pile load tests were carried out at test sites 1 and 2. Three axial compressive load tests carried out at test site 1 were performed using Procedure A (Quick Test) and one test was performed using Procedure B (Slow Test). A total of four axial compression tests were carried out at test site 2 including two tests using Procedure A and two tests using Procedure B. Four axial tension (uplift) pile load tests were carried out at test sites 1 and 2. Two uplift load tests were carried out at site 1 for piles ST3 and ST21 and two uplift tests were carried out at test site 2 for piles ST5 and ST14. All axial tension (uplift) tests were performed using Procedure A (Quick Test).

5.1 Setup for Axial Compressive and Tensile Load Tests

Figure 3 shows a typical load test setup using six reaction piles. Each axial compression or tension test setup included a total of seven piles including the test pile and six reaction piles. The reaction piles were installed to provide sufficient load reaction at a clear distance from the test pile of at least about 2.3 m. The 400 ton test beam was centered over test pile and supported on four 100 ton reaction beams which in turn were supported on three reaction piles per beam. The arrangements for applying loads to the test piles involved the use of a hydraulic jack acting against the test beam. The connections were designed to adequately transfer the applied loads to the reaction piles and to prevent slippage, rupture or excessive elongation of the connections under the maximum load.

Fig. 3. Typical axial compression test setup

The axial loads were applied at the pile head using two 1800 kN hydraulic jacks situated at the pile head for the case of compression test and situated on the top of test beam between the test pile head and test beam for the case of tension test. The test pile is connected to the load cell through a loading frame consisting of four 50.8 mm diameter all-thread Grade 8 steel bars and 51 mm thick steel plate. The load at pile head was measured using a 7400 kN strain gauge load cell that was calibrated up to 4500 kN. A redundant hydraulic pressure transducer (10,000 psi capacity and 0.25%FS accuracy) was also attached to the hydraulic jack to measure the pressure applied at pile head.

Pile head axial movements were monitored at four points during the test, using two independently supported Linear Displacement Transducer (LDT) gauges (0.05 mm accuracy- 150 mm travel) and two mechanical dial gauges (0.05 mm accuracy- 50 mm travel). The LTDs, oriented in orthogonal directions and mounted with their stems perpendicular to the vertical axis of the test pile cap, were bearing against glass plate affixed to the pile cap. All LDTs, load cell and pressure transducer readings were recorded automatically using a Flex Data Logger system at intervals of 30 seconds throughout the test duration.

5.2 Test Procedures for Axial Load tests (Compression and Tension)

The following specific test procedures using Procedure A for Quick Tests for piles under axial compressive or uplift loads were applied:

1. Apply test loads in increments equal to 5% of the anticipated failure loads and maintain load constant for 5 minutes. Monitor movements using LDTs at intervals of 30 seconds. Monitor movements using mechanical dial gauges at the end of each load increment.
2. Add load increments until reaching a failure load but do not exceed the safe structural capacity of the pile or reaction apparatus. When reaching the failure load, maintain the load for longer period of time to monitor creep behaviour (about 10 to 15 minutes).
3. Unload test pile in five increments and hold for 5 minutes with same monitoring intervals as for loading. When reaching zero load continue monitoring the LDTs readings for 10 minutes to assess the rebound behaviour.

6. TEST RESULTS

6.1 Compression Test Results

6.1.1 Cohesionless Soil –Site 1

The load displacement curves (Figure 4) are used to determine the axial compressive load capacity of the piles. It can be seen from Figure 4 that the load displacement curves for all piles (ST1, ST2, ST21 and ST22) can be characterized into three parts: the first linear part up to a displacement of about 2 mm, followed by a nonlinear component continued up to displacements that varied between about 25 mm and 45 mm, followed by a secondary linear component with a smaller slope. No plunging failure was observed on any test pile at Site 1. Piles continued to resist higher loads up to the end of testing. The applied loads at pile head at the end of initial linear component varied between about 450 kN for Pile ST2 and about 500 kN for pile ST22. Comparing between the load-displacement curves for piles ST1 (with double 762 mm helixes) and pile ST2 (with single 762 mm helix), showed that both piles had a similar performance. This observation contradicted the well established fact (Sakr, 2009) that piles with double helixes generally provide a stiffer response at high displacement levels (i.e. higher loads at the same displacement levels). Reviewing installation records for piles ST1 and ST2 indicate that predrilling process was carried out to depth of 7.6 m for pile ST1 which is deeper than the depth to the top helix, while predrilling for pile ST2 was performed to depth of about 6.1 m (about 3 m above the helix level). Therefore the poor results of pile ST1 are mainly due to the deep predrilling which resulted in disturbing the bearing soil layer at the upper helix level, and losing considerable component of the skin friction along the pile shaft.

There are a large number of failure criteria used to interpret the axial compressive capacities of piles from pile load test results such as the Davisson criterion, Brinch Hansen, L1-L2 method, and FHWA (5%). It should be noted in many situations the design of foundations is controlled by the allowable structural displacements at foundation level. In the present study, the ultimate capacities of helical piles were defined as the load level that produce a displacement equal to 5% of the diameter of the largest helix (Sakr 2009) and the results are presented in Table 4. It can be seen from Table 4, that the ultimate capacities for piles ST1, ST2, ST20 and ST22 were about 2030, 1892, 2533 and 2200 kN, respectively.

Table 4 Summary of axial compression test results

Test ID	Pile Type	Shaft Dia. (mm)	Helix Dia. (mm)	Ultimate Capacity (5%)	
				Load (kN)	Displacement (mm)
Site 1					
ST1	4	324	762	2030	38
ST2	3	324	762	1892	38
ST20	5	406	914	2533	46
ST22	7	508	1016	2200	51
Site 2					
ST6	4	324	762	1912	38
ST7	3	324	762	1540	38
ST13	5	406	914	2292	46
ST15	7	508	1016	2400	51

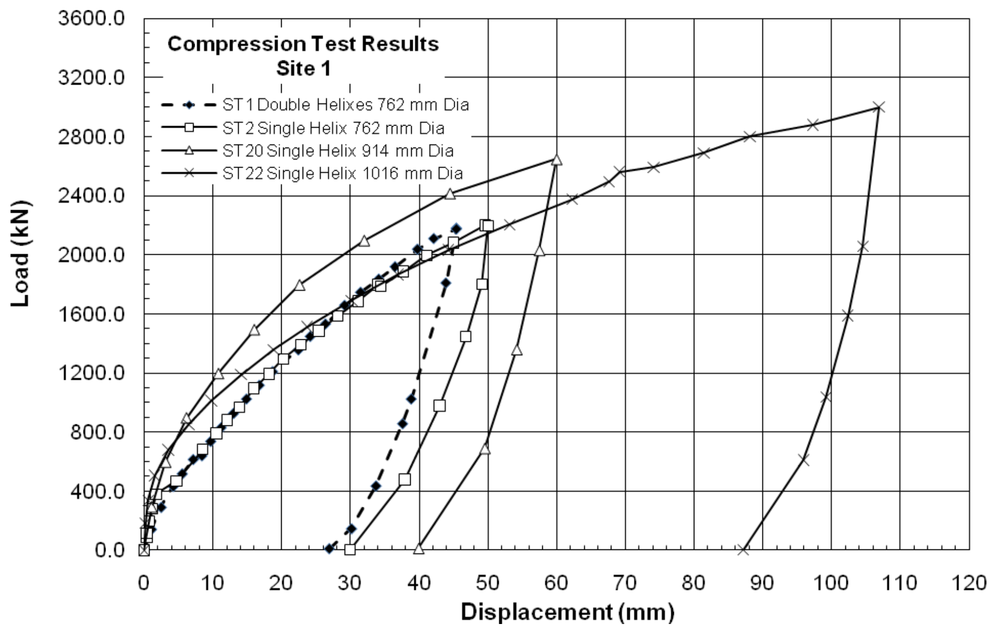

Fig. 4. Axial compression test results – site 1

<u>6.1.2 Cohesive Soil –Site 2</u>

The load-displacement curves for compressive load tests at test Site 2 are presented in Figure 5 and used to determine the load capacity of the piles. It should be noted that load-displacement relationships were established using the loads and corresponding displacements at the end of each load increment to account for any creep effect. This interpretation technique provides a conservative estimate of the axial capacity and takes into account the creep effect. It can be seen from Fig. 5 that the load displacement curves are also characterized by three parts (first linear part up to a displacement of about 2 mm, followed by nonlinear component up to displacement level of about 18 mm to 42 mm and secondary linear component with less slope). The loads at pile head at the end of initial linear component were about 250 kN for Pile ST6 and about 750 kN for pile ST15. All piles at test site 2 continued to resist higher loads up to the end of testing. It can be seen from Figure 5 that pile ST13 (with shaft diameter of 406 mm and single helix 914 mm) showed stiffer response than that of pile ST15 (with shaft diameter of 508 mm and helix size of 1016 mm). The reason for such behaviour is that pile ST13 was tested using a quick test method while ST15 was tested using slow test method and the creep effect was more pronounced for the case of pile ST15.

The ultimate capacities of helical piles at test site 2 were also estimated using 5% criterion and the results are presented in Table 4 as well. It can be seen from Table 4, that the ultimate capacities of piles ST6, ST7, ST13 and ST15 were about 1912 kN, 1540 kN, 2292 kN and 2400 kN, respectively. Comparing between the ultimate capacity of pile ST6 (with double 762 mm helixes) and pile ST7 (with single 762 mm helix diameter), indicates that pile ST6 resisted about 25% higher than ST7. Hence, the use of helical piles with double helixes is preferred since they considerably improve performance.

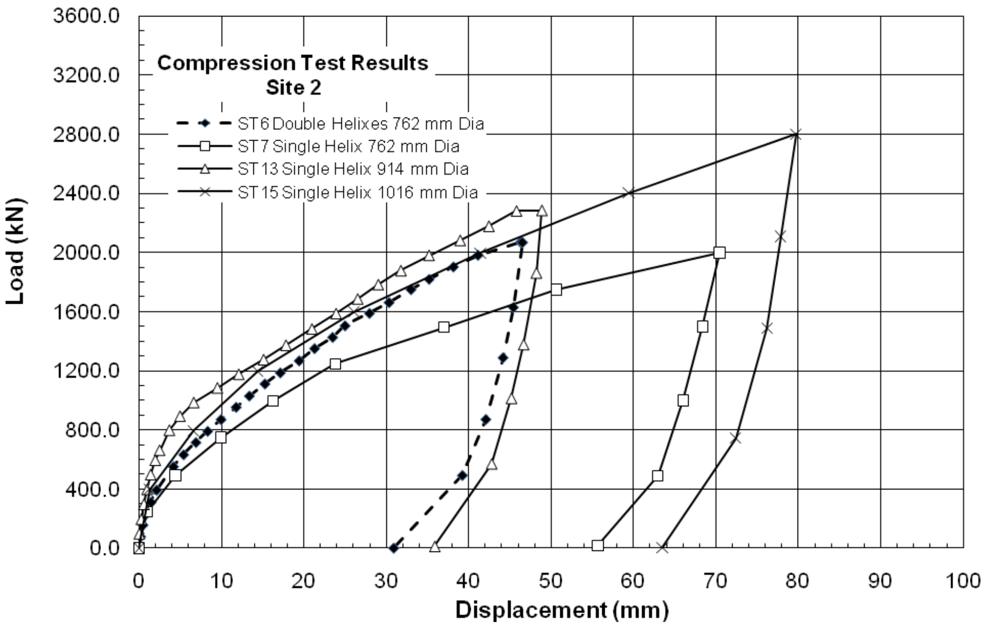

Fig. 5. Axial compression test results – site 2

6.2 Axial Tension (Uplift) Test Results

6.2.1 Cohesionless Soil –Site1

The results of uplift load tests performed at test site 1 are presented in the form of load displacement curves in Figure 6. These curves are used to determine the load capacity of the piles. It can be seen from Figure 6 that similar to compression tests, the load displacement curves can be characterized into three parts: first linear part up to a settlement of about 1 mm to 2 mm, followed by nonlinear component up to displacement level of about 25 mm for pile ST3 and 32 mm for ST21, and secondary linear component with less slope. The loads at pile head at the end of initial linear component were about 150 kN and 450 kN for piles ST3 and ST21, respectively. Pile ST21 was loaded to a relatively large displacement level of about 83 mm to investigate the post failure behaviour. However pile ST21 continued to resist loads up to the final loading increments with no identification of plunging failure. Comparing between the load displacement curves for both piles ST3 and ST21 indicated that the slope of the second linear component of the load displacement curve for pile ST3 (with double helixes) was approximately 200% steeper than that of ST21. This observation suggested again that the double-helix pile perform better than single-helix pile at high displacement levels.

The ultimate tensile capacities of helical piles were estimated using the 5% criterion and the results are presented in Table 5. It can be seen from Table 5 that the ultimate capacity of pile ST3 (Type 4 with double helix) was 1993 kN while the uplift capacity of pile ST21 (Type 5 with single helix) was about 1493 kN.

Table 5 Summary of axial tension (uplift) test results

Site	Test ID	Pile Type	Shaft Dia. (mm)	Helix Dia. (mm)	Ultimate Capacity (5%)	
					Load (kN)	Displacement (mm)
Site 1	ST3	4	324	762	1993	38
	ST21	5	406	914	1497	46
Site 2	ST5	4	324	762	1195	38
	ST14	5	406	914	1680	46

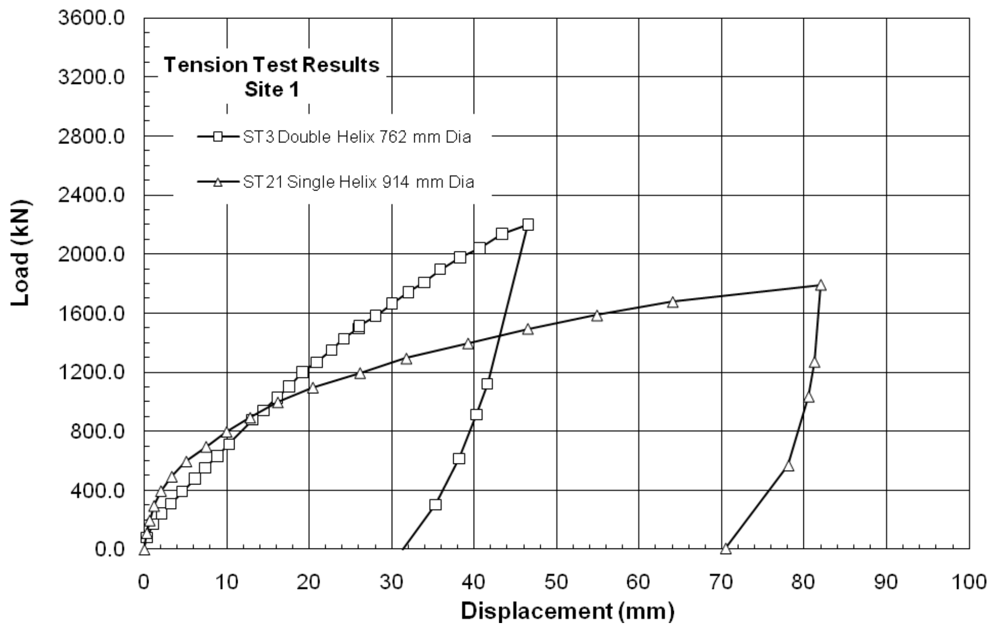

Fig. 6. Axial tension (uplift) test results – site 1

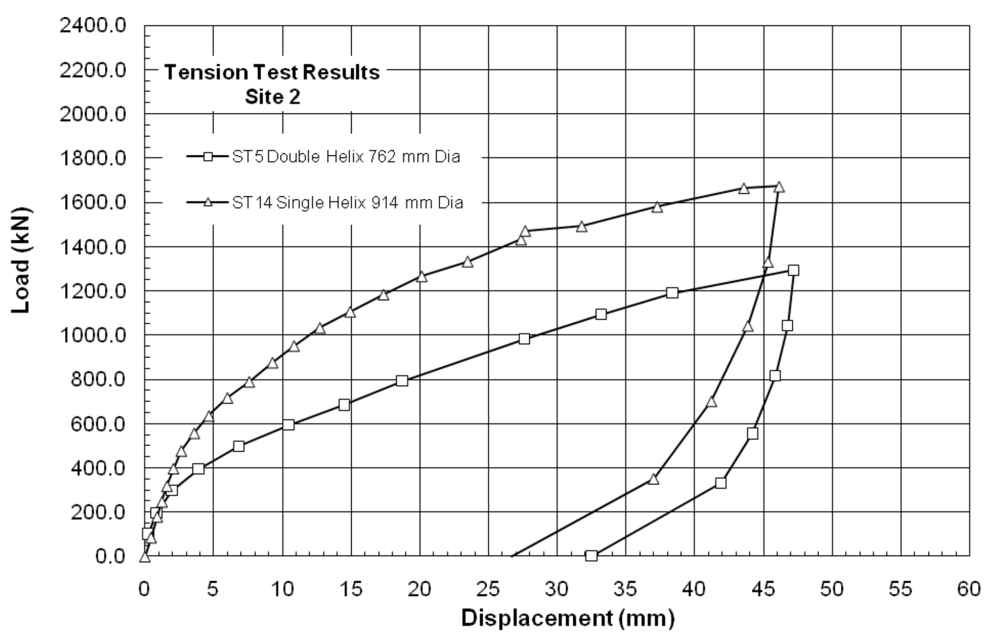

Fig. 7. Axial tension test results – site 2

6.2.2 Cohesive Soil –Site 2

The load-displacement curves for piles ST5 and ST14 tested at site 2 are presented in Figure 7 and are used to determine the load capacities for both piles. It can be seen from Figure 7 that the load displacement curves consisted of the typical three components including initial linear part up to a displacement of about 2 mm, followed by nonlinear component up to displacement level of about 18 mm to 20 mm and secondary linear component with flatter slope. The loads at pile head at the end of initial linear component for piles ST5 and ST14 were about 400 kN for both piles.

The ultimate tensile capacities of helical piles tested at site 2 were estimated using 5% criterion and the results are presented in Table 5 as well. It can be seen from Table 5 that the ultimate capacity of piles ST5 (Type 4) and ST14 (Type 5) using 5% failure criterion were 1195 kN and 1680 kN, respectively. It should be noted that the significant reduction in the uplift capacity of pile ST5 compared to ST15 is likely due to the deep predrill depth of 5.2 m that caused soil disturbance at the helix level and the shallow depth of the top helix. Therefore developing a consistent procedures for installation of helical piles is critical for their performance especially if predrill process is used as part of the procedure.

7. CONCLUSIONS

Helical Piles with relatively large diameters were successfully installed in dense to very dense sand and in very stiff to very hard clay till soil conditions. Based on the results of the full-scale load testing program, the following conclusions may be drawn:

1. The axial compressive capacities of helical piles estimated using 5% failure criterion varied between about 1500 kN and 2500 kN. Therefore helical piles can be used successfully to resist high compressive loads. The uplift capacity of helical piles tested on site was also relatively high and varied between about 1500 kN and 2000 kN. The axial uplift capacities of helical piles are typically at least 60% to 70% of the axial compressive capacities.
2. The high compressive and tensile capacities of helical piles are likely to reduce the number of piles required to support the loads for bridges which generally reduces the foundation costs. The speed of helical pile installation with minimal level of noise is really another differentiating factor for use of helical piles for bridge applications.
3. Helical pile installation is typically performed by screwing the pile into ground without a predrilling process. However, if a predrilling process is used during pile installation, the depth of predrilling should be limited to about 1 helix diameter above the top helix to avoid disturbing the bearing stratum for the most top helix.
4. The use of double helixes is recommended at this site to increase the axial capacities of large diameter helical piles.

8. REFERENCES

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