interaction factor between reference pile 1 and pile j and j = 2,…, n.

2 FULL-SCALE TESTING

The field testing program consisted of performing five compression load tests on four non-instrumented piles of different sizes at two sites: site (A) is composed primarily of sand; and site (B) is mainly clayey soil. Two axial compressive load tests were conducted at Site (A) and three axial compression load tests were conducted at site (B). The piles installed at site (A) had single helix, while those installed at site (B) had double and triple helices. The load tests conformed to procedure A of ASTM D1143 for axial compression testing.

The subsurface soil condition at site (A) included a top 0.3m of an organic soil material followed by a thin brown clay layer that extends 0.5m and consists of silt and sand, and traces of gravel. Underlying the clay layer is a sand layer that extends to 9m below ground surface. The sand ranged from fine grained at the top to coarse grained with increasing depth. The Standard Penetration Test (SPT) blow count number indicated loose to medium dense sand conditions with depth. The natural moisture content was averaged at 20% along depth. The groundwater table was not observed at the time of drilling and the piles were installed and tested during the month of October.

The subsurface soil profile established from the boreholes at site (B) comprises a surficial fill layer of sand and gravel mixed with some organics and extends to 1.5m with an SPT number ranging between 5 and 6. Underlying the surficial layer is medium to stiff brown silt and sand that extends to depths between 2.3m to 4.6m below ground surface with an SPT number varying between 3 and 12. Further deep is a silty clay layer that extends to depths 6.1m and 7.6m below ground surface. The silty clay layer gets softer with increasing depth and the SPT number ranged from 0 to 6. The ground water table was encountered 1.0 m below the ground surface.

The tested piles geometrical properties were representative of typical helical piles geometry in projects that involve light to medium loading conditions and are summarized in Tables 1 and 2 for site (A) and site (B), respectively.

The test results were used exclusively to calibrate and verify the numerical models that were then used to perform the parametric study.

Table 1. Summary of tested piles configurations at site (A)

Test Pile	Depth (m)	Shaft Diameter (mm)	Helix Diameter (mm)
PA-1	5.5	273	610
PA-3	5.6	219	508

Table 2. Summary of tested piles configurations at site (B)

Test Pile	Depth (m)	Shaft Diameter (mm)	Helix Diameter (mm)
PB-1	7.2	178	610x610x610
PB-2	7.2	178	610x610x610
PB-4	3.2	114	406x406

3 NUMERICAL MODELING

A finite element model is developed using the program ABAQUS (SIMULIA, 2009) to simulate the experimental program. The soil continuum is modeled considering a 3D cylindrical configuration and the pile is placed along the axial z-direction of the cylinder. The helix is idealized as a planar cylindrical disk so that modeling of the pile and the surrounding soil can take advantage of the axisymmetric conditions as shown in Figure 1.

Figure 1. Numerical model geometry for a single pile subjected to axial load.

3.1 Model description

The 3-dimensional soil medium is discretized into 8-noded, first order, and reduced integration continuum solid elements (C3D8R). The element has three active translational degrees of freedom at each node and consists of one integration point located at the centroid. The pile is simulated using four-nodes, first order, reduced integration, general-purpose shell elements (S4R).

The boundaries are located such that there is minimal effect on the results. The radius of the soil column extends approximately 33 shaft diameters from the center of the pile shaft. The depth of soil deposits below the lower helix is a minimum of 6.5 helix diameters. The top soil surface is considered as stress-free boundary. The boundary conditions exploited symmetry to reduce the model size. The bottom of the soil cylinder is pinned. The back of the cylinder is constrained in the horizontal plane and is free to move vertically.

The soil is modeled as an isotropic elastic-perfectly plastic continuum with failure described by the Mohr-Coulomb yield criterion. The elastic behavior was defined by Poisson's ratio, ν, and Young's modulus, E. The plastic behavior is defined by the residual angle of internal friction, φ_r, and the dilation angle, ψ, and material hardening is defined by the cohesion yield stress, c, and absolute plastic strain, ε_{pl}.

The pile-soil interface is modeled using the Tangential Behavior Penalty-type Coulomb's frictional model. The soil unit weight is accounted for in the numerical model as an initial stress through the geostatic equilibrium step.

3.2 Calibration and verification

Using some of the test results, the above model properties and configurations, and representative soil properties obtained from the boreholes and the literature, the numerical models are calibrated satisfactorily considering the soil conditions and load test results of piles PA-1 and, PB-1 and PB-2 as shown in Figures 2 and 3. The soil properties used in the analysis are assumed to be the disturbed properties due to pile installation.

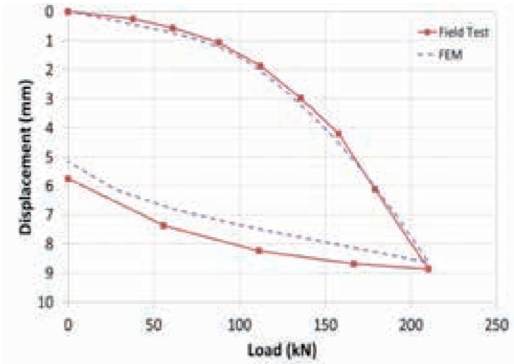

Figure 2. Calibrated numerical model compared to field test of PA-1

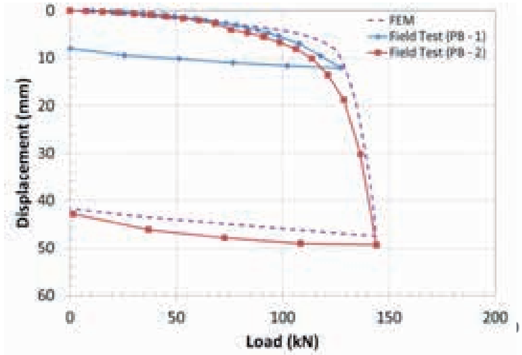

Figure 3. Calibrated numerical model of piles PB-1 and PB-2

In order to verify the ability of the calibrated models to accurately depict the behavior of helical piles under compressive and lateral loading, the calibrated models were utilized (considering the same soil properties and boundary and interface conditions) to analyze the remaining load test data and the results showed satisfactory agreement with actual test results of piles PA-3, and PB-4 as shown in Figures 4(a), and 4(b).

a)

b)

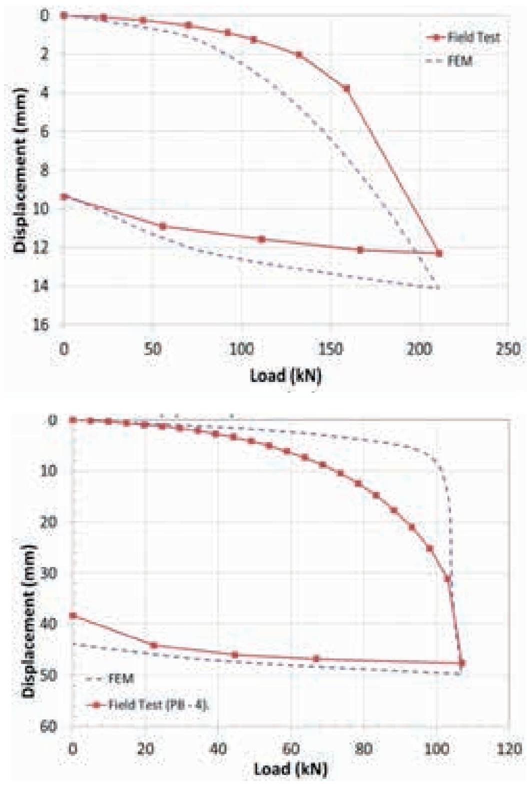

Figure 4. Verified numerical models: a) Pile PA-3; b) Pile PB-4.

Using the same soil properties that are established from the calibration of model considering pile PA-1 test data, the calculated response of neighbouring pile PA-3 is softer than the field test results as shown in Figure 4(a), but the calculated response of pile PB-4 is stiffer than the load test data. This is expected due to the natural spatial variability of soil properties.

3.3 *Parametric study*

Using the previously calibrated and validated models, a numerical parametric study is conducted considering different practical pile configurations and common soil types. The piles considered consist of a 273mm diameter steel pipe that has two 610mm helices attached to it. The inter-helix spacing ratio, S_r, ranges between 1 and 3 helix diameters (i.e. 1*D*, 2*D*, and 3*D*) with a pile embedment depth of 6 m. The piles are modeled as single, two, and four piles in a square arrangement with a center to center spacing, S_p, ranging between *2D* to *10D*.

The pile is modeled as elastic steel with E = 200GPa and ν = 0.3. For piles in sand, the sand is modeled as homogeneous with φ_r = 30° and ψ = 0° to represent loose to medium dense sand. The yield cohesion, c, is 0 kPa to represent purely frictional sand. The sand is assumed to have a bulk unit weight of 20 kN/m^3 and an initial coefficient of lateral earth pressure, K_o, equal 0.5. Moreover, the pile-soil interface friction angle, δ, is assumed to be 0.67φ_r which yields a friction factor of 0.38. Finally, the modulus of elasticity of the soil is assumed to be 100MPa and the soil Poisson's ratio, ν = 0.3.

For piles in clay, it is assumed that the helices are embedded into a very stiff clay layer with undrained shear strength, c_u = 100kPa and E = 50MPa, while the soil above top helix (i.e. along the shaft) is soft clay with c_u = 25kPa and E = 30MPa. The clay is modeled assuming the water level is at the ground surface, and the loading rate is assumed fast enough to invoke undrained conditions. Therefore, Poisson's ratio = 0.49 was considered in the analysis. The adhesion, c_a, between the pile and the soil is estimated from CFEM (2006): for c_u = 25 kPa, c_a = 25 kPa. A friction factor of 1.0 is used indicating that the frictional stresses along the shaft are equal to the contact pressure. However, to account for the adhesion strength, a shear stress limit along the interface is defined at which slippage occurs. This shear stress limit along the interface is c_a.

4 RESULTS AND DISCUSSION

For load-settlement curves with no visually distinctive failure point, as for the case of piles in sand, the failure loads are obtained at a practical settlement level equal to *5%D* (i.e. 30mm). The pile settlement is obtained at a service load equal to the failure load divided by a factor of safety, *FS*, equal to 3.

For a 4-pile group in sand, R_s could be as high as 1.3 at S_p = *2D* and as low as 1.1 at S_p = *5D*. R_s is the greatest at S_p = *2D* and decreases gradually with increasing S_p as shown in Figure 5. It is also found that S_r has a negligible effect on R_s. Moreover, R_s at service load considering *FS* = 2 is larger than R_s for service loads given by *FS* = 4, as shown in Figure 6. It is also found that R_s for a group of piles is not necessarily an algebraic summation of the interaction factors, α_{ij}, of the piles in the group. The existence of other piles in a group (other than the two under consideration) stiffens the soil. Therefore, the interaction factors would decrease relative to the case of a 2-pile group. Basile (1999) made similar observations and concluded that the interaction factors approach may lead to overestimation of pile response. Furthermore, Randolph (1994) stated that the interaction factors should only be applied to the elastic component of settlement since the plastic component of settlement is largely due to localized failure close to the pile and is not transferred to neighboring piles.

It is also found that the empirical equation suggested by Randolph (Equation 1) overpredicts the settlement ratio for four-pile groups by 22% for $S_p = 2D$ and by 45% for S_p greater than *3D*. In addition, using the equation proposed by Randolph and Poulos (1982) to obtain the interaction factor, α_v, for helical piles assuming straight shaft with diameter *D* for $S_p = 2D$ yields largely overestimated interaction effect. On the other hand, using a straight shaft pile diameter of *d* (i.e. helical pile shaft diameter) yields comparable values to the ones obtained by the parametric study.

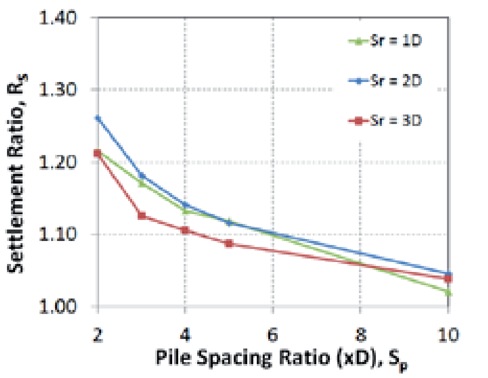

Figure 5. The settlement ratio for 4-piles group in sand with different S_r

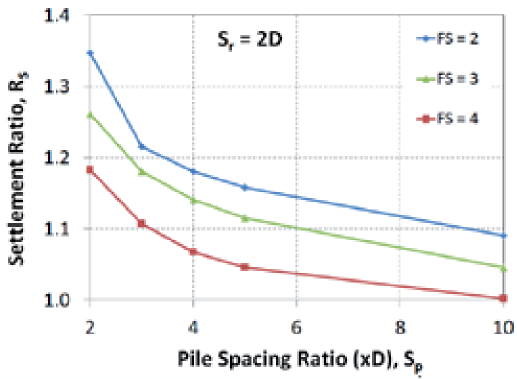

Figure 6. The effect of the factor of safety on the settlement ratio for 4-piles group in sand

For piles in clay, it is found that R_s could be as high as 1.33 for $S_p = 2D$ and as low as 1.1 for $S_p = 3D$, as shown in Figure 7. The settlement ratios are the highest at $S_p = 2D$ and decrease rapidly with increasing spacing. It is also found that S_r has a negligible effect on R_s.

Similar to piles in sand, it is also found that R_s for a group of piles is not a linear algebraic summation of the interaction factors, α_{ij}, of the piles in the group. It is found that the empirical equation suggested by Randolph (Equation 1) over-predicts R_s by 80% for piles spaced at *2D* and by 100% for S_p greater than *3D*. In addition, using Poulos (1979) charts and Randolph and Poulos (1982) equation, (Poulos, 1988), to obtain α_v assuming straight shaft piles diameter of *D* for $S_p = 2D$ is found to overestimate α_v. On the other hand, using the same charts and equation with a straight shaft pile diameter of *d* yields comparable values to the ones obtained by the parametric study.

Finally, in contrast to piles in sand, it is found that R_s at service load considering *FS* = 2 is lower than R_s for service loads given by *FS* = 4, however the effect is negligible.

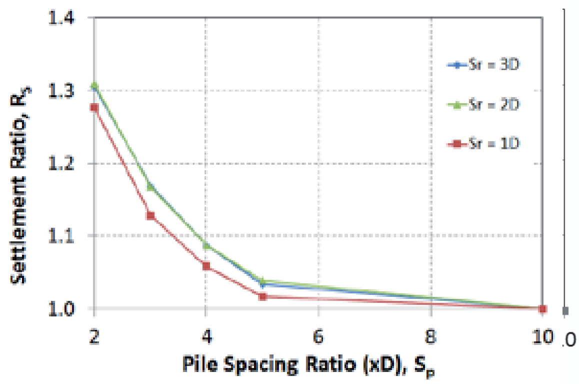

Figure 7. The settlement ratio for 4-piles group in clay with different S_r

5 CONCLUSIONS

The performance of a helical pile group in sand or clay is mainly affected by the piles center to center spacing. The practical range of inter-helix spacing (*1D* to *3D*) has negligible effect on R_s. The factor of safety, *FS*, could significantly affect R_s for piles in sand and has negligible effect for piles in clay. In addition, the settlement ratio, R_s, for a pile group is not simply an algebraic summation of the interaction factors, α_{ij}, of the piles in the group.

Finally, R_s can be conservatively estimated using the methods reported herein using a straight shaft pile with a diameter equal to the shaft diameter of the helical pile. In general, R_s for helical piles with multiple helices spaced at a typical pile spacing of *3D* is in the range of 1.15 to 1.2 for both clay and sand.

6 ACKNOWLEDGEMENT

The authors wish to extend their thanks to the National Science and Engineering Research Council of Canada (NSERC) for the generous financial support. The authors also extend their thanks and appreciation to Helical Pier Systems (HPS) and Almita Piling Inc. for their continuous support.

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