OpenSees Simulations of Axial Behaviour of Single-Helix Piles

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ABSTRACT
Helical piles are widely used in Western Canada for many engineering applications. Soil-helical pile interactions are important for the helical pile industry and have been conventionally investigated using continuum finite element analyses, which require sophisticated modeling of soils and piles. The present research conducted simplified numerical modeling of soil and single-helix pile systems on the platform of the Open System for Earthquake Engineering Simulation (OpenSees). The numerical model adopts the beam-on-nonlinear-Winkler-foundation (BNWF) method for simulating soil-pile interactions. Soil reactions to piles are simulated by a series of q-z, t-z, and p-y springs. The numerical modeling is used to calibrate the performance of in-situ helical piles under axial loads and to understand the load-transfer mechanism during in-situ loading tests. Cone penetrometer tests (CPT) results of each test site are used as the input to parameters of the numerical models. Preliminary results showed that the BNWF method could properly simulate the capacities and load-displacement behavior of single-helix piles.

RÉSUMÉ
Les pieux vissés sont largement utilisés dans l'ouest du Canada pour plusieurs applications en ingénierie. Les interactions sol-pieu vissé sont importantes pour l'industrie du pieu vissé et ont été étudiées grâce à l'analyse par éléments finis, ce qui requiert une modélisation complexe des sols et des pieux. Pour la présente étude, la modélisation numérique simplifiée des systèmes de sol et pieu vissé unique est réalisée sur la plateforme de l'OpenSees (Open System for Earthquake Engineering Simulation). Le modèle numérique adopte la méthode de poutre sur fondation Winkler non linéaire (BNWF, beam-on-nonlinear-Winkler-foundation) afin de simuler les interactions sol-pieu. La réaction du sol est simulée par une série de ressorts q-z, t-z et p-y. La modélisation numérique est utilisée pour calibrer la performance de pieux vissés en place soumis à des charges axiales et pour comprendre le mécanisme de transfert de charge pendant les essais de charge en place. Pour chaque site étudié, les résultats d'essais au piézocône (CPT) sont utilisés comme paramètres d'entrée du modèle numérique. Les résultats préliminaires ont montré que la méthode BNWF pouvait correctement simuler les capacités et courbes charge-déplacement des pieux à hélice unique.

1 INTRODUCTION

Helical piles are widely used in Western Canada as the foundations of houses, commercial structures, transmission towers, and so on. Conventinal appoaches have been developed to study and evaluate the behaviour of helical piles by Gahly et al. (1991), Narasimha Rao et al. (1991,1993), Gavin et al. (2014), Hambleton et al. (2014), and Elsherbiny and EL Naggar (2013) using field or laboratory tests. In the past decades, continuum finite element analyses have been introduced to investigate the load transfer mechanisms of helical piles to more complex soils. Livneh and El Naggar (2008) developed a continuum finite element (FE) model to calibrate the field testing of square shaft helical piles subject to axial loads. Kurian and Shah (2009) built a numerical model to study the load transfer mechanisms of conical tip helical piles under axial loading on the platform of Patron.

Although continuum FE modeling has been applied in the helical pile research, the sophisticated nature of FE modeling makes the simulations relatively expensive and is often beyond the capability of design offices.

The beam-on-nonlinear-Winkler-Foundation (BNWF) method is an efficient and effective modeling method for the soil-structure interaction research. For example, EL Naggar et al. (2005) developed a BNWF model to simulate the sesmic response of off shore piles against nonlinear ground motion input. Brandenberg et al. (2007) developed a BNWF model to calibrate a series of dynamic centrifuge tests on pile groups. Kim et al. (2007) conducted a comparison amongst 1-D, 2-D and 3-D BNWF models to simulate a conventional pile behavior under axial loading. Although the BNWF method has been widely used for simulating conventional driven or cast-in-place piles subject to axial or lateral loads, the method has not been applied to study the soil-helical pile interactions.

The Open System for Earthquake Engineering Simulation (OpenSees 2015) offers a simple yet reliable solution to the numerical modeling of helical pile behaviour in cohesive and cohesionless soils. In OpenSees, according to the BNWF method, reactions of soils to pile foundations are simulated using a series of q-z, t-z, and p-y springs, which represent vertical bearing resistance, shaft friction, and lateral soil resistance, respectively.

The present study investigated the soil – helical pile interaction in the axial direction using the BNWF method on the platform of OpenSees. The main objectives are to: (*i*) simulate the behavior of single-helix piles under static axial loading using a BNWF model on OpenSees platform, and (*ii*) evaluate the efficiency and reliability of

CPT based method of estimating soil parameters for numerical modeling. The paper first briefly presents the results of recent in-situ axial load tests of helical piles installed in cohesionless and cohesive soils. The paper then described the 2-D numerical model developed in OpenSees framework to simulate the tested single-helical piles. The paper discussed the CPT-based method of selecting parameters for the numerical modeling. Results of OpenSees simulations were compared to the in-situ testing results that validated the use of the BNWF method in the research of soil - helical pile interaction.

2 IN-SITU SITE CHARACTERIZATION

In-situ load tests of single-helix piles were conducted at cohesionless and cohesive sites located in Alberta. The test results were used for the calibration of the numerical models developed in this study. Cone penetration tests (CPT) were carried out to identify the subsurface characteristics of the Site 1 at the University of Alberta Farm and Site 2 that is a sand pit near Bruderheim, Alberta. CPT logs and the interpretations are shown in Figure 1.

The CPT results show that at Site 1, the top 5.0 m layer consists of uniform clay, underlain by interbedded silty clay and clayey silt from 5.0 m to 7.0 m. The ground water table was 4.8 m below surface. At Site 2, below the top soils were interbeded clean sand and silty sand to a depth of 4.4 m, underlain by clayey silt to silty clay from 4.4 m to 5.6 m, overlying a mixture of sand to silty sand from 5.6 m to 6.0 m. The ground water table was 3.0 m deep (Figure 1).

The soil type behavior (SBT) was classified after Robertson et al. (1986). The CPT interpretation of cohesionless soils followed the guideline of Robertson and Campanella (1983) with an upper limit of 28° for clay and 32° for silt recommended by Robertson and Cabal (2012).

Details of the in-situ load testing program and soil characterization are presented in the companion paper (Li et al. 2015).

3 DEVELOPMENT OF NUMERICAL MODELS

The numerical model consists of an elastic shaft and three sets of soil elements including the p-y, t-z, and q-z springs. The pile shaft below ground surface and above helical plate is divided into certain numbers of one inch segments with a node at each demarcation point. Each pile node is connected to a fixed node via a corresponding spring node. All the pile segments are modeled by elastic uniaxial steel material since the pile shaft is far from being yielded during the load testing. The length of pile shaft below helical plate is neglected in the modeling as ineffective length (Narasimha Rao et al. 1991, Zhang 1999), which does not contribute to the skin friction resistance. Figure 2 shows a sketch of the model.

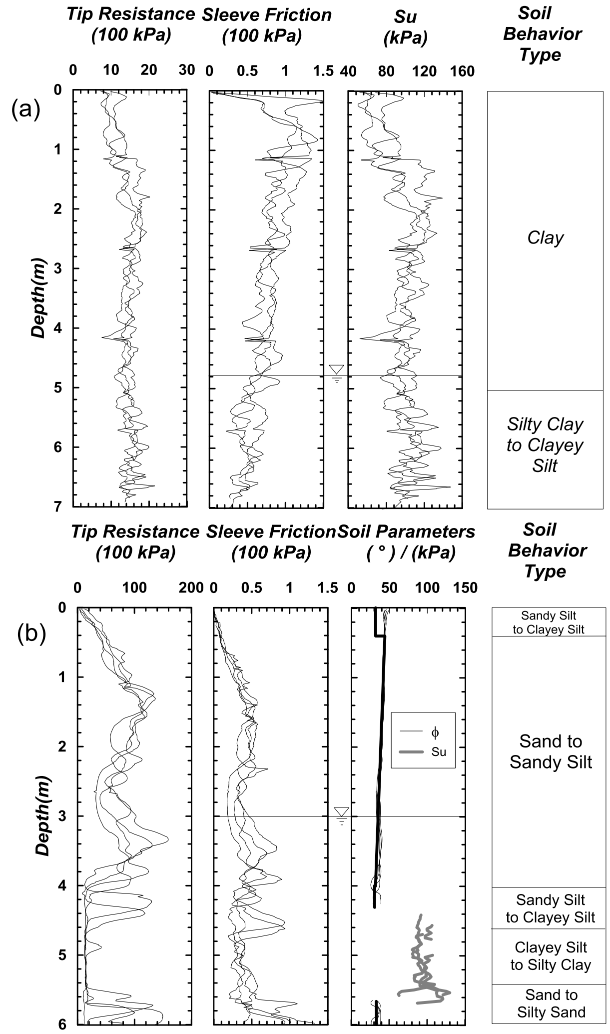

Figure 1. CPT profile of (a) Site 1 at the University Farm and (b) Site 2 at the Sand Pit in Bruderheim

Boulanger et al. (1999, 2003) described the development of these soil spring materials and implemented the materials into the OpenSees platform. The t-z spring named TzSimple1 in OpenSees, q-z spring (QzSimple1), and p-y spring (PySimple1) are suitable for non-liquefiable soils subject to static, cyclic, and dynamic loading conditions.

The present research focuses on the simulations of axial behaviour of helical piles using the BNWF method; under the axial loading, the lateral soil resistance has no significant effects on the behaviour of the helical piles. Thus, the p-y springs and the parameters of p-y springs are not described in this paper although the p-y springs are implemented in the numerical model to properly constrain the pile.

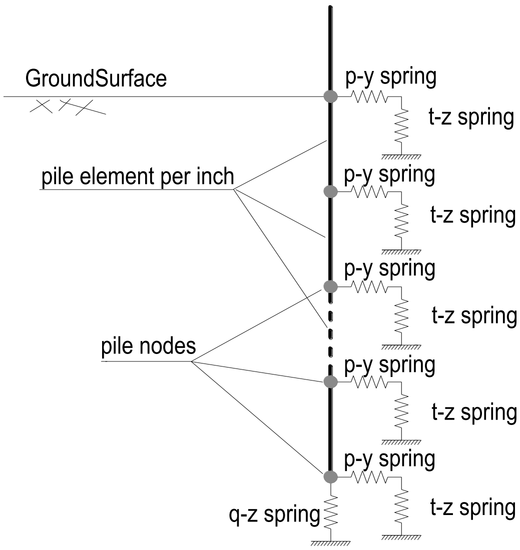

Figure 2. Numerical model configuration

3.1 q-z spring material

The soil springs QzSimple1 are used to simulate the bearing resistance at the helical plate location. The QzSimple1 has four input parameters:

- q_{ult} — the ultimate capacity of q-z spring
- z_{50} — the displacement at which half of q_{ult} is mobilized under static loading
- suction — uplift resistance is equal to suction×q_{ult}. Default = 0.0. The value of suction must be 0.0 to 0.1.
- qzType — an arguments that identifies the choice of backbone q-z relations, Reese and O'Neill (1987) for clay and Vijayvergiya (1977) for sand

Suction is occasionally assigned to the end bearing q-z spring to account for tip suction force of conventional piles. Narasimha Rao et al. (1993) estimated the suction force of the helical plate and recommended to neglect the suction. Thus in the present modeling, suction is set zero.

For sands, Vijayvergiya (1977) proposed an exponential representation for the q-z curve shown in Figure 3 together with the backbone recommended by Reese and O'Neill (1987) for pile in clay.

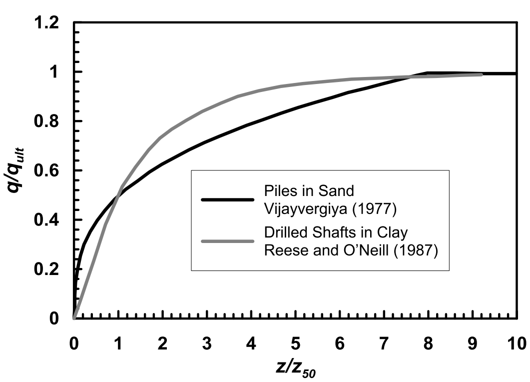

Figure 3 Backbones of q-z spring material adopted in OpenSees (after Boulanger et al. 2003)

To obtain the ultimate capacity q_{ult}, Meyerhof (1976) recommended a set of end bearing factors against friction angle for drilled piles in sand (Table 1). For drilled pile in clay, a classic bearing factor of N_c = 9.0 was adopted. Vijayvergiya (1977) also recommended the critical displacement, z_c, in sand ranging from 3 to 9 percent of the diameter of pile tip (helix). The critical displacement is the first point at which the resistance reaches the ultimate capacity. And z50 is approximately 0.125 times of z_c. As to clay, Aschenbrener and Olson (1984) recommended that z_c was about 1 percent of pile tip (helix) diameter and z_{50} varied within it. During the simulation, all these factors or coefficients were calibrated to approach the load-displacement curves measured from the load testing.

Table 1 . Bearing factor values for driven and drilled piles (After Meyerhof 1976)

Friction Angle, φ (°)	N_q, for Driven Piles	N_q, for Drilled Piles
28	20	8
30	25	12
32	35	17
34	45	22
36	60	30
38	80	40
40	120	60
42	160	80

When considering uplift loading, considerably less about the backbone has been known. Thus a proportionally reduced ultimate capacity was adopted in this study.

3.2 t-z spring material

The t-z spring model named TzSimple1 has three input parameters:

- t_{ult} — the ultimate capacity of t-z spring
- z_{50} — the displacement at which half of t_{ult} is mobilized under static loading
- tzType — an arguments that identifies the choice of backbone t-z relations, Reese and O'Neill (1987) for soft clay and Mosher(1984) for sand

The original backbones of the two types of soils are presented in Figure 4.

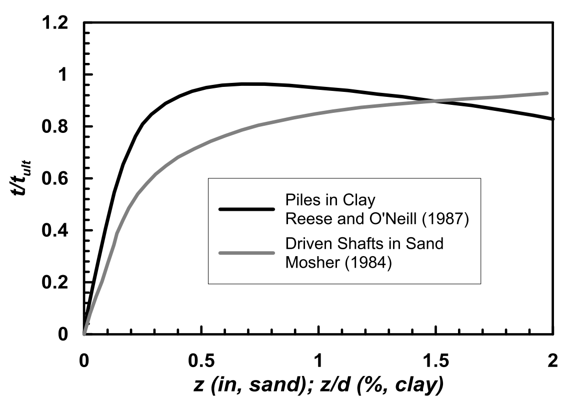

Figure 4 Backbones of t-z spring material adopted in OpenSees (after Boulanger et al. 2003)

Mosher (1984) recommended a design chart to approach the ultimate side friction in sand, say, t_{ult} of t-z spring material (Figure 5). The relative depth is the ratio of pile depth (z) over pile shaft diameter (d). To deal with ground water table, an effective depth z' was proposed as a substitute of z. The effective depth was obtained by dividing the effective vertical stress at a point by the effective unit weight. And z_{50} was calculated by Equation 1:

$$z_{50} = t_{ult}/k_f \quad [1]$$

where:

k_f = the initial slope of t-z curve by Mosher (1984)

Table 2 presents typical values of k_f provided the internal friction angle.

Table 2. The k_f values as a function of friction angle (after Mosher 1984)

Friction Angle of Sand (°)	k_f (kPa/m)
28-31	11310-18850
32-34	18850-26390
35-38	26390-33930

For clay, Coyle and Reese (1966) conducted a series of load tests on instrumented piles in clay and recommended a design chart for ultimate side friction shown in Figure 6.

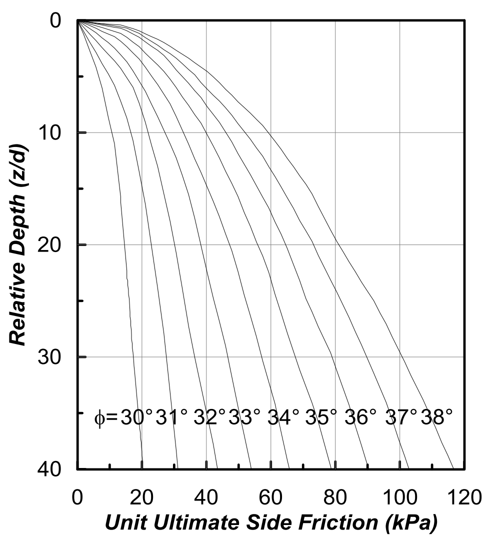

Figure 5. Ultimate side friction of steel shaft in sand (after Mosher 1984)

3.1 Pile geometry and material

The present research tested three types of single-helix piles (Li et al. 2015) with shaft diameters ranging from 7.3 cm to 11.4 cm and pile lengths from 2.44 m to 4.57 m. The detailed pile dimensions and sketches are shown in Table 3 and Figure 7, respectively.

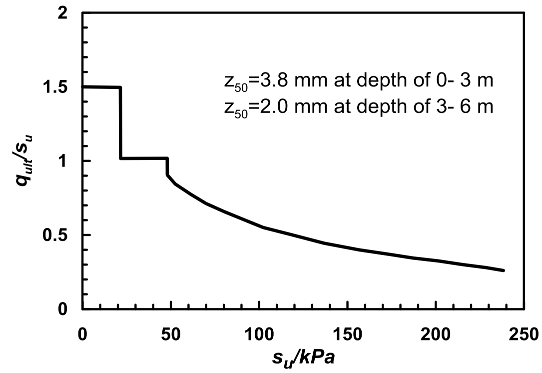

Figure 6 Ultimate side friction as a function of undrained shear strength (after Coyle and Reese 1966)

Table 3. Geometries of helix piles used for in-situ load tests and numerical modeling

Pile Type	L (m)	d (cm)	D (m)	H (m)	P (cm)
1	2.44	7.3	0.305	1.83	7.6
2	3.05	8.9	0.356	2.44	7.6
3	4.57	11.4	0.406	3.96	7.6

The wall thickness of the pile shaft, a circular steel pipe, was 7.8 mm. The deformation of the steel pipe during axial loading tests was negligible compared to the pile settlement. Therefore the pile shaft was considered as an elastic body in the numerical modeling.

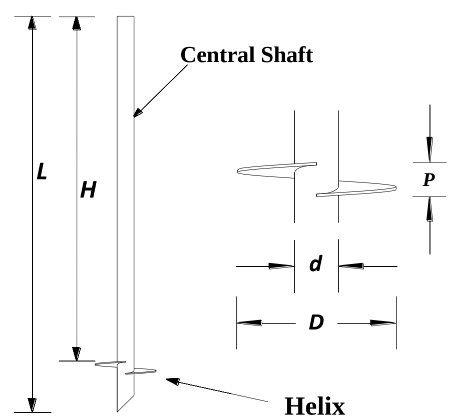

Figure 7. Sketch of the single-helix piles used for in-situ load tests and numerical modeling (not to scale)

4 NUMERICAL MODELING RESULTS

The undrained shear strength and friction angle profiles at the testing sites obtained from the CPT logs were used as the input to the parameters of the numerical models. The parameters of each spring material were generated from the CPT input using the approaches summarized in the previous section and adjusted to calibrate the load testing results. Four typical load-displacement curves corresponding to different soil conditions and different loading directions were presented in Figure 8.

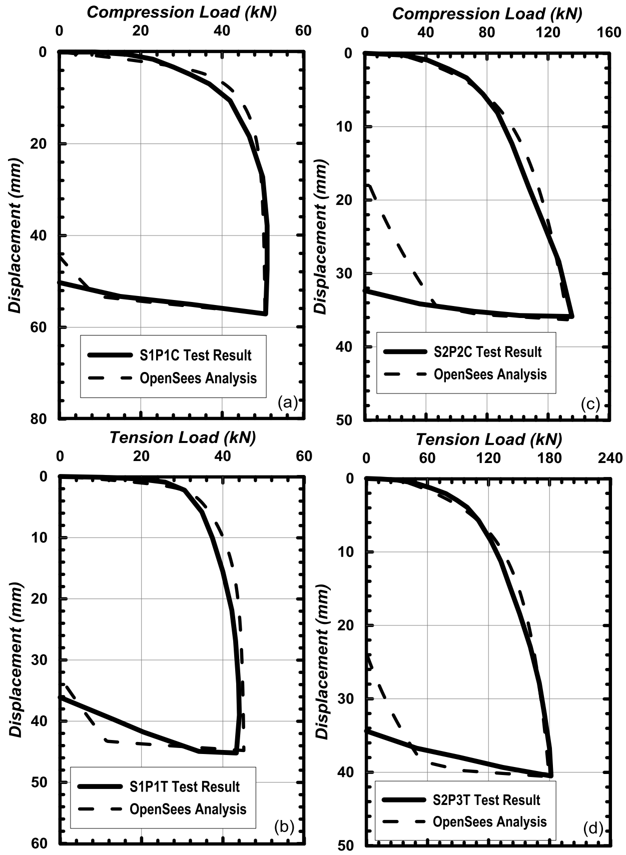

Figure 8. Comparison of numerical modeling to the in-situ test results of selected helical piles

The BNWF numerical models were calibrated against the four selected load-displacement testing. The following points could be observed from the comparisons in Figure 8.

i) The selected load-displacement curves are consistent with results of the BNWF modeling in OpenSees, although the stiff clay condition at Site1 was simulated by soil-pile interactions for soft clay, which is the only available in OpenSees so far.

ii) The stiffness of the elastic portion of the load-displacement curves obtained from clay (Figure 8 a, b) was underestimated. The underestimation was likely due to the selection of backbone curve for q-z spring material. The backbone curve summarized from soft clay was not fully capable of simulating the tests in stiff clay.

iii) The compression test calibration had a better agreement than tension test calibration in the early unloading phase. Despite the wild end of unloading, the pile resistance against compression had a higher initial stiffness during unloading. The most suspicious cause was vertical earth pressure acting on the helix to make it easier to settle (unloading tension) and harder to bounce (unloading compression).

iv) A considerable deviation of each unloading curve, obtained from both numerical modeling and load testing, was observed. To explain this deviation, one of the calculated curves is decomposed into q-z spring response and the sum of all t-z springs' response presented in Figure 9. It is seen from Figure 9 that q-z spring (helical plate bearing) has a steeper unloading slope and greater residual displacement than the sum of all t-z springs (the total skin friction).

At the meantime, another character is observed from Figure 9 that all the t-z springs are not mobilized to ultimate limit state until about 50 mm displacement, which is significantly greater than the displacement for q-z spring to mobilize to ultimate limit state.

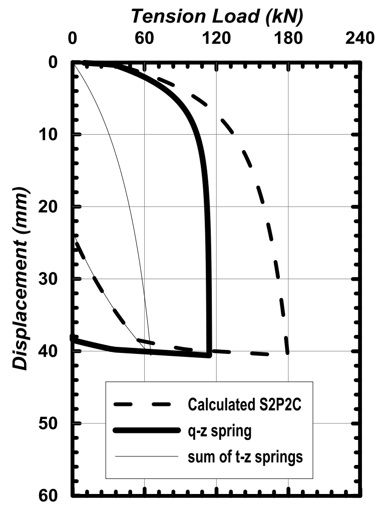

Figure 9. The components of numerical load-displacement curve for S2P2C

v) For the cases in cohesionless site, Site 2 presented in Figure 8 (c) and (d), the deviations of the ends of unloading phase were large. The cause of this inaccuracy was the overestimation of z_{50} for t-z springs, namely the shaft friction. Therefore the friction angle-based method for estimating cohesionless soil-pile interactions was queried. Nevertheless, the most important components, namely the springs, of the BNWF models in OpenSees do not require any soil parameter input, which provides an opportunity to develop an approach passing over characteristic soil parameters from CPT cone tip resistance to soil-pile interaction.

CONCLUSIONS

A numerical model was developed using the BNWF method in the OpenSees framework to simulate the behavior of three types of single-helix piles subject to axial compression or tension loading. The following conclusions may be obtained:

i) The BNWF method in the OpenSees framework is capable of producing the high quality simulation of the single-helix pile under axial static loading.

ii) The CPT-based method of selecting soil-pile interaction parameters for the numerical models in OpenSees is efficient and effective.

iii) The helical plate bearing is mobilized to the ultimate limit state sooner than the shaft skin friction. The residual displacement of the plate bearing is greater than that of shaft friction.

iv) An approach directly from CPT raw data to the parameters of the BNWF model is possible in OpenSees.

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