Observations & Conclusions from the Bahamas case study

Figures 10 and 11 show the variation in strain readings for a 24-hour period when the engine is being moved over the slab. The negative and the positive values in the strain readings show the compressive and tensile stresses experienced by the slab during the movement of the 298 metric ton engine.

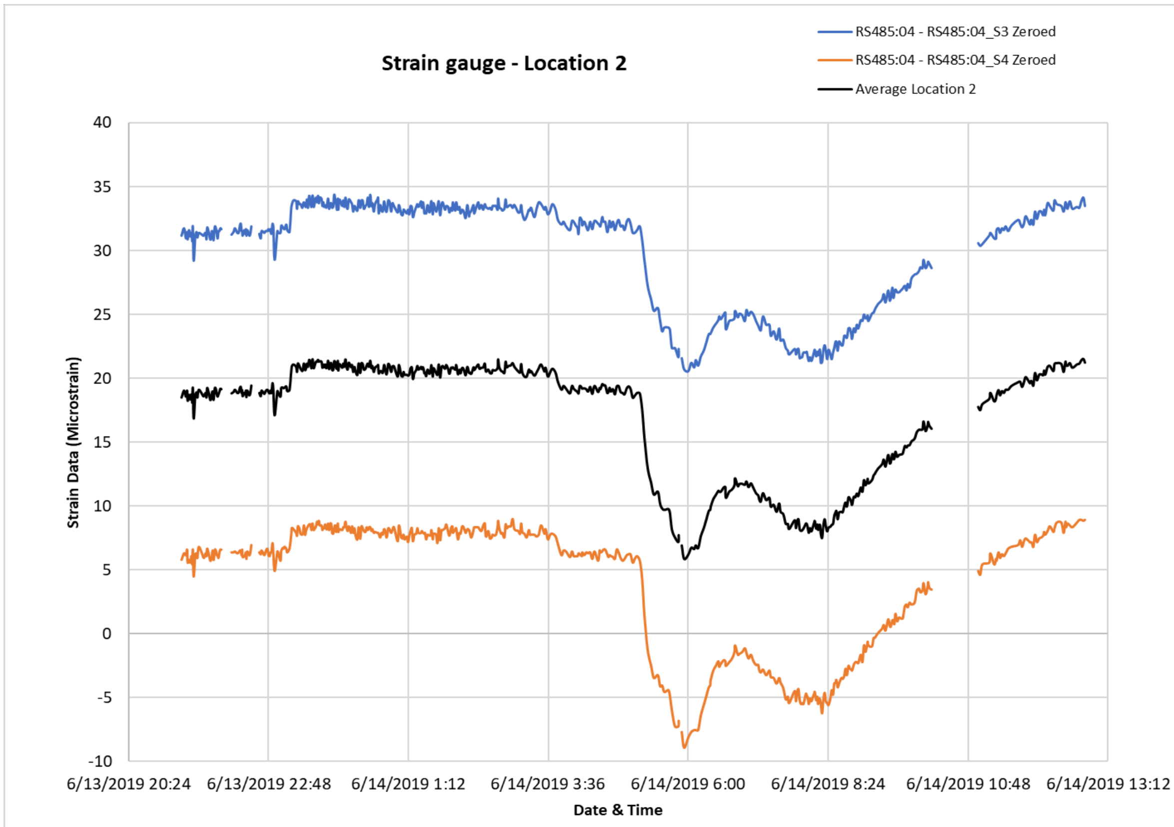

Figure 11: Life cycle monitoring data collected for 24-hour period at Location 2

CONCLUSIONS

The development of embedded sensors has made accurate long-term measurement of strain, temperature, corrosion potential in concrete deep foundations and in superstructure elements possible. As all the cabling is embedded into the concrete, it minimizes the risk to the data collection systems, Sensor equipment and data transmission systems from any construction activities. This technology makes it possible to document and maintain the records from the pre-cast stages and continue to acquire data after construction. Based on these measurements, design parameters, changes in loading; the effects of possible extreme events, corrosion rates and other anomalous data can be evaluated in real time. This process allows owners with sufficient time to plan remedial actions which will prevent or at least minimize the risk of catastrophic failure.

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Retrofitting Uplift Capacity of Telecommunication Tower Foundation with Helical Piles in Dense Granular Soils

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ABSTRACT

These days, it is typical for the height of existing telecommunication towers to be increased to accommodate for the ever-increasing cellular and data demands. This results in considerable increases in uplift force of the tower foundation. To resist the uplift force, in dense granular soils, concrete cast-in-place (CIP) tension piles are typically used adjacent to the tower foundation with a galvanized steel frame to connect the piles to the foundation. However, CIP piles are not only expensive but also require access to larger equipment such as drilling rigs and concrete trucks. These factors can cause additional costs and operational issues during the construction process. Therefore, in this study, as an alternative method to resistance of uplift force, the applicability of helical piles were investigated experimentally and numerically. Six helical piles were installed using a modified installation technique with high torque. The pullout force was measured as the uplift resistance. Furthermore, the total increased uplift capacity of the foundation was analyzed based on 2-dimensional axisymmetric numerical modeling parametrized with soil properties of the site, and a parametric study was performed. These findings suggest that helical piles can be an effective method of resisting the uplift force in dense granular soils created by an extension of a telecommunication tower section compared to CIP piles.

INTRODUCTION

Helical piles that have one or more pitched beating helices attached to the pipe have been vastly used for structures subject to uplift, lateral and compressive loads such as bridges and lighthouses. Recently, the piles have been used for various sustainable and renewable energy applications such as wind turbines, communication towers, solar farms, and other light structures (Perko 2009, Elsherbiny and El Naggar 2013). For example, due to an elevated demand in cellular technology and data, an increase in the height of telecommunication towers results in considerable increases in uplift force to the tower foundation. Helical blades are notable for easy installation as well as a more significant bearing surface that drastically increases the uplift capacity compared to cast-in-place tension piles (Deeks et al. 2005; Dijkstra et al. 2008; Shalabi and Bader 2014). Helical piles are installed in the ground by applying torque to the head. By monitoring the installation torque, the axial pile capacity can be estimated (Livneh and El Naggar 2008; Hoyt and Clemence 1989).

The purpose of this study is to investigate the effectiveness of helical piles in dense soil with high installation torques in providing the necessary axial tensile capacity for a communication tower. This is accomplished by developing a numerical model and comparing its results to the pull-out test performed at the site location. Furthermore, numerical simulation of the pile was conducted to evaluate the effect of selected variables, including the number of helices and soil density to influence on the pile bearing and displacement.

METHODOLOGY

Helical Pile

In the current study, six helical piles were installed to support the addition of a segment to a communion tower in Port Wing, Wisconsin (WI), United States of America (USA), to accommodate for the ever-increasing cellular and data demands as well as providing the additional uplift capacity for the tower. The piles are categorized as CHANCE “SS5-1 1/2” and are 3.05 meters long. The helix configurations are 0.20, 0.25, and 0.30 meters (m) in diameter, respectively, which are located at 1.5, 0.8, and 0.05 meters from the tip of the pile. The final torque reached during installation was approximately 5.4 kN·m, which is considerable and based on previous research studies (Livneh and El Naggar 2008; Hoyt and Clemence 1989), high uplift capacity can be expected.

Pull-out Test

To calculate the uplift capacity a pull-out test was conducted in accordance with ASTM D3689 while utilizing the “Procedure A - quick test method” associated with ASTM D1143 on a single helical pile on site beside the communication tower as shown in Figure 1. The design uplift capacity for the helical piles are 200 kN. During the test, axial tension is increased to 125% of the design uplift capacity (250 kN) and maintained for 10 minutes before unloading. The deflection at 100% of design uplift capacity was 6.6 millimeters (mm) and 10.2 mm at 125% of design uplift capacity.

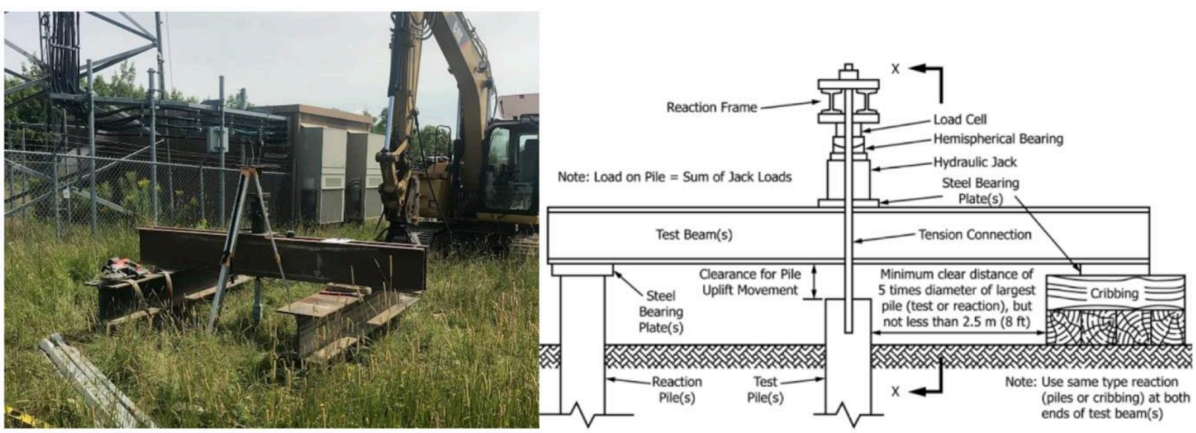

Figure. 1 Tensile load test setup at the site and typical setup supported on test beams (ASTM 3689)

Soil Properties

Three boring holes were drilled on site in Port Wing, WI at the approximate surface elevation of 30 m and the depth of 6 m in accordance with Standard Penetration Test (SPT) (ASTM D1586). The soil profiles are composed of approximately 1.5 m of stiff clay overlying 4.5 m of dense sand. Based on the high SPT blow counts (N_{60} of 30-50), the soil was considered as a dense sand for the numerical modeling.

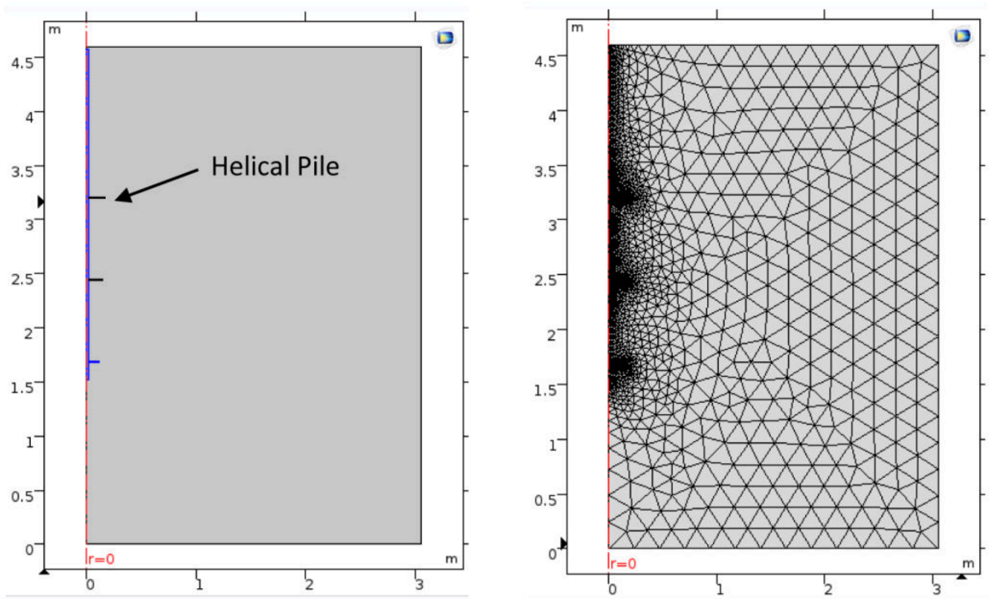

Figure. 2 Modeling geometry and mesh

Numeric Modeling

COMSOL Multiphysics was used to estimate the uplift capacity of the helical piles. COMSOL is a cross-platform finite element analysis, solver, and multiphysics simulation software. A two dimensional (2-D) axisymmetric model was developed while assuming a single layer of cohesionless dense sand for the soil profile. Clay soil is not considered in the simulation as the helices are positioned in sand. Based on the deformation of the pile itself during the tensile test in the field, a linear elastic behavior is assumed for the pile. Due to the large strains in the soil body, an elasto-plastic model with Mohr-Coulomb criterion was used for dense sand. A Coulomb friction model was used for the soil-pile interface with the coefficient equal to 0.1. The soil parameters used for the Mohr-Coulomb model are shown in Table 1.

The helical pile is 3 m long with a diameter of 0.05 m. The three helices positioned at the bottom half of the pile. The diameters of the helices are 0.2, 0.25, and 0.3 m, respectively, with 0.75 m spacing between each one. The boundaries of the model were placed at a distance more than ten times the diameters of the helical blades. The bottom boundary was set at more than the depth of five times the diameter of the helical blade below the pile top (Yang 2006). The boundary at the bottom of the soil is fixed, and the right side is assigned as a roller. The top surface of the soil is free as well as the left side's axisymmetric line. The uplift load is applied at the top of the pile. Figure 2 shows the model geometry and mesh. The material properties pile used in the numerical model are steel with a Young's Modulus equal to 200 GPa and a Poisson's ratio equal to 0.3.

Comparison of Load – Displacement Behavior

Figure 3 displays the load – displacement plot obtained from the numerical simulation along with the tensile test results carried out at the site. A convergence study was performed on very coarse, coarse, medium, fine and very fine meshes. The fine mesh showed the most agreeance with the experiment values. The small difference in the load – displacement plot is acceptable considering the error associated with heterogeneity of soil properties in the field and the degree of uncertainty and limits of precise prediction of soil *in situ* properties.

Table 1. Soil parameters used in the analysis

Parameters	Value
Unit weight ϒ (kN/m^3)	19
Young's modulus E (Mpa)	40
Poisson’s ratio ν	0.3
Angle of internal friction ϕ	30

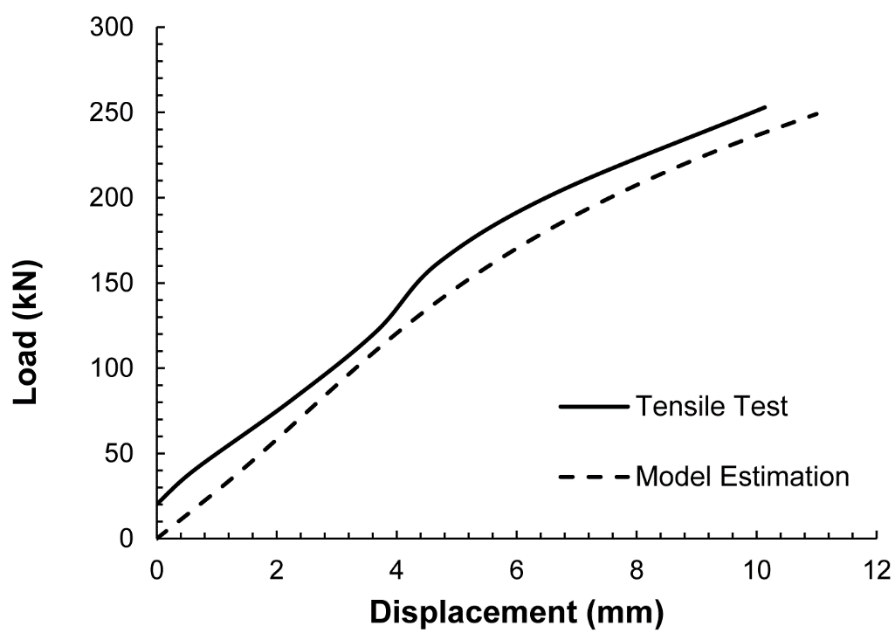

Figure. 3 Validation of numerical model with field test data

RESULTS AND DISCUSSION

Uplift Capacity

Perko (2009) supported the idea that the soil resistance mobilized above the helix during uplift is similar to the bearing resistance mobilized beneath the helix. Other researchers have assumed the failure bulb as an inverted truncated cone (Nazir et al. 2014; Ghaly et al. 1991). This can also be observed in the stress distribution formed beside each helix in our model as shown in Figure 4. The bulk of mass of soil above each helix inside the failure bulb, directly affects the axial capacity in tension (George 2017) and can be estimated from the model. The failure wedge in our model extends to 6.5B (B being the diameter of the pile), Ghaly, Hanna, and Hanna (1991) suggested values between 4B to 6B depending on the soil density and inner friction angle.

George (2017) modeling came to a value of 8.5B for the length of the failure wedge.

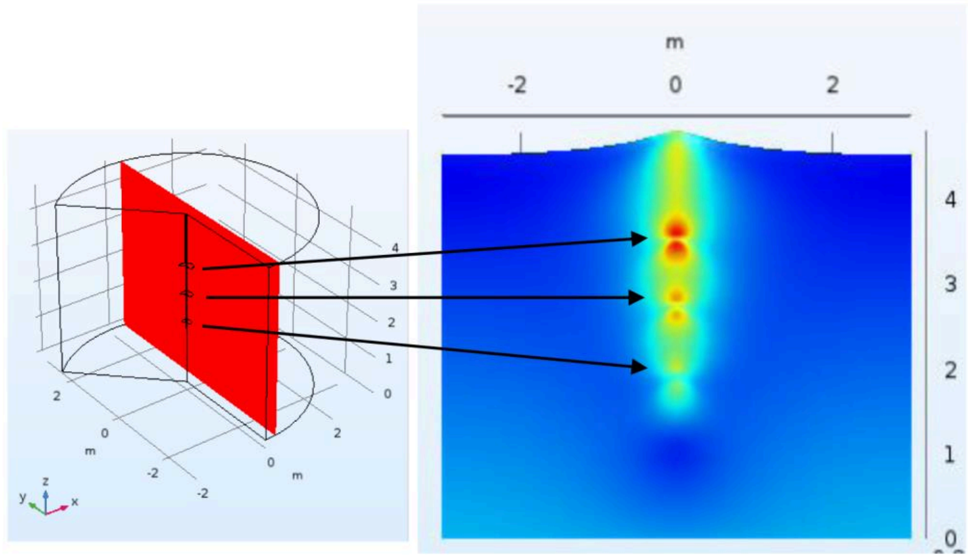

Figure. 4 Stress distribution in the specific soil plane beside the helical pile.

Using the six helical piles reduced the overall costs of the foundation by 70% compared to its counterpart CIP piles as well as reducing the construction time in half. Due to limited space and accessibility on the site from the existing tower, fencing, and equipment, helical piles were considered as more efficient and practical. Helical piles require only small motorized equipment for installation as well as no need for concrete as seen in other methods of reinforcement. The findings in this study suggest the effectiveness of helical piles resisting an increase in the uplift force due to a tower extension in dense granular soils was a viable alternative when compared to CIPs.

Effect of Numbers of Helices and Helix Diameter

Figure 5 shows the variations of total displacement along a vertical line 0.2 m from the pile for three different helix configurations:

1. Three helixes of 0.2, 0.25, and 0.3 m-diameters
2. Three helixes of 0.3 m-diameters
3. Single helix of 0.2 m-diameter at the bottom of the pile

The displacement of the two piles with three helixes are very similar as observed in Figure 5; however, installation of a pile with smaller helix diameters at the bottom is easier than installation of a pile with helices of the same size. In addition, the pile with a single helix displays the same range of total displacement, suggesting that the helix located at the bottom plays the most significant role when it comes to uplift capacity and tensile forces. Based on this parametric study, it is suggested that single helix piles that are easier to install, require less torque, and provide nearly the same uplift capacity as piles with more helices. Ultimately, the position of helices plays a greater role in the ultimate axial tensile capacity than the diameters and number of helixes.

Effect of Sand Density

Due to the failure bulb discussion in the previous section, it is expected that the density of the soil above the helices directly affects the uplift capacity of a helical pile. To demonstrate this idea the density of the site soil profile is reduced to 16 kN/m^3 from 19 kN/m^3. Figure 6 shows the displacement of the soil profile 0.2 m from the pile for loose and dense sands. As seen in the figure, the uplift capacity of helical piles is greatly affected by the density of the soil profile.

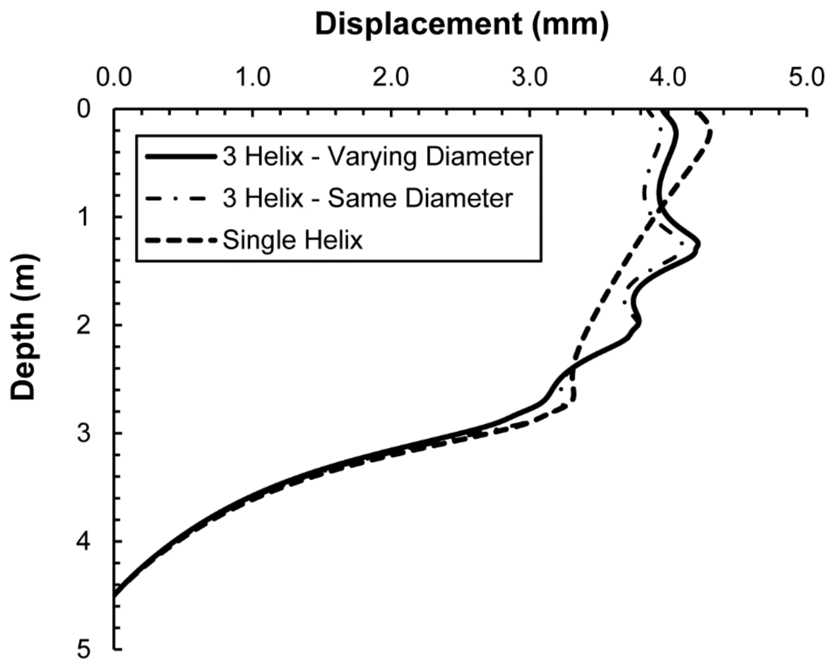

Figure. 5 Displacement versus depth for three piles with different helix configurations – three helixes of different diameters (0.20-0.25-0.30 m), single helix (0.20 m) and three helixes of the same size (0.30 m)

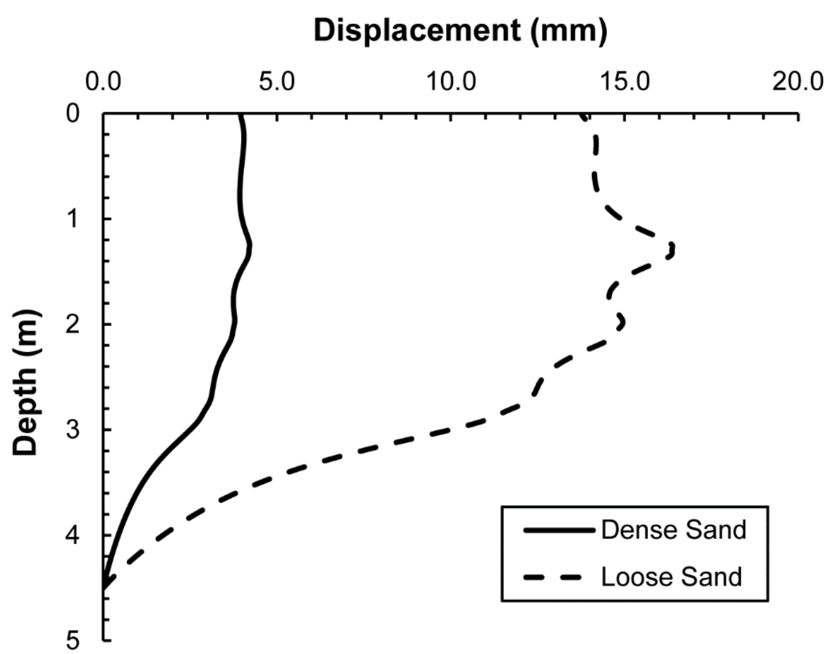

Figure. 6 Comparison of displacement for the helical pile with three helixes in dense and loose sand conditions

CONCLUSION

Six helical piles, two per leg, were installed on a three legged self-support communication tower located in WI, USA. The purpose of the piles were to support additional uplift capacity gained from a newly installed segment atop the tower. The helical piles, each with three helixes at the lower half of the piles, provide an uplift resistance of 200 kN due to their high installation torque and the presence of dense soil. The pull-out test and numeric modeling results present helical piles as a viable replacement for traditional CIP piles. Helical Piles cost less and require smaller installation equipment. Total displacements computed from the present analysis are in concordance with the field test results. In the parametric study, the soil density, number of helices, and helix diameters were studied. The effectiveness of a helix depends on the depth it is placed rather than the diameter of the helix itself. The density of the soil layer plays a major role in the axial tensile capacity of helical piles. Single helix piles are more efficient, practical, and are easier to install, while still providing the same uplift capacity as the three-helix cases demonstrated in this study.

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Efficient Pile Distribution for Piled-Raft Foundations for Tall Buildings

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ABSTRACT

The pile foundations are structural units that transfer the superstructure loads to deep levels where a satisfactory supporting ground or sound formation are encountered. It is quite common to have a group of piles supporting a heavy concentrated load. These groups are designed to support all exerted forces. Piles subjected to lateral loads or pull out forces due to wind or expansive soils need to be checked for stability and adequate strength. The pile settlement is a significant factor that needs to be kept within a small differential range. The work presented in this paper was conducted for a tall building proposed for construction at site underlain by limestone formation. It was aimed at comparing uniform distribution of piles under a thick raft and a rectangular tube-like distribution in which piles are concentrated under exterior shear walls. The structure also included two central columns supported by pile groups. Resistance to different loading conditions including vertical, lateral, and pull-out forces were studied and compared for the two pile configurations. The efficient pile distribution was selected. Comments on the efficient system and the choice criterion are given.

INTRODUCTION

Piled raft in tall buildings is a foundation system influenced by the characteristics of three elements: raft, piles and soil. In some cases, piled raft needs to be considered to avoid excessive settlement or tilting. Piled raft system can be cost effective foundation solution when the near surface ground is not suitable. Design method for piled raft are generally based on two methods: first method is the conventional pile foundation method, where it assumes that the total load from the superstructure is directly taken by piles, and the raft does not take any load, according to most standards, the piles must be designed with a safety factors in the order of 2 to 3 (Phung 2016). The second method is for piled raft foundation where it is assumed that piles and raft share the load and reduce the overall settlement. Reul et al. (2004) studies show that for the same total pile length smaller average settlement is achieved with longer piles rather than with a higher number of piles. Karim et al. (2013) reported that, when the raft is under uniform loading or core-edge loading, the differential settlements can be most efficiently reduced by installation of piles only under the central area of the raft. Modeling pile and raft system in PLAXIS 3D analysis (2018) confirmed that spacing between the piles directly affects the interaction between piles. The pile raft load carrying percentage for the case of two groups with constant length and pile diameters is decreasing by about 23% when the spacing between piles increases from 3 to 10 times the pile diameter. This is due to group action as stated by de Freitas Neto et al (2013). For raft in which the relative spacing between piles was 3, the piles carried 97% of the loading applied. For the raft with relative spacing between piles of 5, the percentage of the load carried by the piles was 86 % and finally with a relative spacing of 10, the percentage carried was 68%. In general it can be stated that if the spacing ratio is decreased then more piles are required and lower load is carried by a single pile. Tuan (2016) stated that Converse-Labarre formula underestimates the group efficiency. As a result, formula of Converse-Labarre also underestimates the load-bearing capacity of pile groups. Salgado et al. (2014) have studied the increase in group efficiency with increasing pile spacing. It was found that group efficiency increases as the pile slenderness ratio Lp/Bp decreases. Moreover, group efficiency in soil profiles with stiffness increasing with depth is found greater than that for the case of uniform soil profile. Jabbar Noman et al. (2019) stated that the group efficiency is increased with increasing the spacing between piles with a maximum value more than unity at 4D pile spacing and beyond this spacing the efficiency is decreased with increasing pile spacing and getting closer to unity. The term "group efficiency" depends originally on parameters such as, type of soil, method of piles installation, i.e. whether cast-in-situ or driven piles or other construction methods. These factors affect the group efficiency of piles. The spacing between groups of pile relies on factors such as overlapping stresses of adjacent piles, cost factor and the pile group efficiency. More details on pile groups are presented by Hannigan et al (2016). The objective of this paper is to compare between two piled-raft configurations for a tower proposed on highly fractured limestone.

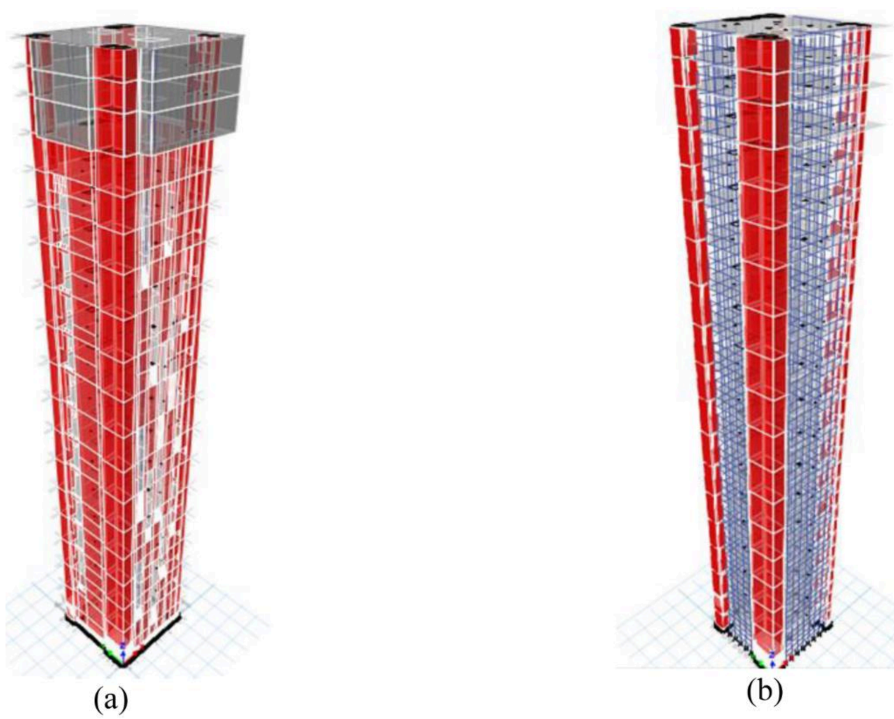

Fig. 1. ETABS models of the tower. (a) Shear wall system (b) Tubular system.

THE CASE STUDY AND STRUCTURAL OVERVIEW

The In this study, a vertical continuous vulcanization (VCV) tower is to be constructed in Riyadh, Saudi Arabia. The proposed tower is 162 m high with a base of 25 m x 25 m and 20 stories having variable heights, including eight-meter height underground basement. This tower will be used to produce high voltage cables to supply the demand of Saudi Arabia which requires more energy to meet the demand of the tremendous population growth. The tower will be