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A Numerical Approach on Bearing Capacity of Drilled Shafts Embedded in Clay

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Abstract

This study numerically investigates the bearing capacity of drilled shafts (bored piles) in clay using FLAC^{2D}. The results obtained in this study are compared with centrifuge test results. The results of the empirical relationships available in the literature are compared with the results of the present numerical study. A series of analyses is also conducted to assess the effects of various soil and pile parameters on the magnitude of tip and side resistance of bored piles embedded in clay. These parameters include the soil elastic modulus, pile length and diameter, undrained shear strength, unit weight, and Poisson's ratio of soil. Furthermore, the coupling effect of soil undrained shear strength and elastic modulus of soil on tip resistance are investigated. The results show that the lower value of soil elastic modulus results to lower effect of soil undrained shear strength. The effect of soil undrained shear strength on tip resistance is approximately constant (about 83% for a change of soil undrained shear strength between 25

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to 200 kPa) for the range of elastic modulus between 20 and 180 MPa. Also, a new equation is proposed to estimate the bearing capacity factor of N^*_c.

Keywords: Bearing capacity, Side resistance, Tip resistance, Drilled shaft, Bored pile, Numerical modelling, clay, Sensitivity analysis, FLAC

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Introduction

For many geotechnical applications, drilled shafts and driven piles, sometimes referred generally as piles, are used to transfer loads from superstructures to the underlying soil layers. In this regard, predicting the axial capacity of piles is among the top interests of geotechnical engineers. The axial bearing capacity of a drilled shaft is composed of two components, namely side (skin) resistance and tip resistance. This study is an investigation on bearing capacity of drilled shafts embedded in clayey soils. There are some recommendations available in the literature to estimate the bearing capacity of drilled shafts embedded in clay. The most well-known relationship to estimate the end bearing capacity is Equation 1. Vesic (1977) [1] has proposed Equations 2 to determine the N_c^* factor. In this equation, I_r is the soil rigidity index as determined from Equation 3. In this equation E_s is soil elastic modulus and s_u is soil undrained shear strength. O'Neil and Reese (1999) [2] reported Figure 1 to estimate soil rigidity index from undrained shear strength

$$q_p = s_u \, N_c^* \tag{1}$$

$$N_c^* = 1.33[(\ln I_r) + 1] \tag{2}$$

$$I_r = \frac{E_s}{3s_u} \tag{3}$$

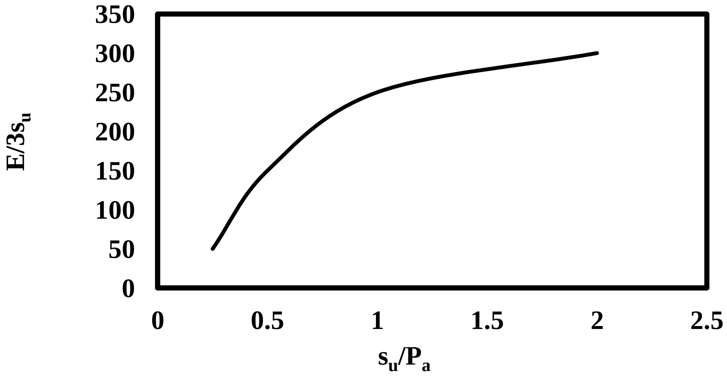

Figure 1. values for soil rigidity index based on undrained shear strength (O'Neil and Reese (1999))

The general relationship to determine the side resistance of drilled shafts in clay is Equation 4 (called alpha method). Based on more than 200 field-test results of drilled shafts, Kulhawy and Jakson (1989) [3] recommended Equation 5 to determine the magnitude of α. However, O'Neil and Reese (1999) [2] proposed Equation 6 to determine the parameter of α. Also, Meyerhof (1976) [4] recommended the value of 0.36 for the parameter of α.

$$f_s = \alpha\, s_u \tag{4}$$

$$\alpha = 0.21 + 0.25\left(\frac{P_a}{s_u}\right) \leq 1 \tag{5}$$

$$\begin{cases} \alpha = 0.55 & \dfrac{s_u}{P_a} \leq 1.5 \\ \alpha = 0.55 - 0.1\left(\dfrac{s_u}{P_a} - 1.5\right) & 2.5 \geq \dfrac{s_u}{P_a} \geq 1.5 \end{cases} \tag{6}$$

Aoki and Velloso (1975) [5], Philipponnat (1980) [6] and LCPC (French method) [7, 8] suggested Equations (7), (8) and (9) for estimation of tip resistance based on CPT results. Aoki and Velloso (1975) [5] and Philipponnat (1980) [6] also recommended Equations 10 and 11 for determination of side resistance. In these equations, q_c is the value of CPT at the pile tip, and $\overline{q}_c$ is the average of q_c along the length of the pile.

$$q_b = 0.286\, q_c \tag{7}$$

$$q_b = 0.5\, q_c \tag{8}$$

$$q_b = 0.375\, q_c \tag{9}$$

$$f_s(\text{kPa}) = 8.6\, \overline{q}_c(\text{MPa}) \tag{10}$$

$$f_s(\text{kPa}) = 17\, \overline{q}_c(\text{MPa}) \tag{11}$$

Numerical modelling

This study uses FLAC ^{2D} [9] to model the bearing capacity of drilled shafts in clay. In this study, the axisymmetric option is used for this three dimensional condition to reduce the number of elements in the solution procedure. Based on sensitivity analyses, width and height of the model are chosen to be 25 times of pile radius and 2.5 times of pile length, respectively. The bottom boundary is restrained in both X and Y directions, and the side boundaries are restrained in X direction. FLAC provides interface elements to be used as a contact of two different material surfaces. An interface is represented as a normal and shear stiffness between two planes which may be in contact with each other. Therefore, two interface elements have been used between the tip and side of pile and the surrounding soil. In this study, the normal and shear stiffness of the interface elements are assumed to be $3*10^8$ Pa/m and $1*10^8$ Pa/m, respectively. These values are chosen based on sensitivity analyses. Also, they are in the range of stiffness values recommended in FLAC manual. The cohesion parameter for interfaces is assumed to be 0.55 times of soil undrained shear strength. The friction angle of interface elements is assumed to be zero, since the soil is undrained clay. The element dimensions are 10*10 cm in the vicinity of the symmetry axis. These elements become larger as their distances from symmetry axis are increased. Finally, they reach to a size of 10*50 cm at the right boundary. Figure 2 shows the mesh used together with its boundaries and the location of interfaces.

Also, the elastoplastic Mohr-Coulomb model is implemented for the surrounding soil, and elastic model is used for the pile material. Table 1 shows the elastic model parameters required for the pile. The elastoplastic Mohr-Coulomb model uses the soil undrained strength (s_u), elastic modulus (E_s), Poisson’s ratio (ν) and unit weight (γ) of soil. Bowles (1996) [10] proposed a range of 0.4 to 0.5 for Poisson’s ratio (ν) of clay. In this study, the value of Poisson’s ratio assumed to be equal 0.45.

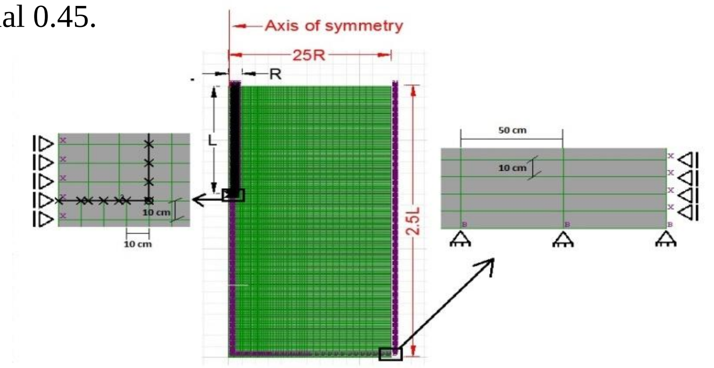

Figure 2- An instance of mesh, boundary conditions and the place of interfaces.

Table 1. Pile elastic parameters

Elastic modulus (E_p) (GPa)	Poisson's ratio (ν_p)	Unit weight (γ_p)(kN/m^3)
30	0.2	2500

Also, the total bearing capacity of drill shaft is the average of vertical stresses (σ_y) in the elements associated under the pile head, and the tip resistance is the average of vertical stresses (σ_y) at the pile tip. It should be noted that the side resistance can be calculated with three different methods. First, side resistance is equal to the subtraction of tip resistance from total bearing capacity. Second, side resistance can be estimated from a code proposed in FLAC manual for interface elements. This code estimates the average of mobilized shear stresses in the interface elements. Third, side resistance is equal to the average of shear stresses (σ_{xy}) at the soil element in contact with the pile shaft. In this study, all of these three approaches lead to the same side resistance results with less than 0.5% error. Therefore, this study uses the code proposed in FLAC manual.

Verification

Horikoshi and Randolph (1996) [11] conducted a centrifuge test in clay having an undrained shear strength given by Equation 12. In this equation, z_p is the depth of the soil. This centrifuge test was performed at the University of Western Australia and its model scale was 1/100 with a nominal centrifugal acceleration of 100g. Also, Horikoshi and Randolph (1998) [12] considered other soil and pile parameters as presented in Table 2. The same assumptions are used in this study.

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The comparison of the results of this numerical study with the centrifuge test results reported by Horikishi and Randolph (1996) is shown in Figure 3. This figure displays acceptable agreement between the results of the centrifuge test and this study.

$$S_u(\text{kPa}) = 33 + 1.2z_p(\text{m}) \tag{12}$$

Table 2. Soil and pile parameters used for verification

Pile length (m)	Pile Diameter (mm)	S_u (kPa)	γ (kN/m^3)	E_s (MPa)	ν
15	320	Equation 12	17.3	20	0.4

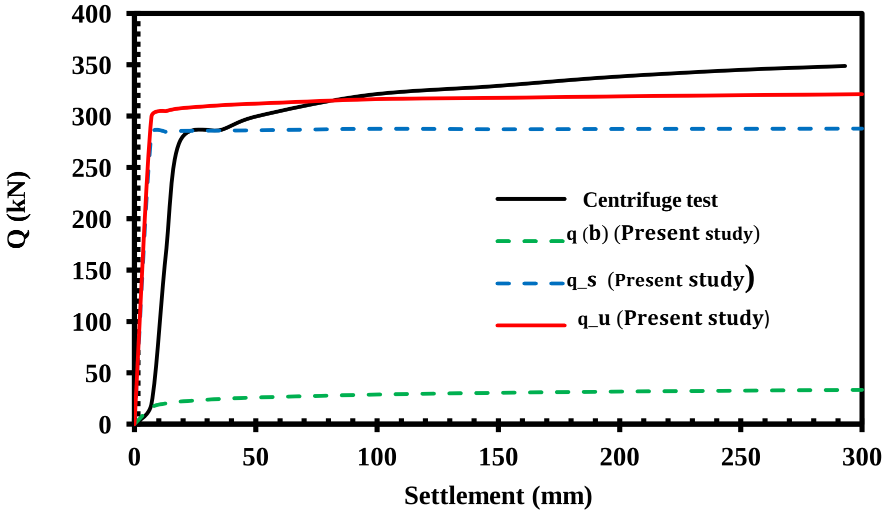

Figure 3. Comparison of the results obtained in this study with the results of the centrifuge test conducted by Horikoshi and Randolph (1996)

Comparing different methods

Bowles (1996) [12] indicated that the load-settlement curve

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related to tip resistance in any soil does not meet any pick point. Therefore, it is necessary to select a criterion for the determination of the ultimate values of tip resistance. Fleming et al. (2008) [13] recommended that the displacement needed for full mobilization of tip resistance was in the range of 5 to 10% D, where D is the pile diameter. According to Whitaker and Cooke (1966) [14] failure load corresponds to a settlement of 10% pile diameter. Reese and Wright (1977) [15] and O'Neil and Reese (1999) [2] recommended the value of 5% for the ratio of settlement to diameter (S/D). In this study, the settlement criterion is selected to be 10% of the pile diameter. Also in this numerical study, load-settlement curve related to side resistance of clay reaches its ultimate value in the range of 0.3 to 30 mm and stays constant for larger values of settlement. It should be noted that the larger values of undrained shear strength and also lower values of elastic modulus needs larger amount of settlement. Therefore, the pick point of load-settlement curve is considered as the ultimate side resistance.

As discussed in the Introduction section, there are different methods to estimate the side resistance. Equations 7 to 11 are based on CPT results, respectively. Therefore, a correlation must be used to convert the value of CPT (q_c) to the undrained shear strength of soil (s_u). In this regard, Equation 13 has been used for this conversion.

$$s_u = \frac{q_c - \sigma_0}{N_K} \qquad (13)$$

In these equations, σ_0 is the total vertical stress, and N_K is in the range of 11 to 19 [1]. According to Mayne and Kemper (1988) [16], the value of N_K for electric cone is 15. Also Based on Anagnostopoulos et al. (2003) [17] field tests in Greece, the value suggested for N_K is 17.2. This study uses the value of 15 for N_K.

In order to compare each method, a series of analyses has been performed with the pile length and diameter of 10 m and 0.8 m, soil unit weight of 18 kN/m^3, Poisson's ratio of 0.45, and undrained shear strength of 25, 50 100, 150 and 200 kPa. O'Neil and Reese (1999) [2] recommended associated values of soil elastic modulus of 4, 22.5, 75, 124 and 180 MPa respectively. This study uses their recommendations.

Figures 4 and 5 show the comparison of the results of tip resistance obtained in this study with relationships based on geotechnical parameters and CPT, respectively. These Figures show that the results of equations proposed by Vesic (1977) and Philipponnat (1980) are close to the result of present study. Also, it should be noted that the correlation (Equation 13) used to correlate the values of undrained shear strength to q_c have an important role in the results shown in Figure 5. It is evident that different correlations can lead to different results for undrained shear strength which in turn will affect the results of the comparisons in this approach. Also, Figures 6 and 7 show the comparison of the results of side resistance obtained in this study with relationships suggested by other researchers which are based on geotechnical

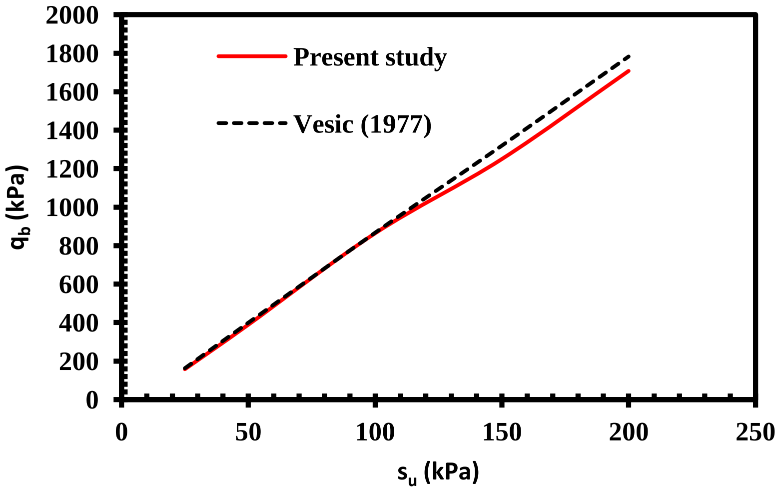

Figure 4. Comparison of tip resistance obtained in this study with relationships suggested by other researchers which are based on soil geotechnical parameters

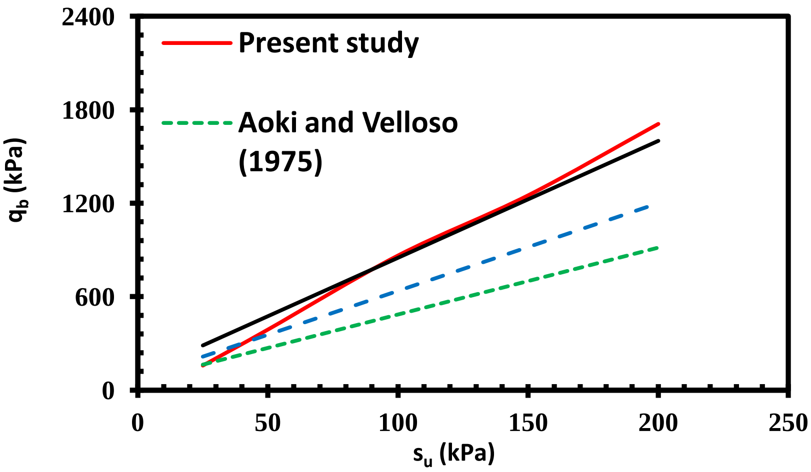

Figure 5. Comparison of tip resistance obtained in this study with relationships suggested by other researchers which are based on CPT results

parameters and CPT, respectively. These figures show that the results of present study are close to the equation proposed by O'Neil and Reese (1999). Figure 7 shows that Aoki and Velloso (1975) propose lower values of the side resistance.

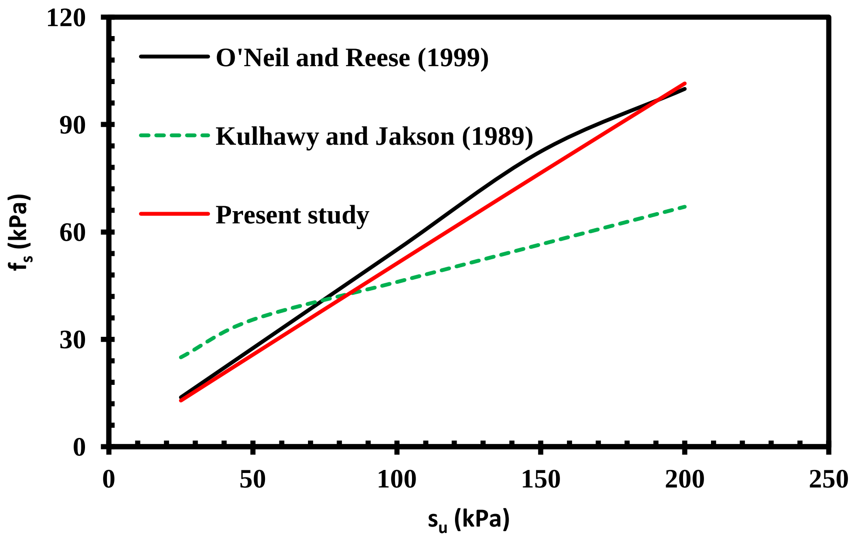

Figure 6. Comparison of side resistance obtained in this study with relationships suggested by other researchers which are based on soil geotechnical parameters

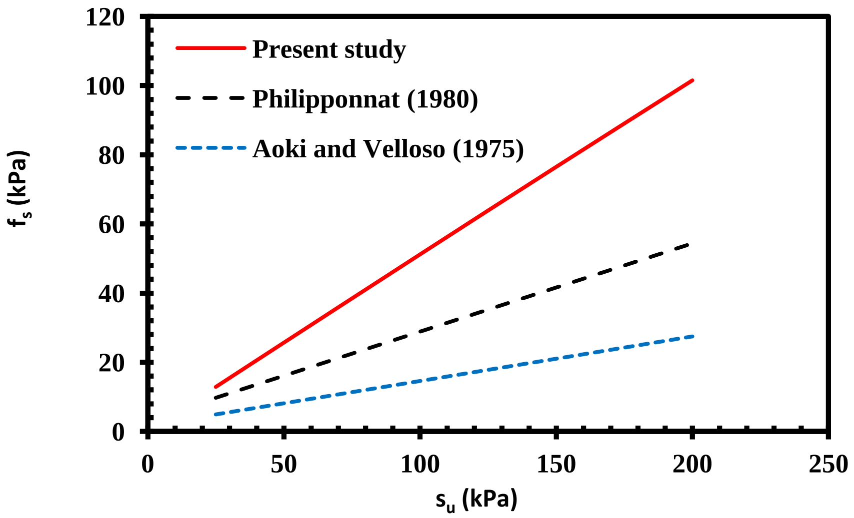

Figure 7. Comparison of side resistance obtained in present study with relationships based on CPT

Sensitivity analyses

In order to study the effects of pile and soil parameters on side resistance, the values reported in Table 3 are considered as initial parameters. In each analysis, only one of these parameters has been changed. Figures 8 to 19 show the results of sensitivity analyses. In these figures, the value of tip resistance obtained in this study has also been compared with Vesic (1977), and the value of side resistance has been compared with O'Neil and Reese (1999).

Table 3. Initial parameters of pile and soil used for sensitivity analyses

L (m)	D (m)	s_u (kPa)	ɣ (kN/m^3)	E_s (MPa)	v
10	0.8	100	18	75	0.45

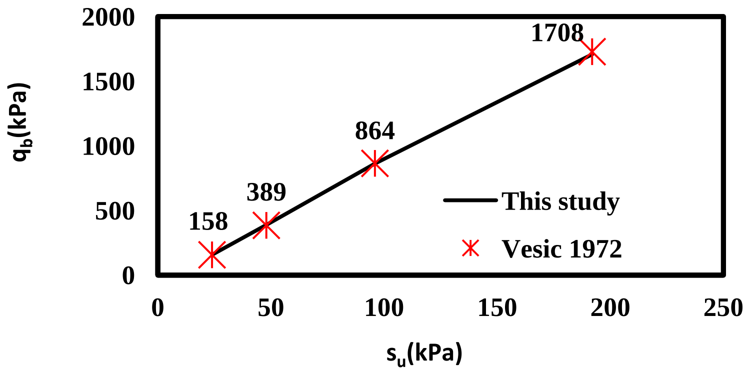

Figure 8. Effect of soil undrained shear strength on tip resistance

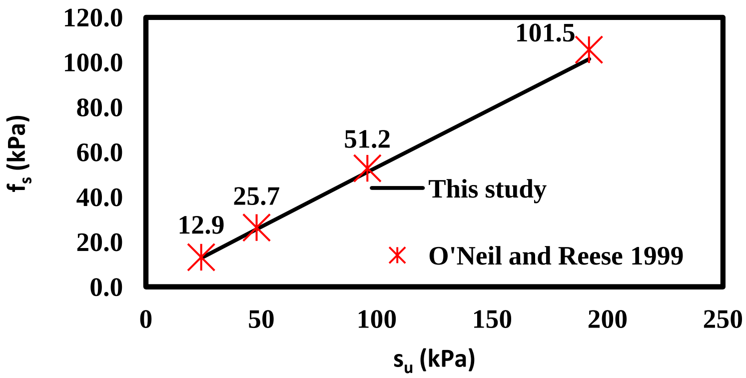

Figure 9. Effect of soil undrained shear strength on side resistance

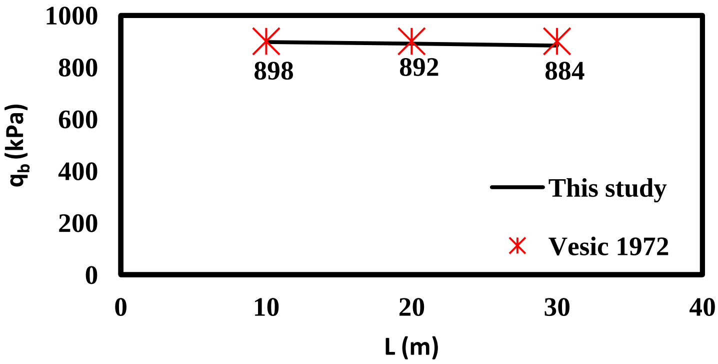

Figure 10. Effect of pile length on tip resistance

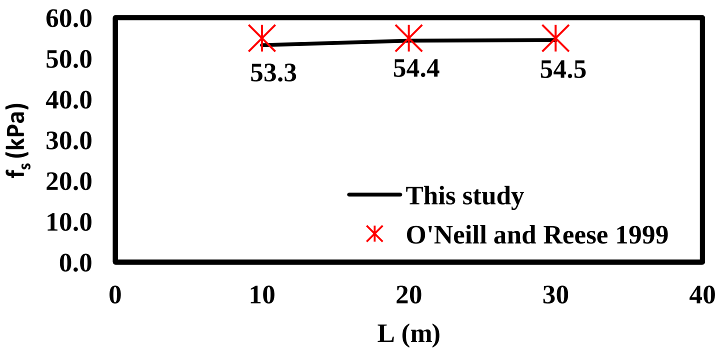

Figure 11. Effect of pile length on side resistance

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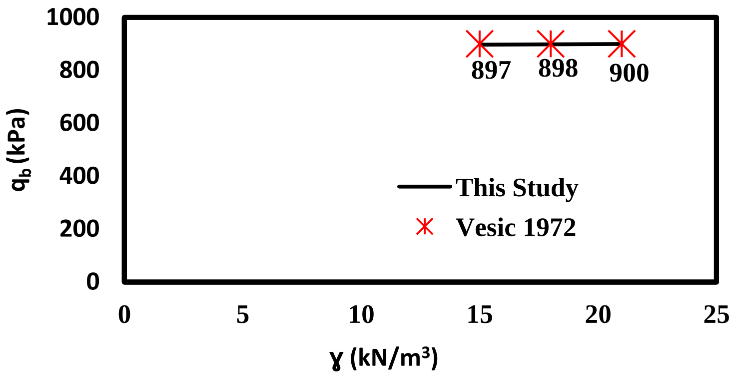

Figure 12. Effect of soil unit weight on tip resistance

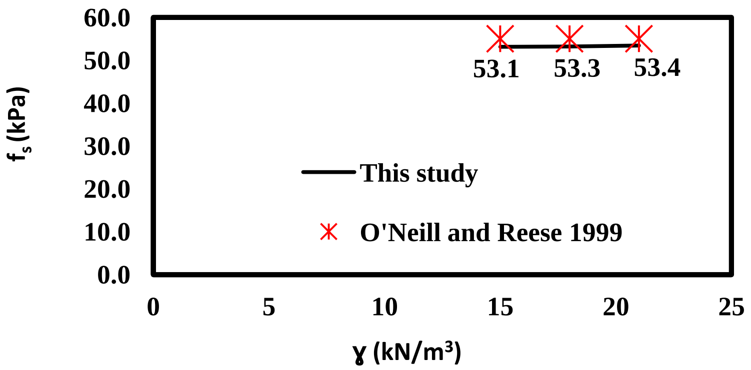

Figure 13. Effect of soil unit weight on side resistance

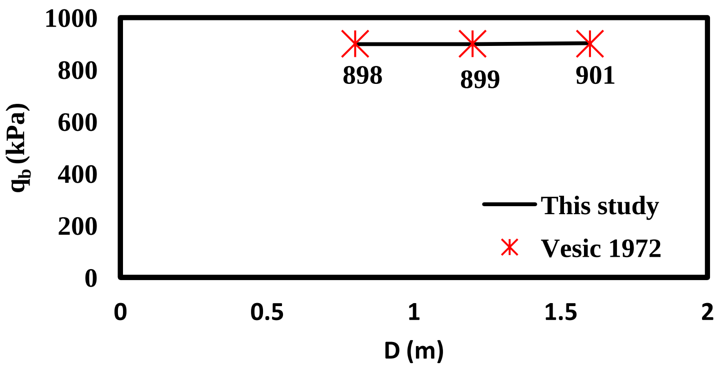

Figure 14. Effect of pile diameter on tip resistance

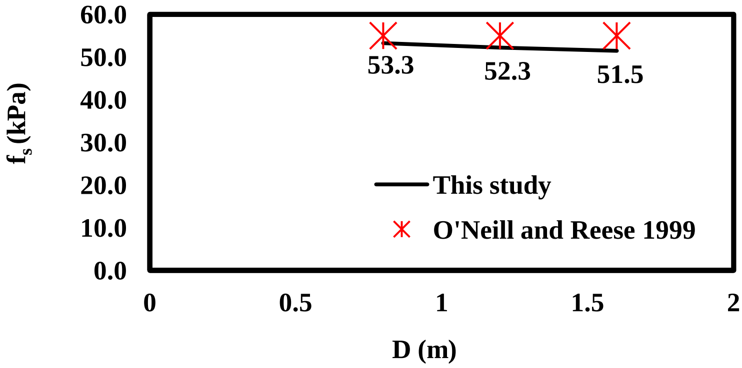

Figure 15. Effect of pile diameter on side resistance

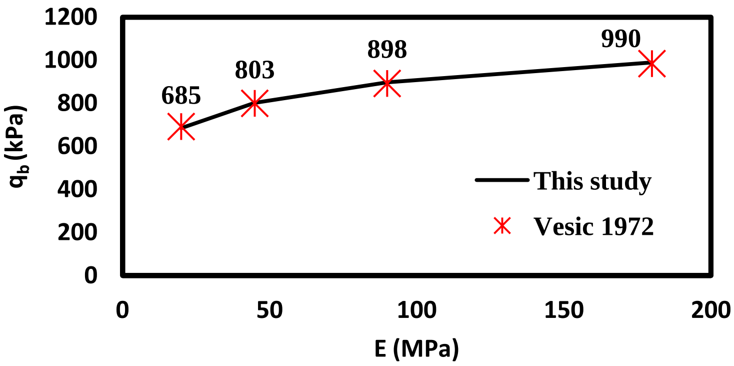

Figure 16. Effect of soil elastic modulus on tip resistance

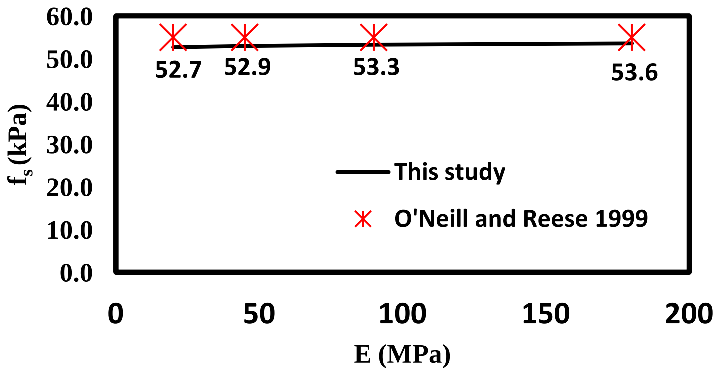

Figure 17. Effect of soil elastic modulus on side resistance

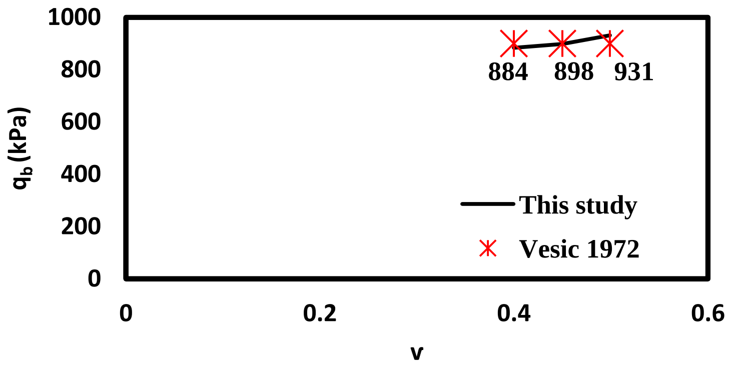

Figure 18. Effect of soil Poisson's ratio on tip resistance

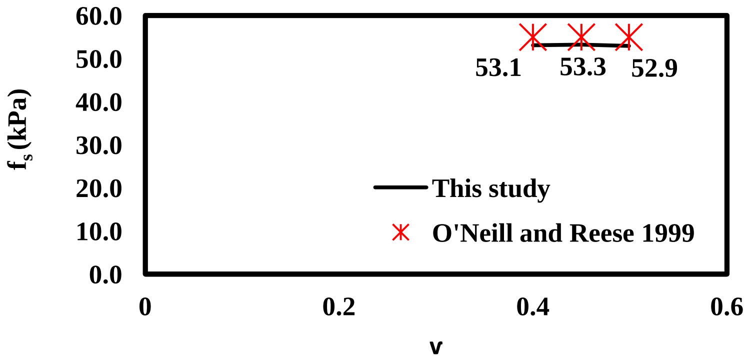

Figure 19. Effect of soil Poisson's ratio on side resistance

These figures show that the results of Vesic (1977) and O'Neil and Reese (1999) are close to this study. Also, undrained shear strength has major effect on side resistance, and the most effective parameters of clay on tip resistance are undrained shear strength and elastic modulus. Therefore, the coupling effect of undrained shear strength and elastic modulus is investigated in the following section.

Coupling effect of undrained shear strength and elastic modulus of clay on tip resistance

In this section, some analyses with the undrained shear strength of 25, 50, 100, 150 and 200 kPa with associated elastic modulus of 5, 20, 75, 125 and 180 MPa has been performed to investigate the coupling effect of undrained shear strength and elastic modulus of clay on tip resistance. Figure 20 shows the results of these analyses. In these analyses, pile length and diameter are 10 m and 0.8 m, respectively. The effects of soil undrained shear strength in each elastic modulus with the change of undrained shear strength from 25 to 200 kPa are 59.5%, 80.5%, 82.9%, 83.4% and 83.6%, respectively. These results indicate that the minimum effect of soil undrained shear strength. occurs around the elastic modulus of 5 MPa (60%). The effect of soil undrained shear strength. on tip resistance is approximately constant (about 83% for a change of soil undrained shear strength between 25 to 200 kPa) for the range of elastic modulus between 20 and180 MPa.

According to Equation 1, the value of tip resistance must be divided into the value of undrained shear strength to determine the value of N^*_c. With this consideration, Figure 20 is converted to Figure 21. This figure shows that the value of N^*_c increases with an increase of elastic modulus and a decrease of soil undrained shear strength. In this regard, a suitable parameter for determination of N^*_c is the soil rigidity index ($\frac{E_s}{3s_u}$) (Equation 3). With this assumption, Figure 22 can be considered to investigate the effect of rigidity index on tip resistance. This figure also shows the best curve that could be fitted on these data. According to this best curve, the authors recommend Equation 14 to estimate the value of N^*_c.

$$N_c^* = 1.55 \times \mathrm{Ln}\left(\frac{E_s}{3s_u}\right) \quad (7)$$

Equation 14 should be compared with Equation 2 proposed by Vesic (1977). Therefore, some cases with different values of undrained shear strength and elastic modulus have been considered in Table 4. Table 4 also is the comparison of N^*_c proposed in this study with N^*_c proposed by Vesic (1977). This Table shows that the value of N^*_c proposed in this study are close to the values obtained from Vesic (1977) suggestion. The agreement in this comparison is improved for higher values of undrained shear strength.

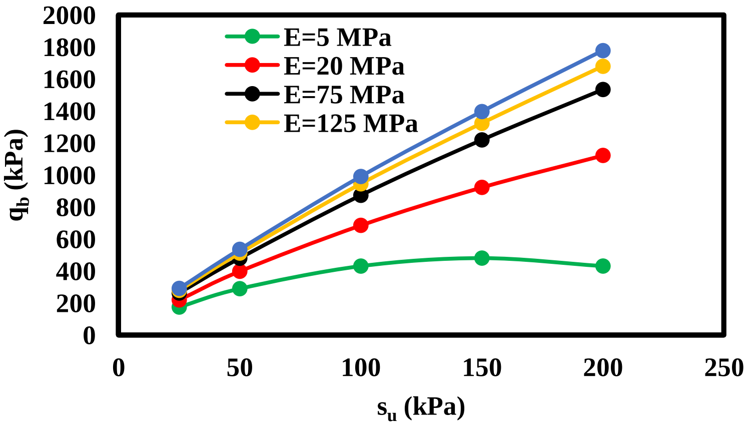

Figure 20. Coupling effect of soil undrained shear strength and elastic modulus on tip resistance

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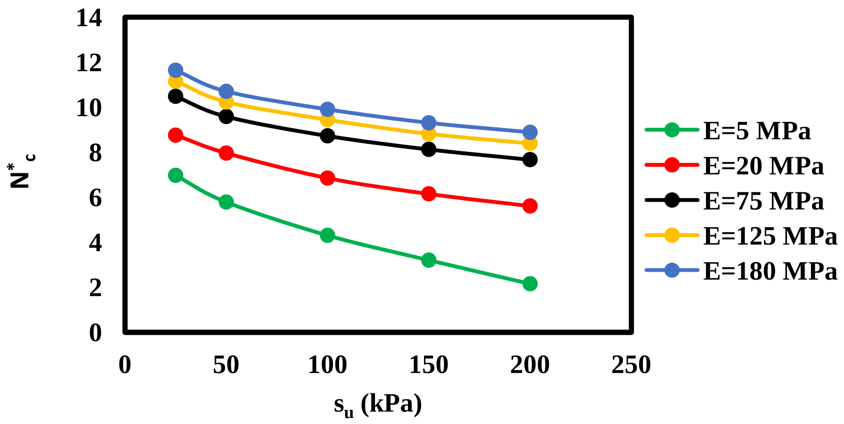

Figure 21. Coupling effect of soil undrained shear strength and elastic modulus on the factor of N^*_c

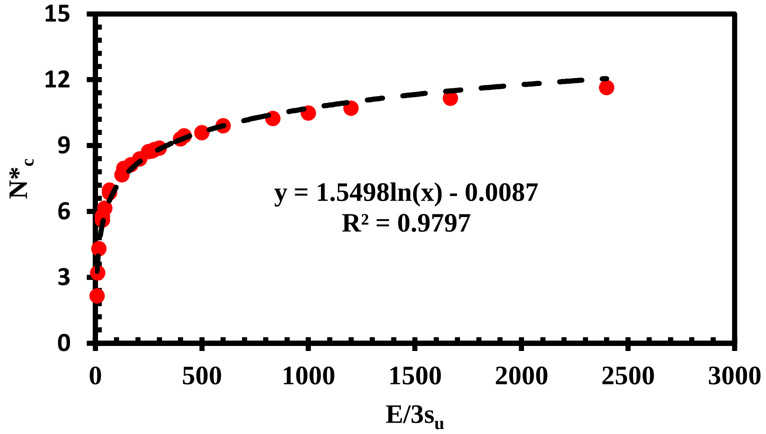

Figure 22. Best curve for values of N^*_c

Table 4. Comparison of N^*_c proposed in this study with N^*_c proposed by Vesic (1977)

s_u (kPa)	E (MPa)	N^*_c Vesic (1977)	N^*_c This study	Error (%)
25	3.75	6.5	6.1	7.18
50	22.5	8.0	7.8	2.85
100	75	8.7	8.6	1.33
200	180	8.9	8.8	0.84

Conclusions

A numerical investigation is performed to study the bearing capacity of drilled shafts embedded in clay. The following specific conclusions can be drawn from this study:

1- Among all the relationships which can estimate the tip resistance of drilled shaft in clay, the relationships suggested by Vesic (1977) and Philipponnat (1980) have the closest results to this numerical study.

2- Among all the relationships which can estimate the side resistance of drilled shaft in clay, the relationship suggested by O'Neil and Reese (1999) has the closest results to this numerical study.

3- Sensitivity analyses were performed in this study, and the results show that undrained shear strength has major effect on side resistance, and the most effective parameters of clay on tip resistance are undrained shear strength and elastic modulus.

4- The effects of soil undrained shear strength in each elastic modulus with the change of undrained shear strength from 25 to 200 kPa are 59.5%, 80.5%, 82.9%, 83.4% and 83.6%, respectively. These results indicate that the minimum effect of soil undrained shear strength occurs around the elastic modulus of 5 MPa (60%). The effect of soil undrained shear strength on tip resistance is approximately constant (about 83% for a change of soil undrained shear strength between 25 to 200 kPa) in the elastic modulus of 20 to 180 MPa.

5. A new relationship to estimate the bearing capacity factor of N^*_c was recommended in this study. ($N_c^* = 1.55 \times \mathrm{Ln}(\frac{E_s}{3s_u})$). The

results of this relationship is close to the values obtained from Vesic's (1977) equation.

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