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2D and 3D Numerical Simulation of Load-Settlement Behaviour of Axially Loaded Pile Foundations

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Abstract Reliable prediction of settlement behaviour of axially loaded piles is one of the major concerns in geotechnical engineering. Therefore, this paper focuses on the finite element solutions of load-settlement behaviour of a single pile and pile group using PLAXIS numerical package. Three different types of analysis were incorporated: a linear elastic analysis, a complete nonlinear analysis and a combined analysis. The pile case history with settlement measurements made during field pile load test was considered to validate the single pile load-settlement simulation, and the same load test result was extended to simulate the load-settlement behaviour of pile group using RATZ analytical approach. The single pile analysis results suggest that realistic load-settlement predictions can be drawn by considering complete soil as Mohr-Coulomb model at lower working loads, and incorporation of an interface zone thickness of two times pile diameter using Hardening-Soil model is required to simulate the load-settlement behaviour at higher working loads. The group pile analysis results provide a better load-settlement prediction when incorporating an interface zone thickness of pile dimeter from the pile shat using Hardening-Soil model while leaving the remaining soil as Linear-Elastic material.

Keywords: *Hardening-Soil, Interface zone, Linear-elastic, Mohr-Coulomb, nonlinear, RATZ approach*

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1. Introduction

Nowadays, lot of attention has been paid on axially loaded pile foundations to construct Civil Engineering infrastructures due to their high load bearing capacity and applicability in various kinds of geological scenarios. As settlement plays a controlling role in the design of pile foundations, number of studies [1-23] have been undertaken by several researchers to capture the exact solutions for settlement. All these methods for the settlement analysis of pile foundations can be classified under three popular categories including (i) Experimental Method, (ii) Analytical methods and (iii) Numerical methods. Few researchers [1,2,3] have conducted experimental study on the settlement behaviour of axially loaded piles. Analytical method on pile group settlement and concept of 'Interaction effect' in pile groups have been introduced and pioneered by Poulos [4]. Continuously, many researchers [5-13] have conducted analytical study for the vertical deformation of pile foundations using different techniques such as theoretical load-transfer curves, discrete layer approach, elastic solutions, theory of pile-soil interactions and hybrid approaches. However, in this method, problems have often been solved by making number of simplifying assumptions regarding the geometry and material properties [14].

Nowadays, researchers intend to provide immediate and suitable solutions through numerical analysis for various field problems and those scenarios can be used for the same kind of field problem in the future as well. In the past decade, there was a rapid increment in developing finite element packages, and it has been concluded that the solutions obtained from finite element analysis are more reliable and accurate [9,15-23]. Zakia et al. [18] have conducted a finite element study on the effect of modelling parameters in settlement predictions using PLAXIS 2D. Based on their findings, they concluded the following: (i) modulus of interface is very much closer to modulus of the soil that in contact with pile, (ii) interface reduction factor provides good agreement when the value is between 0.8 and 0.9, and (iii) modelling the soil completely as Mohr Coulomb model with consideration of interface provides better settlement. Ju [20] has carried out settlement analysis using PLAXIS 3D finite element package for pile-group located in sleech strata. Based on the findings, it was concluded that a combination of nonlinear and linear elastic analysis (using Hardening-Soil model and Linear-Elastic) leads more realistic settlement predictions and a value of half of the pile width as the interface thickness would be sufficient to capture the load transfer mechanism of a pile group. Jian-lin et al. [21] have conducted a FEM study on settlement prediction of pile foundation in deep clayey soil deposit. They proposed a useful equation to correct the compression modulus obtained from laboratory test to simulate deep in-situ soil conditions. Alnuiam et al. [19] have developed a finite element model (FEM) using PLAXIS 3D package to study the settlement behaviour of pile in Toyoura sand and they concluded that Mohr-Coulomb model is the best material model to predict the settlement behaviour of pile in Toyoura sand. Fuchun et al. [17] have carried out settlement analysis of pile groups using PIGLET finite element package and it was concluded that finite element analysis is a best method to predict the settlement behavior of group piles with adequate consideration of soil nonlinearity and interaction effect.

However, reliable settlement prediction of pile foundations at typical working loads still remains as one of the major geotechnical engineering problems [3,20]. Also, selection of suitable material models and modelling parameters based on the geological conditions and loading conditions still remains as a huge challenge in numerical simulations of settlement behavior of pile foundations. Therefore, major aim of this study is to analyze the settlement behavior of the axially loaded piles numerically by accounting the realistic nature of the problem for the safe and economical design.

2. Adopted Pile Case History

Results of static pile load test conducted in Woodland, north western part of Singapore, is adopted for this research study. The stratigraphy of Woodland comprises mainly the silty sand deposits called Old alluvium. Old alluvium is one of the major stratigraphy of Singapore that covers about 15% of the total area of Singapore and consists predominantly silty sand with fines content of about 20% to 30% [24]. Site investigation programme including Standard penetration tests was carried out to provide the required engineering information and description of subsurface soil stratigraphy. The detail of the sub soil profile in the pile test location is given in Table 1. The angle of friction (Ø') and effective cohesion (c') values shown in Table 1 were obtained using the Standard Penetration Test (SPT) results based on the correlations developed for old alluvium [24]. The site investigation revealed that the ground water table is at a depth of one meter below the ground surface.

The tested single pile was precast circular concrete piles, 48 m long, driven to a depth of 47.5 m in the ground with a free-standing length of 0.5 m above the ground surface. Static load test was undertaken to the reference pile up to 21, 000 kN (300 % of the working load) after three months from the installation. In this research study, the load-settlement behaviour obtained from this static pile load test result was numerically simulated.

For the numerical simulation of settlement behavior of group pile, a hypothetical 2 × 2 square pile group with a symmetric arrangement (Figure 1(a)) was considered. The geometry of individual piles in the pile group and the stratigraphy (Figure 1(b)) were considered as similar to that considered in the single pile analysis. Critical design spacing of the pile group to be located in sandy soils is found to be four times the pile diameter [25] and hence, a spacing of four times the pile diameter was used in this study. Material properties of the pile and pile cap are summarized in Table 2.

Table 1. Summary of Engineering Information of subsurface soils

Layer	Depth (m)	Soil description	Moisture content (%)	Bulk density (kN/m^3)	Dry density (kN/m^3)	k_h (×10^{-8} m/s)	c' (kN/m^2)	Φ' / °	Elastics modulus (MPa)	Poisson's ratio (v_s)	Dilatancy angle (Ψ)/ °
1	0 – 2.4	Silty sand	22	20.3	16.6	18.8	5	32	21	0.3	5
2	2.4 – 5.4	Silty sand	22	20.3	16.6	18.8	10	34	22	0.3	5
3	5.4 – 8.4	Medium dense silty sand	22	20.3	16.6	18.8	10	34	28	0.3	5
4	8.4 – 14.4	Medium dense silty sand	18.2	20.7	17.6	6.4	10	34	28	0.3	5
5	14.4 – 26.4	Medium dense silty sand	16.3	20.3	17.6	3.8	10	34	42	0.3	4
6	26.4 – 41.4	Dense silty sand	16.3	20.3	17.6	3.8	15	34	56	0.3	3
7	41.4 – 44.4	Very dense silty sand	16.3	20.3	17.8	3.8	15	34	70	0.3	2
8	44.4 – 47.4	Very dense silty sand	16.3	20.3	17.8	3.8	15	34	63	0.3	2
9	Below 47.4	Siltstone	16.3	20.3	17.8	3.8	15	34	63	0.3	2

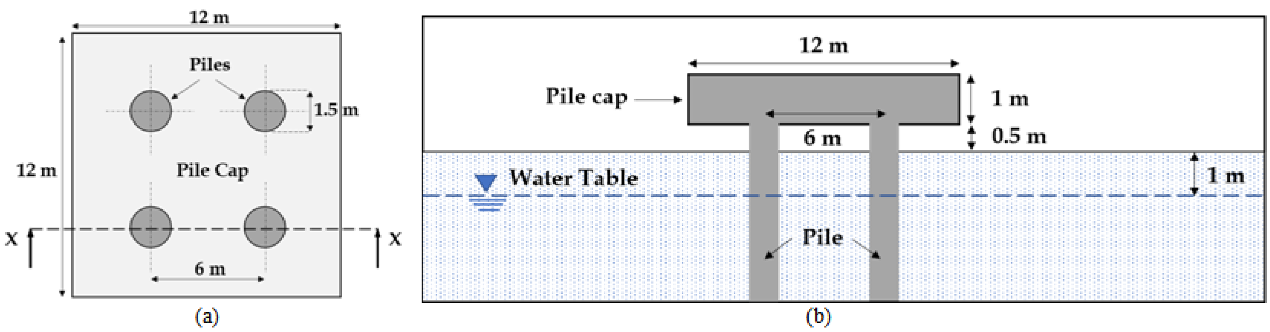


Figure 1. **(a)** Plan-view of 2 × 2 square pile group, and **(b)** Sectional view X-X

Table 2. Properties of pile and pile cap

Properties	Pile	Pile cap
Unit weight (kN/m^3)	24	24
Elastic Modulus (GPa)	30	30
Poisson's ratio	0.2	0.2

3. Research Methodology

3.1. Adopted Finite Element Material Models

Numerical simulation of settlement behavior of axially loaded single pile and group pile were investigated using PLAXIS 2D and PLAXIS 3D finite element packages respectively. In both investigations, three primary finite element material models were incorporated and they are (i) Linear Elastic (LE) model, (ii) Mohr Coulomb (MC) model and (iii) Hardening Soil (HS) model. Linear Elastic (LE) model is based on Hookes's Law of isotropic elasticity. It involves two basic elastic parameters i.e. Elastic Modulus (E) and Poisson's ratio (υ). Although the linear elastic model is not suitable to model the soil, it may be used to model the stiff volume of soil or stiff formulations in the soil. In this study, the piles and pile cap were modelled using LE material model. Mohr Coulomb (MC) model is one of the nonlinear models adopted in this research study. It is a simple nonlinear model which is based on soil parameters that are known in most practical situations. This involves five input parameters: Elastic modulus (E) and Poisson's ratio (υ) for soil elasticity, friction angle ($\emptyset$) and cohesion (c) for soil plasticity and dilatancy angle (ψ). However, all the non-linear features of soil behaviour are not included in this model.

The Hardening Soil model is an advanced nonlinear model adopted for the simulation of soil behaviour. It has been formulated in the framework of classical theory of plasticity. The total strains are calculated using a stress level dependent stiffness with a hyperbolic stress-strain relationship that defers for virgin load and unloading/reloading. In Hardening soil model, stiffness is described much more accurately by using three additional stiffness input parameters: triaxial loading stiffness (E_{50}), triaxial unloading stiffness (E_{ur}) and odometer loading stiffness (E_{oed}). It has been concluded that the values approximately satisfy the following equations: $E_{ur} = 3E_{50}$ and $E_{oed} = E_{50}$ [20,26]. Hence, these equations were used to find out the parameters required for HS model. All these stiffness values are specified at a given reference stress (p^{ref}) and are assumed to vary directly with the horizontal effective stress raised to the power of m. Generally, the reference stress (p^{ref}) at the stiffness measurement (laboratory level) falls between 100 - 200 kPa. The stiffness power in stiffness laws (m) lies between 0.5 and 1. Normally consolidated coefficient of earth pressure at rest (K_0) and the Poisson's ratio for unloading and reloading (v_{ur}) and the failure ratio (R_f) are also required input parameters for the Hardening Soil model. It is suggested that, values of v_{ur} of 0.2 and R_f of 0.9 are appropriate for the HS model at drained condition [13]. The best estimate parameters of each layers adopted for the HS model are summarized in Table 3.

3.2. Finite Element Analysis of a Single Pile

Finite Element Analysis of a single pile was performed using PLAXIS 2D numerical package in axisymmetric conditions with two degrees of freedom of translation per node. Three different types of FE analyses were considered in this single pile finite element study: (i) a LE analysis where all the soil was assumed to be linearly elastic, (ii) a Complete Non-Linear (CNL) analysis where soil was completely modelled using the HS or the MC model, and (iii) a combined analysis (NL-LE) where soil close to the pile shafts (interface zone) was modelled using the HS model, while soil in the remaining area was modelled considering it as either LE or MC material. Three different sizes of interface thicknesses were selected for the combined analysis: interface thickness of (i) D/2, (ii) D and (iii) 2D from the pile shaft, where D is the pile diameter.

The soil was modelled using triangular elements with 15 nodes with an elastoplastic law behaviour obeying the Mohr-Coulomb failure criterion. The lateral sides of the computational domain were taken far enough from the pile in order to avoid the boundary effect. Moreover, the models of single pile for different type of analysis were made using the same working area of 70 m × 25 m. At the bottom level of the model, all movements were restrained, whereas, at the lateral sides, lateral movements perpendicular to the boundary were prohibited. Default interface and interface strength reduction factor (R_{int}) were incorporated in the simulation and R_{int} value of 0.85 was adopted [18]. The size of the elements was to be as small as possible close to the pile shaft because of high stress gradient, which can capture better pile behaviour. The geometry of the Finite element model is shown in Figure 2(a). The undeformed and deformed FEM mesh are shown in Figure 2(b) and Figure 2(c) respectively. Prescribed displacement was applied at the pile head and variations of applied load with settlement were obtained. The computed results from all the three types of analysis were compared with the field test results in order to identify suitable model for the soil.

Table 3. Hardening Soil (HS) Parameters

Layer	Depth (m)	E_{50} (MPa)	E_{oed} (MPa)	E_{ur} (MPa)	p^{ref} (kN)	m	v_{ur}	R_f
1	0 – 2.4	15.6	15.6	46.8	100	0.7	0.2	0.9
2	2.4 – 5.4	16.3	16.3	49.0	100	0.7	0.2	0.9
3	5.4 – 8.4	20.8	20.8	62.4	100	0.7	0.2	0.9
4	8.4 – 14.4	20.8	20.8	62.4	100	0.7	0.2	0.9
5	14.4 – 26.4	31.2	31.2	93.6	100	0.7	0.2	0.9
6	26.4 – 41.4	41.6	41.6	124.8	100	0.7	0.2	0.9
7	41.4 – 44.4	52	52	156	100	0.7	0.2	0.9
8	44.4 – 47.4	46.8	46.8	140.4	100	0.7	0.2	0.9
9	Below 47.4	46.8	46.8	140.4	100	0.7	0.2	0.9

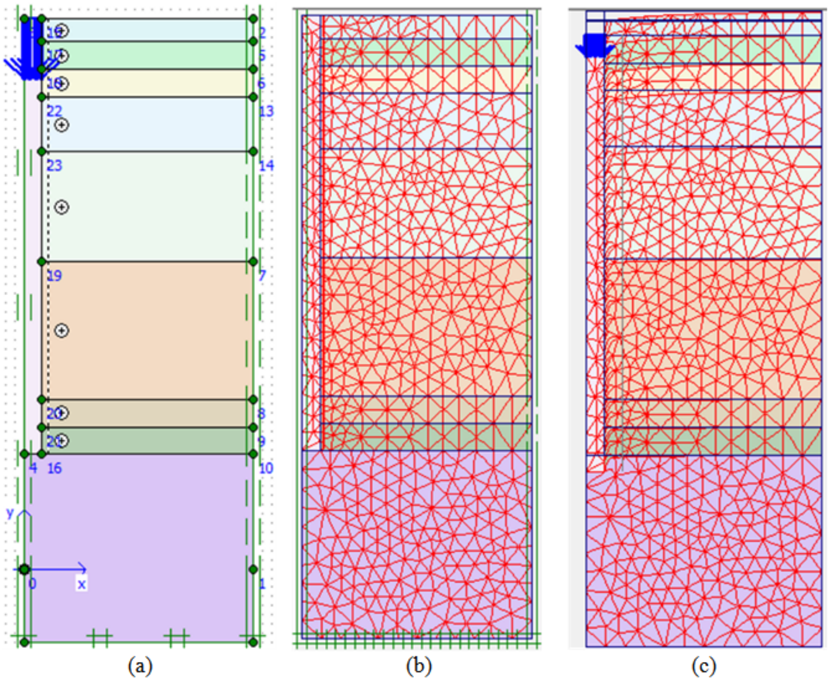


Figure 2. (a) Geometry of the Finite Element Model **(b)** FE mesh and **(c)** deformed mesh for complete LE analysis

3.3. Finite Element Analysis of a Pile Group

PLAXIS 3D Numerical package was used to perform the Finite Element study on settlement behaviour of axially loaded pile group. In this 3D analysis, the soil was modelled by 15-node wedge elements. Similar to the single pile analysis, three different types of FE analyses were performed in the numerical study including: (i) a LE analysis where both the soil adjacent to the pile shafts and between the piles were assumed to be linear elastic model, (ii) a complete nonlinear (CNL) analysis where both the soil adjacent to the pile shafts and between the piles were modeled using either HS or MC model (iii) a combined analysis (NL-LE/ NL-NL) where the soil close to the pile shafts (Zone A in Figure 3) was modelled using HS model, while the soil in the remaining area (Zone B in Figure 3) was modelled either as LE or MC material. Three different size of interface thickness for Zone A were selected for combined analysis: (i) Zone extended to a distance (d) of D from the pile shaft, (ii) Zone extended to a distance (d) of D/2 from the shaft and (iii) Zone extended to a distance (d) of D/4 from the shaft, where D is pile diameter.

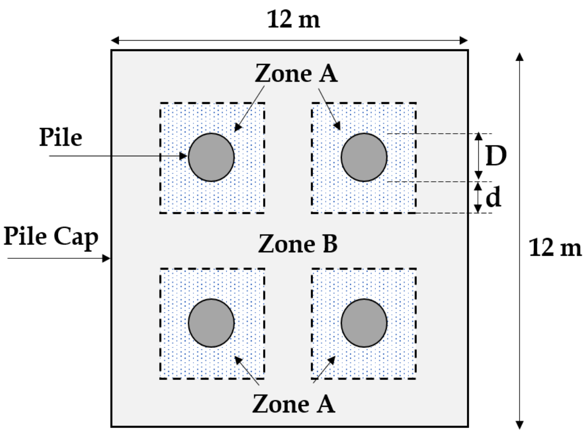


Figure 3. Pile group layout of combined analysis

The fine mesh analysis was undertaken in all over the numerical study. The meshed geometry and meshed pile foundation are shown in Figure 4. The models of pile groups for different type of analysis were performed using the same working area of 70 m × 25 m × 25 m. At the bottom level of the model all movements were restrained, whereas at the lateral sides, lateral movements perpendicular to the boundary were prohibited.

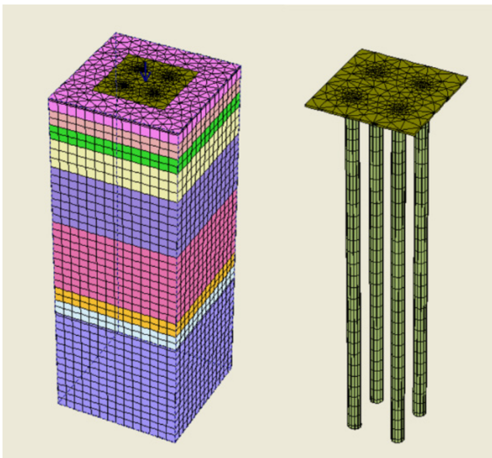


Figure 4. Meshed geometry of group pile foundation

Initially, the results of 3D finite element analysis were compared with well-established RATZ approach [11], and based on the outcome, the best finite element model was selected for any further analysis of settlement behaviour of pile group. Study on the effect of pile spacing on group load-settlement behaviour was also considered in this research study. In addition, settlement analysis was conducted for following three spacing (centre to centre): (i) S = 2D, (ii) S = 4D and (iii) S = 8D, where S is the center to center spacing and D is the pile diameter. Moreover, group settlement ratios and pile group efficiencies for the above models were determined and compared with previous studies.

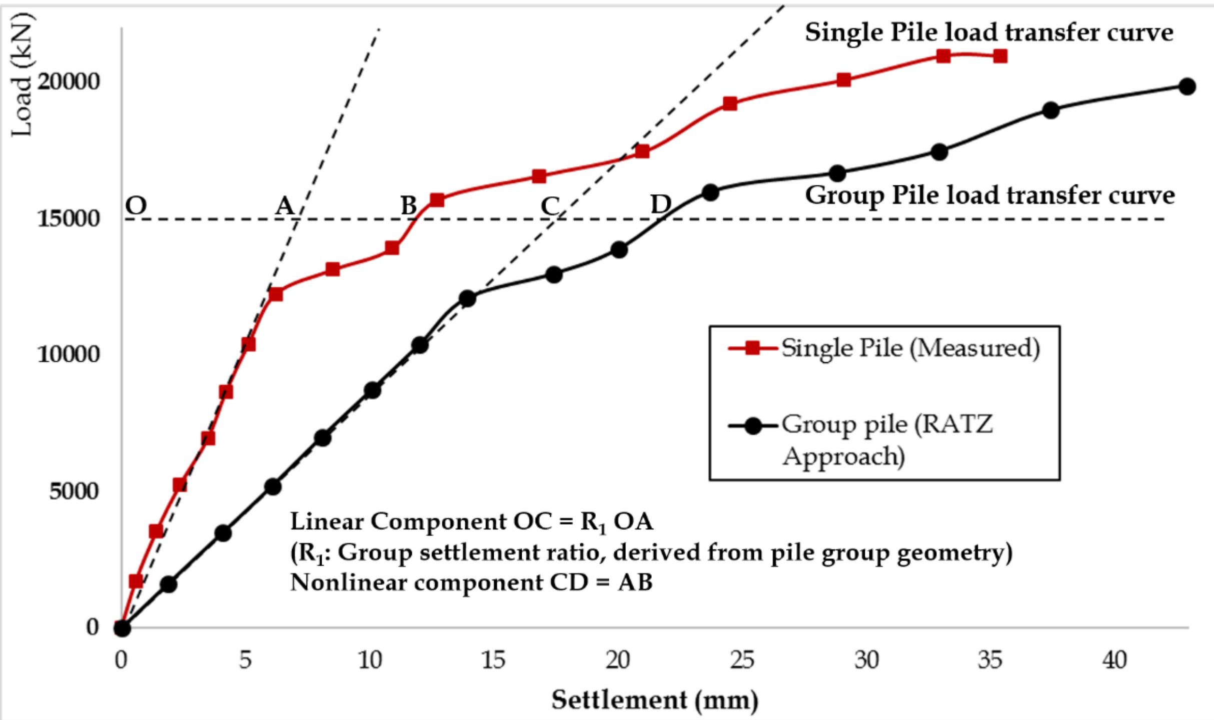


Figure 5. Load-settlement curve of the pile group obtained from RATZ approach

4. Results and Discussions

4.1. Load-settlement Behaviour of a Single Pile

Predicted load-settlement plots using complete LE, MC and HS analyses are shown in Figure 6 together with the measured field test results. Based on the field test results, at a typical working load of 7,000 kN, measured settlement of a single pile is 3.5 mm. For the same working load, settlement predicted from the HS model of FEM analysis is 8.2 mm and settlement values predicted from LE model and MC model for the same working load are almost the same, and it is 6.8 mm.

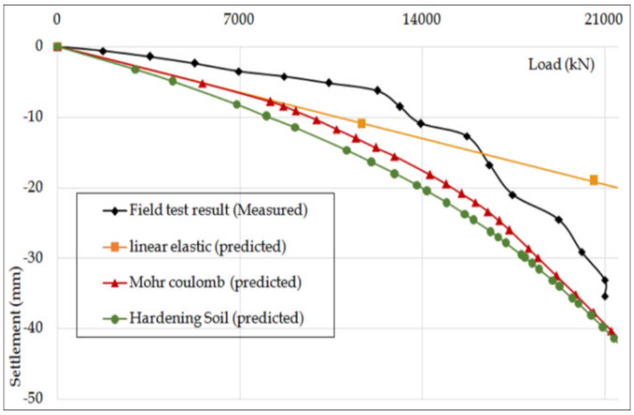


Figure 6. Comparison of load-settlement plots obtained from FEA and field test result for single pile analysis

There is a remarkable difference between the calculated and the measured settlement for the single pile, indicating the calculation significantly over predicted the pile head settlement or underestimated the pile head stiffness. The variation between the predicted and measured values is found to be due to the ignorance of deep-insitu effect in Modulus, as suggested by Jian-lin [21]. Actually, the Modulus obtained from the laboratory tests significantly vary from the in-situ modulus of deep soil and sometimes the difference is in the order of several times [21]. Generally, settlement of shallow foundations using soil compression modulus tested under pressure 100−200 kPa does not have too much error. However, calculated settlement values for deep piles are often associated with large differences in measured values [21] as deep in-situ soil stiffness is always higher than that of samples used to obtain the laboratory elastic modulus. Therefore, Equation [1] proposed by Jian-lin et al. [21] was used to increase the accuracy of the Elastic modulus in the settlement analysis.

$$E_{s,z} = E_{s,0.1-0.2}\left(z / h_0\right)^{1/\beta} \tag{1}$$

where z is the depth of soil layer (m), h_0 is reference depth (generally 1 m), $E_{s,0.1-0.2}$ is laboratory obtained compression modulus under the pressure of 100 - 200 kPa, and β is the plasticity of the soil depending on the liquid limit (LL) and plasticity index (PI) values from the data, from which the value of β can be obtained using the specifications of the BS code. All the FEM analyses were repeated with the corrected values of modulus and the obtained load-settlement results are shown in Figure 7.

According to the Figure 7, LE and MC models predict better prediction than the HS model below the working load of 13, 000 kN. However, beyond the load of 13,000 kN, LE and MC models fail to capture the real behaviour of settlement and underestimates the settlement, and it might be due to the ignorance of soil nonlinearity. On the other hand, HS model overestimates the settlement during the entire analysis due to its advanced nonlinearity.

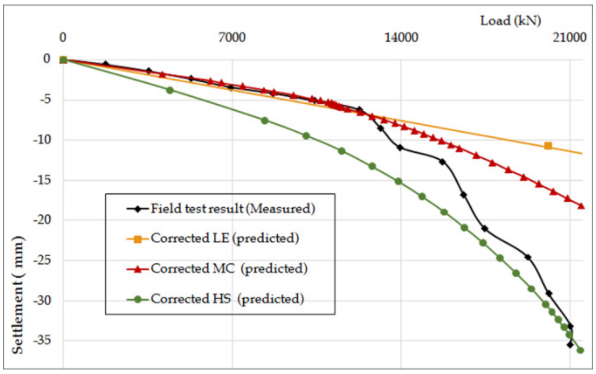


Figure 7. Comparison of load-settlement plots obtained from FEA and field test result for a single pile using corrected modulus

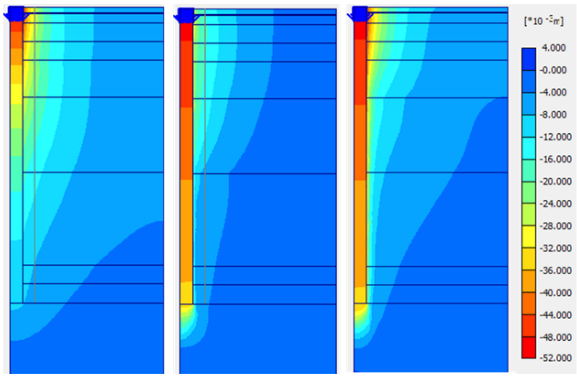


Figure 8. Settlement contours of soil obtained when it was modeled as complete (a) LE model, (b) MC model and (c) HS model

Figure 8 shows the settlement contours obtained for different analyses. It can be seen that linear settlement contours appear when soil is modelled as complete LE (Figure 8a) and this confirms the fact that the LE model disregards the nonlinearity of soil. When soil is modelled as complete MC model (Figure 8b), a few nonlinear contours are obtained in the settlement zone and this simple nonlinearity is adequate to predict the settlement behaviour at lower working loads up to about 13,000 KN. Zakia et al. [18] also have concluded that MC model is good enough to predict the load-settlement behaviour at lower working loads. On the other hand, when soil is modelled as complete HS model (Figure 8c), a large zone of settlement compared to that obtained from the MC model (Figure 8b) appears, showing clearly the advanced nonlinear contours. Modelling the soil completely using HS model leads for the underestimation of the soil stiffness everywhere due to its advanced nonlinearity and thus, complete HS model simply over predicts the settlement of pile.

Therefore, it is very clear that, by using a combination of Nonlinear and Linear (HS-LE) models and Nonlinear and simple Non-Linear (HS-MC) models, it would be possible to predict realistic load-settlement behaviour of settlement single pile. Results obtained from HS-LE analysis and HS-MC analyses (where soil within the interface thickness was considered as HS model) are presented in Figure 9 and Figure 10 respectively.

It can be seen that combined analysis using HS-LE models fail to provide an accurate prediction and the results appear to be related only to the average prediction. However, a combined analysis using HS-MC models provides a better prediction when the load is more than twice the working load. In this case in which a HS-MC model has been used, a HS zone of twice the diameter of the pile shaft with the remaining zone by the MC model, shows adequate agreement with the field test result than any of the other models considered.

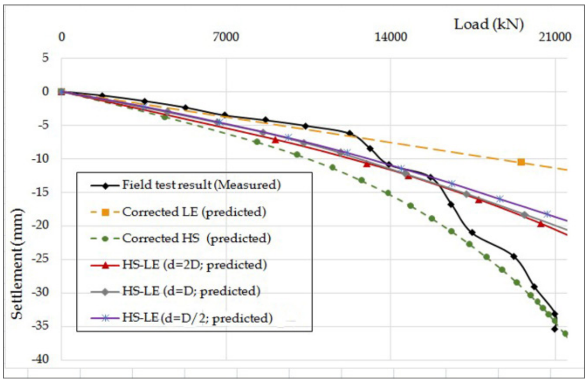


Figure 9. Comparison of load-settlement behaviour obtained from HS-LE analysis and measured Field Test result

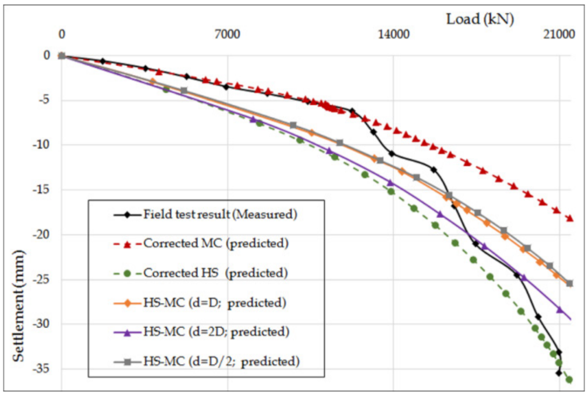


Figure 10. Comparison of load-settlement behaviour obtained from HS-MC analysis and measured Field Test result

4.2. Load settlement Behavior of Pile Group

Predicted load-settlement plots using complete LE and NL analyses and their comparison with the pile group load-settlement curve obtained from RATZ analytical approach are shown in Figure 11. For comparison purposes, curves are plotted as the mean pile head load (defined as the load applied on the rigid cap divided by the number of piles in the group) versus pile head settlement for pile group.

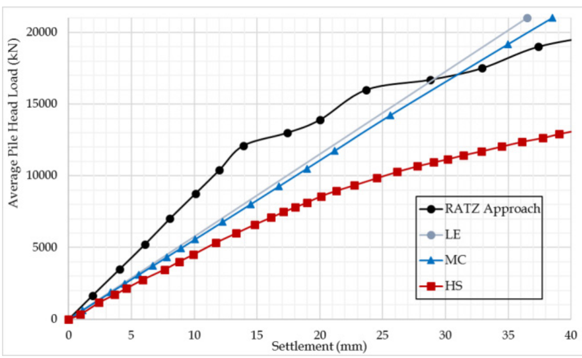


Figure 11. Comparison of load settlement behaviour of pile group from linear and non-linear FE analysis and RATZ analytical approach

Based on the results (Figure 11), it is obvious that there is a significant difference between the calculated and the RATZ prediction curve for the settlement of pile group, indicating the Finite Element results significantly over predicted the pile head settlement or underestimated the pile head stiffness. The contrasts between the RATZ prediction and Finite Element analysis are because of the ignorance of deep in-situ effect in modulus of soils. Therefore, Equation proposed by Jian-lin [21] was adopted to increase the accuracy of the settlement prediction as similar to that illustrated in the single pile load-settlement analysis.

Therefore, FE analyses were repeated using the corrected modulus calculated using the Equation (Equation [1]) proposed by Jian-lin [21]. Figure 12 shows the computed settlement results from all the three types of models (LE, MC and HS) and their comparison with the RATZ prediction.

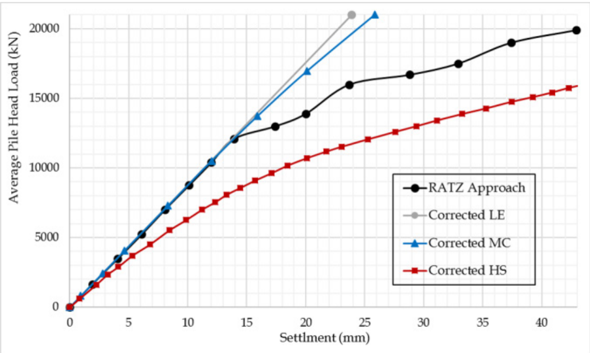


Figure 12. Comparison of load settlement behaviour of pile group from linear and non-linear FE analysis with corrected modulus and RATZ analytical approach

Based on the comparison between RATZ prediction and Finite element analysis results (Figure 12), it can be noticed that the prediction of the pile head settlement is improved. Although, LE and MC models show better agreement in settlement of pile group with RATZ prediction at lower working load (< 12,000 kN), they failed to predict a better behaviour at higher working load (> 12,000 kN). MC model is a simple nonlinear model in which all the nonlinear parameters of the soil are not incorporated, and hence MC model is only adequate to predict a limited level of settlement behaviour (up to 12,000 kN) accurately.

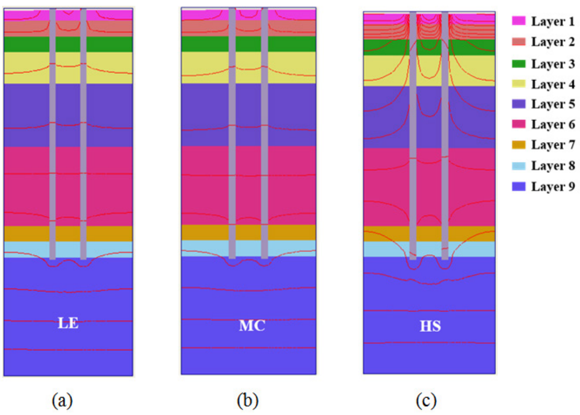


Figure 13. Settlement contours of the considered pile group from (a) LE, (b) MC and (c) HS analysis

Although the HS model over predicted the settlement or under estimated the stiffness, it can capture the nonlinear behaviour, which is the precise behaviour of soils.

The settlement contours obtained from the Finite element analysis using LE, MC and HS model are given in the Figure 13. As the nonlinearity affects the interaction of the pile group, LE and MC models fail to predict the settlement behaviour of pile group (Figure 13a and Figure 13b). The soil nonlinearity and the interaction effects are clearly captured in the HS model as shown in Figure 13c and incorporation of both nonlinear effect leads to real prediction of the settlement behaviour of group piles.

However, the calculated Finite Element results provides only a limited level of agreement with the analytical RATZ results, and based on the observations, it can be deduced that combined models can better predict the load-settlement of pile group. In reality, it is now well-recognized that the modulus of a soil mass decreases with increasing strain level. For a group of piles, it would be expected that the strain level will increase as the pile shaft is approached and hence the stiffness of the soil at this narrow zone close to the pile shaft is smaller than that between the piles at some distance from the pile shaft.

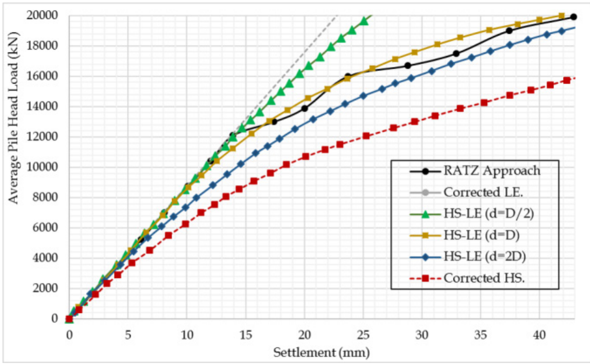


Figure 14. Comparison of load settlement behaviour of pile group from HS-LE analysis and RATZ analytical approach

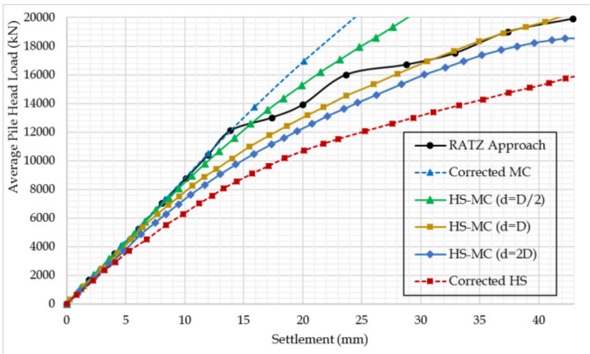


Figure 15. Comparison of load settlement behaviour of pile group from HS-LE analysis and RATZ analytical approach

Therefore, to account this stiffness variation, two types of combined analysis were considered: (i) HS-LE and (ii) HS-MC (where soil within the interface zone was considered as HS model). Results of the settlement of pile group using HS-LE and HS-MC for different interface thicknesses are given in Figure 14 and Figure 15 respectively. Based on the results from HS-LE finite element results (Figure 14), considering a nonlinear interface thickness (d) equal to the pile diameter (D) while keeping the remaining zone as LE, is sufficient to predict the load-settlement behaviour of pile group. At the average typical working load of 7,000 kN, HS-LE (d = D) provides the settlement prediction of 8 mm, while the settlement predicted from RATZ approach is 8 mm. Moreover, at the average ultimate load of 20,000 kN, settlement predicted using HS-LE (d = D) Finite element model is 42 mm, whereas the RATZ analytical approach predicts the settlement of 43 mm at the same load.

Similarly, based on the observations from HS-MC finite element results (Figure 15), considering an interface thickness equal to the pile diameter (d = D) from the pile shaft for non-linear Hardening-Soil (HS) model while keeping the remaining zone as Mohr-Coulomb (MC) model, provides a better agreement with RATZ analytical approach results. At the average typical working load, the settlement predicted by the HS-MC (d = D) is 8.5 mm, while the similar settlement from RATZ prediction is 8 mm. Also, at the average ultimate load of 20,000 kN, the settlement prediction of the above Finite element model is 42 mm, while the settlement from the RATZ approach is 43 mm. Combined model considering the nonlinear interface thickness of half the pile diameter (d = D/2) under predicts the settlement, while using a nonlinear interface thickness equivalent to two times pile diameter (d = 2D) over predicts the group pile settlement. Reason for the under estimation is due to the inadequacy of the zone considered as HS model, where pile-soil interaction takes place. The over estimation of settlement is due to the incorporation of larger zone as HS model, which actually reduces the stiffness of the soil in a larger area unnecessarily.

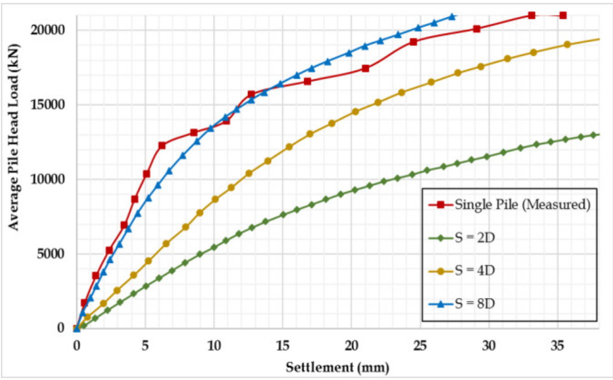
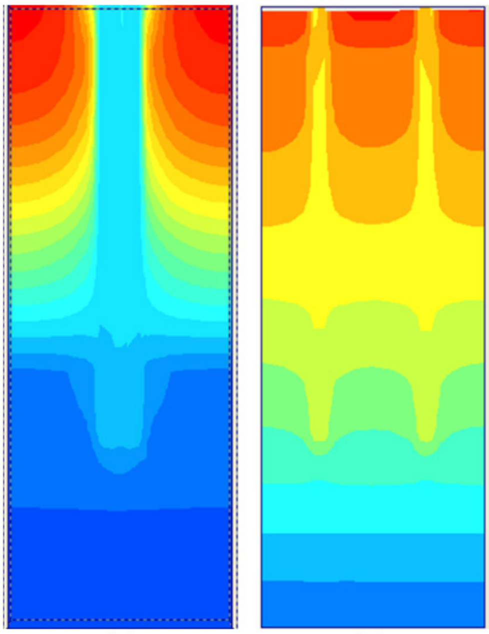


Figure 16. Load-settlement relationships of pile group with different spacing

Moreover, a study on the effect of pile spacing on group load-settlement behaviour was conducted and the results obtained from the analysis are given in Figure 16. The predicted settlements for the average working load of 7,000 KN for S = 2D, S = 4D and S = 8D are 13 mm, 8 mm and 4 mm respectively. Based on the results, the settlement of the pile group decreases when the spacing increases. When the spacing is increased by four times, the settlement decreases to approximately one third.

From Figure 16, it can be stated that the pile group stiffness decreases as the pile spacing decreases. The stiffness of the pile group with 4D spacing is 1.5 times higher than the stiffness of pile group with the spacing of 2D. Also, the stiffness of pile group with spacing 8D is more than 2 times of that of pile group with the spacing of 2D. This result is also supported by Pressley and Poulos [27] and it has been shown that, at closer spacing, the block failure mechanism occurs (Figure 17a) with significant plastic zones being developed below the group and full pile-slip is developed only along the outer piles. Based on the results (Figure 17), when the spacing goes to the value of eight times diameter (8D), group settlement behaviour reaches closer to the settlement behaviour of that of single pile. As the pile spacing increases, the failure mechanism gradually changes to the "single pile" mode (Figure 17b), where full pile-soil slip occurs along whole individual pile.



(a) (b)

Figure 17. Vertical deformation contours of pile group with the spacing of **(a)** 2D and **(b)** 8D

5. Conclusions

Numerical simulation of load-settlement behavior of axially loaded single pile and pile group was simulated using PLAXIS 2D and PLAXIS 3D finite element packages. Based on the findings of this study, the following conclusions can be drawn;

1. For settlement prediction of a single pile, Mohr Coulomb (MC) model is the best soil model to predict the realistic behaviour at lower working load (<13,000 kN). At the same time, considering a combined non-linear model, where hardening soil (HS) model is used within an interface thickness of two times the pile diameter while keeping the remaining soil as MC, will better predict the load-settlement behaviour at higher working load (>13,000 kN).
2. To predict the load-settlement behaviour of pile group, HS model should be used within the interface zone with the thickness equals to the pile diameter from the pile shaft while keeping the remaining zone as LE or MC would be sufficient.
3. When numerical simulations are done to predict settlements realistically, the understanding and the selection of the proper material model is very

important. Material model selection will not only depend on the characteristics of soil, but also will depend on the loading conditions of the structure.

4. Stiffness of the pile group decreases as the spacing of the piles in a pile group decreases. At the closer spacing, block failure mechanism is developed with full pile-soil slip being developed only along the outer face of the outer piles while widespread plastic zones form beneath the entire group. As the pile spacing increases, the amount of pile-soil slip along the inner piles increases and the region below the piles in which plasticity develops diminishes. Also, at a spacing of about eight times diameter (8D), failure of the pile group is governed by the failure of a single pile.

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