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NUMERICAL SIMULATION OF SETTLEMENT BEHAVIOUR OF AXIALLY LOADED PILES USED FOR HIGH-RISE BUILDING

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Abstract: The reliable prediction settlement of pile foundation at typical working load remains one of the major geotechnical engineering problems. In this research, settlement behaviour of a pile foundation located in sandy-silt, under the loads from high-rised building is simulated in 2D using a finite element program (PLAXIS). Three different types of analysis were investigated: a linear elastic (LE) analysis where the soil was assumed as linear-elastic material, a simple nonlinear analysis where the soil was completely assumed as Mohr-coulomb (MC) model and an advanced nonlinear analysis where the soil was completely assumed as Hardening-Soil (HS) model. A comparison was done between the predicted settlement from Finite element analysis and field settlement values. Based on the results of analysis, it is suggested that although complete MC model shows good agreement with the settlement behaviour obtained from field static load test at lower working loads, MC model is not adequate to capture the settlement prediction at higher working loads. In addition, modelling the soil completely using HS model is required to capture the safe settlement prediction at higher working loads. Finally, this scenario can be applied for the similar problems in settlement prediction using numerical methods.

Keywords: hardening-soil model; linear-elastic; mohr-coulomb model; settlement; axially loaded piles;

1. Introduction

The settlement of pile group is the controlling factor in design because the primary purpose of pile foundation is to limit the deformation of structures [1, 2, 3]. In present days, heavy high raised buildings and transportation structures such as highway and bridges are being constructed on pile foundations. Demand of the axially loaded piles has been increasing with the time due to their high bearing capacity and applicability in various kind of geological strata.

In past decades, a variety of analytical, experimental and numerical based studies have been adopted by the researchers [4-7] to predict the actual settlement behaviour of pile foundation. Based on their outcome, a number of approximate solutions have been provided for different scenarios. McCabe and Lehane [4] have conducted a field experimental study on settlement behaviour of axially loaded driven piles in soft clay-silt. Horikoshi et al. [5] have developed laboratory based study on settlement behaviour of piles. Rational analyses of pile group displacements were pioneered by Poulos [6] who introduced the concept of 'interaction factors' in pile groups. Sonia and Desai [7] proposed a useful result based on various field tests for the ultimate point resistance and skin resistance depending on the type of the piles installed in the cohesion less soil. However, formulas described above have not been able to fit and predict the entire process of the "load-settlement" curves [3, 8]. Therefore, the intension towards the real prediction of the settlement of pile foundation moves to numerical based methods.

Numerical simulation is widely used in geotechnical engineering analysis to solve complex problems [9]. In the past decade, there was a rapid increment in developing finite element packages, and solutions obtained from finite element analysis would have been more suitable and accurate. Zakia et al. [9] have conducted finite element study about 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 between 0.8 and 0.9, and (iii) modelling the soil completely as Mohr Coulomb model with consideration of interface provides better settlement. Jun Ju [10] carried out settlement analysis using finite element approach in PLAXIS 3D for piles in sleech soil. Based on the findings, it was concluded that a combination of nonlinear and linear elastic analysis leads more realistic predictions of settlement than complete nonlinear analysis of soil. Jian-lin et al. [3] 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 and it was suggested to implement for deep soft soil at numerical simulations. Alnuiam et al. [11] have developed a finite element model (FEM) using PLAXIS 3D package to study the settlement behaviour of pile in Toyoura sand and proposed that Mohr Coulomb (MC) model is the best model to predict the settlement behaviour of pile in Toyoura sand.

Despite many previous studies in this area, the reliable prediction of settlement of piles at typical working load remains one of the major geotechnical engineering problem [4, 10]. As it is required to check the settlement behaviour of the axially loaded piles for the safe and economical design, major aim of this research study is to simulate the settlement behaviour of vertically loaded piles used for high-rised buildings

2. Pile case history

For this research study, pile load test results of north western part of Singapore (Woodland) is adopted. The stratigraphy comprised of Old alluvium that consists of sand, silt, and clay but predominantly silty sand with fines content of about 20% to 30% [12]. Old alluvium is one of the major stratigraphy of Singapore and it covers about 15% of the total area of Singapore [12]. A site investigation including Standard penetration tests was carried out to provide the required engineering information and description of subsurface soil and basic structure of stratigraphy. Standard penetration test results are summarized in Figure 1 and description of subsurface soil is given in Table 01. In addition, Water table also was found to be at

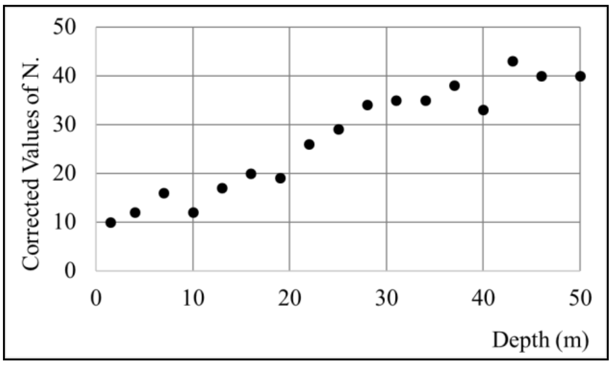


Fig 01: Corrected average SPT values with depth

depth of 1 m below the ground surface. Basic subsurface soil parameters of multi layered soil were already investigated by Li et al. [13] and proposed with respect to the depth, and they are summarized in Table 2.

The reference single pile used at this site 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. The additional properties of the reference pile are tabulated in Table 3. Static load test was performed to the reference pile up to 21, 000 kN (300 % of the working load) after three months from the installation.

3. Research Methodology

3.1 Soil models used

(a) Linear Elastic (LE) model

The numerical simulation of the pile load test was performed through the Finite Element code PLAXIS 2D. Very basic LE model is one of the model developed in PLAXIS package and it was adopted to perform linear elastic analysis to the material. The LE model is based on Hookes's Law of isotropic elasticity. It involves two basic elastic parameters i.e. Young's Modulus (E) and

Poisson's ratio (v). 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 [14].

(b) Mohr Coulomb (MC) model

MC model is one of the nonlinear model adopted in this research study. It is based on soil parameters that are known in most practical situations. This involves five input parameters, i.e. Elastic modulus (E) and poisson's ratio (v) for soil elasticity, (friction angle) φ and cohesion (c) for soil plasticity and dilatancy angle. However, not all the non-linear features of soil behaviour are included in this model.

The LE and MC model parameters for the Old alluvium are summarized in Table 2 Orihara and Khoo [12] have conducted a number of Standard Penetration tests and triaxial tests for the corresponding piston samples that were consolidated isotropically to a stress of 100 kPa. Based on their test results, they have correlated the strength parameters (c and Φ) with SPT values for the Old alluvium. In this research, correlations developed by Orihara and Khoo [12] for the drained strength parameters were adopted.

(c) Hardening Soil (HS) model

The HS model is an advanced nonlinear model adopted for the simulation of soil behaviour. Jun Ju [10] and Schanz et al. [15] described in detail about Hardening Soil model (HS) and is 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 Mohr Coulomb model, limiting state of stresses are described by means of the friction angle, cohesion and dilatancy angle. But in Hardening soil model, stiffness is described much more accurately using three different stiffness input parameters such as triaxial loading stiffness (E50), triaxial unloading stiffness (Eur) and odometer loading stiffness (Eoed). In most of the cases, the values approximately satisfy Eur=3E50 and Eoed=E50 [10, 14] and the same were adopted to this research study as well.

Hence, E_{oed} could be related to E_s (Elastic Modulus) as given in Equation [1] [14, 16].

$$E_{oed} = \frac{(1+v_s)(1-2v_s)}{(1-v_s)} E_s \quad [1]$$

Table 01: Stratigraphy description of the soil

No. of layers	Depth (m)	Corrected Avg. SPT values	Soil description
1	0 – 2.4	10	Silty sand
2	2.4 – 5.4	12	Silty sand
3	5.4 – 8.4	16	Medium dense silty sand
4	8.4 – 14.4	16	Medium dense silty sand
5	14.4 – 26.4	25	Medium dense silty sand
6	26.4 – 41.4	35	Dense silty sand
7	41.4 – 44.4	45	Very dense silty sand
8	44.4 – 47.4	40	Very dense silty sand
9	Below 47.4	40	Siltstone

Table 02: Parameters used for LE and MC model

No. of layers	Moisture content (%)	Bulk density (kN/m³)	Dry density (kN/m³)	k (×10⁻⁸ m/s)	c′ (kN/m²)	Φ′/ °	Elastics modulus (MPa)	Poisson's ratio (v_s)	Dilatancy angle (Ψ) / °
1	22	20.3	16.6	18.8	5	32	21	0.3	5
2	22	20.3	16.6	18.8	10	34	22	0.3	5
3	22	20.3	16.6	18.8	10	34	28	0.3	5
4	18.2	20.7	17.6	6.4	10	34	28	0.3	5
5	16.3	20.3	17.6	3.8	10	34	42	0.3	4
6	16.3	20.3	17.6	3.8	15	34	56	0.3	3
7	16.3	20.3	17.8	3.8	15	34	70	0.3	2
8	16.3	20.3	17.8	3.8	15	34	63	0.3	2
9	16.3	20.3	17.8	3.8	15	34	63	0.3	2

Table 03: Properties of pile foundation

Pile properties	Unit weight (kN/m³)	Diameter (mm)	Length (m)	Elastic Modulus (GPa)	Poisson's ratio
Value	24	1500	48	30	0.2

Table 04: Soil parameters for HS model

No. of layers	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	15.6	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

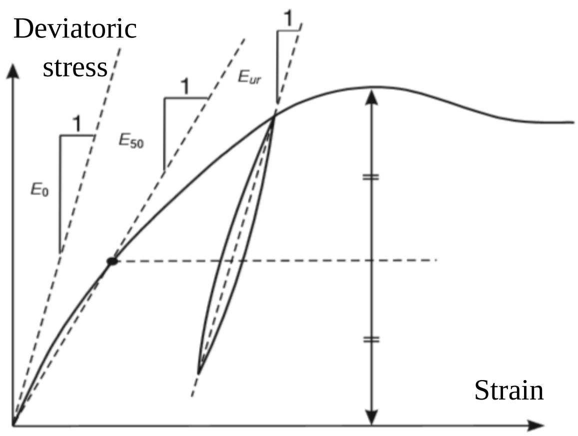


Figure 2: Hyperbolic stress-strain relationship used in Hardening Soil model [10].

where υs is Poisson's ratio. In addition, Poisson's ratio for loading and unloading (vur), lateral earth pressure coefficient, K0 (= 1-sinØ) and failure ratio (Rf) are also required input parameters for HS model. The total strains are calculated using a stress level dependent stiffness (defined by a power law with a coefficient m as given in Equation [2]) [10, 14, 16] with a hyperbolic stress-strain relationship that differs for virgin load and unloading/reloading (Figure 2).

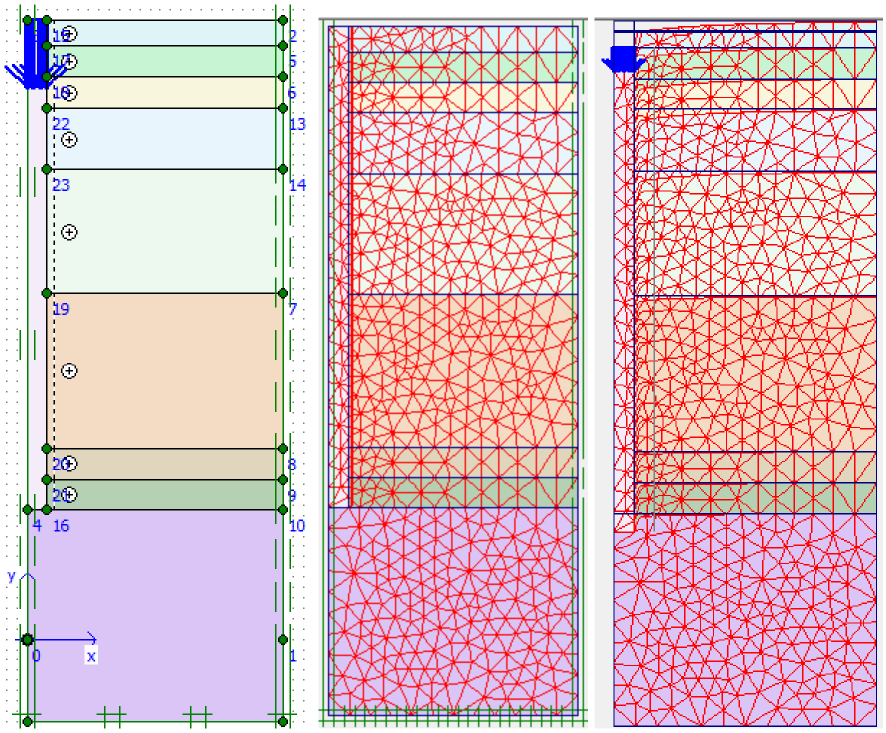
$$E = E^{ref}\left(\frac{c\cot\emptyset - \sigma_1'}{c\ \cot\emptyset + p^{ref}}\right)^m \qquad [2]$$

Wherein c is cohesion, m is power in stiffness laws and can be defined between 0.5 and 1, φ is angle of internal friction, E^{ref} is modulus at p^{ref} and E_s is modulus at σ_1'. It is suggested that, values of v_{ur} and R_f are 0.2 and 0.9 respectively are appropriate for the HS model at drained condition [12]. The best estimate parameters of each layers adopted for the HS model are summarized in the Table 4.

3.2 FEM analysis

PLAXIS 2D was used to simulate the settlement behaviour. In this program, modelling was carried out in axisymmetric conditions with two degrees of freedom of translation per node. The soil was modelled by 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 have been made using the same working area of 70 × 25m. At the bottom level of the model, all movements were restrained, whereas, at the lateral sides, lateral movements perpendicular to the boundary were prohibited.

A linear elastic non-porous and isotropic material model is used to represent the piles. Three different type of FE analyses were performed: (i) a linear elastic (LE) analysis where the soil was assumed as linear-elastic, (ii) a simple nonlinear analysis where the soil was completely assumed as Mohr-coulomb (MC) model and (iii) an advanced nonlinear analysis where the soil was completely assumed as Hardening-Soil (HS) model.



(a) (b) (c)

Figure 3: (a) Finite Element Model (b) mesh and (c) deformed mesh

The geometry of the Finite element model is shown in Figure 3(a). 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 element size could be larger near the boundaries with the condition that the distribution of stresses and settlements were not to change significantly by increasing or decreasing. The size of such boundary element. The undeformed and deformed FEM mesh are shown in Figures 3(b) and 3(c) respectively. Default interface and interface strength reduction factor (R_{int}) was incorporated in the simulation and adopted R_{int} value was 0.85 [9] and the positive interface arrangement is shown in Figure 3(a). The simulation sequence included an initial step in which the initial stress condition was established. Prescribed displacement was then applied at the pile head and variations of applied load with settlement were obtained. The computed results from all the three types of models were compared with the field test results in order to identify suitable model for the soil.

3.3 Numerical convergence analysis

Numerical convergence test was adopted to determine proper number and type of elements using different mesh densities, in addition to graphical observation. The analysis was begun with the simulations with coarser elements. Due to difference between the numerical and field experimental curves, element size was refined until getting the proper one. Fixed point method was adopted and the numerical convergence rate was estimated by using the equation [3] for each refinement of meshing to verify the results obtained from Finite element method.

$$e_k = |U_h - U| = Ch^{p} \qquad [3]$$

Whereas p is the order of Numerical analysis, U is the actual field result, Uh is the result from numerical analysis, ek is the error, C is constant and h is the step of the analysis. A better convergent result was obtained when the 15 node elements with average element size of 1.16 m were used. Moreover, all the type of analysis showed the best level with the above type (very fine) meshing and it was noticed that further refinement would not affect the results significantly.

4. Results and Discussion

The predicted settlement behaviours using complete LE, MC and HS analyses are presented in Figure 4 together with the measured field test results. Based on the field test results, at a typical working load of about 7, 000 kN, the measured settlement of 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 it is more than 2 times the field measurement. In addition, settlement values predicted from Linear elastic (LE) model and Mohr Coulomb (MC) model for the same working load are almost the same, and it is 6.8 mm. Based on these observations, it can be noticed that the settlement derived from LE and MC analyses is about 2 times higher than the field measurement for typical working load, and there is a marked difference between the calculated and the measured curves 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 may be due to the ignorance of deep-insitu effect in Modulus, as suggested by Jian-Lin [3]. Actually, the Modulus that obtained from the laboratory tests significantly vary from the insitu-Modulus of deep soil and sometimes the difference is in the order of several times [3]. Settlement of shallow foundations using soil compression modulus tested under pressure 100−200 kPa generally do not have too much error. However, settlement calculated values for deep soft soil of deep piles are often associated with large differences in measured values [3] as deep-insitu soil stiffness is always higher than that of samples used to obtain the laboratory elastic modulus. Therefore, Equation [4] proposed by Jian-lin et al. [3] was adopted to increase the accuracy of the settlement prediction. Elastic modulus obtained from laboratory tests were modified using Equation [4] as proposed by Jian-Lin [3].

$$E_{s,z} = E_{s,0.1-0.2}(z/h_0)^{1/\beta} \qquad [4]$$

where z is the depth of soil layer (m), h_0 is reference depth (generally 1m), $E_{s,0.1-0.2}$ 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 and plasticity index (IP) values from the data by which the value of β can be obtained using the specifications of the BS code. For the silty sand β value is defined between 3.5 and 5 [3]. Figure 6 shows the comparison between the laboratory modulus values and the corrected modulus values using equation [4].

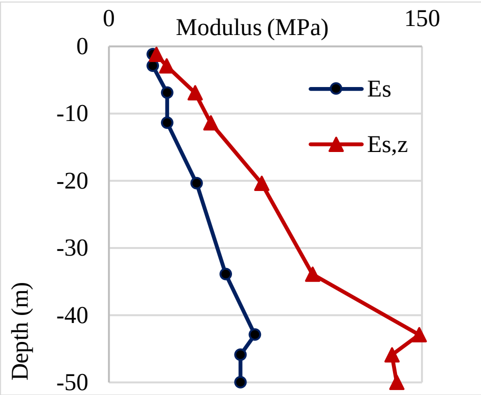


Figure 6: Comparison between experimental compression modulus (Es) and corrected modulus (Es,z)

All the FEM analysis was repeated with the corrected values of modulus and the obtained results are shown in Figure 5. According to Figure 5, predicted settlement from complete HS model is 6.2 mm at the working load of 7,000 kN, while the settlement predicted from Linear elastic (LE) model and Mohr Coulomb (MC) model for the same working load are 4 mm and 3 mm respectively. The obtained settlements values using the corrected modulus are very much closer to the measured settlement value of 3.5 mm. LE and MC models predict better prediction than the HS model below the working load of 13, 000 kN and this result is supported by the conclusion proposed by Zakia et al. [9]. 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 at low working loads (up to 3000 kN) for the entire analysis due to its advanced nonlinearity. Although HS model over predicts the settlement, incorporation of HS model is necessary to predict the settlement at higher working loads.

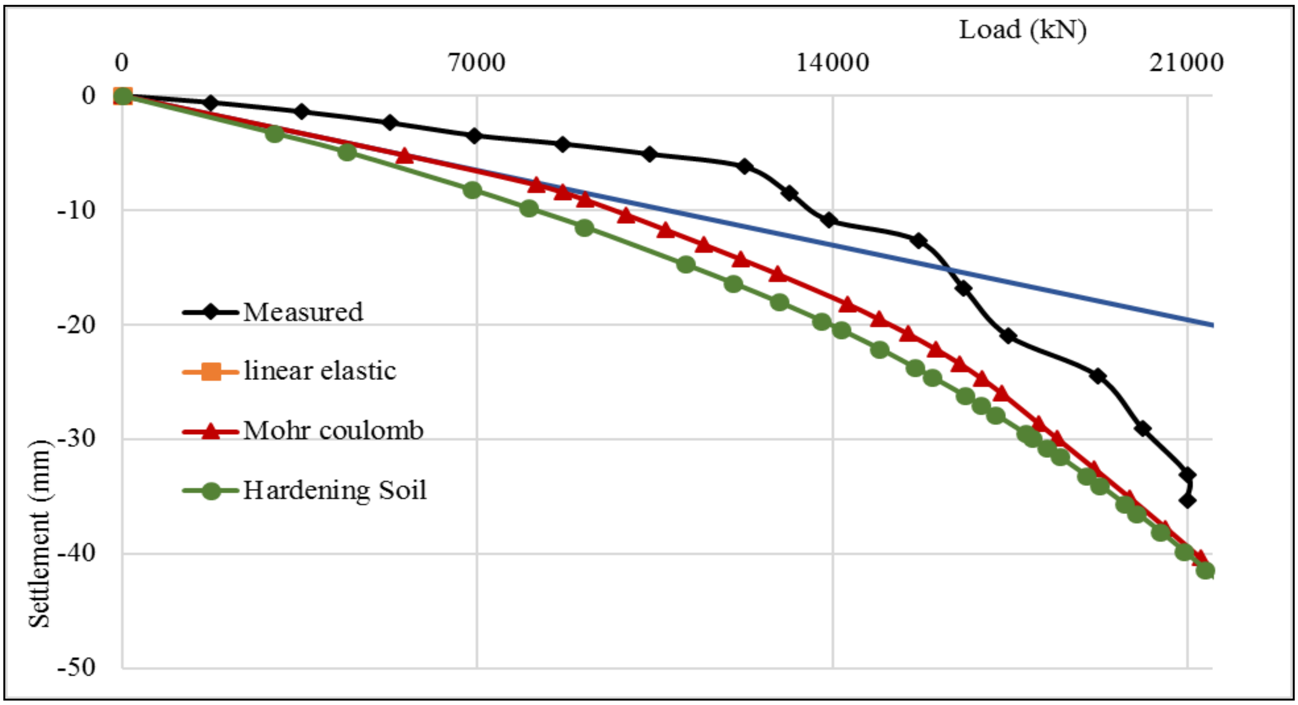


Figure 4: Comparison between Field test results and Predicted settlement from LE and NL

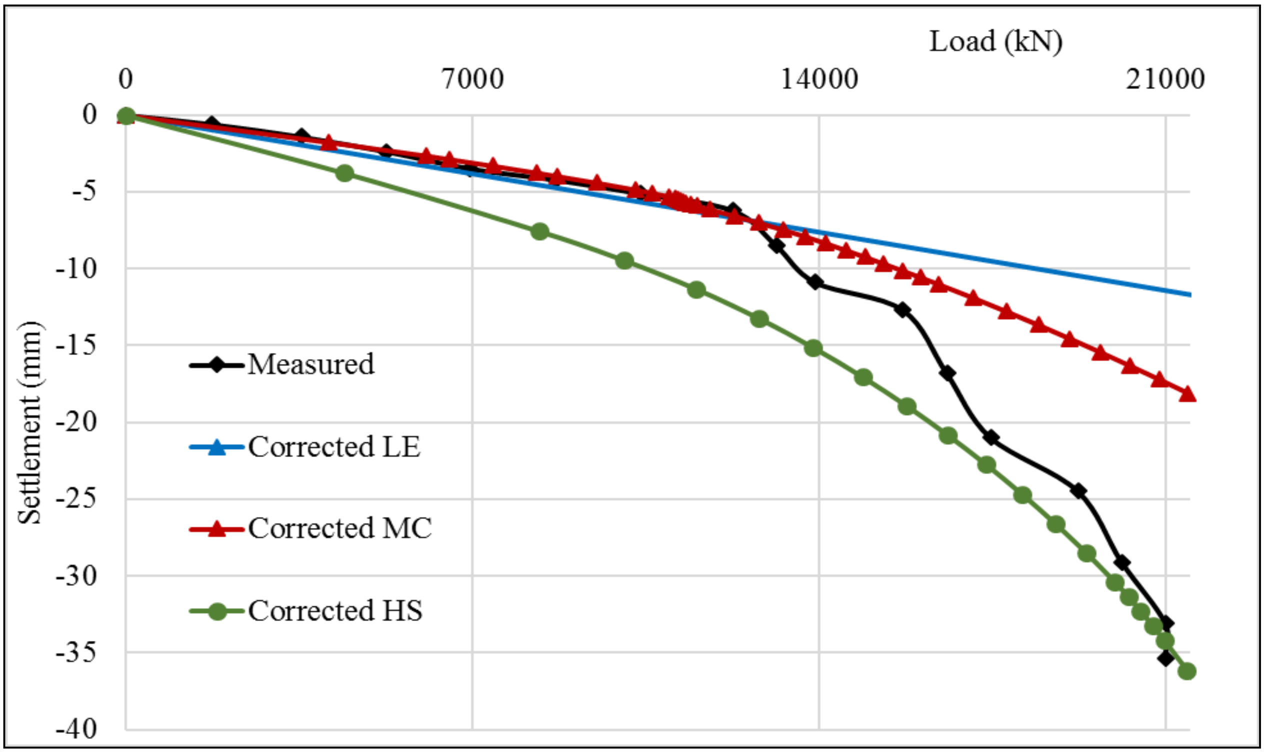


Figure 5: Comparison between Field test results and Predicted settlement with Corrected modulus Es,z

5. Conclusions

In this research, settlement behaviour of a single axially loaded pile was simulated with PLAXIS 2D using three different material models such as linear elastic (LE), Mohr Coulomb (MC) and Hardening Soil (HS) models. The following conclusions are drawn based on the outcomes of this research:

1. The numerical method cannot be used for the pressure between 100−200 kPa of the soil compression modulus $E_{s,0.1-0.2}$, as settlement calculated values for deep soft soil of deep pile are often associated with large differences in measured values. Therefore, Modulus correction formula is necessary to incorporate in numerical analysis in order to obtain accurate prediction values of settlement.
2. 2. MC model is the best soil model to predict the settlement of a single pile and its simple nonlinearity is adequate to predict the realistic settlement prediction at lower working loads up to about 13,000 kN.
3. 3. In addition, using HS model is adequate to capture better prediction for the working load above 15,000 kN. Therefore, HS model can be suitable in simulation of massive and high rised structures.
4. On the whole, in the numerical simulations to predict the realistic settlement, understanding and selection of material model is very important. Material models selection is actually not only depending on the characteristic of soil, but also depending on the type and loading conditions of the structure.

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