A Numerical Analysis of Anchored Diaphragm Walls for a Deep Basement in Kuala Lumpur, Malaysia.

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ABSTRACT: The performance of 1.2 m thick anchored diaphragm walls for a 6-level deep basement in Kuala Lumpur, Malaysia, is reviewed. The depth of the basement excavation ranges from 24.5 m to a maximum of 28.5 m and the walls were constructed in residual soils derived from the Kenny Hill Formation. The finite element method (FEM) utilising computer program PLAXIS, was used to back-analyse the deep basement excavation supported by anchored diaphragm walls. The residual soil was modelled using "Hardening Soil Model" and undrained analyses were carried out incorporating the steady state groundwater seepage calculation for comparison. The effects of soil berms on the performance of the wall were also modelled in the FEM analyses. The lateral movements of the wall and settlement of the retained ground obtained from the FEM analyses were compared with the measurements taken from the instruments installed at the site. Finally, the differences between design assumptions and actual field conditions are discussed and some suggestions on the use of FEM for the analysis of the deep excavation in tropical residual soils of Malaysia supported by anchored diaphragm walls are also presented.

1 INTRODUCTION

In the expensive and congested urban area of Kuala Lumpur, Malaysia, deep basements have been extensively constructed to effectively utilise the underground for car parks and other usage. In this paper, the performance of anchored diaphragm wall system for a 6-level basement, is reviewed. The depth of basement excavation was generally 24.5 m to 26 m and reaching a maximum of 28.5 m at the two tower locations. Temporary ground anchors of 5 to 7 levels were selected as the support system for the 1.2 m thick diaphragm wall and the total plan length of the diaphragm wall is 735 m with wall depth of 42 m to 44 m.

Finite element method (FEM) have proved to be of significant help in estimating wall and ground deformations at each stage of excavation. Although significant progress has been made in numerical modelling of deep excavations, some refinements in the selection of appropriate constitutive models which have a significant influence on the results in terms of the overall deformation behaviour and the selection of in-situ stiffness parameters still need to be addressed.

In this paper, the finite element method (FEM) utilising computer program PLAXIS, was used to back-analyse the measured lateral movement profiles of the two selected wall panels, namely Wall A and Wall B as shown in Figure 1. The residual soil was modelled using the "Hardening Soil Model" and undrained analyses were carried out incorporating steady state groundwater seepage calculation for comparison. The effects of soil berms on the performance of the wall were also modelled in the FEM analyses.

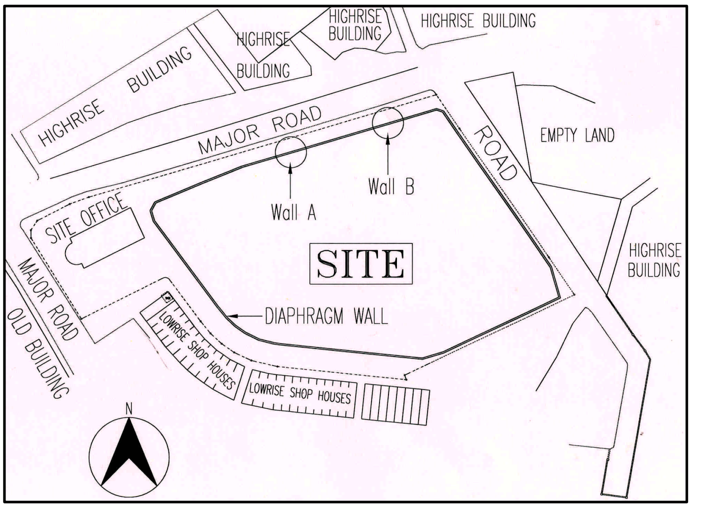

Figure 1. Site plan.

The lateral movements of the wall and settlement of the retained ground obtained from the FEM analyses were compared with the measurements taken from the instruments installed at the site. Finally, the differences between design assumptions and actual field conditions are discussed and some suggestions on the use of FEM for the analysis of the deep excavation in tropical residual soils of Malaysia supported by anchored diaphragm walls are also presented.

2 GENERAL GEOLOGY AND SUBSOIL CONDITIONS

2.1 *General Geology*

General geology map of Selangor, Malaysia indicates that the site is located in the boundary between the Kenny Hill Formation and Kuala Lumpur Limestone. The subsurface investigations carried out at the site confirmed that the site is on residual soils derived from the Kenny Hill Formation overlying a highly variable karst marble surface, known locally as the Kuala Lumpur Limestone (marble).

The Kenny Hill Formation consists of Carboniferous to Triassic meta-sediment interbedded between meta-arenite and meta-argillite with some quartzite and phyllite. Due to intense weathering process of tropical climate, the meta-sediment has already been transformed into residual and completely weathered soils (Grades V and VI). The Kuala Lumpur Limestone (marble) is of middle to late Silurian Age. This sedimentary limestone sequence was folded and metamorphosed during the Devonian Age. In general, the limestone bedrock is associated with subsurface karstic features such as pinnacles, solution channels and cavities.

2.2 *Subsoil Conditions*

The subsurface investigations carried out comprises of rotary wash boring boreholes along the perimeter of the diaphragm wall and inside of the site. Field testing like Standard Penetration Tests (SPT) and pressuremeter tests were also carried out in the boreholes. Standpipe piezometers were installed in the boreholes to measure changes of groundwater level with time for design. The groundwater level measured was generally located at about R.L.+36.0 m before excavation. In addition, the disturbed and undisturbed soil samples collected from the boreholes were also tested in laboratory to acquire the necessary soil parameters.

The typical subsoils profile at the site, as shown in Figure 2, indicates that the Kenny Hill residual soils mainly consist of stiff to hard clayey SILT, and silty CLAY with sand or gravel. In between these materials are layers of medium dense to very dense clayey and silty SAND and GRAVEL.

Although the subsoil profile presented in Figure 2 shows variable succession of strata and for analysis purposes, the subsoils were divided into two major components, namely predominantly Granular materials and predominantly Cohesive materials for the design and back-analysis of the two selected typical panels of the diaphragm walls with different ground anchor configurations (Wall A and Wall B). These two panels were located on one side of the site as shown in Figure 1. Based on the type of subsoils and variation of SPT′N′ values, the subsoils were further divided into four different layers in the analyses (Gue & Tan 1998a).

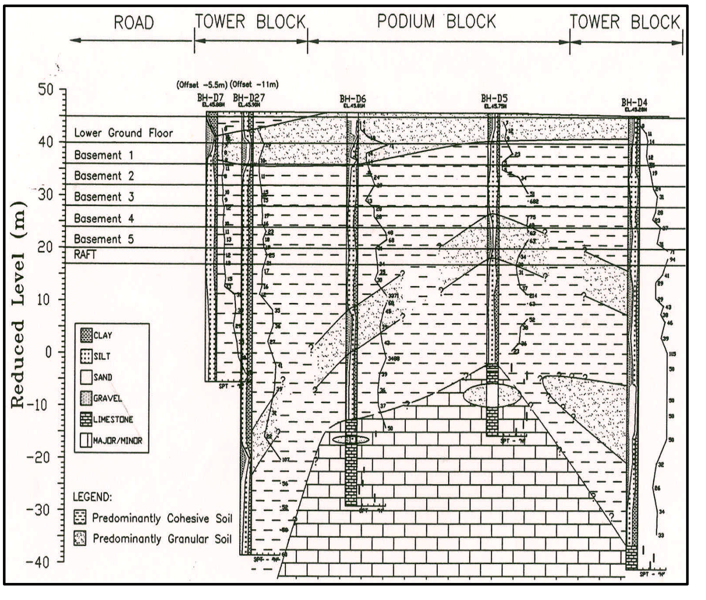

Figure 2. Typical subsoil profile (from Tan 1997).

3 FINITE ELEMENT ANALSIS

3.1 *Introduction*

Finite element method (FEM) utilizing computer program "PLAXIS", was used to simulate the basement excavation. Figure 3 shows the plane strain condition FEM model of the Wall A in initial stage utilising 6-node elements under 2-D plane strain condition.

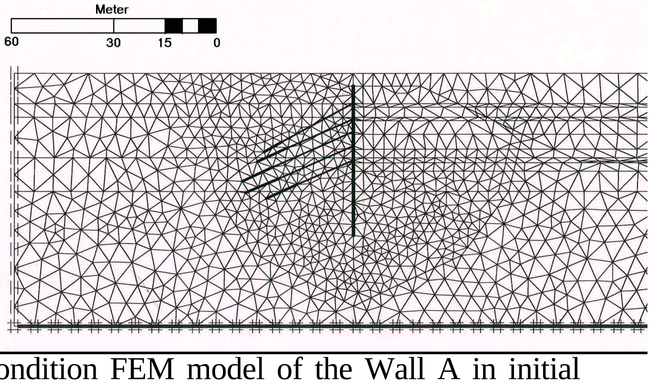

Figure 3. Finite element modelling of excavation.

Table 1. Soil parameters for FEM back-analysis.

Layer	SPT′N′ (blow/300mm)	γ_b (kN/m^3)	E’ (kN/m^2)	E’$_{ur}$ (kN/m^2)	c’ (kN/m^2)	ϕ’ (degree)	ψ (degree)	K_O
> R.L. 32m	10	18.5	20,000 (35,000)	60,000	0	31	2	1.0
R.L.32m to R.L. 20m	28	18.5	56,000 (98,000)	168,000	5	32	2	0.8
R.L.20m to R.L. 10m	34	19.0	68,000 (119,000)	204,000	5	32	2	0.8
< R.L. 20m	42	19.0	84,000 (147,000)	252,999	5	34	4	0.8
Note : The E’ values in brackets are the stiffness used in the original design of the diaphragm wall (Gue & Tan 1998a).								

Table 2. Details of ground anchors.

Prestressed Ground Anchors at	Wall A					Wall B			
	Level-A	Level – B	Level – C	Level – D	Level – E	Level – A	Level – B	Level – C	Level – D
Reduced Level (m)	36.2	31.5	27.8	23.5	19.3	36.3	28.7	24.5	20.7
Angle of Inclination (degree)	30	25	25	25	25	30	25	25	25
Working Load (kN)	740	730	1008	1129	891	879	811	1148	1157
Horizontal Spacing of Anchor (m)	1.57	1.05	1.05	1.05	1.05	1.57	1.05	1.05	1.05
Prestressed Lock-Off Load (kN)	689	626	847	873	715	671	624	901	924

The “Hardening Soil” model (Vermeer & Brinkgreve 1998) was used to model the residual soils and the 1.2 m thick diaphragm wall was modelled as beam elements. The free length of pre-stressed temporary ground anchors was simulated as node-to-node spring while the fixed length grout body was simulated as slender objects with a axial stiffness but with no bending stiffness and can only sustain tensile forces only. In addition, the interface elements were introduced to simulate the soil-structure interactive behaviour.

The key input soil parameters (all in effective stress) are listed in Table 1 and the details of the prestressed ground anchors are listed in Table 2. The effective Young’s Modulus (E’) of the soils were interpreted from pressuremeter tests performed at different depths in the boreholes. The effective unloading/reloading stiffness (E’$_{ur}$) is taken as three times the efffective Young’s Modulus which tally with the unload-reload cycle of pressuremeter tests. The initial coefficients of earth pressure at rest, K_o were also obtained from pressuremeter tests.

In the back-analysis, the soil stiffness which has considerable influence on the wall movement has been modified from the values used in the original design (Gue & Tan 1998a). The original design of the diaphragm wall was carried out using computer program FREW, a quasi-finite element program which allows the interation between the wall and the soil to be modelled. The full details of the computer program FREW are illustrated by Pappin et al. (1986).

In the numerical modelling, the diaphragm wall was asumed as “wished-in-place” after the bulk earthworks excavation in the middle of the site but before the start of the excavation immediately infront of the wall. For each stage of excavation in the undrained analysis, steady state groundwater calculation was performed.

The excavation sequences simulated in the FEM analysis are as follows :

0. Bulk earthworks excavation at the centre of the site.
1. Installation of diaphragm wall and excavate to 1st level (R.L.35.5 m for Wall A and R.L. 36.0 m for Wall B).
2. Installed and Prestress Anchors (Level-A) and excavate 2nd level (R.L.31.0 m for Wall A and R.L. 28.0 m for Wall B).
3. Installed and Prestress Anchors (Level-B) and excavate 3rd level (R.L.27.0 m for Wall A and R.L. 23.6 m for Wall B).
4. Installed and Prestress Anchors (Level-C) and excavate 4th level (R.L.22.5 m for Wall A and R.L. 20.0 m for Wall B).
5. Installed and Prestress Anchors (Level-D) and excavate 5th level (R.L.18.5 m for Wall A and final level R.L. 16.7 m for Wall B).
6. Installed and Prestress Anchors (Level-E) and excavate 6th level (final level R.L.16.7 m for Wall A).

Figure 4 shows the bulk earthworks excavation carried out at the centre of the site before installation of the wall.

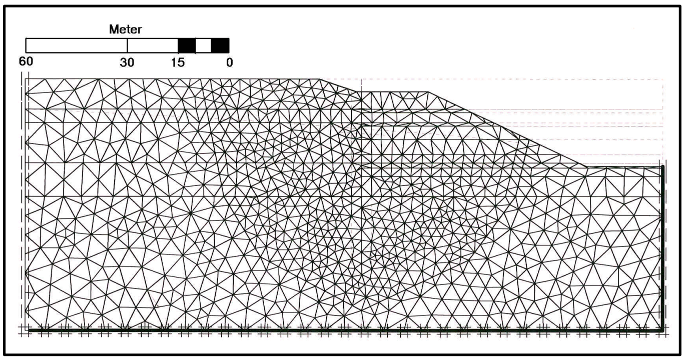

Figure 4. Modelling of bulk earthworks excavation before installation of wall.

3.2 *Results and Discussion*

The back-analysed results presented in this paper include relative lateral displacement of the walls and settlement of the retained ground behind the walls due to excavation.

Figures 5 and 6 show the relative lateral displacement measured from inclinometers installed at Wall A and Wall B respectively. In these figures, the back-analysed wall displacements were also presented. Generally the FEM results match the measured wall profile well for all stages of excavation for both Wall A and Wall B except for slight under-estimation of the wall top displacements. In view of a good match between the measured and back-analysed wall relative lateral displacement profile, it is evident that residual soils can be properly modelled using the "hardening soil" model for the deep excavation problem.

The ground surface settlement measured from settlement markers installed perpendicular to the wall in the retained ground and the FEM back-analysed results for Wall B are presented in Figure 7. In the FEM analysis, the maximum ground settlements are located at the distance of 15m to 20m from the wall and indicates close agreement with the field measurements. The analysed settlement profile overestimated the ground surface settlement for Stage 0 after bulk earthwork excavation at the centre of the site. The results improved for Stages 1 to 3 where the analysed and measured ground surface settlement profiles tally well for a distance of 50m before the analysed results overestimated the settlement. The reason for smaller measured settlement after a distance of 50m is due to the presence of highrise buildings supported by deep foundation obstructing the propagation of the ground settlement. In the last two stages (Stage 4 and Stage 5), the analysed results underestimated the ground surface settlement which might be due to consolidation of the subsoil due to lowering of ground water level caused by seepage flow during the process of excavation. In order to model this effect, coupled consolidation analysis needs to be carried out.

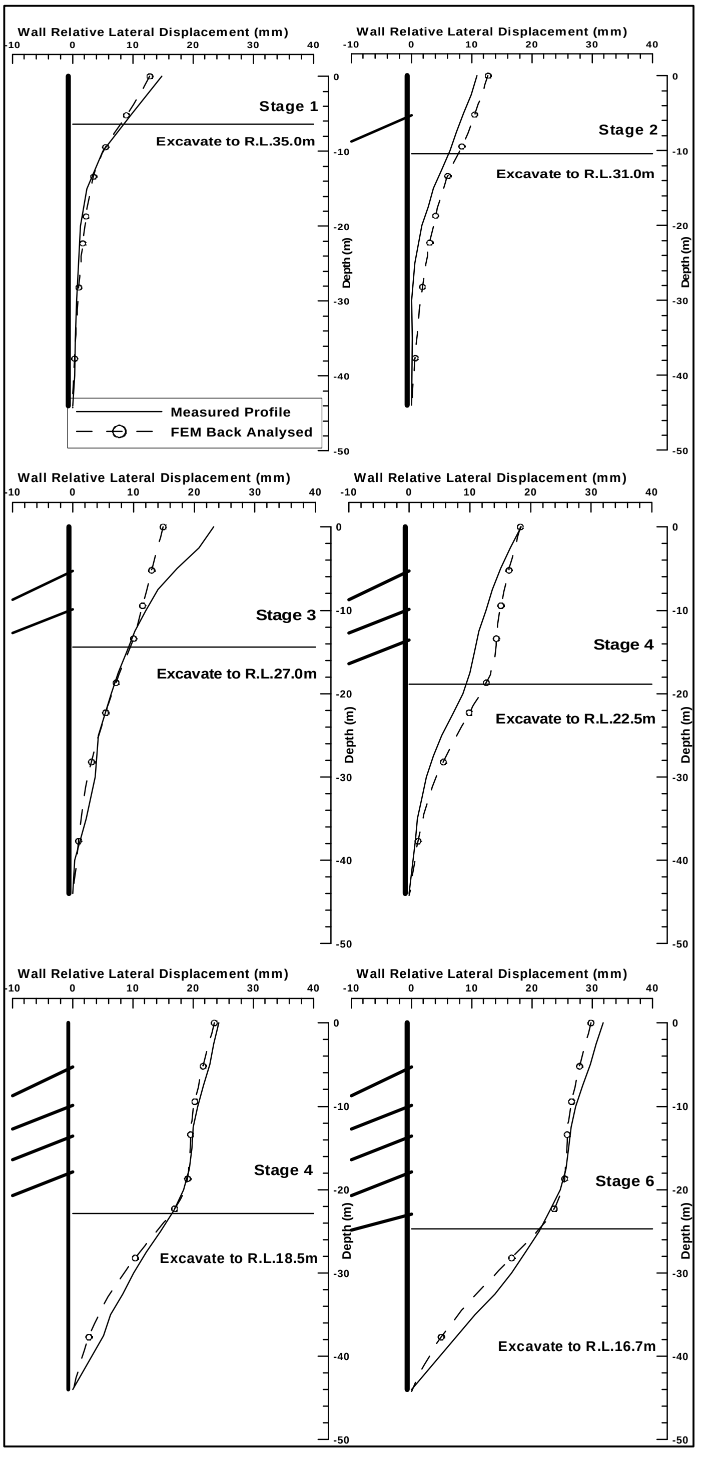

Figure 5. Relative lateral displacement profile for Wall A.

Figure 8 shows the analysed and measured ground surface settlement profiles plotted in dimensionless settlement profiles recommended by Clough & O'Rourke (1990). It is observed that the settlement profile generally extended up to a distance of 5 to 12 times the depth of excavation instead of 3 times presented by Clough & O'Rourke (1990) for stiff to very hard clay. This is because residual soil is usually of higher permeability than clay therefore seepage flow might have caused some lowering of groundwater level in the retained ground which can extend to distance well beyond 3 times the depth of excavation. For excavation in soft clay, the extent of the ground settlement can extend to as far as 50 times the depth of excavation if the control of seepage was not properly carried out (Gue & Tan 1998b).

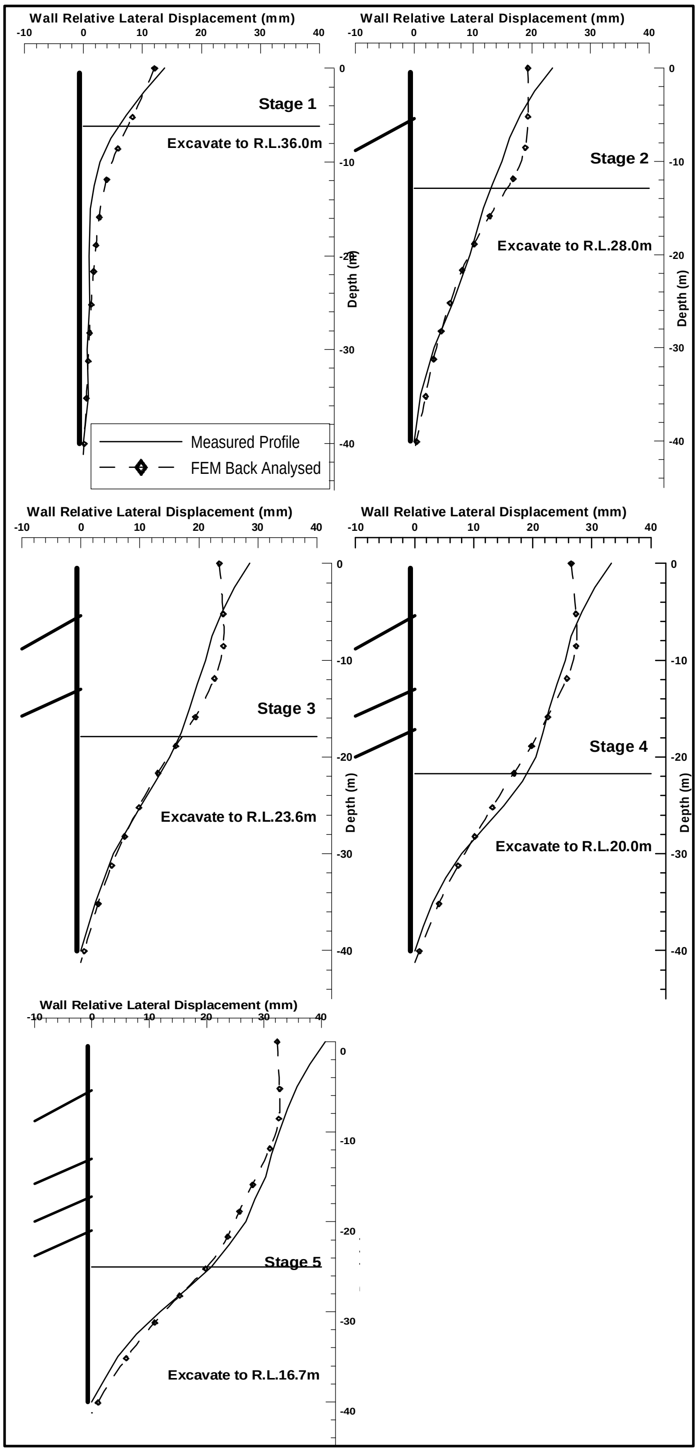

Figure 6. Relative lateral displacement profile for Wall B.

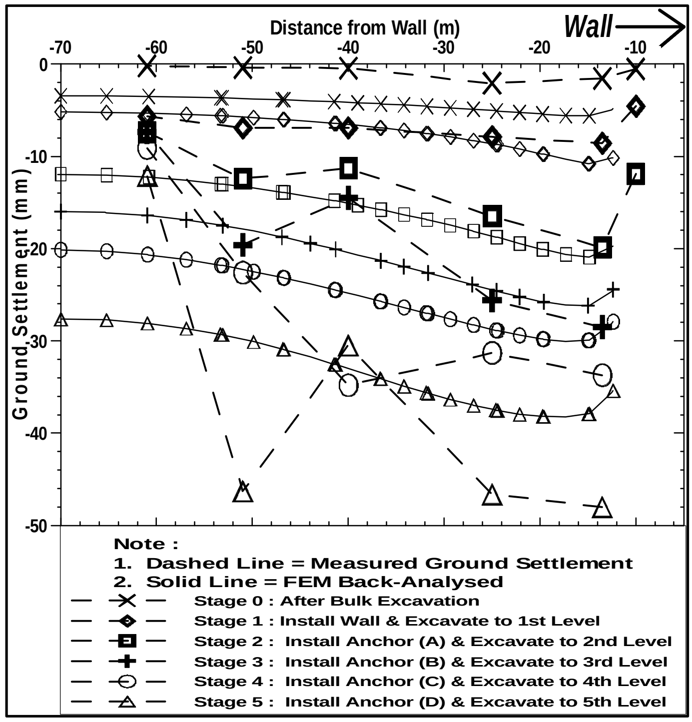

Figure 7. Ground surface settlement profile.

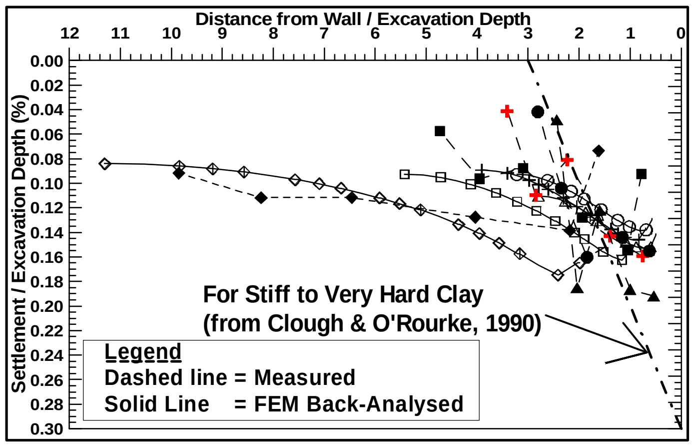

Figure 8. Dimensionless ground surface settlement profile.

On the other hand, the percentage of magnitude of maximum settlement over the depth of excavation for both measured and back-analysed is about 0.2% which is smaller than the 0.3% presented by Clough & O'Rourke (1990).

4 CORRELATIONS

In Malaysia, the standard penetration tests (SPT) commonly carried out at site were correlated to obtain strength and stiffness parameters of the residual soils for design because it is economic and easily available. Usually this was based on the contention that the SPT'N' values (blows/300mm), which were obtained extensively at site, adequately represent the mass of the residual soil which is very heterogeneous thus makes sampling and testing for representative parameters difficult. Pressuremeter tests are also carried out at times to refine the selected soil parameters.

From the FEM back-analysis, the suggested correlations between SPT'N' values and the effective Young's Modulus (E') and effective unloading/reloading stiffness (E'_{ur}) used in the "Hardening Soil" model are as follows :

$$E' = 2000 \times SPT'N' (kN/m^2) \quad (1)$$

$$E'_{ur} = 3 \times E' = 6000 \times SPT'N' (kN/m^2) \quad (2)$$

5 CONCLUSIONS

In conclusion, the displacement patterns of the walls and ground surface settlement profile obtained from the FEM back-analyses agree reasonably well with the measured values. Therefore, it is evident that residual soils can be properly modelled using the "hardening soil" model for the deep excavation problem.

It is also observed that, for deep excavation in residual soils, the ground settlement can extend up to a distance of 5 to 12 times the depth of excavation. The percentage of magnitude of maximum settlement over the depth of excavation is about 0.2% only for the diaphragm wall properly supported by multi-levels pre-stressed ground anchors.

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