Deformation Behaviour of a Retaining Wall for a Deep Basement Excavation with Semi-Top Down Method

Sofiana B. Talha
SSP Geotechnics Sdn Bhd, Selangor, Malaysia

ABSTRACT: The deformation behaviour of 1.0 m and 0.8 m thick retaining walls supporting a maximum excavation of about 27.0 m for a luxurious apartment block with an eight level mechanical basement car park in the City of Kuala Lumpur is presented in this paper. The area of excavation was about 34.0 m long and 20.0 m wide which exposes the residual soils from the Kenny Hill Formation. This paper presents the comparison between the measured deformation profile with those predicted using a quasi-finite element program "FREW". The choice of retaining system and construction method is also discussed.

1 INTRODUCTION

The site is located in between Jalan P. Ramlee and Jalan Puncak that leads to the KL Tower. The project comprises a 27 storey luxurious apartment block over the site with a deep basement car park. The basement is to be an 8 level mechanical car park with a void at the center for the movement of the car lift (see Figure 1 for plan and elevation) and is founded by a 2.5 m thick raft. The basement excavation was generally 21.0 m to 27.0 m deep. The sides are supported by reinforced concrete diaphragm walls of a plan size of about 20.0 m x 34.0 m, which are 0.8 m and 1.0 m thick. The maximum reduced level at the top of the wall is RL46.0 m and the lowest is RL40.0 m. Toe of the wall is at RL9.0 m. The choice of the retaining system and construction method is discussed. Instrumentation and monitoring is highly essential in this project to verify the design assumptions and ensure the integrity of the basement structures and adjacent structures during construction. It is also to ensure that the ground movement induced on adjacent structures is within acceptable limits in view of the close proximity of high rise buildings and roads. A comparison is made between wall movements predicted using the computer program FREW (Papin et al., 1986) and those actually measured using inclinometers at 2 selected wall panels. The results are presented and discussed.

2 GEOLOGY AND GROUND CONDITIONS

Regionally, this site is underlain by metasedimentary residual soils of the Kenny Hill Formation, a sequence of interbedded sandstones, siltstones and shales/mudstones. In general, the Kenny Hill Formation distributes as a broad synclinal belt of 7.0 to 10.0 km wide running from Kuala Lumpur southward through Petaling Jaya and further to the south for at least another 30.0 km. The residual soils at this site form from the in situ weathering of parent rock of phyllite, shales and schist mainly consist of sandy clayey SILT and clayey silty SAND and GRAVEL. 6 boreholes were conducted in this 1554 m^2 site and the typical subsoil profile in latitudinal section is exhibited in Figure 2. Only one borehole exhibit the presence of rock at 78.30 m bgl @ RL-32.83 m, exposing SCHIST and it is well below the lowest excavation level. The groundwater table at this site has been monitored during the soil investigation works, which indicated to be at around RL36.50 m with pore water pressure increasing hydrostatically. No significant variation in ground water level is expected on this site.

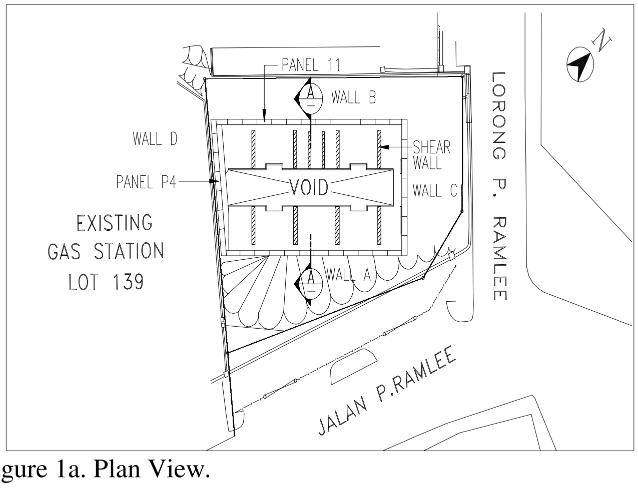

Figure 1a. Plan View.

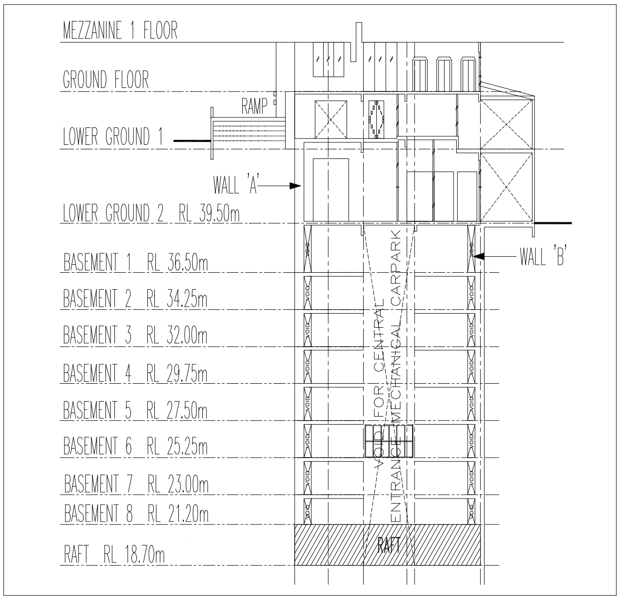

Figure 1b. Elevation view: cross section A-A.

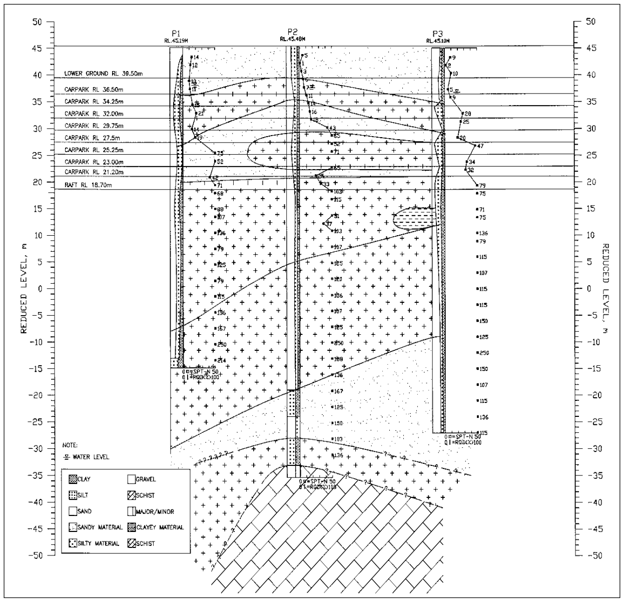

Figure 2. Subsoil profile in cross section A-A.

3 DESIGN PARAMETERS AND APPROACH

Figure 3 exhibits the summary results for Standard Penetration Test (SPT 'N') for all boreholes. The result shows typical trend of SPT 'N' value increases with depth. The prediction of wall movements was primarily based on parameters derived from SPT 'N' correlation. This was based on the contention that the SPT 'N' values adequately represent the mass of the material. Hence, a moderately conservative design line was chosen as shown in Figure 3. The approach of identifying the envelope or design line is outlined in CIRIA Report 104 (Padfield and Mair, 1984). The analyses has to consider the short term and long term behavior of the soils, taking account of construction periods and likely consequence due to unforeseen delay.

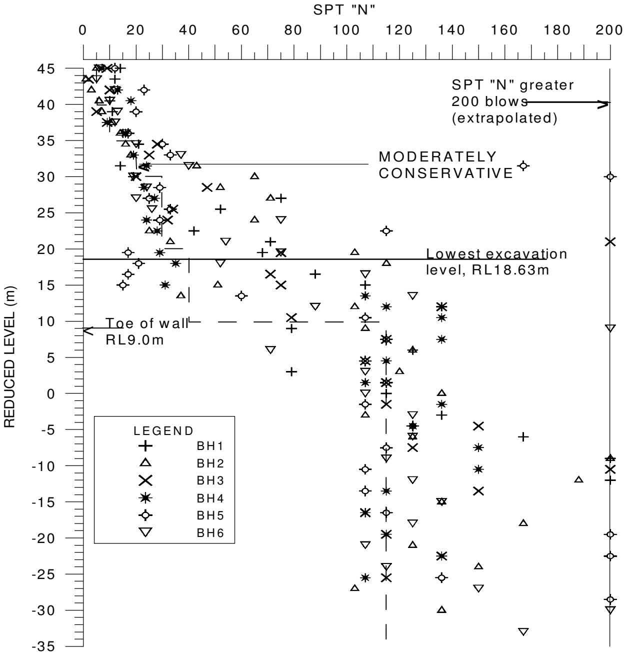

Figure 3. SPT 'N' variation.

For design purposes, the subsoil is divided into predominantly sandy materials for the first 5.0 m which were modeled to behave as 'drained condition'. The rest of the subsoil which is predominantly silty materials were modeled as the following as suggested by CIRIA Report 104 due to the uncertainty about the applicability of a total stress analysis:
Case 1: 'undrained condition' on both sides of the wall.
Case 2: 'drained condition' on retained side, 'undrained condition' on excavation side
Case 3: 'drained condition' on both sides of the wall.
The bending moment envelope of all the 3 cases were used in the reinforced concrete design for the diaphragm wall. The design parameters for FREW input are given in Table 1.

Table 1. Moderately conservative soil parameters for design.

DEPTH (m)	N	γ (kN/m^3)	E' (MPa)	ϕ'	S_u (kPa)	E_u (MPa)
0.0–5.0	5	18.0	12.5	29^o	-	-
5.0-10.0	10	18.0	25.0	30^o	40	30.0
10.0-15.0	20	18.5	50.0	32^o	80	60.0
15.0-25.0	30	19.0	75.0	34^o	120	90.0
25.0-35.0	40	19.5	100	36^o	160	120
>35.0	100	20.0	125	40^o	400	300

Stroud (1974) suggested the correlation between SPT 'N' and undrained shear strength as $S_u = f_1$ x SPT 'N', where f_1 (kPa) ranges from 4 to 6. Based on the interpreted results of Unconsolidated Undrained Triaxial Test (UU) and Isotropically Consolidated Undrained Triaxial Test (CIU) conducted on 75mm diameter undisturbed soil samples from boreholes at this site, the equation $S_u = 4$ x SPT 'N' (kPa) was adopted, which is in the lower range as recommended by Stroud. Unfortunately, pressuremeter test was not conducted at this site. However, the correlation of SPT 'N' values with E_{pm}, Pressuremeter Modulus values, for Kenny Hill Formation was obtained from Toh et al. (1987). In the design, the relationship between Undrained Young's Modulus E_u and S_u was taken as $E_u = 750S_u$ kPa, whereas for Drained Young's Modulus (E') was taken as 2500N kPa.

3 RETAINING WALL SCHEME

In choosing the type of retaining scheme, the following factors must be considered:

a. the site constraint
b. the condition of the soil/ground, total excavation depth and area
c. the control on ground movement
d. the importance of water tightness as the location of ground water table is about 20.0 m above the final excavation level
e. the availability of machines and contractors' experience in the country to construct the proposed structure
f. the construction feasibility, monitoring and control during construction

After considering the factors listed above, reinforced concrete diaphragm walls are found to be most feasible as the basement retaining system. Sheet piles were rejected due to lack of control in large ground movements during construction, also due to poor in water tightness control and installation difficulty. Contiguous bored pile was also ruled out due to insufficient water tightness as well as the tolerance. As for secant pile wall, local contractors have limited experience on this method. It is also difficult to construct and control during construction.

4 BASEMENT CONSTRUCTION METHOD

Three distinctive methods of constructing the proposed 20.0 m x 34.0 m basement 'box' have been identified for the project. In all these method, the perimeter wall is first constructed.

<u>A. Top down with internal shear wall constructed as diaphragm wall:</u> In this method, internal diaphragm walls are constructed at the same time as the perimeter wall. Excavation can begin as soon as the diaphragm walls achieve the minimum cube strength required in the design. Floors are constructed in top-down manners where excavation would have to be carried out in a very restricted headroom.

<u>B. Conventional strutting</u>: In this method, the foundation for the internal shear wall will be constructed with empty bores to the pile cap level. This could be in the forms of either large diameter bored pile or short panel of diaphragm wall (barrette). Successive strutting precedes excavation until the lowest level. Floor and internal shear wall are then constructed bottom-up.

<u>C. Semi-top down</u>: This is essentially a hybrid of the above two methods. The foundations for the internal shear walls are constructed from present ground level with empty bores to the pile cap level, as in the conventional method. In addition, plunged stanchions (king posts) are constructed to support the floor slab to be constructed in top-down manners. Here, the slab and the temporary struts spanning the central access void acts as props for the perimeter retaining wall. In other words, the system depends on the floor slabs and struts to transfer lateral force in the temporary conditions with the vertical support for the floor slab provided by the stanchion. Shear walls are then constructed bottom-up. Eventually, when all the shear walls are able to act on their own, the temporary struts are then removed.

The likely deviation of the internal walls, if constructed as diaphragm wall, is unacceptable and the space between internal walls could not be enlarged. The potential risk of difficulty to rectify excessive out of alignment is considered too high to take. Hence, method 1 was ruled out. Two possible methods were then the semi-top down and the conventional strutting. The semi-top down method appears to have a shorter construction period by three months. Thus, semi-top down method was opted.

4.1 *Construction sequence*

The following construction sequence has been envisaged in the design by using FREW:

Stage 0: Assume the wall in place
Stage 1: Excavate to RL38.4m
Stage 2: Install temporary strut at RL39.0m, excavate to RL33.7m
Stage 3: Construct slab and install strut at RL34.25m (B2), excavate to RL29.2m
Stage 4: Construct slab and install strut at RL29.75m (B4), excavate to RL26.9m
Stage5: Construct slab and install strut at RL27.50m (B5), excavate to RL24.6m
Stage 6: Construct slab and install strut at RL25.25m (B6), excavate to RL22.5m
Stage 7: Construct slab and install strut at RL23.00m (B7), excavate to final level, RL18.63m to build raft

This program has been calibrated against established finite elements programs (e.g. safe from OASYS and ACFEP from Imperial College, London). Full details of the assumptions and method of analysis utilising the program FREW are illustrated by Pappin et al. (1986).

5 DISCUSSION OF RESULTS

The movements of selected two diaphragm wall panels are presented here; Panel P4 and Panel P11 (Figure1). Panel P4 of Wall D is retaining the adjacent Petrol Station ground at RL46.0 m near the boundary. The thickness of panel P4 is 1.0 m. Panel P11 of Wall B, 0.8 m thickness, is retaining the ground at RL40.0 m. The original ground level of this site was approximately RL45.0 m. The first temporary strut was installed at RL39.0 m. Hence, Panel P4 were cantilevered by 7.0 m, where as Panel P11 were strutted almost at the top of the wall. The measured movements for both panels are respectively shown in Figures 4 and 5. It was found that the wall movements in Case 3 (drained condition on both sides of the wall) were the closest to the measured condition. Both the other two cases over-predicted the wall deflections. Only Case 3 wall movements are shown here in the Figure 4 and Figure 5. For the cantilevered Panel P4, the magnitude of movement at top of the wall as predicted by FREW is much larger in all stages of excavation except for the last stage, Figure 4. Similar pattern was also observed by Tan (1997). As for panel P11 where struts were located very near at the top of the wall, prediction is in good agreement with the measured movement.

6 CONCLUSION

Two diaphragm walls of different thicknesses were analysed. In the initial stage of excavation, before the struts were introduced, cantilever type deflections were observed with maximum values at the ground surface. It is also noted that theoretical predictions over-estimated the deflections compared to the actual ones. As the excavation progressed, the deflection pattern varied and the maximum deflection occurred approximately at the excavation base level. It is observed that the deflection of the 0.8 m wall is more than the 1.0 m thick wall at the lowest excavation level due to lower wall stiffness. Theoretical predictions of the wall with struts at near the top of the wall match well with the field behaviour. In a condition where it takes 3 to 4 weeks to reach the next excavation level because of space constraint, 'drained condition' on both sides of the wall gives better wall movements prediction in Kenny Hill Formation residual soils. With the benefit of more documented case studies, designers' confidence in their models and chosen parameters will be enhanced resulting in more optional designs and cost savings.

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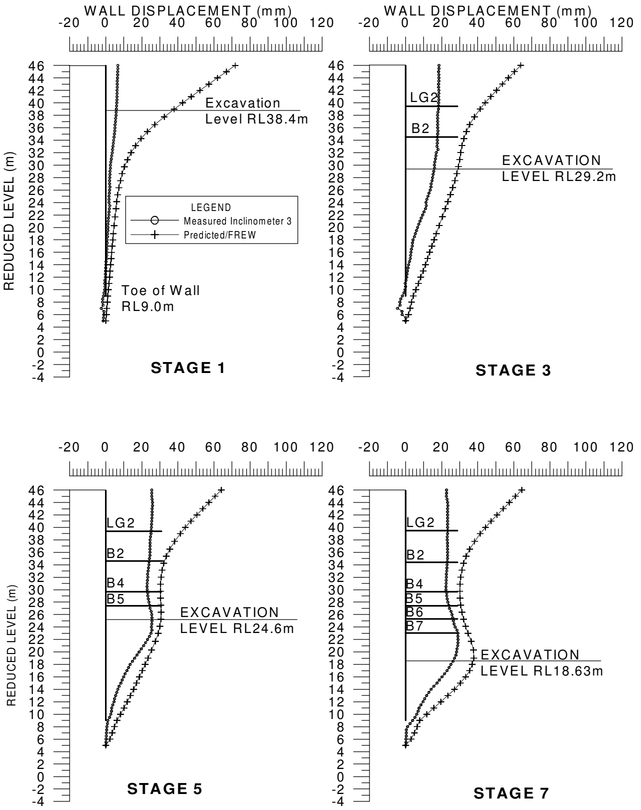

Figure 4. Wall movements of Panel P4.

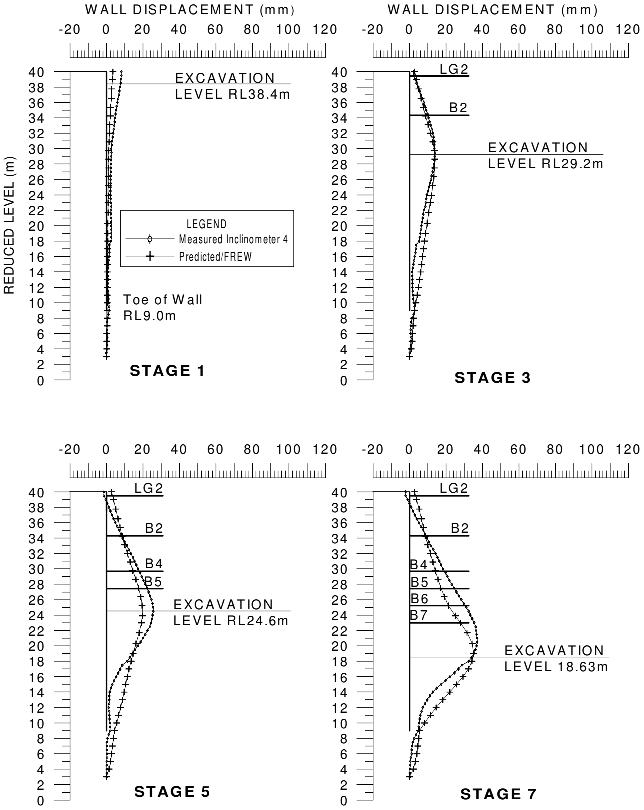

Figure 5. Wall movements of Panel P11.

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