Back Analyses and Performance of Semi Top-Down Basement Excavation in Sandy Alluvial Deposits

S. S. Liew & S. J. Gan

G&P Geotechnics Sdn Bhd, Wisma G&P, Bandar Tasik Selatan, Kuala Lumpur, Malaysia.
gnp-geo@gnpgroup.com.my

Abstract: This paper presents a case study of 11m deep 3-level basement excavation using semi top-down method with ring slab at B1 level. 600mm thick diaphragm wall of 15m deep was designed as the retaining structure for the basement. The overburden soils within the excavation are predominantly silty sand, probably from mine tailing deposits, overlying the Kenny Hill formation. Comprehensive instrumentation has been planned and implemented to monitor the performance of the excavation. These instruments consist of five inclinometers within the diaphragm wall, two inclinometers and five observation wells behind the walls, two piezometers beneath the lowest basement slab, strain gauges in the diaphragm wall and ring slab. Back-analyses on the wall deflections and ground settlement have been carried out to calibrate the soil stiffness with respective to various stages of excavation. From the instrumentation results, the overall performance of the excavation is satisfactory indicating the effectiveness and economy of this proposed solution.

1 INTRODUCTION

Recently, a lot of luxury condominiums are mushrooming at the vicinity of Kuala Lumpur City Centre (world's tallest twin tower) to cater for the residential market for young professionals and expatriates who love city life and convenience. Basements are normally adopted to effectively utilise the underground space for car parks and housing of utilities with aesthetical purposes.

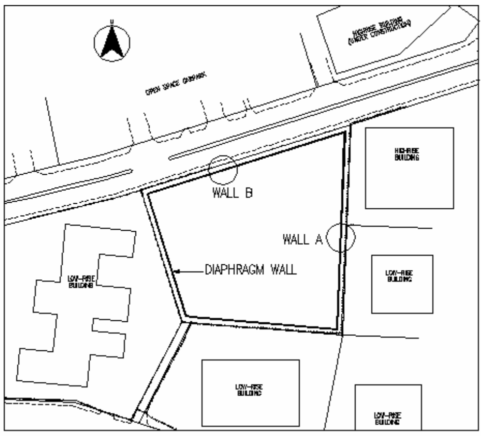
The development consists of one block of 20-storey tower block with three levels basement. The depth of basement excavation ranges from 9m at the southern boundary to 11m at the northern boundary due to variation in the existing ground level. The depth of diaphragm wall ranges from 14m to 16m. Semi top-down construction sequence was adopted by utilising the permanent slab at B1 level as the temporary propping to the surrounding diaphragm wall while excavating to the final excavation level. The 325mm thick B1 slab which has an opening to facilitate subsequent excavation works to final excavation level is functioning like a "ring slab" which is able to resist the lateral load from the diaphragm wall. The B1 ring slab which is also a temporary working platform is temporarily supported by API pipe which is plunged into the installed bored piles.

In view of close proximity of the adjacent buildings and roads to the site, the diaphragm walls are designed to limit their displacements to minimise the influence to the adjacent buildings and properties. Finite element method (FEM) utilising computer software, PLAXIS, was used to predict the lateral wall displacement and retained ground settlement. In this paper, two diaphragm wall sections have been re-analysed based on the actual sequence of works and time duration at site. Wall A and Wall B as shown in Fig. 1 were selected for the re-analyses. The difference between the predicted lateral wall displacement profile and ground settlement behind the wall obtained from the re-analyses and the actual measured value at the site are presented and discussed.

2 GENERAL GEOLOGY AND SUBSOIL CONDITIONS

2.1 *General Geology*

General geological map of Selangor, Malaysia indicates that the site is underlain by Kenny Hill Formation. Subsequent subsurface investigation (SI) shows that the overburden material consists of weathered Kenny Hill Formation, mainly sandy silty CLAY and clayey silty SAND. These coarse-grained and fine-grained soils were derived from weathering of meta-arenite (Sandstone) and meta-argillite (Shale) rock respectively.



Fig, 1 Site plan

2.2 *Subsoil Condition*

Fig. 2 shows the typical subsoil profile across the site. In general, the overburden was divided into two major components. The top 11m of alluvial deposits which mainly consists of very loose to loose (SPT-N < 10) silty/gravelly SAND and clayey/silty SAND (possible mining material) underlain by very hard stratum (SPT-N > 50) of Kenny Hill Formation consisting of mostly clayey

SILT material. The subsoil stratification across the site is generally uniform. The SPT-N profile with depth is shown in Fig. 3.

Observation wells were installed in the boreholes to measure changes of groundwater table with time for design and construction monitoring. The measured groundwater level across the site is approximately at RL34.5m, which is about 3.5m to 5m below existing ground level.

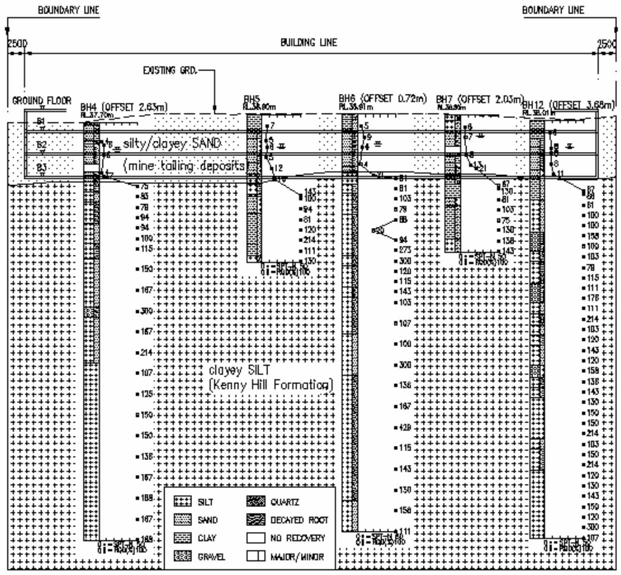


Fig. 2 Typical subsoil profile

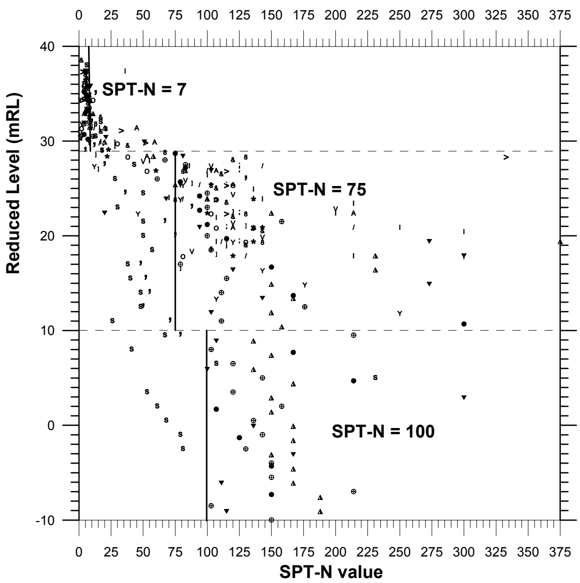


Fig. 3 SPT-N plot with depth

3 FINITE ELEMENT ANALYSES

3.1 *Introduction*

The simulation of the basement excavation processes was carried out using PLAXIS, which is a 2-D Finite Element program suitable for the analysis of deformation problems both in soil and rock. Fig. 4 shows the FEM model of the Wall A at the initial stage utilising 15-node elements under 2-D plane strain condition. Effective stress analyses using Hardening Soil model is adopted to model the soil behaviour in the retaining wall analyses. The soil material is assigned with undrained behaviour and coupled with consolidation analyses to simulate the actual soil behaviour from undrained to drained condition due to dissipation of excess pore water pressure. The undrained soil model coupled with consolidation analyses is able to predict a more representative wall displacement and wall bending moment relative to construction sequence as compared to conventional drained analyses.

The wall is modelled as beam element while the basement slab is modelled as fixed end anchor support. The wall interface elements were introduced to simulate the soil-structure interaction behavior. The original subsoil parameters adopted in the reanalyses are tabulated in Table 1.

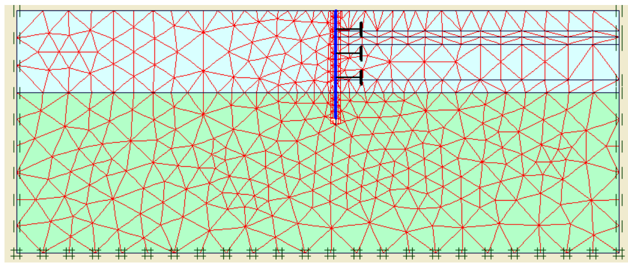


Fig. 4 Finite element modelling of excavation

Table 1 Initial subsoil parameters for FEM analyses

Subsoil Parameters	Above RL28m	Below RL28m
SPT-N	7	75
γ_d (kN/m^3)	16	16
γ_s (kN/m^3)	18	18
k_x, k_y (m/day)	0.086	8.64E-4
E' (kN/m^2)	14,000	150,000
E_{ur}' (kN/m^2)	42,000	450,000
c' (kN/m^2)	1	10
ϕ' (°)	30	35
R_{inter}	0.8	0.8
K_o	0.5	0.43

As shown in Table 1, the effective Young's Modulus (E') and the effective unloading/reloading Young's Modulus (E'_{ur}) are correlated based on 2000×SPT-N and 6000×SPT-N as recommended by Tan *et al* (2001) in view of no other in-situ tests, like pressuremeter tests have been performed at site. It is often appropriate to adopt back-analysed effective Young's Modulus on good quality case history in comparable condition.

Wall interface elements, R_{inter} of 0.67 and 0.5 are commonly adopted for sandy and silty material respectively. However, according to Gaba, A R *et al* (2002), the maximum friction angle, δ_{max} that can be mobilized against the surface of rough wall (concrete cast against soil) would be expected to be the critical state angle of shearing resistance of the soil, ϕ_{crit} as the soil adjacent to the wall will be disturbed by installation of wall. Therefore, the R_{inter} of 0.8, which is a ratio of critical state angle of shearing, ϕ'_{crit} over the angle of shearing at peak, ϕ'_{peak} can be adopted for concrete cast against soil (diaphragm wall) to achieve optimistic design provided the diaphragm wall is not subjected to high vertical load which causes the wall settles relative to the soil.

The excavation sequences simulated in the FEM analyses for both Wall A and Wall B are shown in Table 2 and Table 3 respectively. The excavation sequences are simulated based on actual construction at the site and actual excavation depth in stages (without unplanned excavation) based on weekly instrumentation monitoring programme. As no significant surcharge was observed at the retained ground during the construction period, the 10kPa surcharge as modelled in the original analyses for wall design is not considered in the re-analyses in order to simulate the serviceability condition of the site.

Table 2 Sequence and duration of excavation stages for Wall A

Step	Description of step	Time (months)
1	Install Diaphragm Wall	4
2	Excavate to RL35.38m	2
3	Install B1 Strut Slab	0.5
4	Excavate to RL33.43m	1
5	Excavate to RL30.83m	0.75
6	Excavate to RL29.3	3
7	Install B3 Slab	0.5
8	Install B2 Slab	120

Table 3 Sequence and duration of excavation stages for Wall B

Step	Description of step	Time (months)
1	Install Diaphragm Wall	5
2	Excavate to RL36.2m	0.75
3	Excavate to RL35.27m	1.25
4	Install B1 Strut Slab	0.5
5	Excavate to RL34.29m	0.5
6	Excavate to RL29.3m	3
7	Install B3 Slab	1
8	Install B2 Slab	120

3.2 *Results and Discussions*

The re-analysis results presented in this paper include relative wall lateral displacement and settlement of the retained ground behind the wall. Figs. 5 and 6 show the wall displacement obtained from both re-analyses based on the parameters as shown in Table 1 and actual measurement from inclinometers at Wall A and Wall B respectively. There is generally good and reasonable match between the predicted and measured wall lateral displacement profiles. Therefore, it is evident that the behaviour of the alluvial and residual soils can be correctly modelled using the "hardening soil" model for excavation analyses.

Also, undrained condition coupled with consolidation analyses is able to give good prediction on wall lateral displacement. However, it can be seen that the lateral wall displacement of Wall A at initially cantilever condition (Fig. 5) was slightly overestimated while the final wall lateral displacement is slightly underestimated. These may possibly indicate that the soil exhibits higher stiffness than the adopted stiffness value when the soil operates at smaller shear strain at the early stages of excavation. Additional analyses have been performed to match the actual measured wall lateral displacement at cantilever stage by varying the soil stiffness. The analyses results found that the lateral wall displacement with soil stiffness of 3000×SPT-N at an initial cantilever stage of excavation matched reasonably well with the measured wall lateral displacement. With further excavation stage, the soil stiffness reduced gradually to 1850N at the last excavation stage. It has been a well known phenomenon on reduction of stiffness with increasing shear strain in most geotechnical problems. As such, the reduction of stiffness is anticipated in this particular case and is consistent with the well known phenomenon.

As shown in Fig. 6, the predicted Wall B lateral displacement profile matched well with the measured profile until the stage 3 excavation (Step 5) to RL34.29m. When the excavation reached final excavation level, the wall still behaved like a cantilever wall despite the strut slab at B1 level has been already in place to support the diaphragm wall. The wall displacement profile is different from the predicted profile. However, the wall lateral displacement profile for both predicted and measured seems to be converged in long-term condition. It seems that the wall deformation has not fully developed in Step 6.

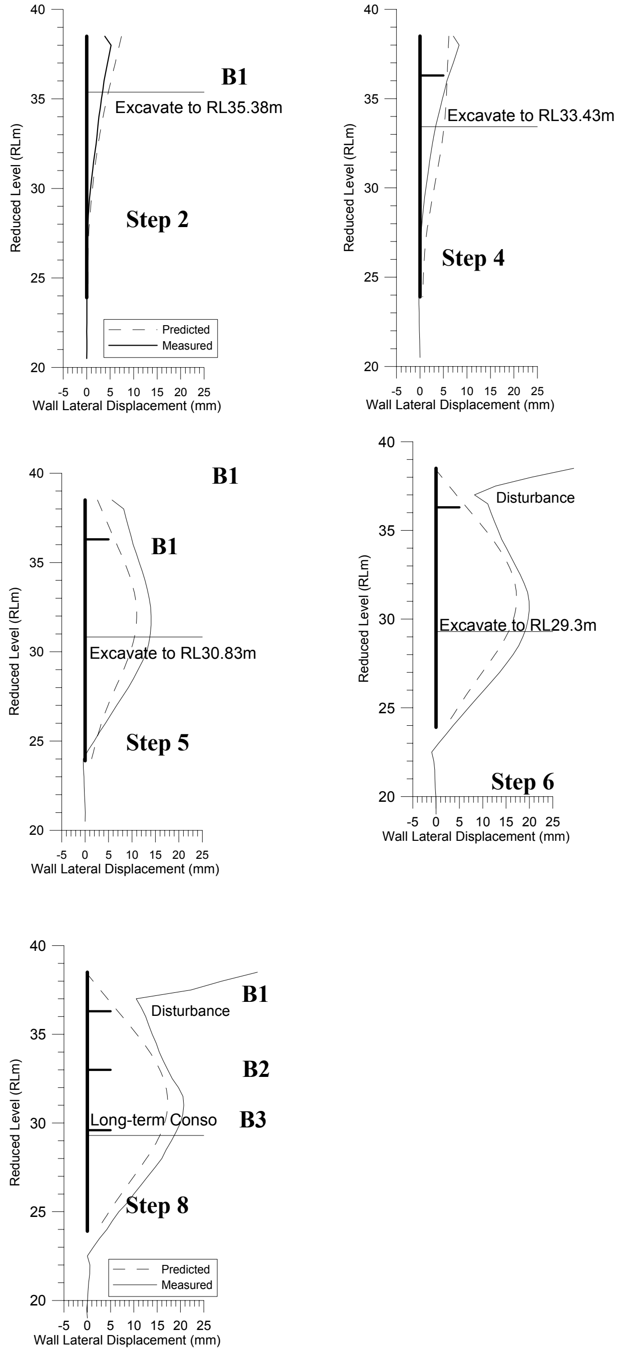


Fig. 5 Wall lateral displacement for Wall A

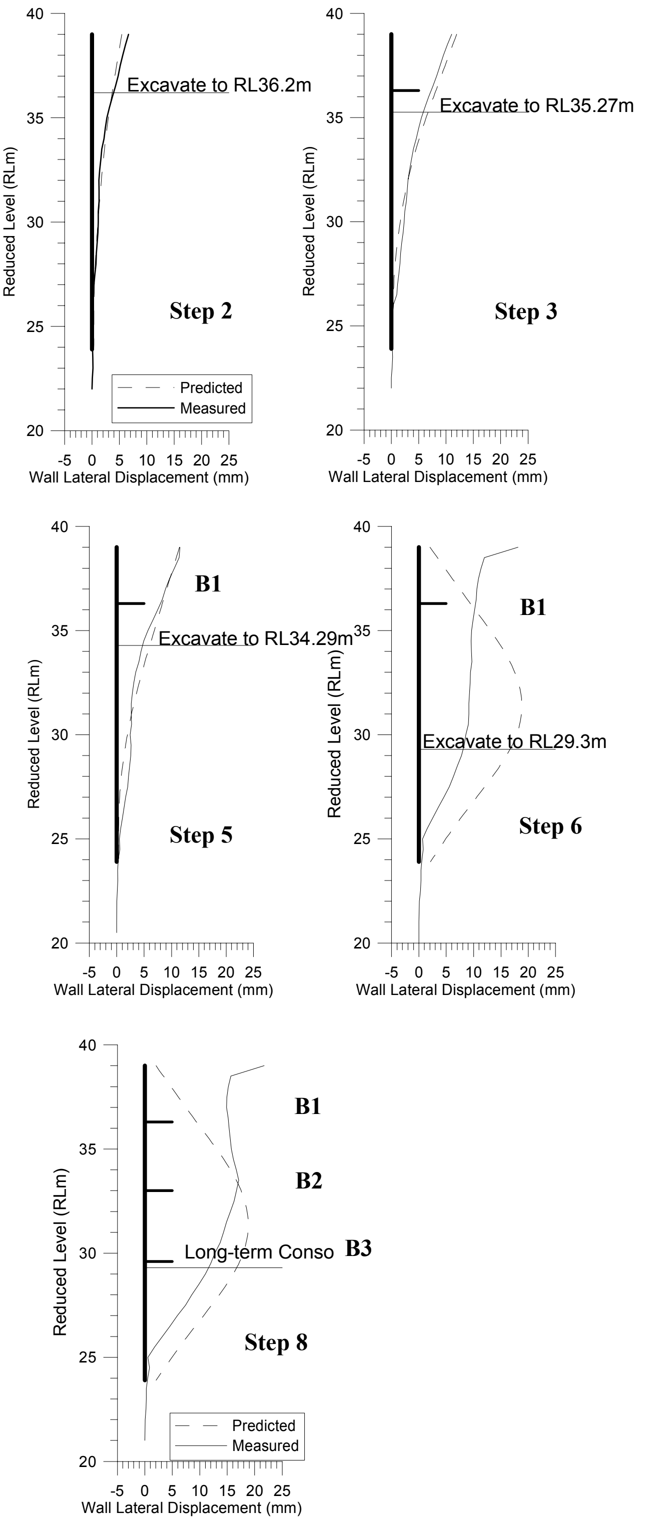


Fig. 6 Wall lateral displacement for Wall B

The maximum wall lateral displacement for both Wall A and Wall B is about 20mm. Given an average maximum excavation depth (D) of 10m, the maximum wall lateral displacement is about 0.2% of maximum excavation depth. Therefore, the semi top down method of construction by utilising B1 slab as propping is proven to be effective in controlling the wall lateral displacement throughout the entire excavation stages.

The ground surface settlements for both wall sections measured from the settlement markers installed a long section perpendicular to the wall at the retained ground and predicted values for Wall A and Wall B are presented in Figs. 7 and 8 respectively. In the finite element analyses, the maximum ground settlements are located at a distance of 5m from the wall, which is half of the maximum excavation depth and tally reasonably with the field measurements. The predicted maximum ground settlement behind the wall is about same magnitude with the wall maximum displacement. However, the maximum measured ground settlement seems to be in the range of 10mm and is about half of the predicted maximum ground settlement, which is also half of the predicted and measured wall lateral displacements. The percentage of the measured ground settlement is about 0.1% of the maximum excavation depth.

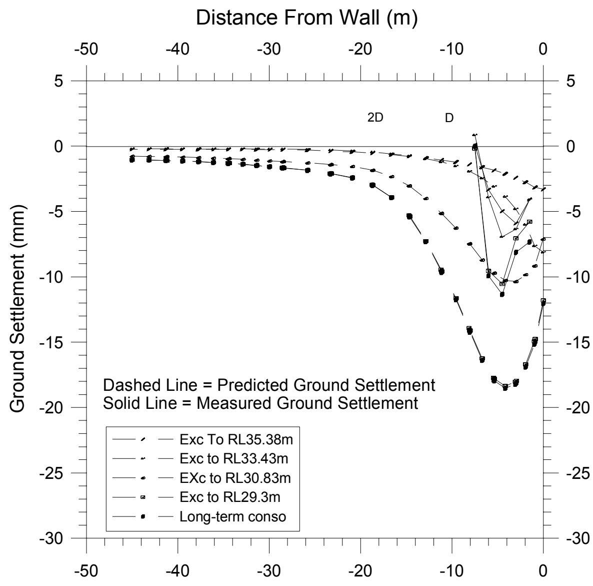


Fig. 7 Ground settlement profile for Wall A

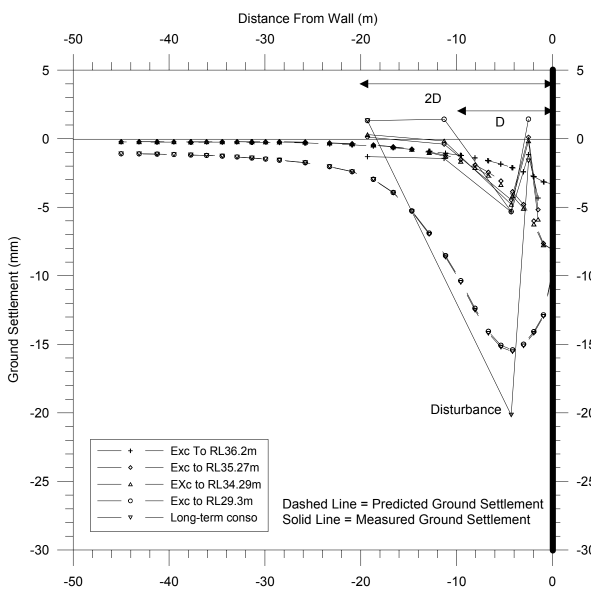


Fig. 8 Ground settlement profile for Wall B

The finite element analyses predicted the distance behind wall to negligible ground settlement would be two times maximum excavation depth, i.e. 20m while the extent of ground settlement was only about one time maximum excavation depth based on the actual ground settlement readings. As evidenced by three (3) observation wells installed at the retained ground, no significant drawdown of groundwater during the excavation was observed behind the wall and this seems to suggest that the wall has effectively cut-off the potential seepage path. In summary, based on the instrumentation monitoring results, the ground settlement behind the diaphragm wall embedded 4m into relatively impervious stiff weathered Kenny Hill material with proper cut-off against water ingress is found to be very localised and generally extended to about one time of the maximum depth of excavation from the wall.

4 CONCLUSIONS

In conclusion, the alluvial and residual soils have been correctly modelled using the "hardening soil" model for this case study of basement excavation. Also, undrained condition coupled with consolidation analyses is able to give a good prediction on wall lateral displacement in time sequence. The FEM predicted wall lateral displacement profile matches reasonably well when comparing to the measured lateral displacement profile. Therefore, the effective Young's Modulus (E') and the effective unloading/reloading Young's Modulus (E'_{ur}) which are correlated based on 2000×SPT-N for initially small shear strain in the earlier excavation stage and 6000×SPT-N respectively are suitable to be adopted for Kenny Hill Formation residual soil. For a more refined FEM modeling, a stiffness reduction with shear strain would be necessary to obtain a more accurate prediction at respective construction stage. In addition, a correct wall interface, R_{inter} also plays an important role to simulate the soil-structure interaction behaviour to give a close estimate on the wall lateral displacement profile and the ground settlement behind the wall.

Both re-analysis results and the instrumentation results also show that the maximum wall lateral displacement is about 0.2% of the maximum excavation depth. The measured ground settlement behind the diaphragm wall embedded 4m relatively impervious stiff material with proper cut-off against water ingress is found to be about 0.1% of the maximum excavation depth and generally extended to about one time of the maximum depth of excavation from the wall. The measured ground settlement is more optimistic than the FEM's predicted ground settlement profile. The maximum ground settlement for both analyses and actual measurement at site coincide at a distance of 5m behind the diaphragm wall, which is half of the maximum depth of excavation.

In summary, the performance of the maximum 11m deep excavation works with ring slab B1 level as a temporary propping system is found to be satisfactory based on the instrumentation monitoring results.

REFERENCES

Gaba, A.R, Simpson, B, Powrie, W. & Beadman, D.R. 2002. CIRIA, 2003, Embedded Retaining Wall: Guidance For Economic Design*, CIRIA Report No. 580, U.K.*.

Tan Y.C. 1997. Deformations of Anchored Diaphragm Walls for Deep Basement at Berjaya Star City, Kuala Lumpur. *Proc. of the Third Asian Young Geotechnical Engineers Conference*, Singapore: pp 121-129.

Tan Y.C., Liew S.S and Gue S.S. 2001. A Numerical Analysis of Anchored Diaphragm Walls for a Deep Basement in Kuala Lumpur, Malaysia. *Proc. of 14th SEAGC*, Hong Kong.