Performance of a 600mm thick diaphragm wall under excavation in firm clay profile

Illikkal Velayudhan Anirudhan[1] and M.K. Annam[2]

[1] Geotechnical Solutions, Chennai, India, 600042, anirudhen@gmail.com
[2] Keller Ground Engineering India Pvt. Ltd., Chennai, India, 600024, madankumar@kellerindia.com

ABSTRACT

Deep excavation for basement construction under restricted setback conditions is necessary these days because of increased underground utilisation in an urban environment. A diaphragm wall with active soil/rock anchors at two levels is adopted for a 12.5m deep basement excavation in mixed soil conditions. The diaphragm wall is resting in weathered rock available at about 19.50m. The first level anchor could be accommodated only at 6.0m depth from the top because of the subsoil conditions and the elevation of basement floor slabs. The performance of the retention system was analysed using 2D finite element program PLAXIS 2D, and the estimated wall deflection was not of cantilever type and the maximum bending moment was on the excavation side. The actual deflection of the wall was measured at different locations during different excavation stages. Cantilever deflection pattern was recorded for the first 6.50m excavation, much different from the estimated deflection profile. The wall deflection increased suddenly in two steps after the site received heavy rains. This paper discusses the estimated & measured wall deflections along with back-analysis and some difficulties faced with the Plaxis 2D analysis for the present case.

Keywords: Diaphragm wall, Plaxis 2D, deep excavation, wall deflection, inclinometer, undrained B

1. INTRODUCTION

Deep excavation for basement construction under restricted setback conditions is required these days because of increased underground utilisation in an urban environment. Mixed soil conditions with sand, weak clay and sandy clay layers and shallow groundwater are seen in many construction sites. The stability of deep excavations is achieved by inclusion of piled walls, sheet piles or diaphragm walls. Cross struts are not encouraged since these elements would cut down the progress apart from the restrains in the subsequent constructions. Top-down construction is not yet prevalent in India, and often the retaining wall needs to be held back by introducing ground anchors at different levels.

Diaphragm wall with active soil/rock anchors at two levels is adopted for a 12.5m deep basement excavation in mixed soil conditions. The sub-soil profile comprised firm to stiff clay layers and sand layers. The diaphragm wall is resting in weathered rock available at about 19.50m. The first level anchor could be provided only at 6.0m depth from the top. The reason for keeping the first level of anchors at 6.5m was to accommodate the elevation of basement floors as per the project specifications. The performance of the retention system was analysed using 2d finite element program PLAXIS 2D, in which all the soil layers were modelled as Mohr-Coulomb (MC). The analysis, however, did not produce expected cantilever deflection pattern for the first 6.50m excavation without anchors.

This paper discusses the analyses of the retention system during the design stage, actual measured performance of the wall and post-construction analysis. The study concludes that the expected cantilever deflection pattern is not captured by 2D finite element analysis using Plaxis 2D and lesser bending moments are estimated when the profile comprises firm to stiff clay layers at shallow depths, and undrained analysis with procedure B is adopted.

A realistic analysis requires to input failure modes manually for capturing all the possible failure conditions. This paper is discussing the performance of the wall under excavation up to 6.50m from the ground level only.

2. SUBSOIL CONDITIONS AND ANALYSIS OF RETENTION SYSTEM

The construction of a commercial building with three basement floors requiring roughly 12.5m excavation is underway. The subsoil comprises firm to stiff high plasticity clay layers of varying consistency up to about 15m followed by sand layers and weathered rock. The stratification along with the essential soil data is presented in Table 1.

The excavation was designed with support of 600mm thick diaphragm wall and soil anchor tie backs at two levels. The 600mm thick diaphragm wall resting in very dense residual soil at about 19.50m depth (-3.40m EL) is used. The first excavation was proposed up to 6.50m depth for installing the anchors at 6.0m depth. Dewatering from the excavation side was planned.

Table 1. Sub-soil stratification at project site

Sl No	Type	Elevation, m	N	c_u kPa	ϕ' (°)	E' kPa
1	Firm S clay	15.8 - 11.8	15	90	-	28800
2	Firm clay	11.8 - 6.8	16	96	-	30720
3	Firm clay	6.8 - 4.3	10	60	-	16800
4	Soft clay	4.3 - 1.3	8	48	-	12000
5	Sand	1.3 - $\overline{0.2}$	65	-	42	90000
6	Sand	$\overline{0.2}$ - $\overline{1.7}$	85	-	42	180000
7	Sandy clay	$\overline{1.7}$ - $\overline{4.0}$	90	400	-	200000
8	w.j. rock	$\overline{4.0}$ - $\overline{9.2}$	300	1000	-	250000

The reason for keeping the first level of anchors at 6.5m was the elevation of the intermediate basement floor levels. Poor soil condition to considerable depth offering poor anchor capacity was also one of the reasons. By experience, 600mm thick D wall with more than 12m embedment and acting as free cantilever was not expected to undergo large deflection.

Preliminary estimations suggested that the cantilever wall deflection is about 35mm as the soil is firm to stiff clay. The retention system was analysed in detail using Plaxis 2D finite element programme. Standard MC model parameters were derived and used in the analysis. The clay layers were modelled as Undrained B soil in which the shear strength is defined from undrained cohesion c_u (ϕ=0) and stiffness parameters as drained effective ones. Sand is modelled as drained type with dilatancy ψ=6°. The soil-wall interface reduction factor was assumed 0.70. The design groundwater table was at 2.0m below the surface, and the water level in the excavation area was lowered to 7.0m depth to facilitate the excavation.

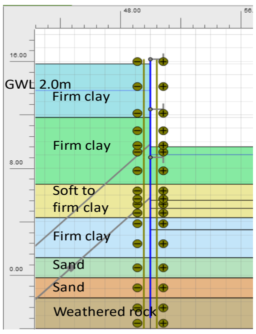

Fig. 1 Plaxis 2D model of excavation

The Plaxis 2D model used in the analysis is shown in Figure 1. The lateral deflection profile and the bending moment distribution of the wall obtained from the study are presented in Figures 2a and 2b. The wall deflection is not typical of a cantilever deflection and the bending moment is developing on the excavation side of the wall.

The estimated wall deflection was 25mm at wall top and 30mm at the excavation level after 6.50m excavation. The reversal of BM distribution could not be understood, and the analysis was repeated using the Hardening Soil (HS) model for the top clay layers using undrained B parameters. The results were very similar with only some differences in the deflection and BM values. Review of the literature did not provide a suitable answer for the BM

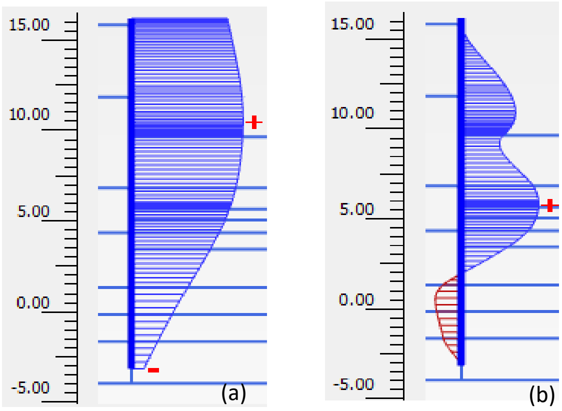

Fig. 2 (a) Wall deflection (b) BM distribution after 6.5m excavation.

distribution on the excavation side for the cantilever wall. The analysis was made for the full depth of excavation with two level anchors, and 20% increase on the estimated BM was recommended. The main steel corresponding to the maximum BM was provided for the full wall depth. Undrained analysis was conducted as the first level excavation and anchor installation was expected to be completed in a short period of four to five weeks.

3. CONSTRUCTION AND MONITORING

The diaphragm wall was constructed in panels of 6m wide using the hydraulic grab and polymer as stabilising fluid. Inclinometers were installed for the full depth of the wall at different locations, mostly towards the centre of the walls. The monitoring of the walls initiated just at the time of excavation. Counter measurements were made using total station observations.

The excavation up to 6.50m depth was initially done for two wall lengths of 65m and 45m. The weather was dry during this excavation. The actual deflection of the wall was measured during different excavation stages by continuous monitoring of the inclinometers. Cantilever deflection pattern was registered for the first 6.50m excavation, much different from the estimated deflection profile and about 32mm deflection at the top and close to 12mm at the excavation level were recorded. The predicted

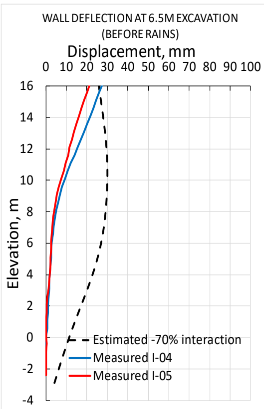

Fig. 3 Predicted and measured wall deflection @ 6.5m excavation

and measured wall deflections are presented in Fig. 3 for comparison.

The construction of anchors and pre-stressing did not progress after the excavation as planned, and the installed ones were not pre-stressed for almost a month. The wall deflection, however, remained more or less stable at about 32mm. The groundwater table stood at 2.0m 2.5m depth. The site received heavy rains at this stage, and the wall started moving and reached a stabilised maximum deflection of about 65 mm at the wall top. The groundwater table was at 0.50m below ground level after the rains. Even though the rains stopped, further work could not commence for another 15 days.

The wall moved to roughly 85mm suddenly after 15 days and then again stabilised at about 90mm at the top. The site observed 20mm to 25mm gap between the wall and the soil during the initial observations of 30mm deflection of the wall. This detachment was for about 3m from the surface. The

Fig. 4 (a) Gap between the wall and the soil @ 30mm wall movement (b) @ 45mm wall movement after rains

ground settlement was not observed initially. Figure 4a is presenting the site observations after the wall moved around 30mm and Figure 4b is when the wall further moved immediately after the rains.

4. BACK ANALYSIS AND DISCUSSION

The wall deflection was more than twice the predicted deflection that warranted a stability study. The difficulty in capturing the cantilever wall movement by Plaxis 2D analysis could have caused structural instability when the excavation depth without lateral support is more as in the present case. It is, hence, necessary to look into the reasons for such unusual results from the Plaxis 2D analysis. Ideally, the excavation without lateral support should have been limited to about 4.0m.

Literature on back analyses of the performance of the instrumented diaphragm walls with struts and anchored tiebacks showed that Hardening Soil (HS) model predicts the wall deflection more realistically than the Mohr-Coulomb (MC) model [Suched Likitlersuang et al. (2013) and Bin-Chen Benson Hsiung and Sy-Dan Dao (2014)]. Accordingly, HS model with undrained B parameters was tried in the present case, but the results were similar.

The problem was again analysed again using Plaxis 2D for replicating the actual performance in the field. The presence of a gap between the wall and the soil developed during the initial stages suggested no interaction between the wall and soil unless until the soil mass moved towards the wall. The presence of water in the gap is also making the interface slippery. There was no evidence of 'softening' due to the dissipation of negative pore pressure. The bending of the wall towards the soil side during the initial analysis is expected to be due to the soil-wall interaction. The soil mesh and the wall glued to each other during the computation. The new study was carried out with zero interaction between the wall and the top soil layer and the estimated wall deflection along with the observed one is presented

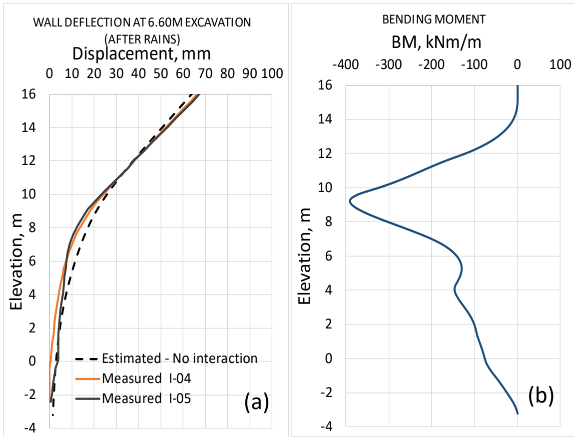

Fig. 5 (a) Estimated and observed wall deflection @ 6.5m excavation and rains (b) Estimated BM distribution

in Figure 5a. The corresponding BM distribution is shown in Figure 5b. The estimated and observed wall deflection were comparable, and the analysis was fruitful in confirming that the bending moments were well within the design limits. However, the sudden movement of the wall further by about 30mm after maintaining a stable position for about 15 days given suspicion to a wedge failure of the soil mass behind the wall. There was minor subsidence behind

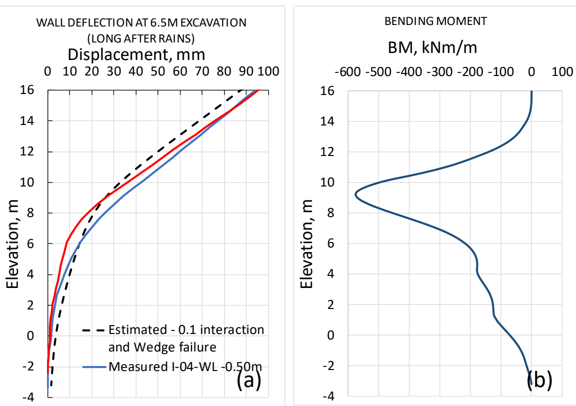

Fig. 6 (a) Estimated and observed wall deflection @ 6.5m excavation and long after the rains (b) Estimated BM distribution the wall partially closing the gap at the surface. The analysis was performed manually inducing 45° wedges. This analysis was necessary for ascertaining the structural stability of the wall. The results of the analysis along with the site measurements are presented in Figure 6a. The corresponding BM distribution is shown in Fig. 6b.

In this case also, appreciable match was observed between the estimated and measured deflection and the BM distribution resulted from the analysis was used to confirm the structural safety. The differential bending deflection at this stage is 1/1030, well within the structural acceptability limit of 1/300.

The analysis was further done from this stage for the remaining excavation steps so that the stability was ensured. Since the further excavations were continued after providing the inclined anchors as designed, the wall deflections and bending moments remained within the estimated values. Figures 7a and 7b illustrate measured wall deflections comparable with the estimated ones.

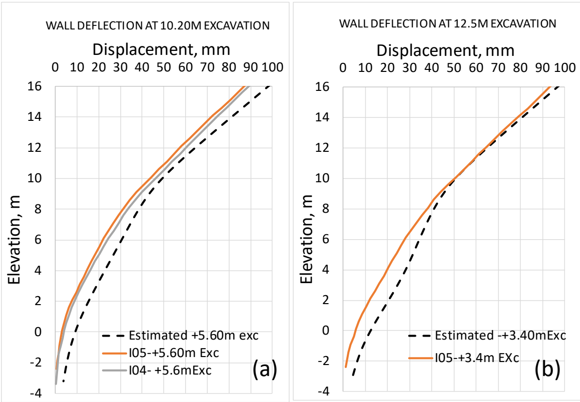

Fig. 7 (a) Estimated and observed wall deflection @ 10.2m excavation and first level anchors (b) @ 12.5m excavation and 2nd level anchors

The assumption of no interaction with the wall may be the right approach, but may not be suited to all site conditions. Figures 8a and 8b illustrate the shear stress distribution on the D wall in the design stage estimation with 0.7 interface factor and zero interface factor for the top layer. The shear stress in the D wall top portion suggests a propping action forcing the wall bend towards soil when interface reduction is 0.7 that is not happening in reality.

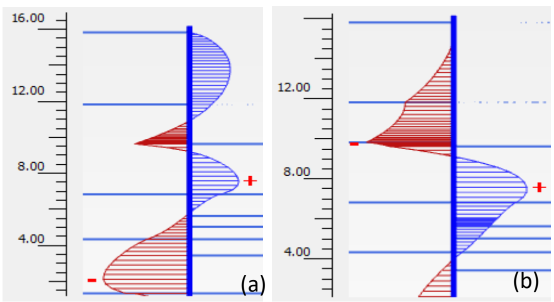

Fig. 8 Shear distribution in the wall @ 6.5m excavation (a) with 0.7 interface constant (b) with 0.01 interface

Further, analyses of a 15m deep wall in a single layer soil profile comprising sand and firm clay with 6.0m excavation were performed assuming 0.7 interface reduction factor. The clay was modelled as undrained B with c_u as the input parameter. The results are presented in Figures 9a and 9b. Even though the wall is cantilevered for 6.0m in both the cases, the BM distributions are on different sides of the wall.

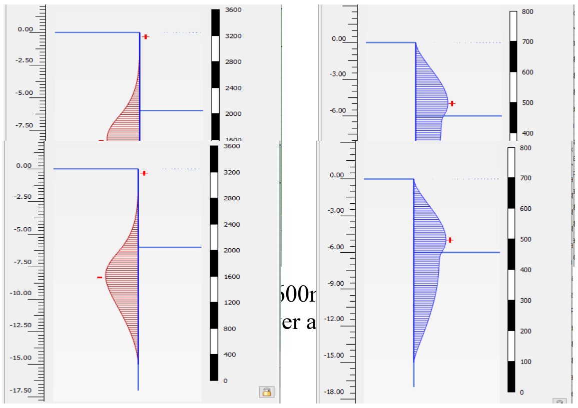

Fig. 9 BM distribution on a 600mm thick wall @ 6.0m excavation (a) in sand layer and (b) in clay layer

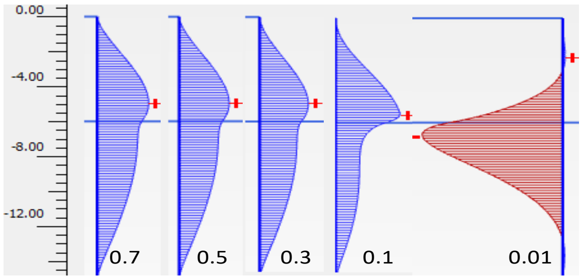

Fig. 10 BM distribution on a 600mm thick wall @ 6.0m excavation for different soil wall interface constants

Analysis was performed with different interface reduction factors for clay, and the results are shown in Figure 10. The analyses show that the BM distribution remains more or less same on the excavation side for interface constants more than 0.01 and it reverses as the interface is almost zero. Since this analysis did not provide any insight into the reasons for the anomaly in the moment distribution, further study is done modelling the clay as undrained A type with c' and ϕ'. It is seen that as the c' increases for accommodating high undrained strength, the bending moments are on the excavation side and the reversal occurs when c' is as small as 10 kPa. The apparent prop action comes into play with large cohesion intercept and always when undrained B model is used.

5. CONCLUDING REMARKS

The Plaxis 2D analysis modelling firm clay with undrained B procedure did not capture the cantilever behaviour of a non-strutted wall. Analyses with zero interface and manually induced wedge failure showed agreement with the actual performance. The present case study reveals the limitations when using undrained B model and the necessity of additional manual input during the design stage for predicting more realistic field performance.

Even though 6.0m cantilever wall is stable in firm to stiff clay, the cantilever length of the wall should have been limited to 4.0m for stability under unforeseen environment as experienced in the present case.

6. REFERENCES

Bin-Chen Benson Hsiung and Sy-Dan Dao (2014) Evaluation of Constitutive Soil Models for Predicting Movements Caused by a Deep Excavation in Sands, EJGE, Vol. 19, pp 17325-17344.

Likitlersuang S, Surarak C, Wanatowski D, Erwin Oh, and A. Balasubramaniam (2013), Finite element analysis of a deep excavation: A case study from the Bangkok MRT, Journal Soils and Foundations by The Japanese Geotechnical Society. Elsevier B.V., Volume 53, Issue 5, October 2013, Pages 756-773.