OBSERVED BEHAVIOUR OF A DEEP EXCAVATION IN TAIPEI

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ABSTRACT

This paper summarises the observed behaviour of a well-instrumented deep excavation in Taiwan, the BL12 underground station on the Nankang Line of Taipei Mass Rapid Transit System. The 16m deep excavation was retained by diaphragm walls, and also was internally braced by seven levels of struts. The major focus of this paper is on the lateral deflection of retaining wall, the associated ground settlement and heave, and the variation of pore pressures and prop loads during excavation.

INTRODUCTION

Owing to space limitations in urban areas, deep excavations are often constructed in close proximity to surrounding buildings and services. In the design of deep excavations, the design of the retaining structure and support system plays a very critical role. In the past, it was normal to use simplified limit equilibrium methods for analysis of the retaining system. More recently designers have started to use several methods which take deformation into account, such as the beam-on-springs methods, beam-on-elastic continuum method, and finite element methods although the latter are not widely used in current design of walls. However, before such methods of analysis can be adopted for reliable use in design it is essential that they be calibrated against high quality case records. In examining case records of diaphragm wall excavations, the data from field monitoring need to be careful reviewed to identify the key aspects of ground behaviour.

In this paper, the observations made during the construction of the Dr. San Yat-Sen Memorial Hall (BL12) Station on Phase 1 of the Mass Rapid Transit (MRT) system in Taipei, Taiwan will be described. Comparison will be made with previous case records, and possible reasons for differences will be discussed. It is hoped that the observation will provide a good reference for back-analysis of the excavation in the future.

THE SITE

Dr. San Yat- Sen Memorial Hall (BL12) Station is located at the junction of the Chung- Hsiao East Road and Kung- Fu South Road in Ta- An District of Taipei City (Moh and Associates Inc., 1995). The length and width of the BL12 station are 256m and 20m respectively, and its structure is a two-level reinforced concrete box extending to 16.2m below the ground level. The adjacent ground is retained by 1m thick, 33m deep diaphragm walls. This station was constructed by what is known in

Taipei as the "semi-bottom-up" method. In this method, excavation is first made down to the level of the 1.5m thick roof slab, which is then cast and provides permanent support to the diaphragm walls. Excavation is continued to the full depth, with five levels of temporary steel props to give lateral support, before the station box is completed. Table 1 shows the typical construction sequence for the BL12 station site, and Figure 1 shows a generalised cross section of the station.

The major soil strata at this site are the Sung- Shan formation. Figure 2 shows the typical soil profile in the BL12 station area (Moh and Chin, 1991), from which it may be seen that the liquidity index of the clay is close to 1 from surface all the way down to a depth of about 30m. In the past pumping from deep aquifers has under-drained this area of Taipei causing considerable consolidation settlements, and pumping still continues albeit at a reduced rate. A profile of undrained shear strength is given in Figure 3 (Chin et al., 1994), from which it may be seen that the undrained shear strength at the base of the excavation i.e., depth of 16.2m, was around 70 kPa. The angle of shearing resistance determined from consolidated undrained triaxial tests has been measured as 27°- 32° for the Sung- Shan formation.

INSTRUMENTATION AND OBSERVED BEHAVIOUR

Instrumentation was installed at the site primarily to ensure the safety of the works and surrounding property during construction. The instrumentation included permanent bench marks and precise levelling markers to monitor surface and building settlements, inclinometers to observe the lateral wall displacements, open standpipes and electrical piezometers, total pressure cells on the diaphragm walls, and vibrating wire strain gauges and load cells to monitor the prop loads.

Figure 4 and Figure 5 show the piezometric levels observed at the BL12 station. The site data shows clearly that the pore pressure was not initially hydrostatic before excavation. A perched water level table is present 1- 2m below ground level (bgl), while the piezometric level was initially 12m bgl in gravel layer at 51m depth. This distribution is consistent with the under-drainage noted above.

Outside the station box, the piezometers PS1 PS2, PS5, PS6 showed that the piezometric levels at 12m bgl were 1- 2m bgl before excavation, and reduced by 2-3m during construction. At greater depths the piezometric levels reduced from 8 to 12m bgl at 32m bgl, and from 10 to 13m bgl at 37m bgl during construction. Surprisingly the piezometric level in the gravel layer at 51m bgl reduced during excavation from 17 to 22m bgl, and then recovered to its original level as the box was completed. Inside the excavation most of the piezometers were damaged during construction, so no pore pressure data is available. However the piezometers beneath the two working shafts, one at each end of the station, provide a little useful information. There the pore pressures reduced as soil was excavated till the end of construction. The piezometric levels in the clay on both excavated and retained sides rose slightly afterward.

Figure 6 shows the field data from inclinometer SID3; in interpreting the inclinometer data it has been assumed that tip is fixed since it extends several metres below the toe of the diaphragm wall. It shows a maximum lateral wall deflection of 47mm at the end of main excavation, which increases to 54mm at the end of construction. It may be noted that small outward movements are recorded after the roof slab was in place during the final stage of excavation. There was no independent check on the convergence of the diaphragm walls, but it seems likely that this set of data is slightly inaccurate.

The maximum surface settlement during the excavation was 64mm, but this increased after the completion of the excavation as the station box was finished (see Figure 7). Movements extended out to a distance of 50m, three times the depth of the excavation.

The field data from the prop load cells and vibrating wire strain gauges at three locations (Districts 2, 6 and 14) is presented in Figure 8 which shows the variation of average prop load at each level for different stages of construction.

DISCUSSION

Lateral wall displacement

In the first stages of construction the diaphragm wall deflected as an unpropped cantilever (see Figure 6). A long time was taken to remove the utilities in the early stages of construction and it appears that the level 1 props were largely ineffective. Even when the 1.5m thick station roof slab was cast this did not immediately arrest the movements. This was probably because in the first few weeks after the slab was cast there were thermal, shrinkage and creep movements which effectively pulled the diaphragm walls inward by around 10mm. At this stage (stage IV) the ratio δ_{hmax}/ D of the maximum lateral wall deflection (δ_{hmax}) to the current excavation depth (D) was nearly 0.6%, which was much larger than the ratio for the remainder of construction.

From then on the roof slab stopped further inward movement of the top of the diaphragm walls, and the deflected profile changed to the more usual propped mode. The ratio δ_{hmax} / D for the remainder of construction varied from 0.2% to 0.28%. In a survey of a number of excavations on Taipei MRT system, Wu et al., (1997) concluded the ratio of (δ_{hmax}/D) was generally 0.07% to 0.2% while at other sites it sometimes reached 1.1%. The reason for this is perhaps that the construction of deep diaphragm walled excavations on the Taipei MRT system had somewhat better quality control than previous construction in the city. The ratio at the BL12 station site is thus similar to earlier cases on the MRT, reported by Wu et al.(1997).

The ratio of diaphragm wall thickness to the maximum excavation depth at the BL12 station site was 6% which is not unusual for such excavations. It has been of interest to compare the lateral displacement profile with that for a slightly shallower excavation in stiff Gault clay at Lion Yard, United Kingdom constructed by top down methods with which the second author is familiar (Lings et al, 1991). There the maximum lateral movements were of the order of 12mm at the end of the 10m deep main excavation. At Lion Yard the ratio δ_{hmax} / D was 0.12% which is only about half of that was observed at the BL12 station site. Furthermore the maximum lateral movement at Lion Yard occurred above final excavation level whereas that at BL12 was deeper. The ratio $\gamma H/s_u$ (where H is the maximum depth of excavation and γ and s_u are the average total unit weight and undrained shear strength of the clay) gives an indication of the likelihood of base heave in a deep excavation. This has been calculated for both sites, and at BL12 is equal to 4.3 (based on s_u = 70 kPa) whereas at Lion Yard it was equal to 1.2 (based on s_u = 150 kPa). With a ratio greater than 4 clearly base stability could have been a problem at BL12 had not the stiffness of the diaphragm walls below the excavation prevented collapse. It should be noted that despite prestress the struts were not able to prevent the large lateral movements below them. O'Rourke (1993) has developed a design chart, refer to Figure 9, which relates expected lateral deflections (δ_{hmax}/ D) to the stiffness of the wall and the factor of safety against base heave. The data from the BL12 station site have been plotted in this way and suggest that the factor of safety against base heave was around 1.4.

Ground settlement behind the wall

The maximum ground settlement behind the wall amounted to nearly 90mm at the end of construction (stage XXIII) which is approximately 0.5% of the excavation depth. For a well constructed excavation in soft to firm clay in which there is low risk of base heave Clough and O'Rourke (1990) suggest that settlements will not exceed 1.0%.

The pattern of settlements is also different to that normally associated with such excavations. Settlements of around 10mm at distance of twice the maximum depth of excavation from the wall are larger than expected, and indeed settlements extended to a distance of up to three times the excavation depth. This may perhaps have occurred because significant plastic deformations were occurring below the base of the excavation, and may also be due to consolidation caused by reduction of pore water pressures in intermediate and underlying sand/gravel layers. Furthermore the maximum settlement was immediately adjacent to the walls and not at a distance of half to three-quarters of the excavation depth behind the wall as generally observed in Taipei (Wong and Patron, 1993) and elsewhere (Clough and O'Rourke 1990). This may be associated with the cantilever profile of the wall in the early excavation stages and the settlements of the wall.

The ratio of the maximum ground vertical displacement (δ_{vmax}) to maximum horizontal displacement (δ_{hmax}) increased from 0.91 at stage IV to 1.55 at the end of monitoring, and is slightly larger than the ratio from former excavation cases in Taipei area (Woo and Moh, 1990). There are at least three possible causes of surface settlement in retained side during the excavation: lateral wall movement, consolidation settlement from the variation of pore pressure, and the creep of soil. A comparison has been made of the volume change of the settlement trough outside the excavation with the volume of the clay moving laterally towards the excavation. During the main excavation the two volume changes were almost the same, but after the end of main excavation, the volume change calculated from lateral wall movements was only half of that calulated from surface settlements. Thus in the later stages of construction it is clear that ground settlements were continuing despite small lateral movements of the walls. Examination of the piezometric records (Figure 4 and Figure 5) show that pore pressures gradually increased during the later stages of construction. Thus there is no sign that consolidation was occurring - if anything the ground might have been expected to swell. Thus the most likely reason for the continuing movements is creep compression strains in the clay; in a lightly over-consolidated clay the stress changes during construction would have taken the state of the clay to the state bounding surface - at which there would be significant creep. This has implications for the design future excavations in Taipei.

Piezometric behaviour

In assessing the piezometric response of the clay it is thought that the clay outside the excavation remained undrained during construction. Thus the modest reductions in pore pressure arose from undrained lateral unloading as the wall moved laterally. It is possible that the response was affected by the presence of a sand layer some 9 metres below ground level.

Inside the excavation one would expect large reductions of pore pressure as the overlying clay is excavated. Unfortunately most piezometers inside the box were destroyed during excavation. Surprisingly the few electrical piezometers beneath the shafts at the ends of excavation which did survived showed only modest reductions of pore pressure, with piezometric levels stabilising quickly at around the final excavation level. This is in marked contrast to those at Lion Yard (Nash et al. 1996) where suctions developed in the clay beneath the excavation which dissipated only slowly. Such a limited response at BL12 could be due to one of several reasons; the clay might have been only partially saturated or alternatively the coefficient of swelling was rather high allowing excess pore pressures to dissipate quickly. Another possibility is that lateral loading at depth resulted in positive pore pressures which compensated for the negative pore pressure caused by vertical unloading. Unfortunately there is insufficient data on this site to draw firm conclusions.

It is rather surprising that significant reductions of piezometric level occurred in the underlying gravel aquifer 50m bgl. It is thought that these were unconnected with the construction of the BL12 station, but they might have influenced the pore pressures in the overlying clay layers and caused some additional consolidation.

Prop loads

The maximum prop load data have been analysed to derive distributed prop load diagrams similar to apparent earth pressure diagrams proposed by Terzaghi and Peck (1948). These are presented in Figure 10 which shows the maximum distributed prop loads at the Stage XV for each level of props derived from observations of vertical sets of props at three locations on the site. A recent study for Twine and Roscoe (1997) for the UK research organization CIRIA has updated Terzaghi and Peck's original design envelopes and the CIRIA distributed prop load diagrams design envelopes are also shown in Figure 10. It may be seen that the prop loads at BL12 fall well within the CIRIA design envelopes (of 1.15 γH) for stiff walls in soft clays where there is a risk of base heave.

Heave and swelling movements

Two possible upward earth movements beneath the excavation might occur in deep excavations. Heave of the soil occurs simultaneously with excavation, and is due to plastic deformation of the clay. The stress relief normally results in negative excess pore pressures in the underlying clay as noted above, and swelling occurs over subsequent months or years as the negative excess pore pressure beneath the excavation dissipates. At Lion Yard (Nash et al, 1996), the upward heave of the base of the excavation during construction amounted to around 30mm whereas subsequent long term swelling has increased this to 110mm some 5 years later, nearly 3.25 times the heave during excavation (see Figure 11). Such upwards movement could have serious consequences for some important underground structures, such as underground MRT stations, tunnels, etc., if it has not been anticipated in the design.

At BL12 no data is available on upwards movement of the clay inside the box. It is likely that large heaves occurred associated with the inward movement of the walls during excavation, but if the piezometers beneath the shafts are believed there were negligible resulting excess pore pressures. In that case it seems unlikely that swelling will have occurred - and indeed there was possibly some further consolidation as the vertical load from the dead weight of the station was reapplied.

Overall performance of instrumentation

While the instrumentation data from the BL12 site is not fully complete the extant data is of sufficient quality that it provides a very useful case record. For back-analysis purposes it is essential to make sufficient measurement of prop loads that the data can be used to carry out an equilibrium analysis of the wall. In this way the net earth pressure and bending moment diagrams may be obtained (Lings et al 1993) for comparison with numerical analysis.

Surface settlement monitoring was successful, but while the majority of movement occurred as a result of main excavation, there was little reliable field data which could be used to assess the effects of diaphragm wall installation. Although it is likely that these movements were small, it would be useful on future projects to monitor lateral and vertical ground movements carefully in the earliest stages of construction since the analysis of wall installation is a very important but difficult part of the design of such an excavation. It is also noted that no measurements were made of vertical movement of the diaphragm walls. Such measurements require the installation of settlement monitoring points immediately after panel construction which is not always convenient.

The inclinometers appeared to have performed satisfactorily although it should be noted that the assumption that they were fixed at their toe probably led to small errors. On future projects it would be advantageous to consider an independent check on horizontal movements at the top of the inclinometers. No measurements of movements of the ground inside the box were made with extensometers; like standpipe piezometers, such instruments are extremely vulnerable. To obtain the results at Lion Yard it was necessary to have full time presence of a researcher (Ng, 1992) on site to liaise closely with the contractor to minimise the risks to the instrumentation.

Two different types of piezometers were installed at BL12 - open standpipe piezometer and electrical piezometers. While the former were very satisfactory for the granular layers there is some possibility that there their response was too slow in the lower permeability clay layers. The electrical piezometers had a rapid response time and performed very satisfactorily in the clay. Ideally for future projects electrical or pneumatic type piezometers would be installed in clay layers both inside and outside the box and strenuous efforts made to maintain them during construction.

The data from the BL12 site shows that by the end of construction, pore pressures had not reached equilibrium and the surface settlement was still increasing. This suggests that on future projects some selective monitoring would be useful after the end of construction until movements and pore pressures stabilize.

CONCLUSIONS

The instrumentation data from the BL12 site presented here is of sufficient quality that it provides a very useful case record. The following main conclusions have been drawn:

1. As with other excavations on the Taipei MRT system, this site had better quality control than former cases in Taipei.
2. At the final excavation stage large lateral movements of the diaphragm wall occurred at depth. The deflected shape of the diaphragm wall is consistent with significant plastic deformations of the clay having occurred below dredge level.
3. The piezometric changes in the clay outside the box were small, and the limited data from the clay beneath the excavation suggests that final excess pore pressures were very small.
4. The creep of the Taipei clay resulted in significant ongoing settlement outside the box even after lateral movement of the wall had ceased at the end of construction.

5. Suggestions have been made for the monitoring of similar projects in the future including measurements of lateral and vertical movement of the top of the wall and piezometric changes and movement of the clay beneath the excavation.

As this paper mentions, such reliable in-situ monitoring data is essential for the calibration of numerical models, and it is planned to undertake such back-analysis in the future.

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Construction Stage	Construction Operation	Construction Period		Day No
		From	To	
I	Construction of Diaphragm Wall		04/03/93	0
II	Excavation to 2.1m bgl	26/03/93	31/03/93	22- 27
III	Installation of level 1 props	04/04/93	04/04/93	31- 31
IV	Excavation to 4.1m bgl	24/04/93	27/04/93	51- 54
V	Casting of top slab	19/06/93	19/06/93	107- 107
VI	Removal of level 1 props	26/06/93	26/06/93	114- 114
VII	Excavation to 6.6m bgl	27/07/93	31/07/93	145- 149
VIII	Installation of level 2 props	14/08/93	14/08/93	163- 163
IX	Excavation to 9.2m bgl	29/08/93	01/09/93	178- 181
X	Installation of level 3 props	02/09/93	09/09/93	182- 189
XI	Excavation to 11.7m bgl	21/09/93	23/09/93	201- 203
XII	Installation of level 4 props	24/09/93	28/09/93	204- 208
XIII	Excavation to 13.9m bgl	04/10/93	07/10/93	214- 217
XIV	Installation of level 5 props	12/10/93	12/10/93	222- 222
XV	Excavation to 16.2m bgl	17/10/93	19/10/93	227- 229
XVI	Casting of bottom slab	07/11/93	07/11/93	248- 248
XVII	Removal of level 5 props	02/01/94	02/01/94	304- 304
XVIII	Removal of level 4 props	03/01/94	03/01/94	305- 305
IXX	Installation of level 7 props	27/01/94	27/01/94	329- 329
XX	Removal of level 3 props	18/02/94	18/02/94	351- 351
XXI	Casting of concourse level slab	18/03/94	18/03/94	379- 379
XXII	Removal of level 7 props	04/04/94	06/04/94	396- 398
XXIII	Removal of level 2 props	04/04/94	09/04/94	396- 401

Table 1 Construction sequence at the BL12 station site

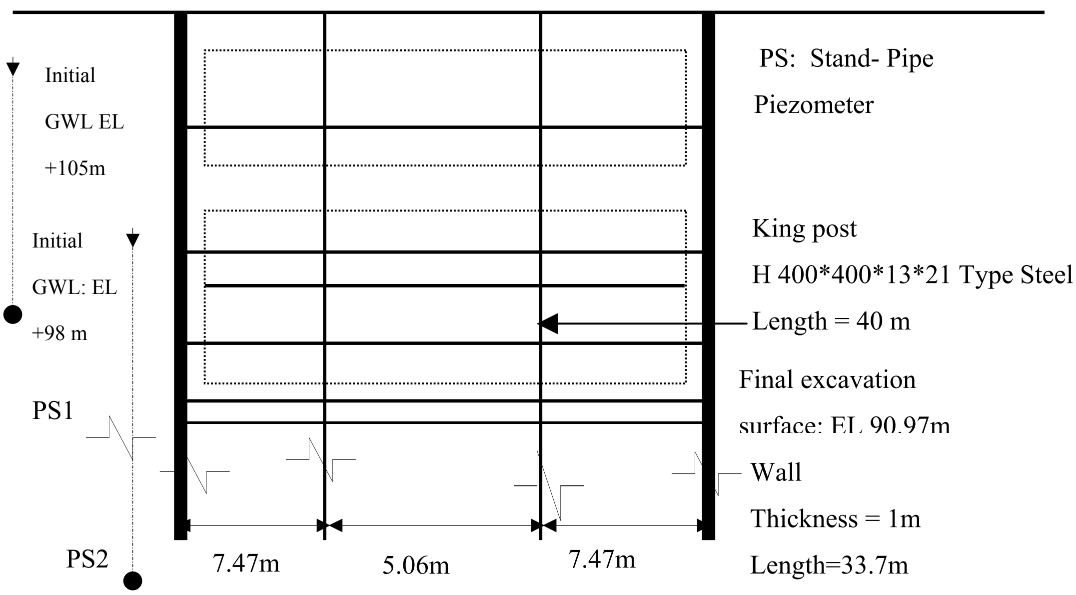


Figure 1 Cross section of the excavation for the BL12 station

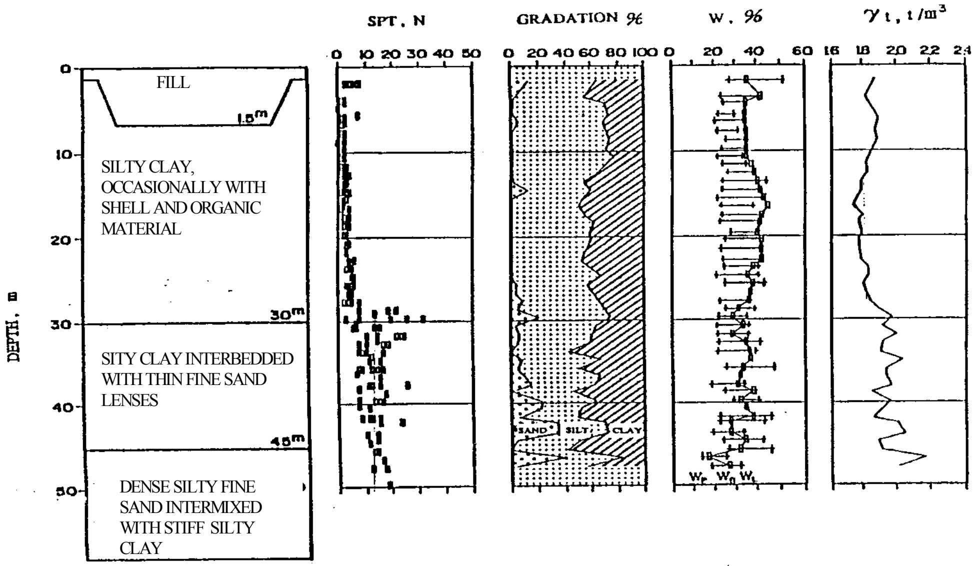
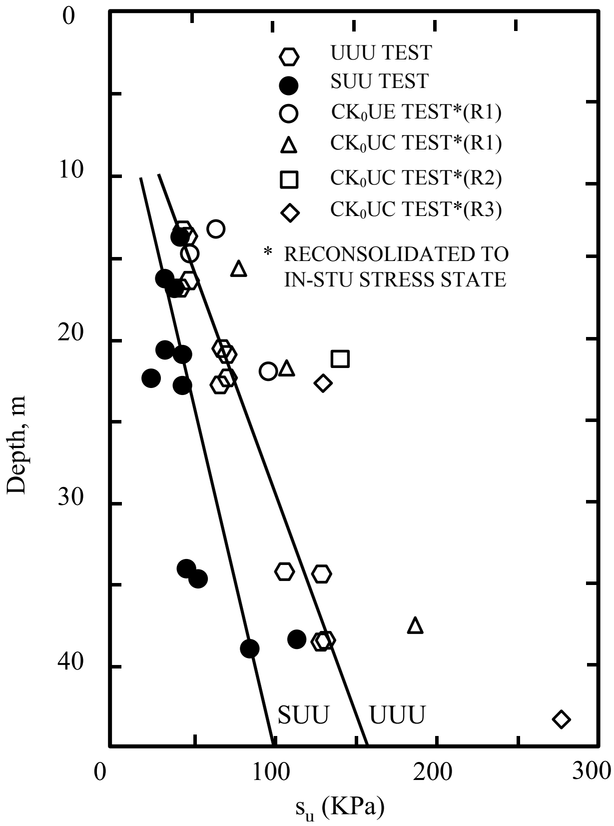


Figure 2 Typical soil profile in BL12 station area

Figure 3 Typical profile of undrained shear strength in Taipei



UUU TEST
SUU TEST
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CK0UC TEST*(R1)
CK0UC TEST*(R2)
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* RECONSOLIDATED TO
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Depth, m
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10
20
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300
su (KPa)

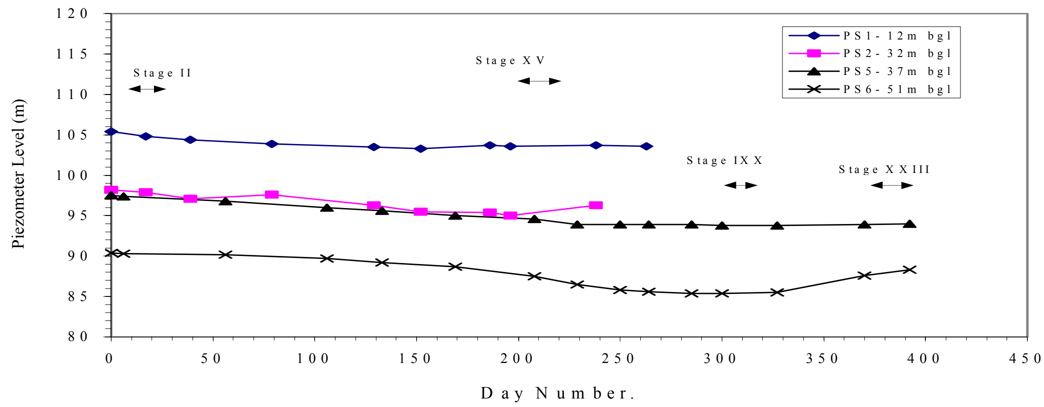

Figure 4 Piezometric level outside the excavation area

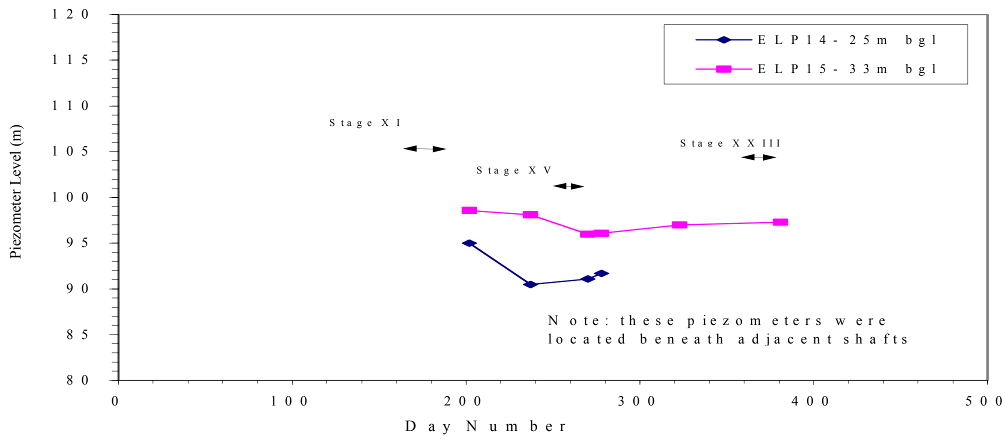

Figure 5 Piezometric level inside the excavation area

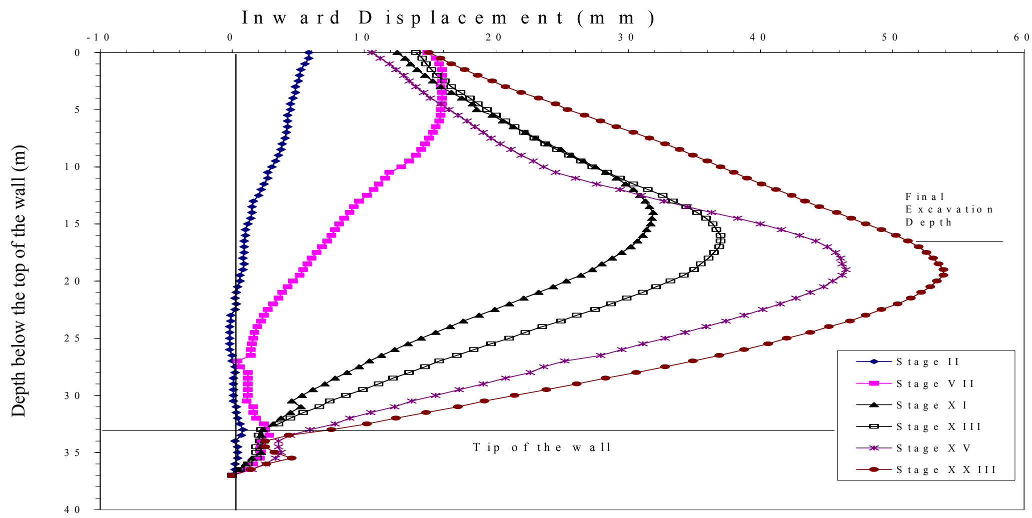

Figure 6 Lateral wall deflection

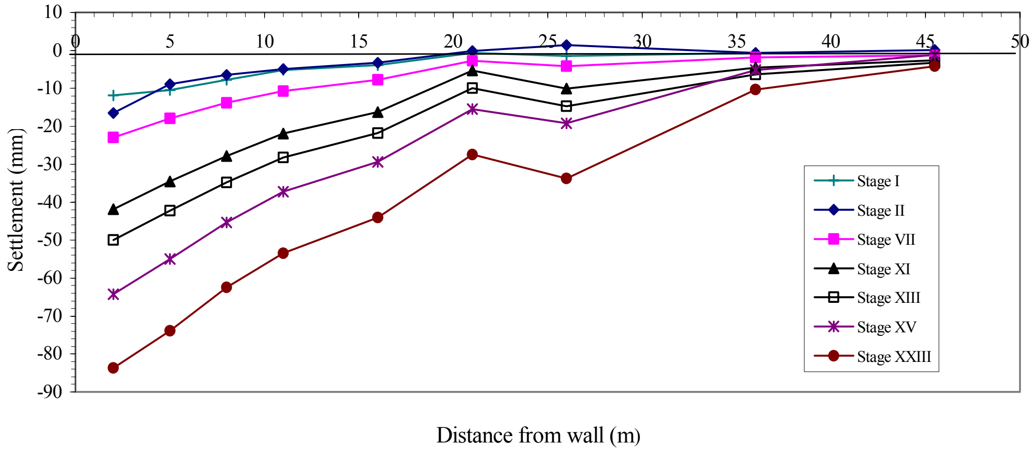

Figure 7 Surface settlements behind the wall

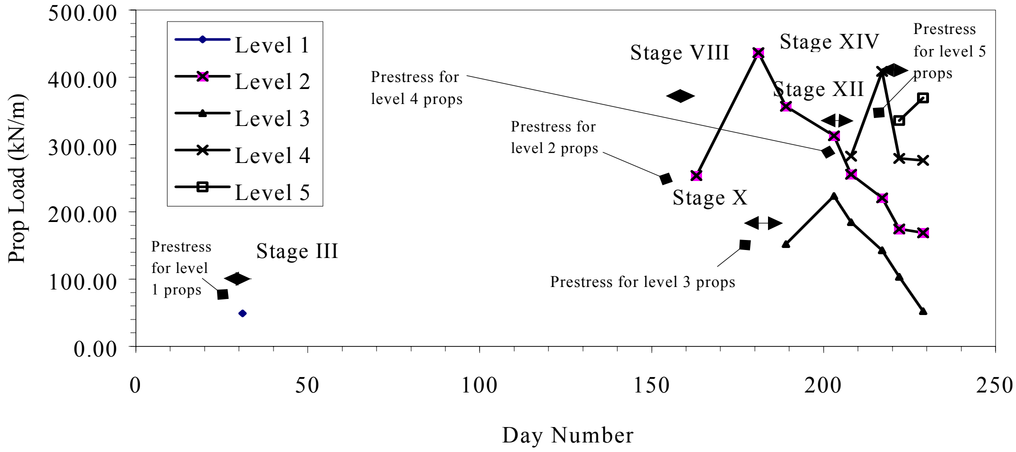

Figure 8 Variation of prop loads at different stage

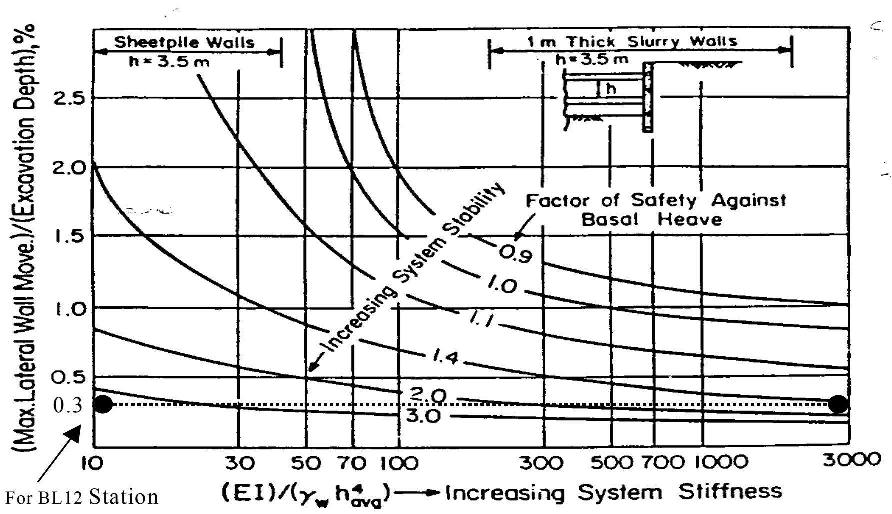

Figure 9. Maximum lateral deflections related to wall stiffness for excavations in soft to firm clays (after O'Rouke 1993)

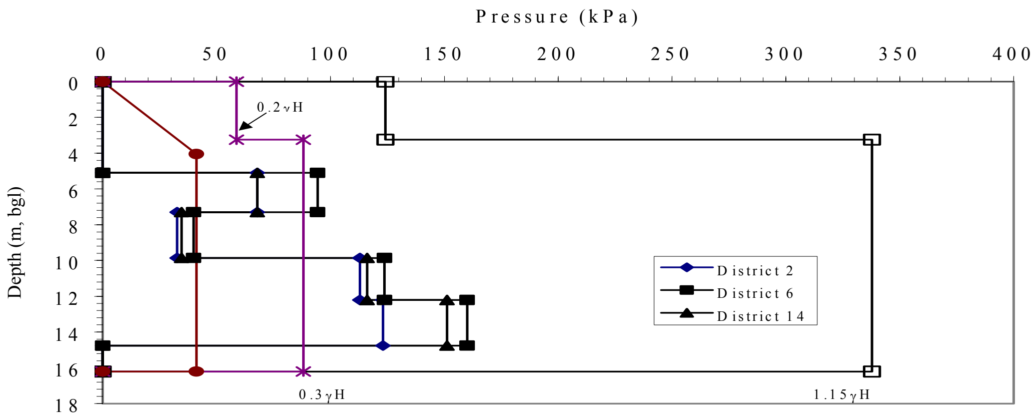

Figure 10 Apparent earth pressures at the Stage XV

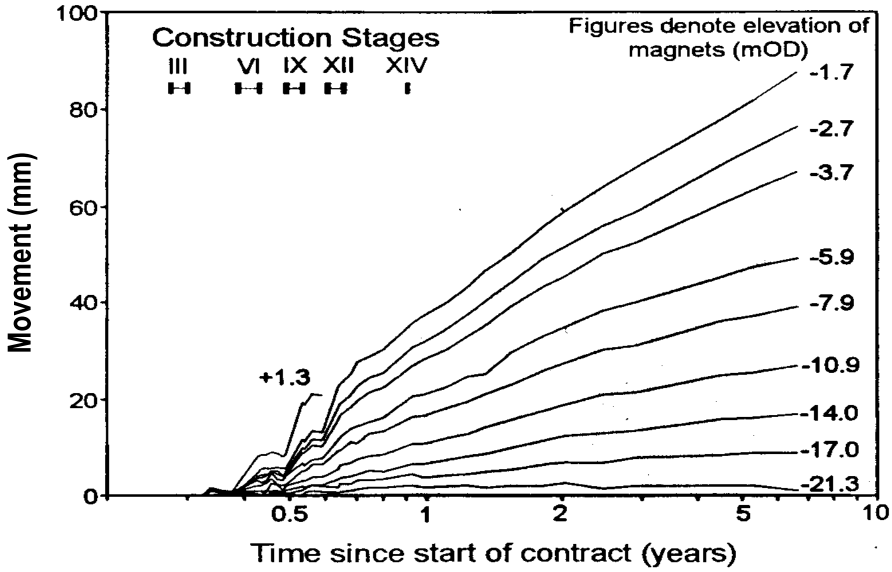

Figure 11 Upward earth movement inside excavation area in Lion Yard