

Case Studies of Support of Open Excavations and Distressed Retaining Walls in Malaysia

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Abstract: This paper presents two case studies on basement excavation with new temporary wall support system, two distressed open excavations in fill ground and two distressed retaining walls. As for the new wall support system, namely jacked-in pipe anchorage system, some promising performance of the support system have been demonstrated through the instrumentation and monitoring results as good temporary support system for basement excavation to replace conventional internal strutting or ground anchorage. The displacement effect of the jacked-in intrusion of steel pipe has stiffened and strengthened the retained earth mass resulting good wall performance. There are two case studies illustrating problems on uncontrolled earthwork construction, such as excessive ground movements for constructed retaining wall or open excavation, particularly at thick fill placed over a natural valley terrain without proper treatment to weak deposits contained in the valley. The natural valley usually collects natural underground seepage, which can cause nuisance and pose difficulties to the basement construction. On the case studies of piled retaining wall, extreme care shall be exercised in design as the lateral earth pressure would be, in some cases, sufficiently large to affect wall stability. Conventional global stability analysis on the retained ground with wall section may overestimate the safety margin. In these case studies, investigation and remedial solution will be presented and lessons learned are also summarized as a design checklist to prevent fundamental design error. Well documented forensic findings can provide good insights on the conservatism in the conventional design and can encourage improvised design with cumulated knowledge and experience from case studies. One should view such painful experience as gaining in good engineering experience for future design improvement.

1 INTRODUCTION

In this paper, efforts are aimed at presenting the useful compilation of case histories in open basement excavation and distressed piled retaining walls with involvement of the author: firstly two cases on new innovation of alternative stabilisation technique involving reinforcing the retained earth mass in open cut excavation, secondly two case studies on distressed open excavation in uncontrolled fill over natural valley and lastly forensic findings on the two distressed piled retaining walls. In all the cases presented in this paper, mechanism of the associated geotechnical processes will be highlighted as lessons learnt and design checklists for future improvised design.

2 NEW WALL SUPPORT SYSTEM

There are two successful case studies presenting herewith the application of unusual temporary wall support system using the jacked-in pipe anchorage. Liew (2000 & 2003) had presented these two cases in details whereas Cheang et al. (1999) has presented the same type of jacked-in pipe anchor in another Malaysia site.

The typical construction sequence of this new anchorage system involves:

- i. Establish a reaction system for the jacking of the steel pipes into the retained ground. In most cases, the retaining wall usually formed part of the reaction system.
- ii. Holes will be cut on the retaining wall at various designated levels for the pipe intrusion.
- iii. The reaction frame is structurally attached to the retaining wall using the high tensile bolts for developing pulling reaction as shown in Figure 1. The insertion angle of the pipe can be adjusted in this stage. In normal cases, the pipe is jacked horizontally with maximum downward inclination of 15° if needed.
- iv. Mild steel pipe of $\phi 114\text{mm}$ (OD) and 4.5mm thick is placed inside the reaction frame with the hydraulic jack at the end of the pipe. Steel bearing plate is secured by latching into the grooves along the reaction frame.
- v. The pipe is then jacked and advanced progressively against the steel bearing plate by repetitive extensions of hydraulic jack at different locations in the reaction frame.
- vi. Jointing of pipe can be carried out when longer penetration of the pipe is required.
- vii. Continuously jacking the pipe until the designed length or 2.0 times of the working capacity of the pipes is reached, whichever is longer. Due design consideration on the contribution from tip resistance during jacking process shall be made and the required penetration length will be reviewed from the estimated skin resistance with adequate safety factor as in normal engineering practice.

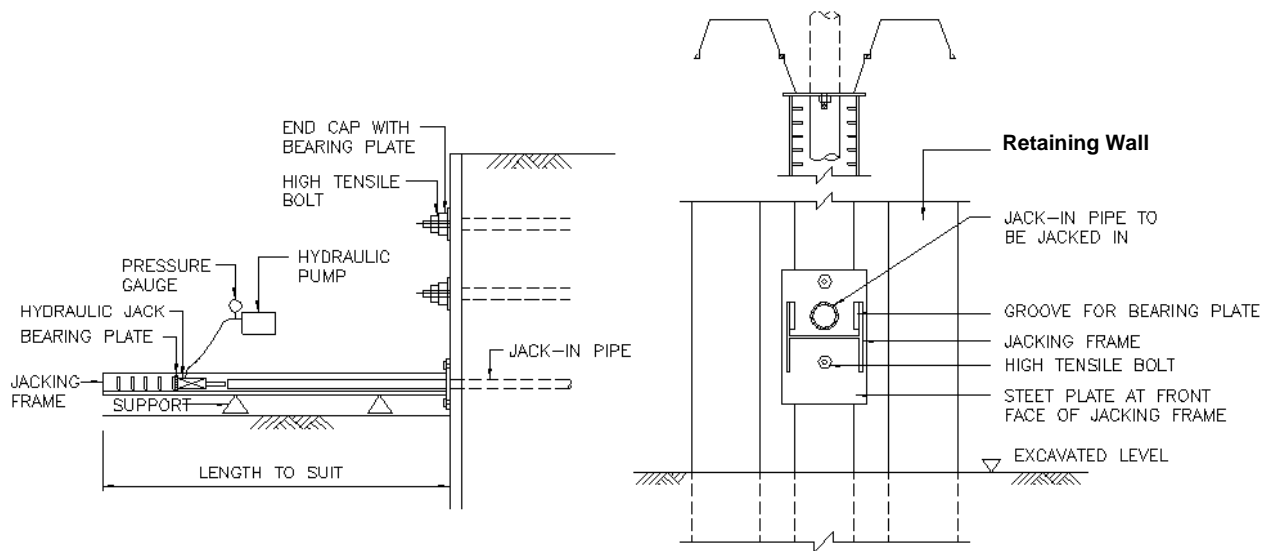


Fig. 1. Reaction Frame for Jacked-in Pipe Anchorage.

2.1 Case A (Liew, 2000)

The site is located in one of the busiest main road in the middle of Kuala Lumpur, Malaysia. The excavation was about 8m away from the main road. Due to the close proximity to the main roads on the south, a row of Section 16W sheet-pile wall supported by two rows of conventional anchorage into the limestone was initially designed to facilitate the open excavation along the main road. The rest of the excavation was open cut with stable slope configuration. Toe pins were provided for sheet-piles with short penetration due to shallow limestone profile at certain areas. Figure 2 shows the plan view of the excavation.

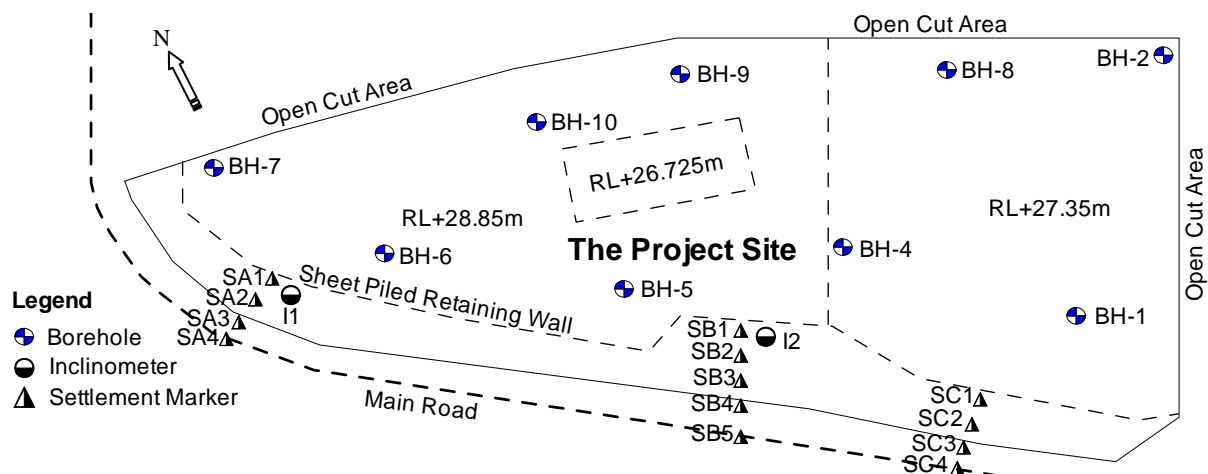


Fig. 2. Plan View of Project Site.

Site Geology & Subsoil Conditions

The general subsurface profile of the site is shown in Figure 3, indicating that the site is underlain by alluvium soils of Quaternary period. The top 3m of the subsoil consists of firm, intermediate plasticity Clay followed by minimum 17m thick of loose, well-graded silty Sand. Kuala Lumpur Limestone of middle to upper Silurian age is found at the average depth of about 20m with significant variation of bedrock profile. The SPT 'N' values at the upper cohesive layer and the lower cohesionless layer are about 8 to 10 and 2 to 24 respectively, but with representative values of 8 to 10. The Unconfined Compression Strength (UCS) of the Limestone ranges from 44MPa to 92MPa. Groundwater table is about 5 to 6m below existing ground level.

Construction

Due to highly unstable hole-stability in the initial ground anchorage construction in the sandy subsoil and occurrence of sink holes on the main road, alternative jacked-in pipe system was proposed to avoid excessive ground disturbance. Three rows of jacked-in pipes were proposed and installed at reduced levels RL+31.0m, RL+28.5m and RL+27.0m respectively. The lateral spacing of the pipes was 1.05m centre-to-centre at the top level and the 0.525m at the two lower levels. Figure 4 shows the installed jacked-in pipes anchorage at the temporary sheet-pile wall.

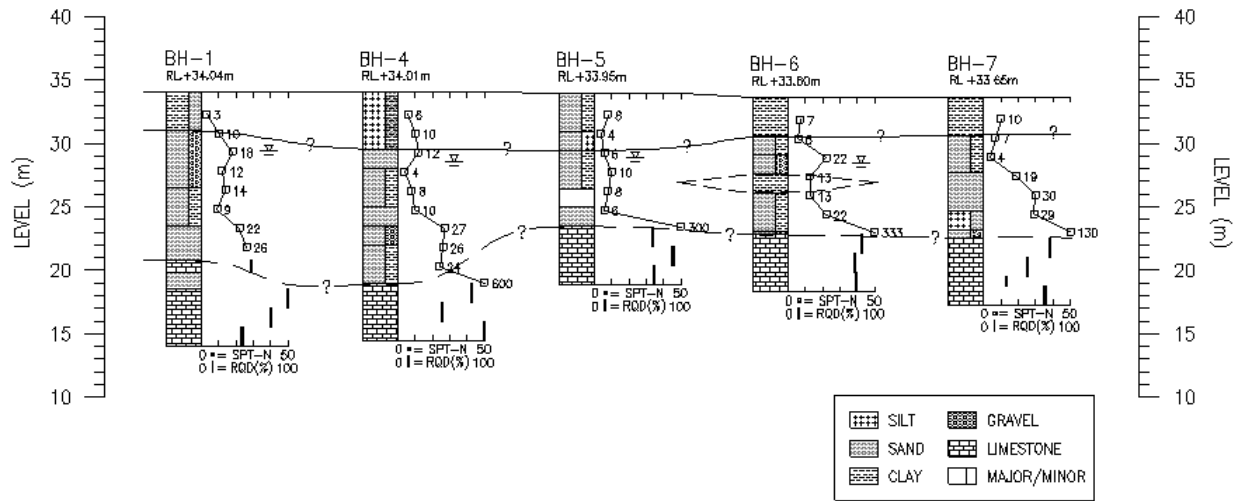


Fig. 3. Subsoil condition of Project Site.



Fig. 4. View of Retaining Wall with Jacked-in pipe Anchorage.

Performance of Temporary Retaining System

Two inclinometers, namely I1 and I2, were installed in the retained soil immediate behind the sheet-pile wall to measure the performance of the retaining system. Inclinometer I1 was installed with 1m embedment into the shallow limestone at about 12m below the ground level whereas Inclinometer I2 was also embedded 1m into the deeper bedrock at about 19m below ground level. The lateral ground movements recorded by the two working inclinometers are shown in Figure 5. The maximum lateral ground movements at the end of excavation are in the range of 106mm to 110mm, which is corresponding to approximately 1.2% of the retaining height. The maximum lateral movements occurred at the top of retained ground. The lateral deflection profile of the retained soil tends to indicate that the wall almost performed like a cantilever wall.

The settlement profiles of the three (3) sets of settlement markers are shown in Figure 6. The maximum settlement is about 218mm, which is located at Marker SA1. This could be attributed to the excessive washing-out of subsoil material during trial installation of the two conventional ground anchors according to the compliance design. In general, the magnitude of settlements behind the wall at the end of excavation is in the range of 125mm at the wall and tapers off after about 20m to 25m away from the wall, which is about 2.2 to 2.8 times of excavation depth. The settlement profiles fall near to the Zone I in the settlement chart by Peck (1969), which is for sand and soft to hard soil with average workmanship. The ground settlement-to-wall deflection ratio ranges from 0.95 (Settlement Markers SB and Inclinometer I2) to 2 (Settlement Markers SA and Inclinometer I1).

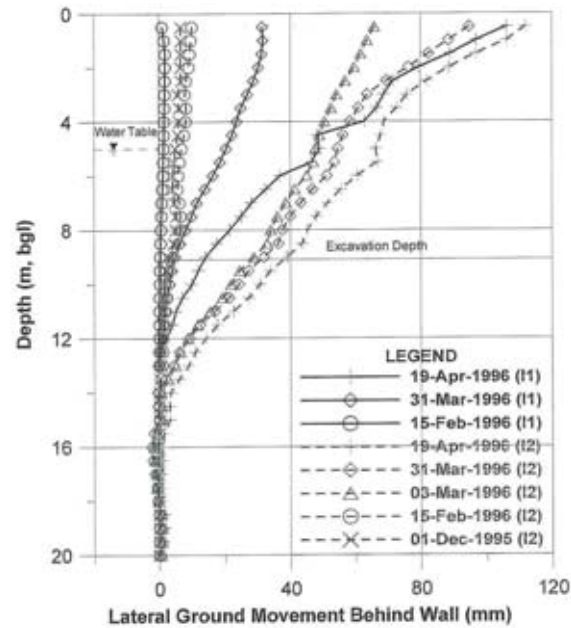


Fig. 5. Wall Deflection Profile of Retaining Wall with Jack-in pipe Anchorage.

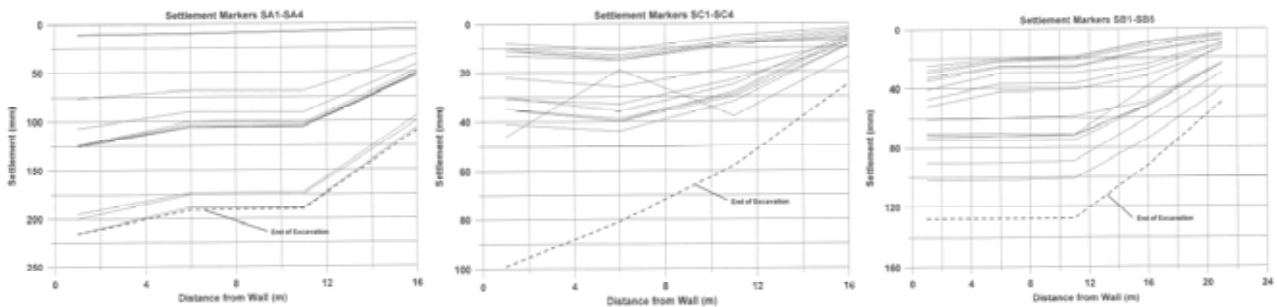


Fig. 6. Settlement Profiles for Markers SA1-SA4, SB1-SB5 and SC1-SC4.

Pull-out Test Results

The following discussions will concentrate on the pull-out test results of this anchorage system as summarized in Table 1:

- Time effects on the pull-out resistance were observed in the test results. Generally, the skin resistance increases with time in the granular soils (lower two rows) whereas the skin resistance seems to significantly reduce with time in cohesive soils (top most row), however there is insufficient data to clearly show the remarkable reduction of skin resistance in cohesive soil.
- Based on the effective stress approach, the calculated skin resistance factor, β , for the jacked-in pipes ranges from 0.44 to 1.32 with majority of the values fall within 0.50 to 0.63. These values are larger than those derived from pile skin friction, which is normally in the range of 0.35 to 1.0 for granular soils (Combarieu, 1985).
- In certain pull-out test results, for instances, Test No. 2 at Level 1 and Test No. 1 at Level 3, low pull-out stiffness is observed at the beginning of the test. This could be due to the locked-in compression stresses in the pipe as the result of compressive jacking. This compressive stress was then released during the pull-out test. It is believed that the occurrence of the locked-in compressive stress can be isolated incidence but, theoretically, it can occur in most of the pipes.

Table 1 Summary of Pull-Out Test Results

Test No	Anchor Level	Reduced Level	Date of Installation	Date of Testing	Time Lapsed (Day)	Pipe Diameter (mm)	Free Length (m)	Fix Length (m)	Ultimate Load (kN)	Skin Resistance Factor, β
T1-1	1st	RL+31.0m	18/03/96	05/04/96	18	100	4	8	122	0.90
T1-2	1st	RL+31.0m	19/04/96	24/04/96	5	114	4	11	280	1.32
T2-1	2nd	RL+28.5m	19/04/96	22/04/96	3	114	2	10	170	0.48
T2-2	2nd	RL+28.5m	24/04/96	04/05/96	10	114	2	10	185	0.52
T3-1	3rd	RL+27.0m	30/04/96	02/05/96	2	114	0	9	180	0.50
T3-2	3rd	RL+27.0m	04/05/96	07/05/96	3	114	0	9	225	0.63

Note : Ground level : RL+34.0m

2.2 Case B (Liew, 2003)

This project involves construction of a temporary shoring system for a 17m deep open excavation using $\phi 750\text{mm}$ and $\phi 900\text{mm}$ contiguous bored pile (CBP) wall and soldier pile (SP) wall with maximum five rows of prestressed ground anchorage support in the original design. Due to the close proximity to the buildings, the designed ground anchorages are inclined at steep angle with relatively short anchor length to avoid hitting the building foundation, which are only 8m away from the wall. Figure 7 shows the plan view of the CBP wall. However, due to the difficulties and slow progress in installing these prestressed ground anchors with full casing method as specified, an alternative proprietary jacked-in anchor system to expedite the works was then proposed at areas where there is no encroachment of the alternative support system to the building foundation. The alternative support system consists of upper seven rows of 18m long and lower two rows of 12m mild steel pipes as jacked-in anchors respectively spaced at 850mm to 1000mm centre-to-centre lateral spacing. These steel pipes were laterally installed by hydraulic jack in between the gaps of CBP wall and SP wall.

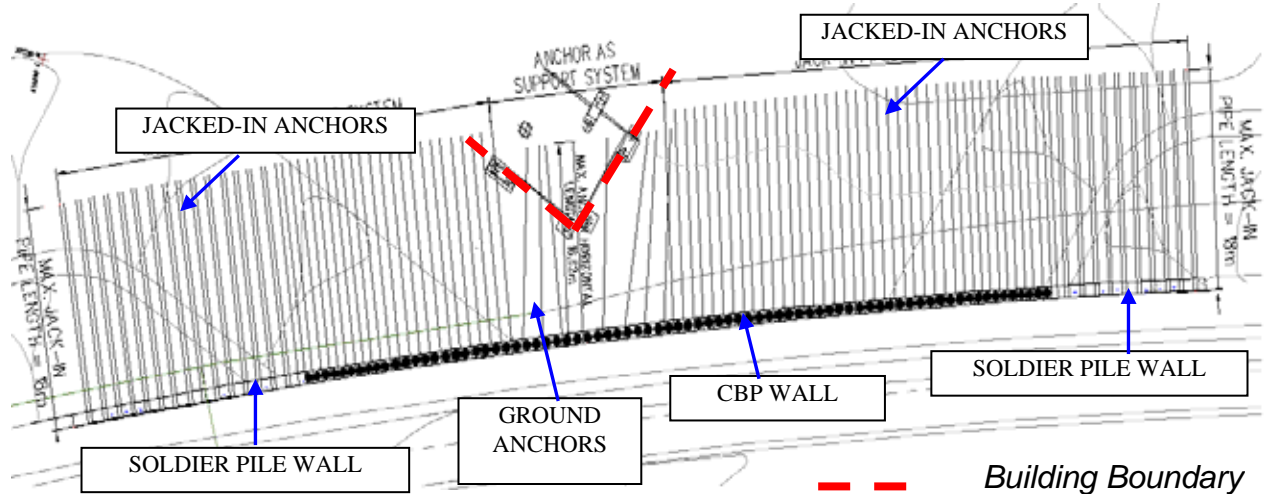


Fig. 7. Plan View of Project Site.

Site Geology & Subsoil Conditions

The project site was initially an undulating palm oil estate underlain by meta-sedimentary Kajang formations and some alluvial deposits consisting of sandy clayey silts at low-lying areas. Fill of about 10m thick with SPT'N values ranging from 5 to 13. Beneath the fill is the sandy/clayey silt with average SPT'N values of 20. Slightly weathered schist is found at the depth of about 40m. It is also expected that shale with intercalation of foliated phyllite, graphitic schist, sandstone and quartzite can be found within this meta-sedimentary formation.

The engineering properties of the subsoil are summarised in Figure 8 and Table 2 respectively. The groundwater measured from the standpipe and during subsurface exploration was about at RL21.8m, which is 12m below the retained ground level of RL34m.

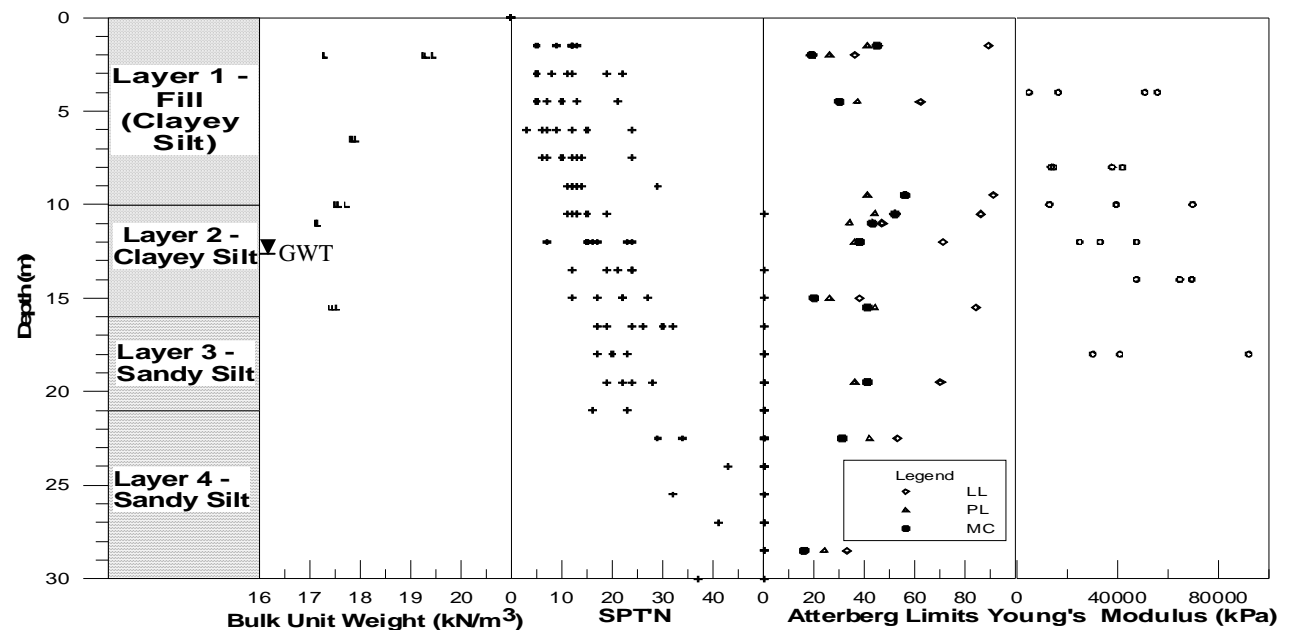


Fig. 8. Stratification and Engineering Characteristics of Subsoils.

Table 2 Engineering Properties of subsoil

Layer	γ_{bulk} (kN/m ³)	γ_{dry} (kN/m ³)	ϕ' (°)	c' (kN/m ²)	ψ (°)	E' (kN/m ²)	E'_{ur} (kN/m ²)	ν_{ur}
1	18.0	14.0	30	4	0	11,130	33,380	0.2
2	18.0	14.0	32	4	2	24,500	73,500	0.2
3	18.5	14.5	32	4	2	45,390	136,170	0.2
4	19.0	15.0	34	5	4	133,500	400,500	0.2

Pull-out Test Results

Pull-out tests have been carried out at different time intervals after installing the jacked-in anchors to verify the development of shaft resistance with time. These pull-out test results are presented in Figures 9, 10 and 11. Not all pull-out tests have mobilised the ultimate capacity as they were only tested to 2.2 times the designated working pull-out capacity.

From Figure 11, the mobilised shaft resistance of these pull-out tests shows an obvious increasing trend with time. This is primarily caused by the increase of effective radial stress surrounding the jacked-in anchor after dissipation of excess pore pressure induced by soil displacement during jacking. It is also expected that the stiffness of the soil will increase indirectly in the similar way. Tan et al (2001) have presented a methodology using cavity expansion method to assess the excess pore pressure response of a jacked anchor inclusion and its dissipation resulting in increase of pull-out capacity.

Strain gauges have been installed on the two selected jacked anchors at levels L4 and L7 respectively to monitor the load transfer behaviour of the shaft resistance mobilized during jacking-in process and pull out tests at various time intervals after installation as shown in Figure 9. As only one strain gauge was installed at each section of L4 jacked anchor, significant flexural effect as a result of compressive buckling could be expected to affect the strain gauge reading during jacking-in process. This has been verified during jacking the L7 jacked anchor, in which the coupled pair of strain gauges has indicated significant flexural effect in the jacked anchors. However, the flexural effect is much minimized in tensioning condition during pull-out test. It was also observed that lower shaft resistance has been mobilised at the L4 jacked anchor as compared to the L7 jacked anchor. For the two instrumented jacked-in anchors, higher mobilized shaft resistance is observed at the middle segment of the anchor during pull out test. Most instrumented segments of jacked-in anchor indicate ultimate shaft resistance has been achieved when the head displacement of jacked anchor reaches 5mm to 10mm. Generally, the pull out load of the two instrumented jacked anchors show increasing trend with time indicating stiffer behaviour.

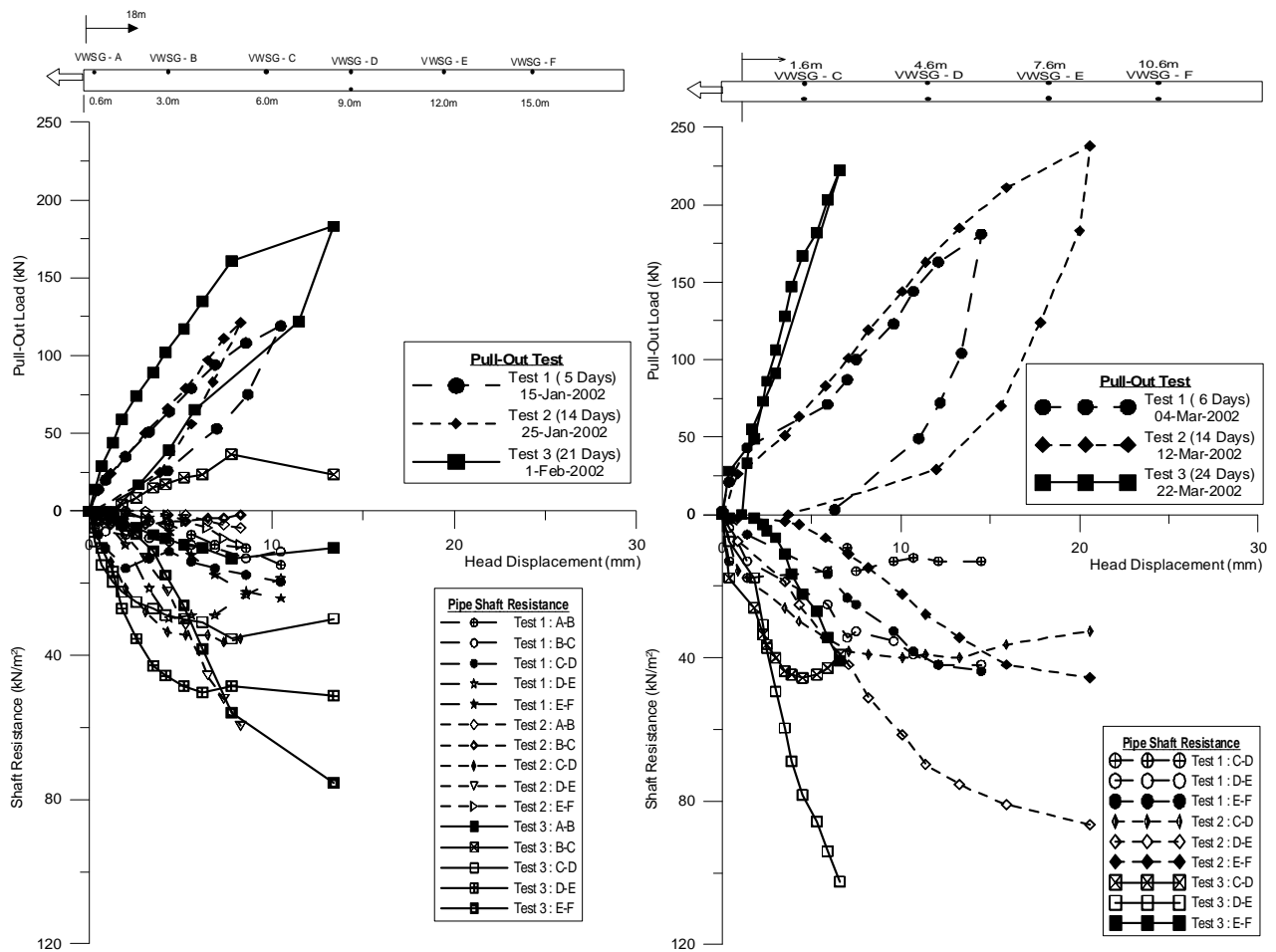


Fig. 9. Pull-out tests at Level L4 and L7 and Interpreted Mobilised Shaft Resistance.

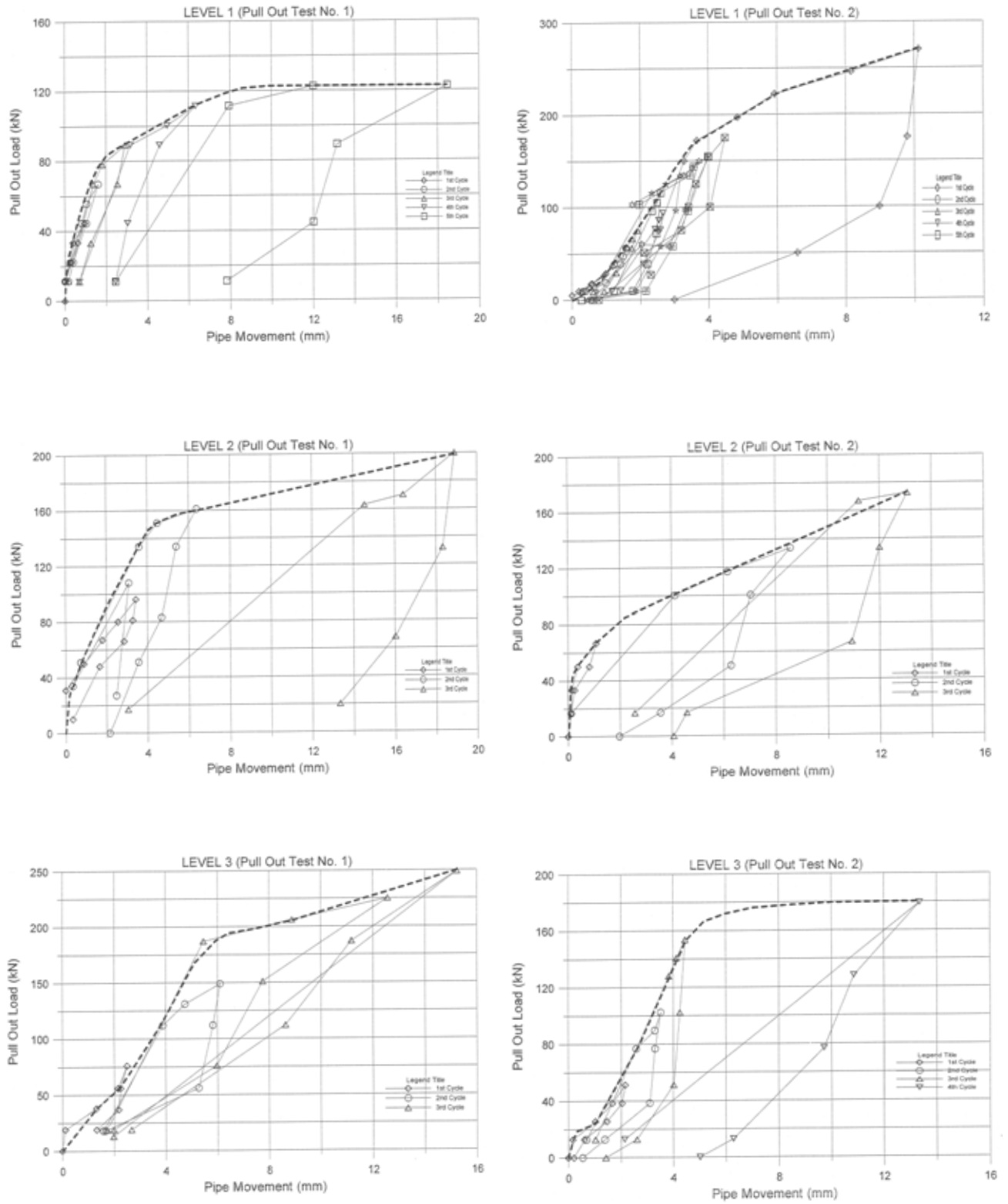


Fig. 10. Pull-out Load and Pipe Movement of Six Pull-out Tests.

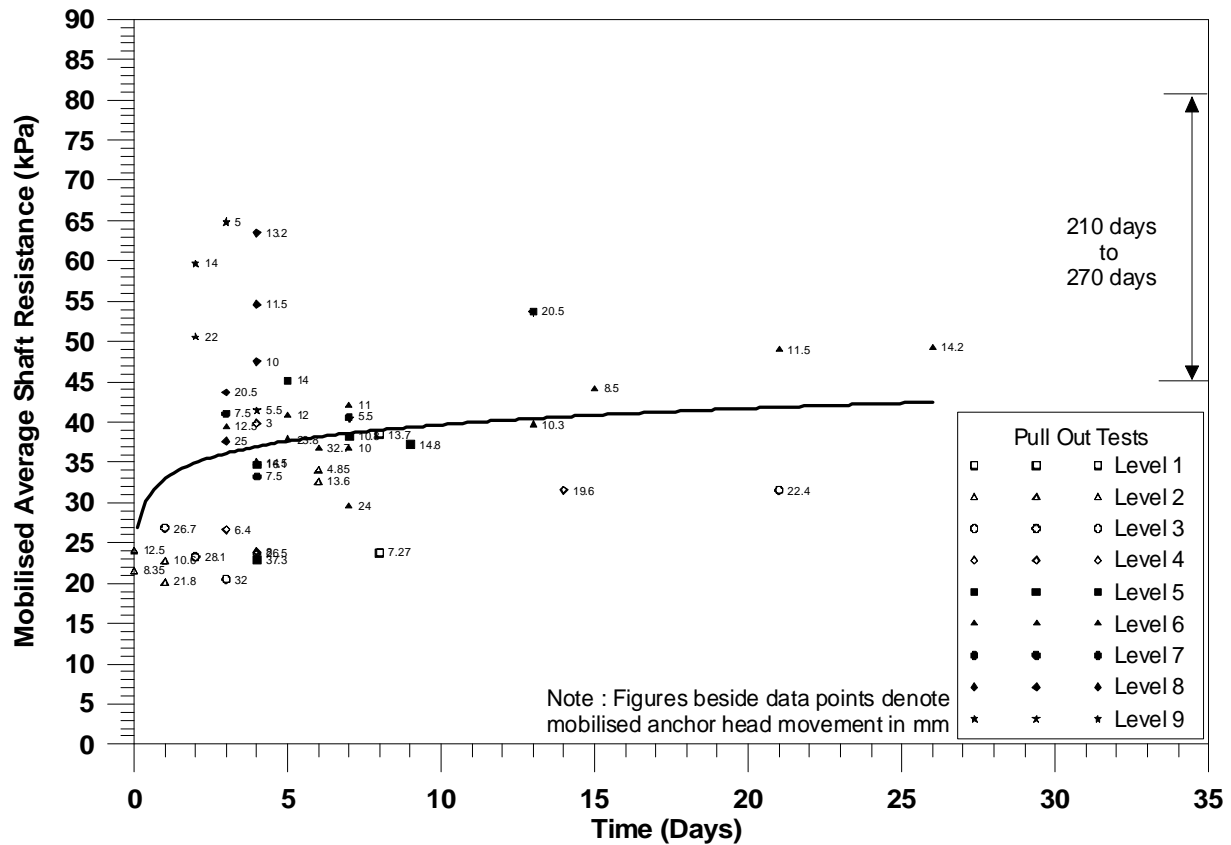


Fig. 11. Time Effect on Pull-out Resistance of Jacked-in Anchors.

Load Cells

It is observable that the jacked-in anchor loads at the anchor-to-wall connection increase drastically from the initial nominal lock-off load with the progressing excavation depth except for the lowest jacked-in anchor, in which there is no significant excavation after installation of the lowest jacked-in anchor as compared to other jacked-in anchors at higher levels. Another reason for that could be due to relatively stiff soil stratum at lower level. The average excavation depth at each excavation stage is about 1.8m. Figure 12 shows the variation of jacked-in anchor load with time.

As for the prestressed ground anchor, designated prestressing loads have been applied to the respective ground anchors during lock-off. However, some ground anchors have shown reduction of prestress as in Figure 13, which could be due to creeping of the relatively short fixed anchor length and potential relaxation of prestress as a result of vertical movement of CBP piles under high vertical load component from the prestressed ground anchor.

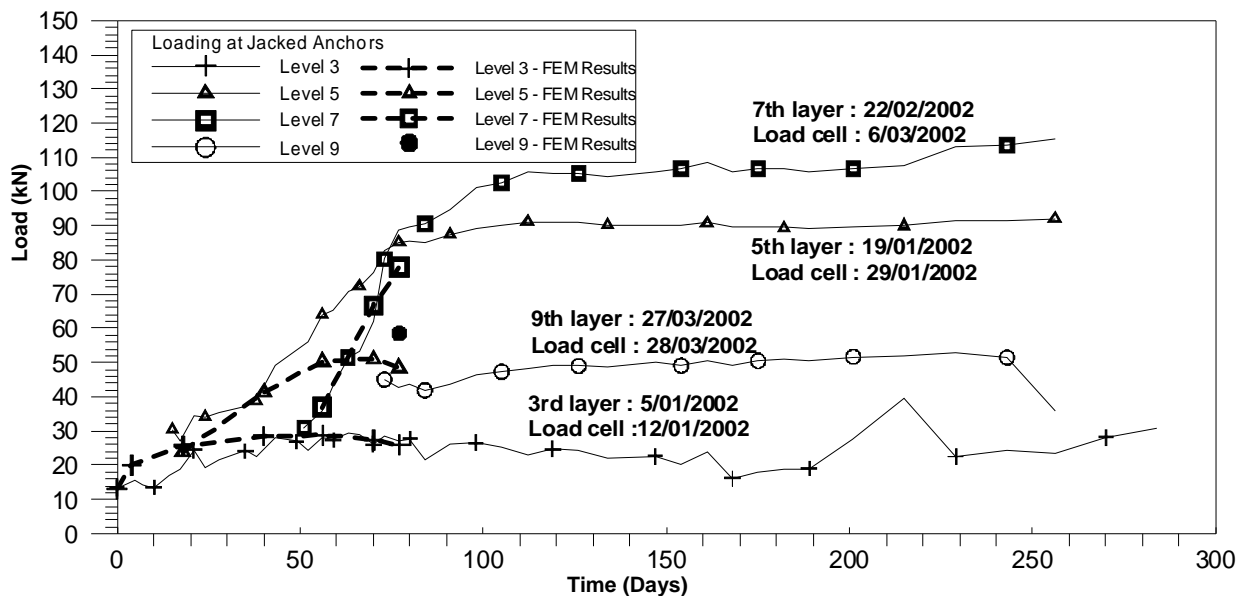


Fig. 12. Jacked-in Anchor Load with Time.

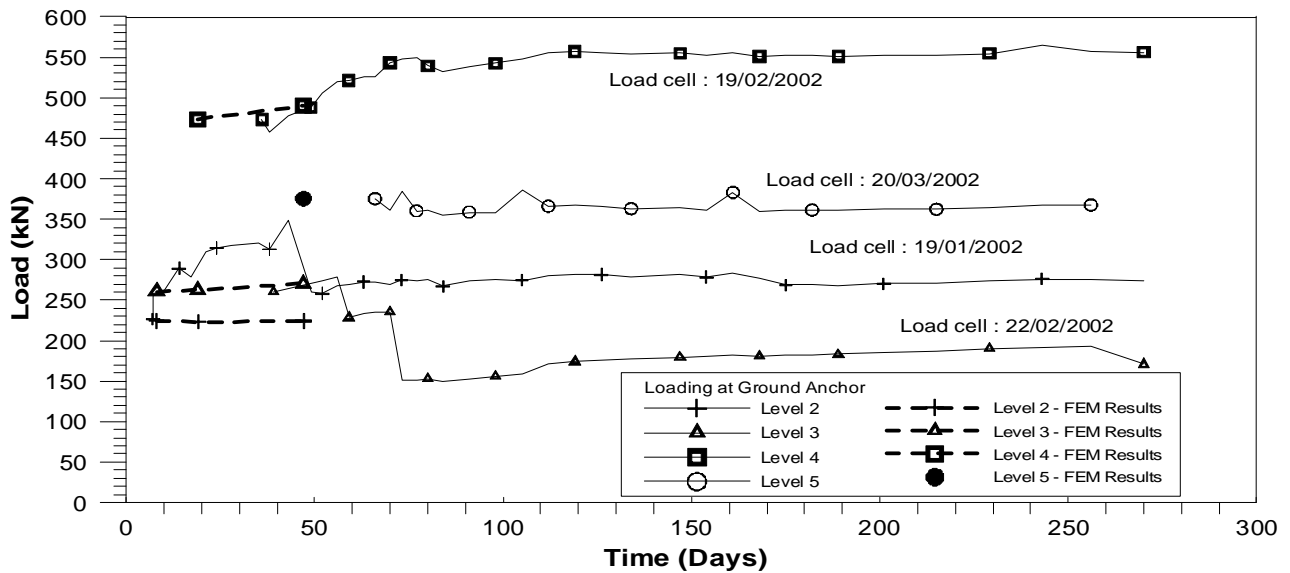


Fig. 13. Prestressed Ground Anchor Load with Time.

Wall Movements

The wall movements at both the jacked-in anchors and prestressed ground anchor walls have been monitored using the inclinometers installed inside the CBP wall. Figure 14 shows the monitored wall movements during various excavation stages at both walls. The jacked anchor wall has moved about 35mm at the final excavation with relatively fixed toe embedment, whereas the prestressed ground anchor wall has moved about 46mm with the wall toe rotated, which implies the overall wall movement could be more and the more passive resistance at the wall embedment has been remarkably mobilised to maintain overall wall stability. The wall deflection-to-excavation depth ratio is about 0.21% and more than 0.27% for jacked-in anchor and ground anchor respectively.

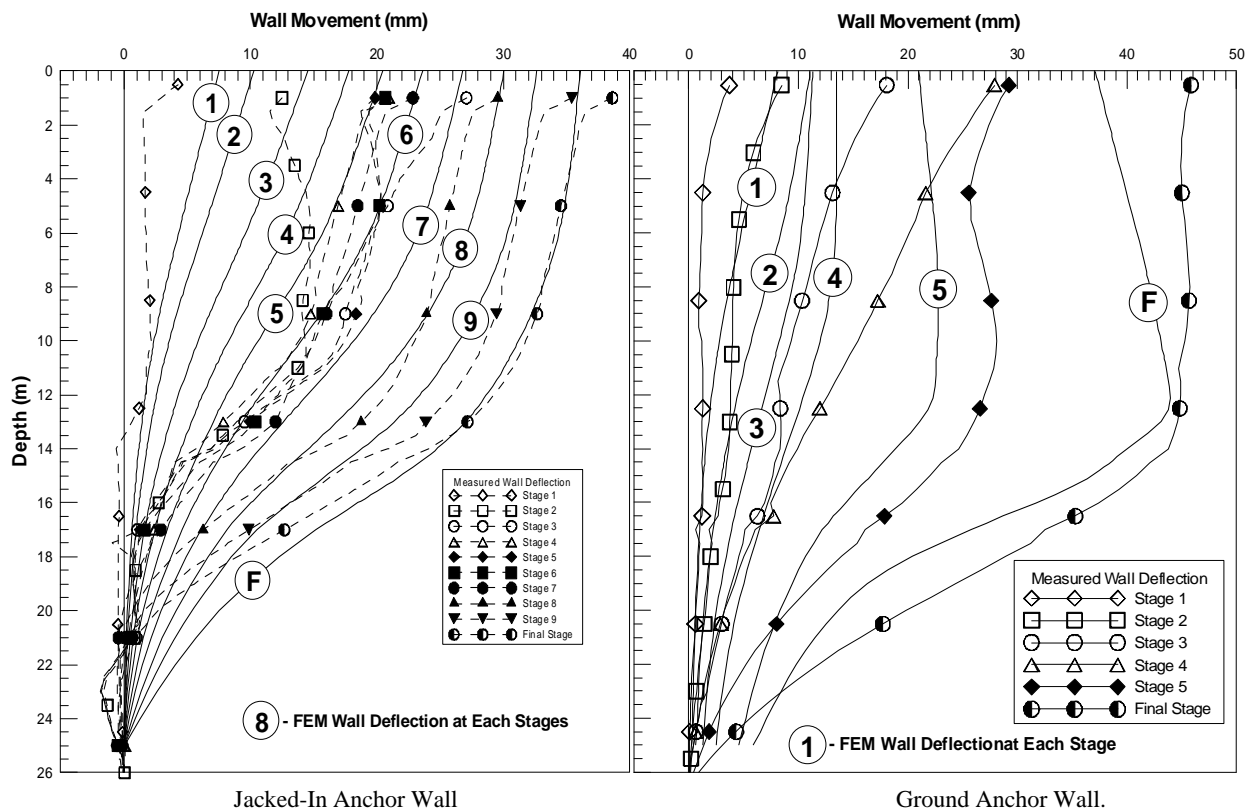


Fig. 14. Wall Movements at Jacked-In Anchor Wall and Ground Anchor Wall.

Settlement Markers

The ground settlement behind the CBP wall at various construction stages is shown in Figure 15. The ground settlement results indicate the performance of jacked anchor support system is far better than the prestressed ground anchor system. Generally, the ground settlement-to-wall movement ratio for jacked-in anchor wall and ground anchor wall is 1.57 and 3.37 respectively.

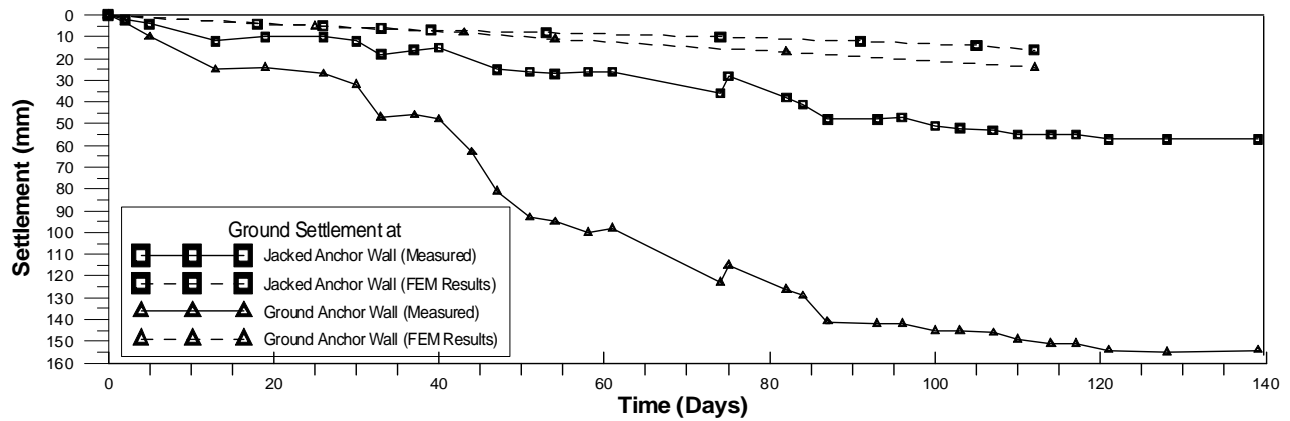


Fig. 15. Ground Settlement behind the CBP Wall.

Back-Analyses and discussions

Two dimensional finite element method (FEM) with 6-node elements was used to model and back-analyse the performance of the CBP wall supported by both jacked-in anchors and prestressed ground anchors. “Hardening Soil” model (Brinkgreve, 2002), which is suitable for residual soil, was used as soil model. The $\phi 900\text{mm}$ CBP wall and jacked anchors were modelled as beam element whereas the prestressed ground anchors were modelled as elastoplastic spring node-to-node element for the free length and geotextile interface element for the fixed length. Interface elements were also applied to the wall-soil and anchor-soil contacts. In numerical modelling, the CBP wall was assumed as “wished-in-place” condition before the excavation started, and the undrained analysis incorporated with groundwater calculation was performed under 2-D plane strain condition for the expected short construction period. Figure 14 shows the lateral wall movements of the CBP wall supported by jacked-in anchors and prestressed ground anchors. In general, reasonably close agreement in the lateral wall movement profile of jacked anchor wall and prestressed ground anchor wall has been achieved during the back-analyses except at the final stage of excavation for the prestressed ground anchor wall where the wall movement is slightly underestimated. This could be due to the load relaxation and creeping of some prestressed ground anchors with time, particularly those at levels 2 and 3 as shown in Figure 13, that are not taken into account in the FEM modelling. Generally, the FEM results match well with the measured lateral wall movement profile in most excavation stages for both jacked-in anchor and ground anchor walls. On the other hand, ground settlement markers installed at about 1m behind the CBP wall indicate larger settlement magnitude compared to the FEM results as shown in Figure 15. Figure 16 shows the dimensionless plot of the analysed and measured ground surface settlement profiles as recommended by Clough & O’Rourke (1990). The maximum wall movement of CBP wall at final excavation is about 0.002 of excavation depth, which is tally well with the average maximum movement of cast-in-situ rigid wall on hard clay and sandy soil layers presented by Clough & O’Rourke (1990). However, Ou et al (1993) has also presented the ratio of maximum wall deflection-to-excavation depth in the range from 0.002 to 0.005 in numbers of deep excavation in Taipei basin.

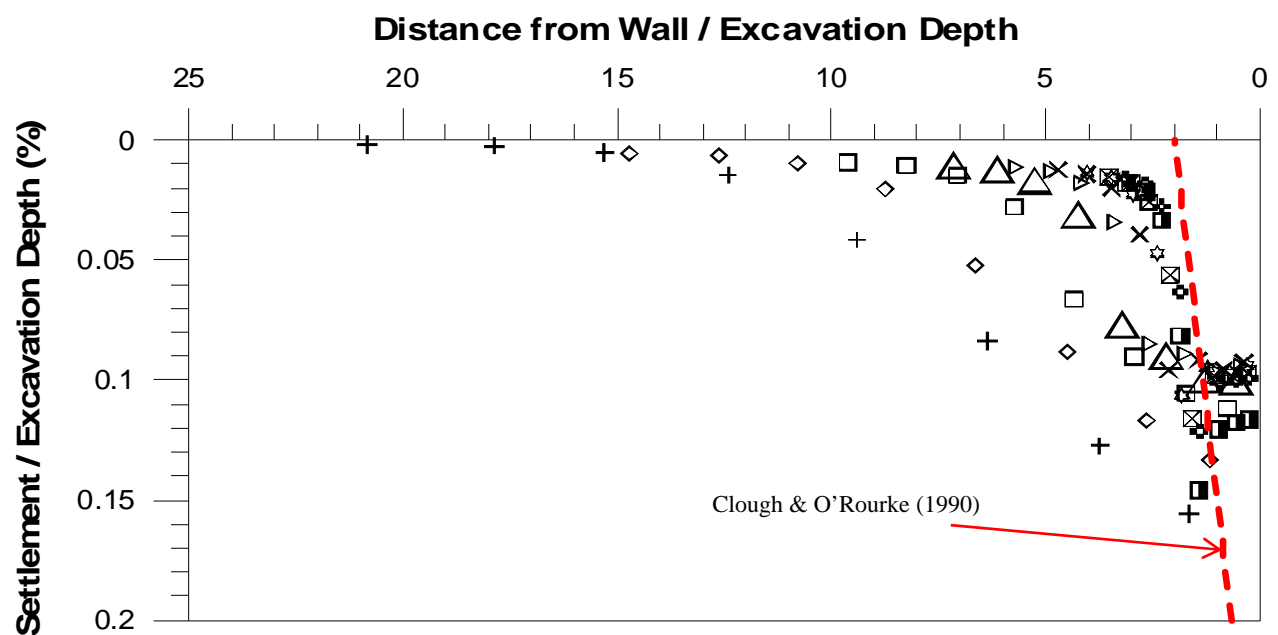


Fig. 16. Dimensionless Ground Surface Settlement Profile.

In Malaysia, it is a common practice to use correlation of Standard Penetration Test (SPT) to obtain both strength and stiffness parameters of the residual soils for geotechnical design, despite some researchers disagree one in-situ test can derive both

parameters simultaneously. In this case study, the FEM back-analysis results show that effective Young's modulus (E') of the upper clayey silt and lower sandy silt subsoils are about $2150 \times \text{SPT}'N$ and $2600 \times \text{SPT}'N$ respectively. The first correlation agreed well with the correlation as proposed by Tan et al. (2002) for 27m deep excavation in the weathered meta-sedimentary Kenny Hill formation. The suggested unloading/reloading stiffness (E'_{ur}) used in the 'Hardening Soil' Model is about three (3) times of effective Young's Modulus (E'). Figure 17 shows the back-analysed effective Young's Modulus in the FEM analysis for the prestressed ground anchor wall superimposed over the interpreted effective pressuremeter (PMT) stiffness modulus, in which the back-analysed E' is generally at lower bound of the interpreted pressuremeter stiffness modulus. Whereas the back-analysed E' for the jacked-in anchor wall is generally about 30% more than that for the prestressed ground anchor wall.

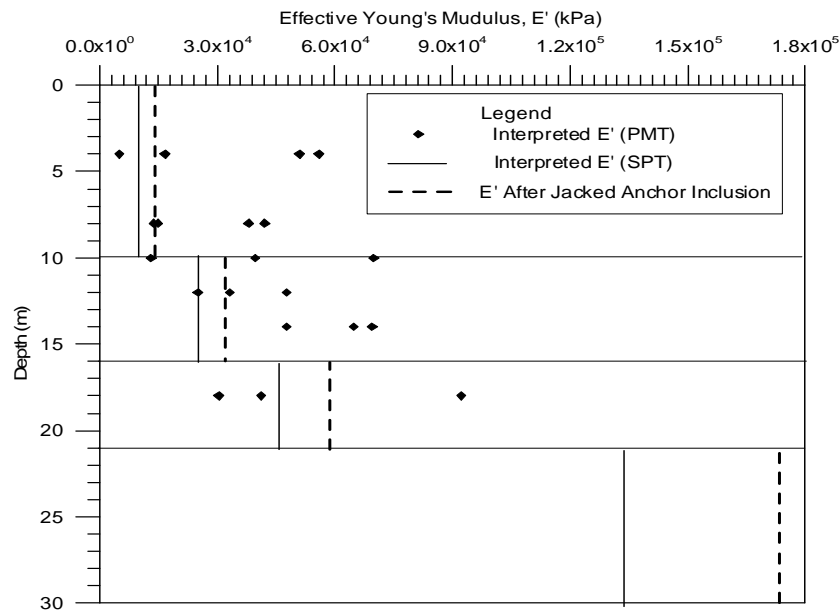


Fig. 17 Interpreted E' from SPT' N and Pressuremeter Test (PMT).

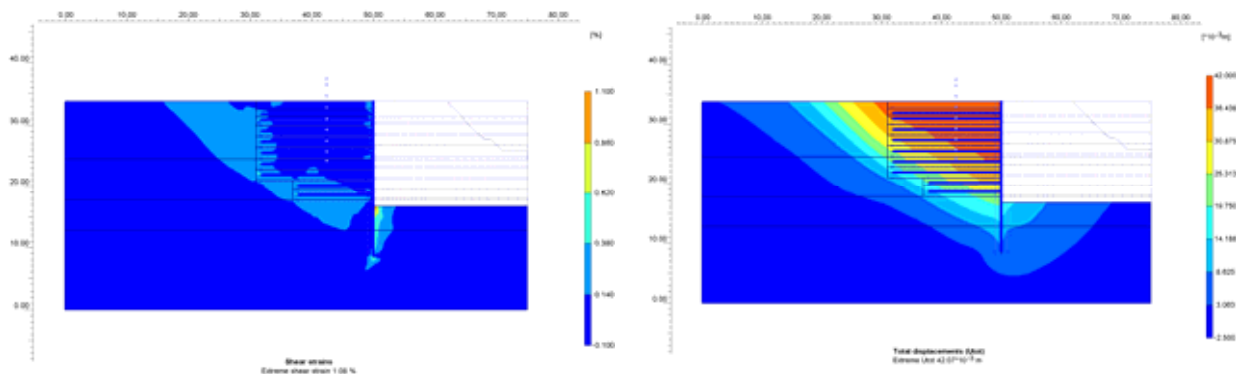


Fig. 18. Soil Shear Strain and Total Ground Displacement within the Jacked Anchor Retaining System.

From Figure 18, the induced shear strains during excavation within the reinforced soil mass as a result of jacked-in anchor inclusion are mostly less than 0.15%. There is a relatively larger shear strains ranging between 0.26% and 0.38% developed along the potential slip surface of the active zone behind the reinforced soil mass. This potential band of slip surface appears to suggest that the reinforced soil mass tends to slide laterally along the base under the huge active earth pressure behind the reinforced mass. The inclusion of jacked anchors has restricted the development of active zone within the reinforced soil mass. In accompany to the wall geometry and the potential failure mechanism, it is expected that huge shear force and flexural stresses would be induced at the embedded wall and the passive zone will be highly mobilized, particularly at the excavation level. In fact, the highest shear strain in the FEM analysis is actually located at the passive zone in front of the wall embedment.

From the total ground displacement contour as shown in Figure 11, it can be observed that the reinforced soil mass has more displacement at the upper portion with gradually reduced trend towards the lower portion, in which the displacement is cumulative.

Conclusions and recommendations

Based on the discussions in the earlier sections, the following conclusions can be made:

- The jacked-in anchor wall behaves as a semi reinforced soil wall with better overall performance as compared to the prestressed anchored wall. Intensive soil-structure interaction can be observed between the soil and the jacked-in anchors. As a result, earth pressure immediately behind the jacked-in anchor CBP wall is much less as compared to the one with prestressed ground anchors. This is because part of the resistance to the active zone within the reinforced area has been provided through the

interfacial resistance of the jacked-in anchors before it is fully transferred to the wall. Therefore, it is conservative to use conventional Rankine or Coulomb earth pressure for assessing the bending stresses of the wall.

- b. Observable shearing zone, which could be developed into slip surface defining the failure mechanism of the retaining system forming the active wedge, has occurred behind the jacked anchor wall.
- c. From the observation of the initial pull out test results, the pull out capacity of the jacked anchor does not appear to be related to the effective overburden stress at the initial stage, rather its mobilised shaft resistance is constricted to a narrow range from 20kPa to 30kPa. However, the thixotropy effect of soil has shown increase in pull out capacity of the jacked-in anchor with time, but remains constant after probably 120 days. The effective increase of pull out capacity is generally in the range of 70% to 90%.
- d. There is also significant stiffening effect after the jacked-in anchor installation, which significantly improve the overall performance of the wall. In this case study, there is an increase in stiffness by about 30%. Therefore, selection of design parameters shall take into consideration of such effect.
- e. The backed-calculated engineering parameters between the ground anchor wall and the jacked-in anchor wall can give indication of the stiffening effect. Soil plug formed within the pipes during jacking in process would have implication of the increase of radial stress on the pipe due to cavity expansion.
- f. The instrumentation and back-analyses have yielded very useful information in this study.
- g. Finite element method can be successfully deployed to analyse the complicated interaction of the entire soil-structure system and therefore to assess the ultimate and serviceability conditions of the retaining system.

There are also recommendations for the future research of this innovative temporary support system as follows:

- a. The reaction system on the retaining wall to derive the pulling reaction force will aggravate the wall deflection. Imprvement can be made utilising the pull out capacity of the adjacent pipe anchors by attaching the reaction system on them, not the wall. This also will release the locked-in compressive forces in the pipe anchors.
- b. The jointing between two pipes should not form an enlargement of hole in the embedded soil. Such enlargement will act as a collar to reduce the redial stress on the pipes, hence reduce the skin resistance.
- c. Strain gauges shall be installed in pairs at the jacked-in anchor section to avoid flexural effect in the interpretation.
- d. Settlement profile behind the wall and even behind the end of jacked-in anchors shall be monitored to confirm the settlement trough above the active wedge, which could be a concern to the structures sit on top of the active wedge.
- e. More inclinometer results are needed behind the jacked-in anchors to indicate the formation of the active wedge.
- f. More researches on the generation of excess pore water pressure and its dissipation around and along the jacked-in anchor during inclusion to the retained soil shall be carried out to assess the set-up of interfacial resistance.

3 DISTRESSED OPEN CUT EXCAVATION IN FILL GROUND

This section presents two case studies of distressed basement excavation in filled ground investigated by the author. The processes of geotechnical investigation, remedial design and construction monitoring are discussed. Finite element analyses were carried out to reveal the associated mechanisms of the distressing ground with progress of excavation. Finally, lessons learned from the investigation are documented as useful mementos for future projects of similar nature.

3.1 Case C (Liew & Khoo, 2008)

Introduction

This case study involves construction of a high-rise mixed development with a five-and-half storey basement car park adjacent to an existing commercial development. Figure 19 shows the layout and the most severe and critical section (Cross-section A-A) of the development. The entire excavation face is about 250m long over an uncontrolled fill to the depths ranging from 7m to maximum of 14.5m.

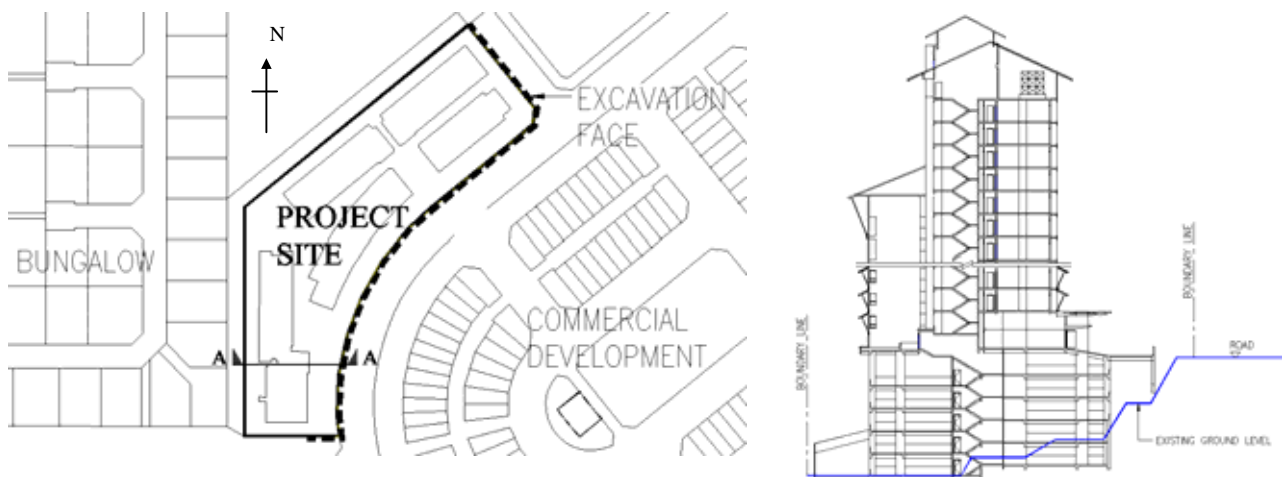


Fig. 19. Layout and Cross-section (Section A-A) of Development.

Site Observations

An inspection was carried out by the project team shortly after observing the tension cracks on the road pavement. The semi-circular crack pattern was generally observed on the top of the excavated slope as shown in Figure 20. Before excavation, water seepage at two locations were also observed indicating high groundwater levels within the filled slope (see Figure 20). At the time of investigation, the excavation at the southern portion of the site was in a more advanced stage. It was not surprising that more ground movements were occurring behind the excavation where the tension cracks were firstly observed.

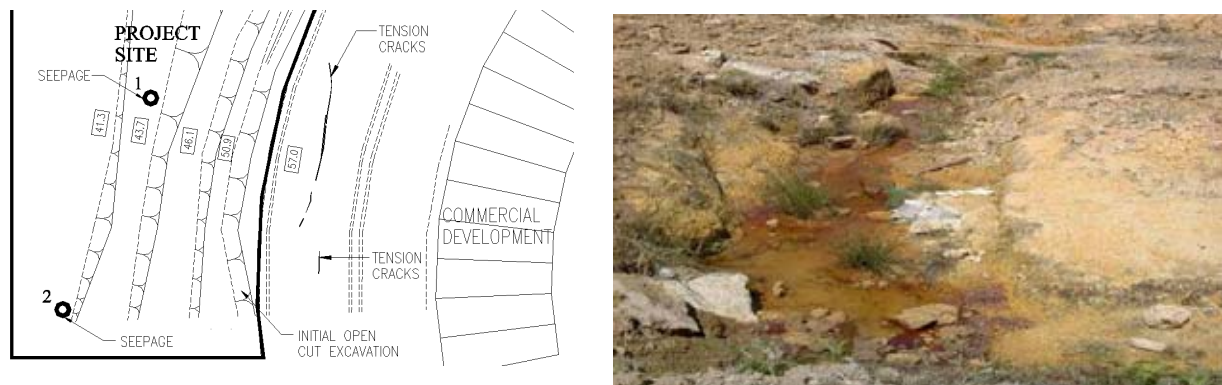


Fig. 20. Tension cracks on Main Road at earlier excavation stage and Seepage at Cut Platform.

Geotechnical Investigation

Information obtained from the detailed site investigation revealed that the slope above the lowest proposed basement level mainly comprised of massive uncontrolled fill. At one location where ground distresses were observed in the initial stage of steep open excavation, a previous natural valley with underground stream was later discovered originating from the north-eastern hilly terrain as illustrated in Figure 21. The adjacent commercial development had subsequently leveled a wide building platform with uncontrolled fill as thick as 15m primarily made up of loose sandy silt overlaying a thin (about 2m thick) deposited soft compressible material at the valley area.

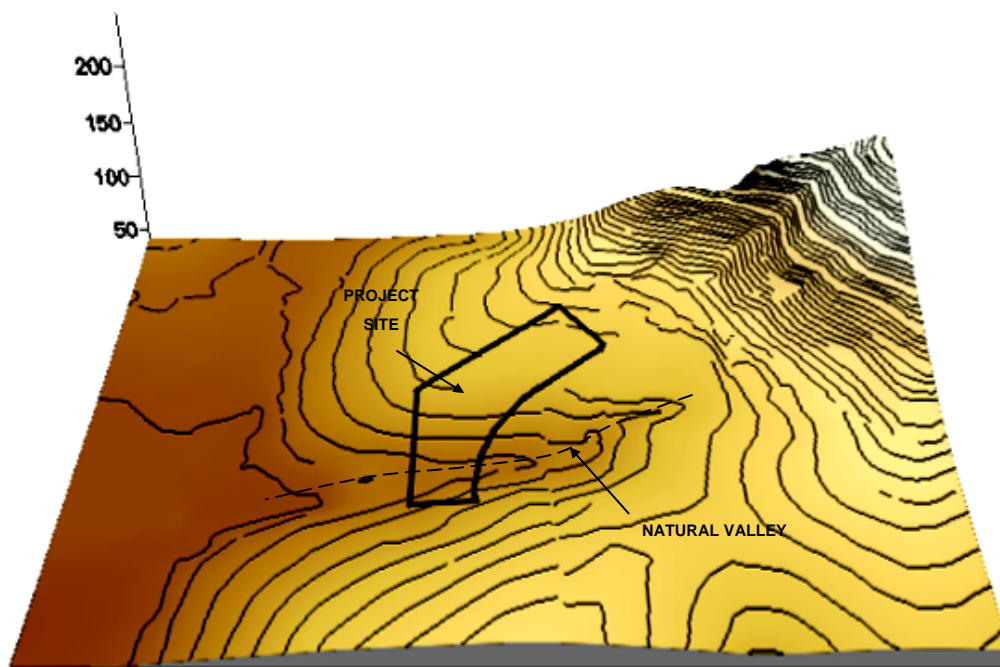


Fig. 21. Three-dimensional original ground contour.

Subsurface investigation (SI) was carried out to establish the subsurface conditions for the geotechnical investigation and remedial design. Figure 22 shows the layout of SI works, instrumentation and the interpreted boreholes logging. The fills material mainly consists of sandy silt with SPT-N values generally ranging from 0 to 20. A layer of soft material was detected at the depth of 12m to 15m below the ground surface in two boreholes (BH-IM1 and BH-IM4) within the distressed soil mass, which was overlying the natural valley in the pre-development topographical condition. The existence of this soft compressible material was further confirmed during an additional subsurface investigation when excessive lateral movements were detected in the subsequent staged excavation.

Groundwater levels were fluctuating and exhibiting seasonal storm responses throughout the construction period. The topographical features of a previous natural valley suggest that collection and concentration of underground seepage may have occurred within the previous valley. This is particularly evident in the soggy and saturated conditions of excavated materials immediately above the valley. Significant seepage was also observed at the previous valley area during excavation.

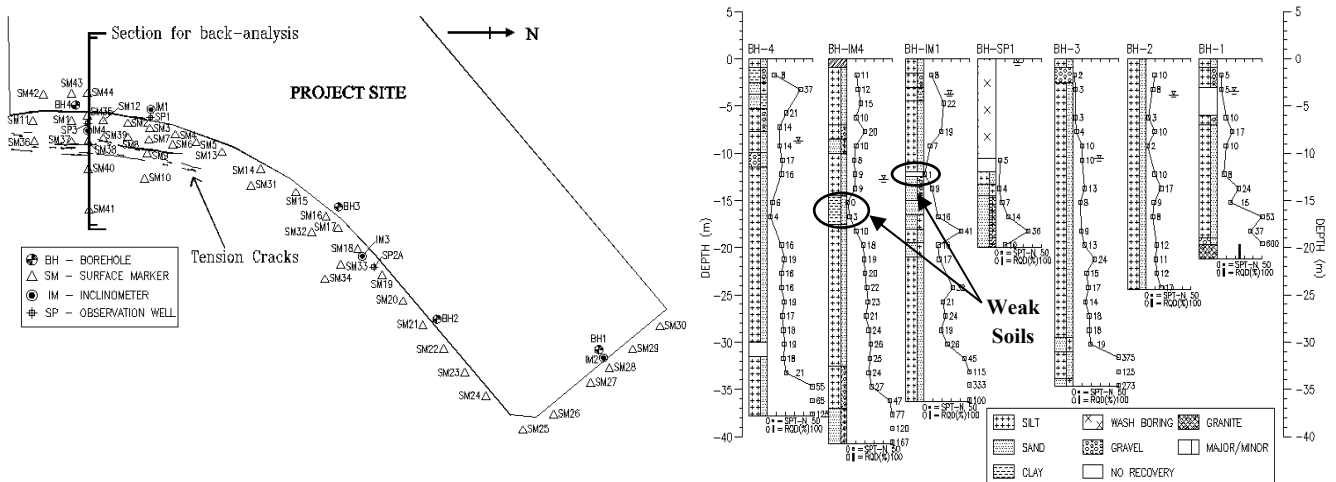


Fig. 22. Subsurface investigation and instrumentation layout and Borehole Logging Profile.

Remedial Design

For remedial and stabilisation works, the primary objective is to improve the safety factor of slopes to an acceptable design requirement. Soil nails with gunite surface was proposed to provide overall stabilisation and lateral support to the excavation for basement construction. The soil nail stabilisation works were designed to cater for a maximum retained height of 14.5m by reinforcing the in-situ saturated loose fill with closely spaced soil nails of varying lengths from 6m to 12m and structural gunite facing with sufficient weepholes/subsoil drains. The soil nailed slope was formed at steep angles of 4V:1H and the nails were installed at horizontal and vertical spacings of 1.25m centre to centre.

At the valley area where excessive creep movement was observed, 12m long FSP IIIA sheet pile walls with two rows of 18m long soil nail anchorages and permanent reinforced concrete props against the basement structure were used to supplement the passive resistance of the excavation in addition to the soil nailed slope on top. The cross-section of stabilisation work at the valley area is shown in Figure 23. Strength reduction method in finite element (FE) analysis was used to assess the original safety factor and the improvement after the proposed stabilisation work at this critical section.

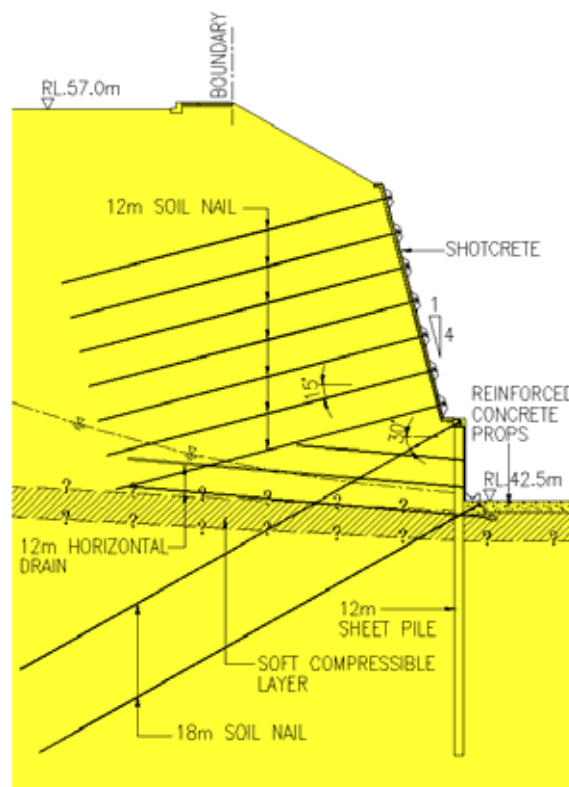


Fig. 23. Cross-section of stabilisation work in the valley area.

Construction

The details and method statement of soil nail stabilisation works to facilitate this deep excavation was discussed by Liew & Khoo (2006). Some of the interesting matters observed during the construction are described herein.

In one location, groundwater was observed continuously flowing out immediately after excavation. This particular location was believed to be the natural water path of the stream as revealed in the pre-development ground contour. Sufficient horizontal drains were installed at this location to release the perched groundwater. Figure 24 shows the water continuously discharging from horizontal drain after installation.

Observable additional ground subsidence was associated with the boring operation of the soil nails. This could be attributed to the excessive ground loss in massive manner of unlined micro tunneling in the loose fill.

When excavation nearly reached the final excavation level, additional field testing such as Mackintosh probing and in-situ penetrating vane shear tests were conducted at the expected soft compressible layer to verify the soil parameters adopted in the remedial design. The localized excavation for pile cap construction further confirmed the existence of this soft deposited material as shown in Figure 25.



Fig. 24. Continuous seepage flow collected in Subsoil Drains.



Fig. 25. Soft compressible material detected at localized pile cap excavation.

Instrumentation Monitoring

The instrumentation program was set up mainly as an alert system for construction safety control and to monitor the interaction performance of the proposed stabilisation system with the surroundings. This instrumentation scheme (see Figure 22) had provided sufficient coverage to monitor the performance of the nailed excavation. The readings of inclinometer, ground settlement and groundwater level were taken weekly. However when critical stages were involved, the frequency of the readings was increased accordingly.

Figure 26 shows the monitored soil nailed slope movements during various stages of excavation. While the cumulative ground settlements behind the soil nailed slope are shown in Figure 27. Groundwater levels had generally been monitored from the installed observation wells throughout the construction period. Figure 28 shows the measured groundwater levels over time.

From the monitoring results, the ground lateral displacement and settlement had stabilised with no appreciable further deformations after the completion of slope stabilisation works. In addition, the groundwater was observed achieving steady state equilibrium even after reaching the final excavation.

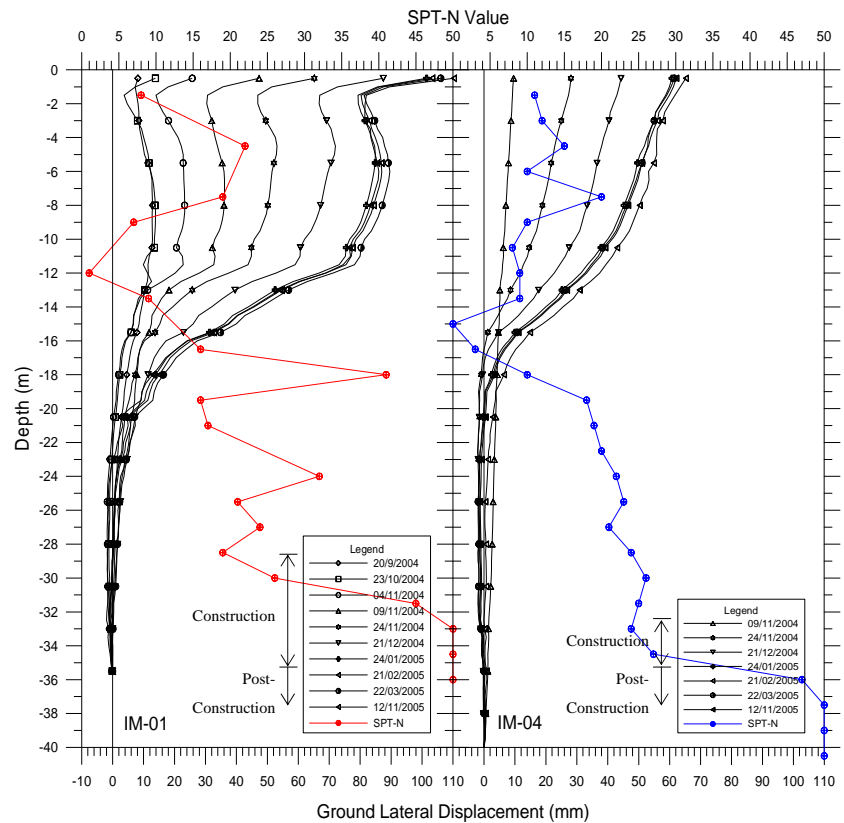


Fig. 26. Measured ground lateral displacements.

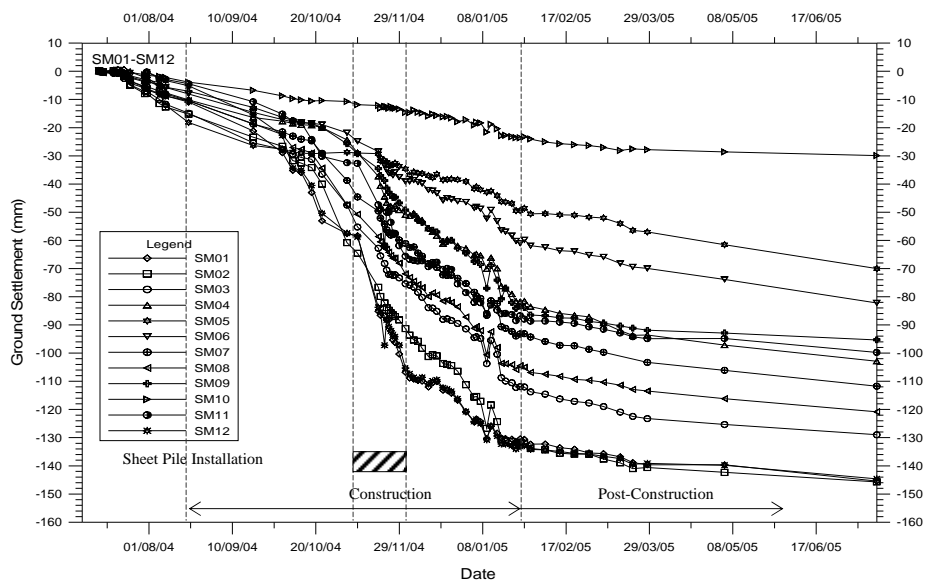


Fig. 27. Measured ground settlements.

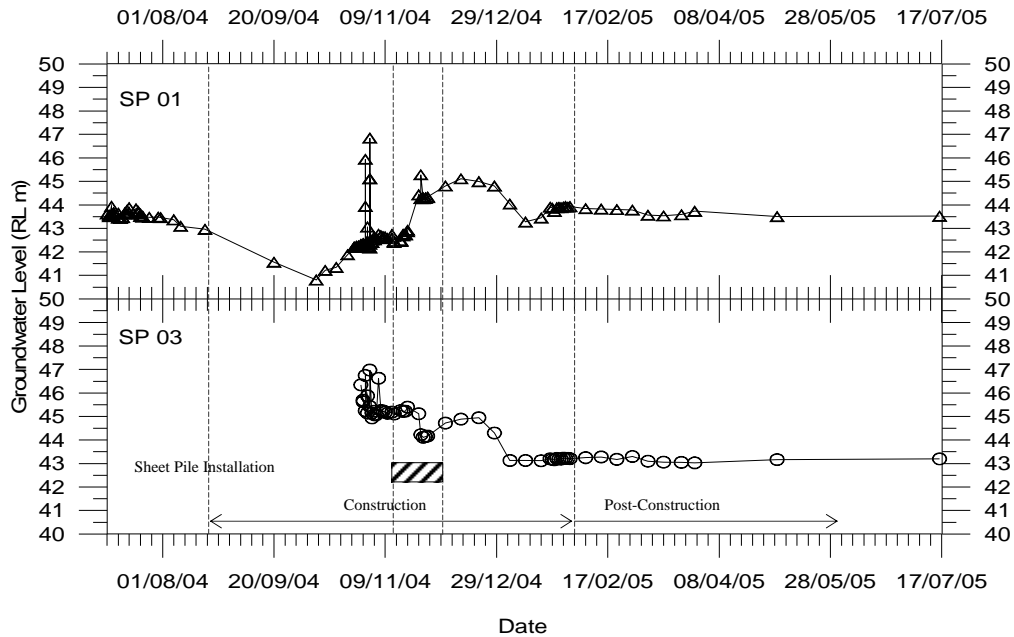


Fig. 28. Measured groundwater levels.

Back Analysis

Numerical modeling using the computer program “PLAXIS” was used to simulate the excavation sequence and installation of nails. The finite element (FE) analyses were aimed at gaining insight into the inherent mechanisms within the excavated slope and subsequently verify the behaviour of ground movements and settlements. The details of the back analyses were discussed by Liew & Khoo (2007).

Subsidence troughs developed on the retained ground surface has been observed from the FE back-analysis results (see Figure 29). The results show that surface subsidence is generally expected at a distance of 8m from the excavated face, which tallies extremely well with the site conditions as demonstrated in Fig. 30. The location of the trough is just immediately behind the end of the soil nail where a relatively large shear strain is developed along the potential slip surface behind the reinforced soil mass. This is fairly close to the formation of an active wedge in the retaining wall design. Two major tension cracks signify the extent of the developed active wedge behind the reinforced slope mass.

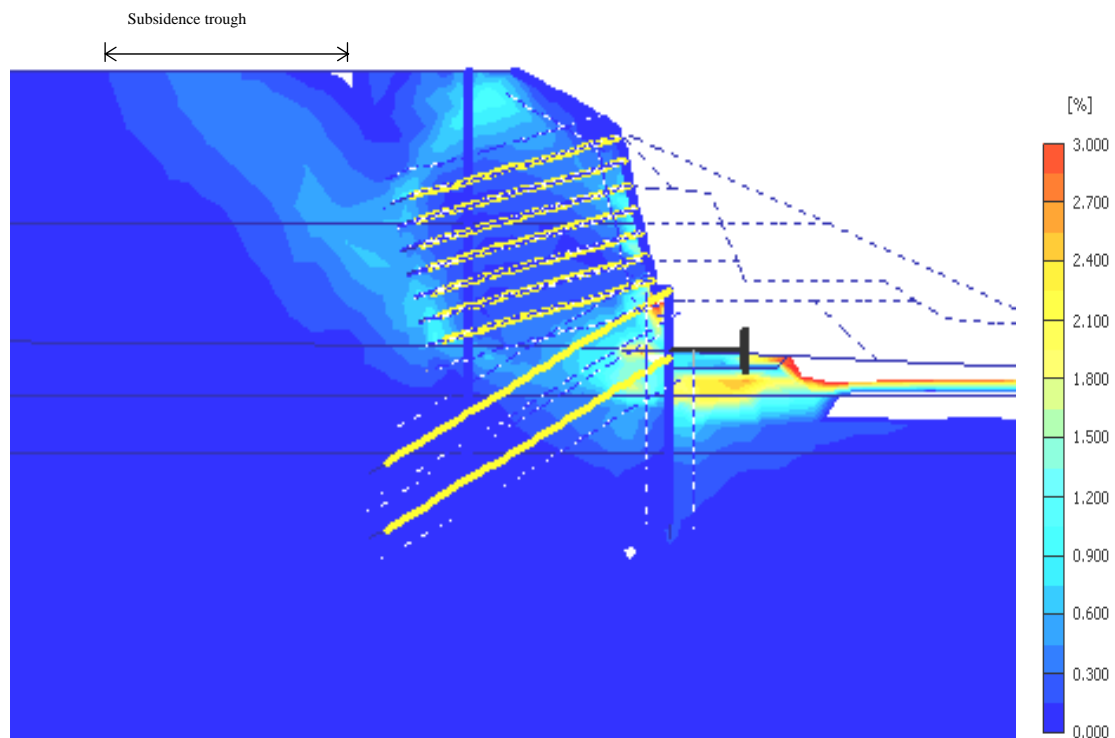


Fig. 29. Shear strain of soil mass within the soil nail reinforcing system.



Fig. 30. Subsidence trough developed on the retained ground.

3.2 Case D (Liew & Khoo, 2008)

Introduction

This case history involved construction of a two-storey basement in an urban area. The scope of investigation was to evaluate the conditions of a distressed temporary shoring structure, investigate the probable causes and subsequently to propose remedial options. Figure 31 shows the location of the project site and the adjacent land lot which had been affected due to ground distresses.

Temporary shoring structure consisting of contiguous bored pile (CBP) wall propped by raking struts against lower basement slab was proposed by the contractor to provide peripheral soil support for the 10.5m deep excavation at western boundary. The designed CBP wall is of 16m long 750mm diameter bored pile with cut off level at RL 54.6m. In order to facilitate the CBP installation, 12m long temporary steel sheet piles (type FSP IIIA) were driven from RL 59.0m at about 0.7m offset away from boundary to provide sufficient working platform and as the temporary shoring support for the exposed 4.4m temporary excavation (from RL 59.0m to RL 54.6m). Figure 32 shows the details and cross-section of the proposed alternative temporary shoring system and permanent retaining wall.

After completion of CBP wall installation, excavation was proceeded from RL 54.6m to RL 52.0m to remove the passive berm. Ground distresses at the adjacent higher platform in the forms of ground subsidence, tension cracks and deviation of the CBP wall were observed at the excavation.

From the site observation, deviation of CBP wall was likely caused by the over-excavation of the temporary passive berm with localized pile cap excavation in front of the wall without installation of the planned raking strut. The incidence had affected the adjacent property lot with considerable ground distresses and also structurally damages the CBP wall.

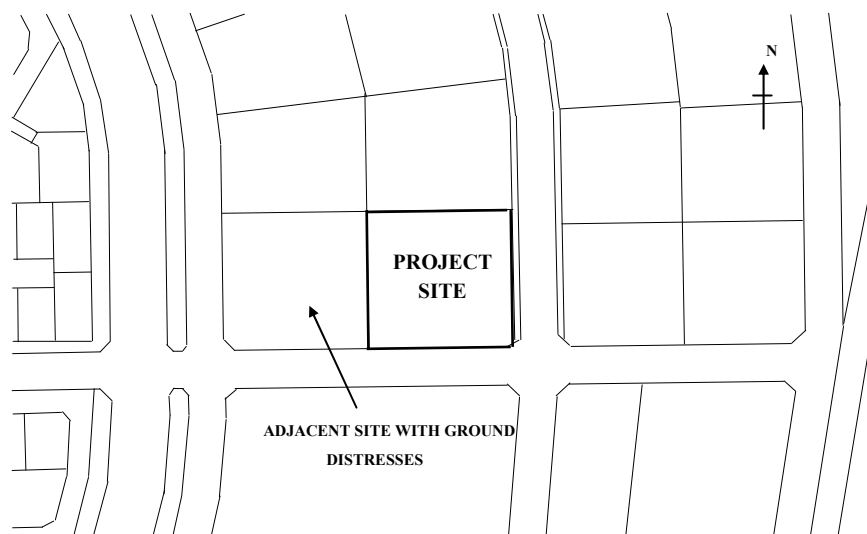


Fig. 31. Site location.

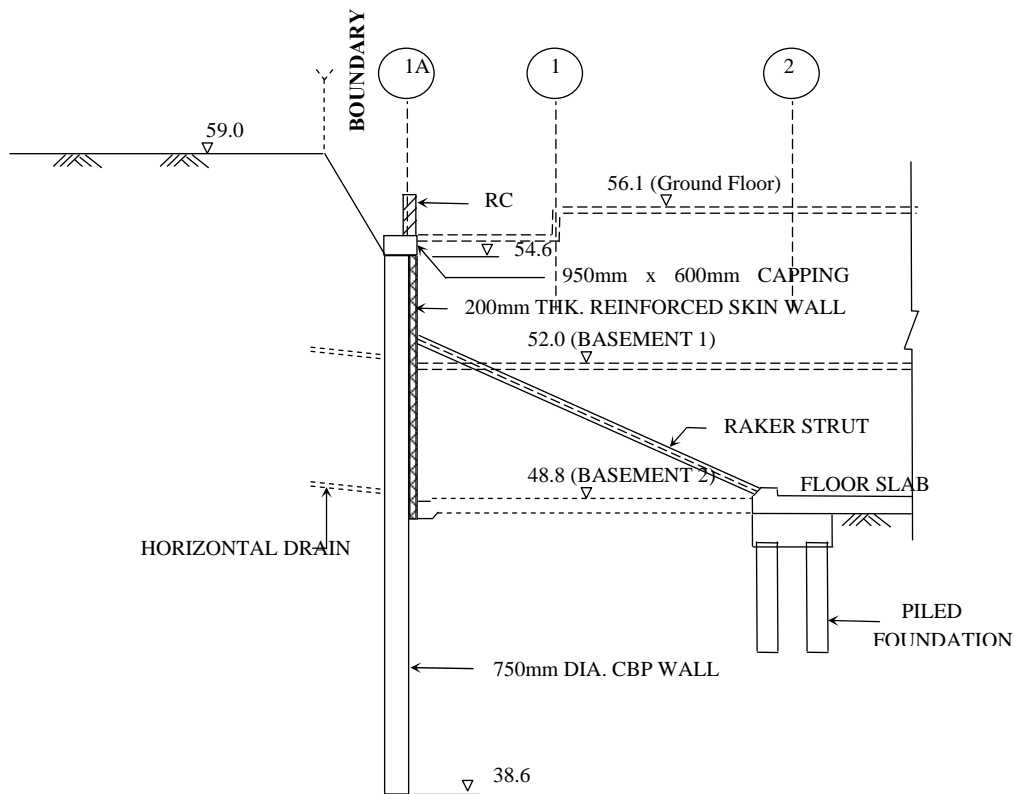


Fig. 32. Cross-section of the proposed alternative retaining walls.

Site Conditions

From the previous as-built earthworks drawings, the ground level before excavation was relatively flat ranging from RL 54.8m to RL 56.5m with a steep soil slope (1V:1H) of about 3m high sloping from adjacent lot (at RL 59.0m) towards the proposed site at the western boundary of the project site.

Based on the topographical survey plan of adjacent lot, which was believed to be the condition before the earthworks, part of the proposed site (i.e. at north-western side) is of sloping terrain from approximately RL 65m to RL 56m. There is a 6m high cut slope existed within this sloping ground as indicated in Figure 33.

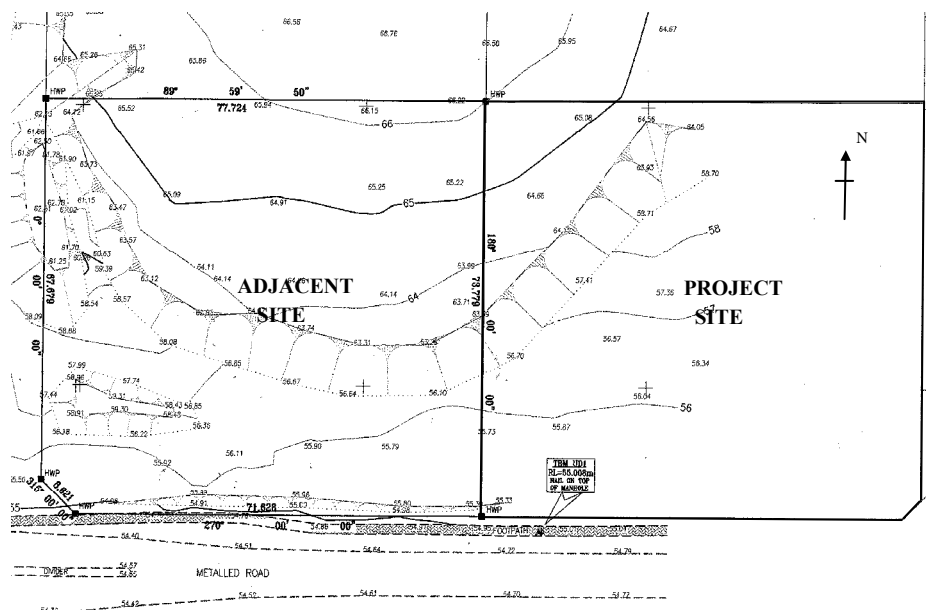


Fig. 33. Topographical survey plan of adjacent lot (subject to disturbance before the finished level).

Based on the pre-development topographical condition as shown in Figure 34, the contour lines for both the adjacent lot and the proposed site range from RL 54m to RL 47m. As such, it is evidenced that earthworks had previously been carried out at these areas to raise the building platform level to RL 59.0m and RL 56.0m for the adjacent lot and the proposed site respectively. Both of

the sites are on filled platforms. Particularly, the distressed area was primarily located at the valley where thicker fill was placed. High potential of saturation of fill due to perched groundwater seepage after filling in the previous valley terrain can be expected if subsurface drainage is not provided.

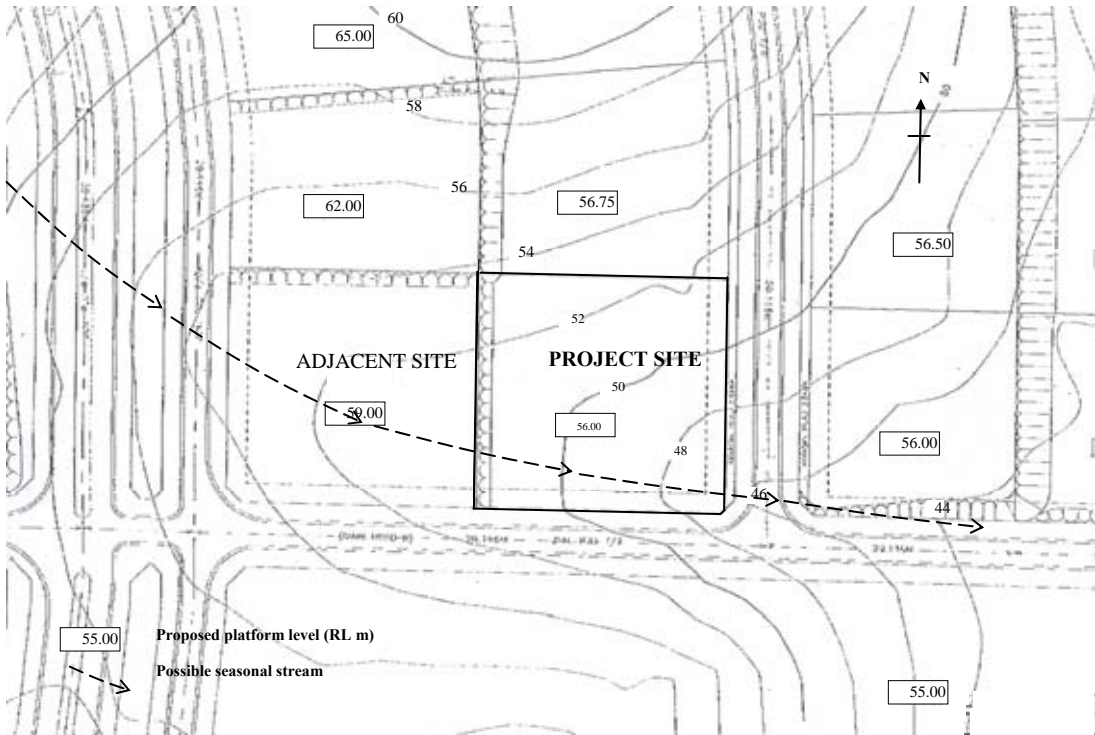


Fig. 34. Regional major earthworks layout with contour lines.

In addition to the topographical map, assessment on the piling information was carried out to reveal the soil consistency profile within the site as shown in Figure 35. It was found that the distressed ground and retaining wall areas correspond well with the expected thicker fill and deeper weathering profile at the valley area as discussed.

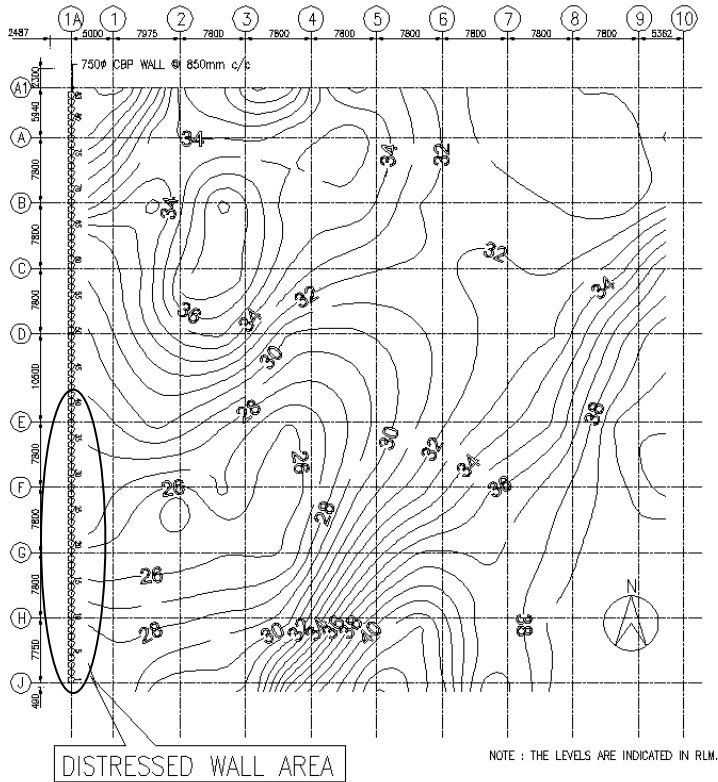


Fig. 35. Interpreted contour of hard stratum from piling information.

Site Inspection and Mapping

It was observed that continuous tension cracks appeared at varying distance away along the sheet pile wall of approximately 73m long. The tension cracks were more distant from the sheet pile wall at the southern end and become closer to the sheet pile walls toward northern end. At the time of inspection, more extensive excavation was carried out at the southern portion than the northern portion. The backyard car park platform of the adjacent lot had shown signs of subsidence and tension cracks as shown in Figures 36 and 37.



Fig. 36. Site conditions of adjacent lot after the incidence of wall and ground distresses (Southern view).



Fig. 37. Site conditions of adjacent lot after the incidence of wall and ground distresses (Northern view).

Site mapping had been carried out on the observed tension cracks and tilted sheet pile wall. Much more tension cracks were observed at the southern region. In this particular location, the sheet pile wall was seriously deviated outward relatively to the designed wall alignment. Efforts were made to map the crack lines by using measuring tape and slope meter. Figure 38 shows the details of mapped tension cracks.

Generally, the tension cracks were measured up to 400mm wide with shear drops between the two dislodged earth blocks. At the critical location (Gridline G), the major crack line was measured at approximately 6.2m from the original fence line. While, the

furthest crack line was measured at approximately 12m from the original fence line. In addition, the overall tilt angle of subsided platform at this area was crudely measured to be about 12° as shown in Figure 36. Sheet pile wall had also moved outward about maximum 1.2m from the designed wall alignment (at Gridline G). On the other hand, the measured pile top deviation of contiguous bored pile wall is also shown in Figure 38.

During the emergency repair work, the vibration effect of re-installing the temporary sheet piles wall by vibro hammer had caused further tension cracks at the backyard car park platform. Therefore, it is suspected that the platform is of filled ground, which might not be well compacted as the effect of vibration can cause soil densification and aggravated the creep movement.

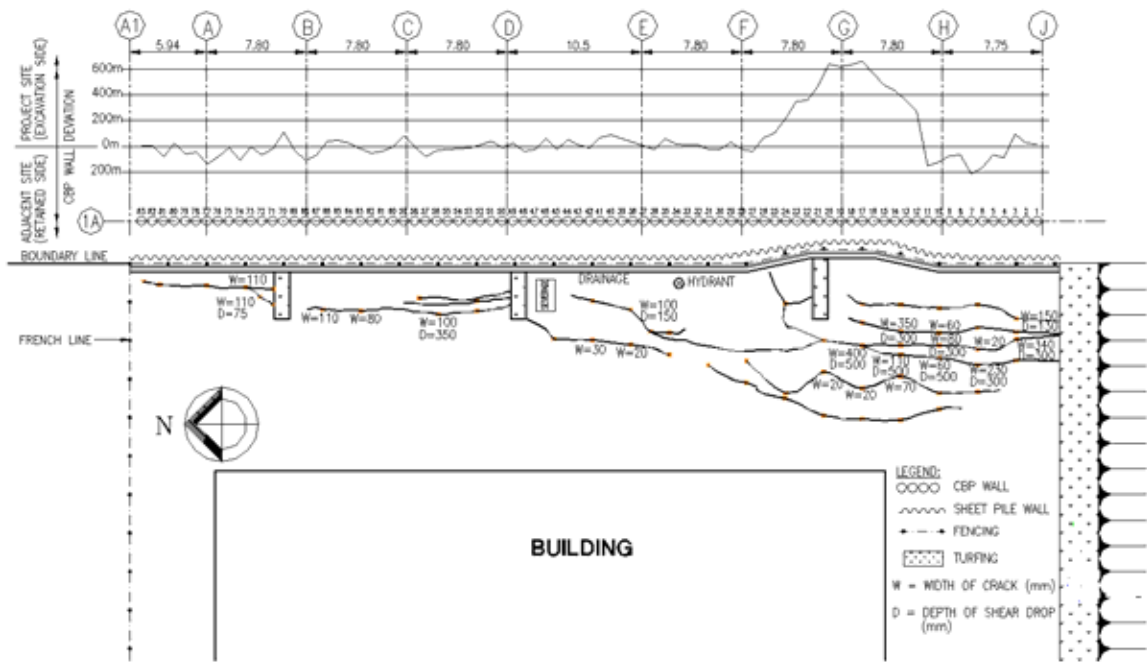


Fig. 38. Crack mapping layout.

Subsurface Investigation

Before the wall and ground distresses, two stages of subsurface investigation (SI) works were carried out at the proposed site. The second stage SI works is the additional SI conducted at the perimeter western boundary for the alternative basement wall design by the contractor. At that time, no much attention was given in identifying the weak deposits between the original ground and the platform backfill. The SI layout is shown in Figure 39.

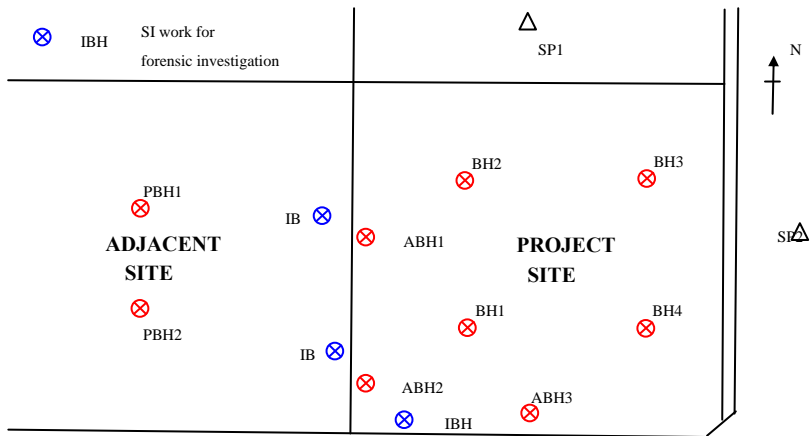


Fig. 39. Subsurface investigation layout.

After the incidence, additional three boreholes were sunk within the distressed wall area to investigate the subsurface profile and to install instruments for construction monitoring. In particular, a layer of 6-9m thick of very soft to soft sandy/silty clay (SPT-N <= 4) was encountered from RL 52m to RL 43m as detected in the few boreholes near to the distressed wall area. Generally, SPT-N values of the subsoil range from 3 to 6 at the top 13m of the subsoil and gradually increase with depth thereafter. This implies that the top layer of subsoil is most probably of fill material underlain by the soft deposits in the valley. A typical cross-section of subsurface profiles at the distressed wall area is shown in Figure 40. The groundwater levels recorded in the boreholes and observation wells were generally higher at the western side and lower towards the eastern side of the project site, which tallies with the flow path of the valley during pre-development condition.

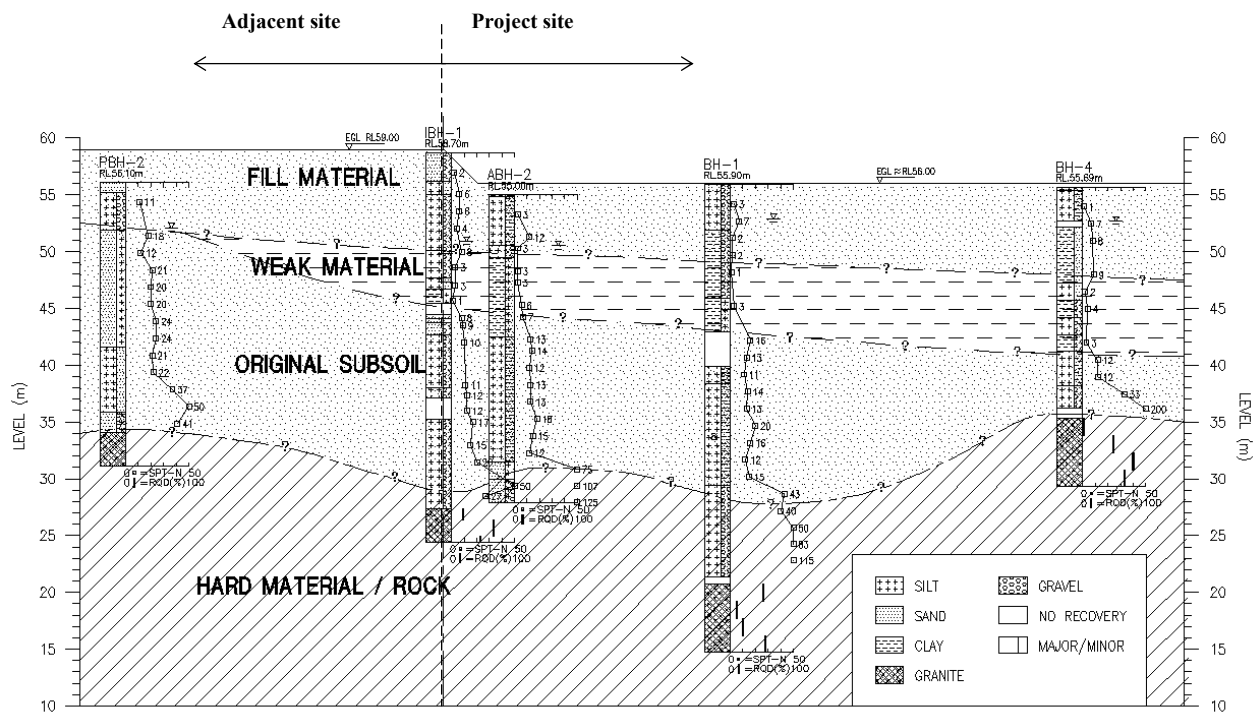


Fig. 40. Typical cross-section of subsurface profiles at the distressed wall area.

Back Analysis

In order to confirm the probable causes of ground distresses and wall movement, Finite Element (FE) analysis using computer software “PLAXIS” was performed independently based on the subsurface profile as shown in Figure 41. The construction sequences of excavation were simulated in the FE analysis.

At one analysis stage where over-excavation in front of the wall was carried out, the analysis results revealed that the retained earth platform displaced excessively in the horizontal and vertical (settlement) directions with the temporary sheet pile retaining wall moving forward. As part of the lateral resistance to the temporary retaining walls by the passive berm was removed before installation of raking strut, over-excavation of this berm had reduced the lateral resistance to the sheet pile wall and subsequently mobilises the resulting strength of the retaining walls from serviceability state condition towards the ultimate limit state condition. The excessively displaced temporary sheet pile wall had induced additional lateral force to the installed contiguous bored piles (CBP) walls. The high induced flexural stress unavoidably damaged the CBP pile and led to excessive ground distresses. The results of FE analyses (see Figure 41) reasonably well agreed with the measured wall movements and ground deformations (e.g. tension cracks, settlement and depression).

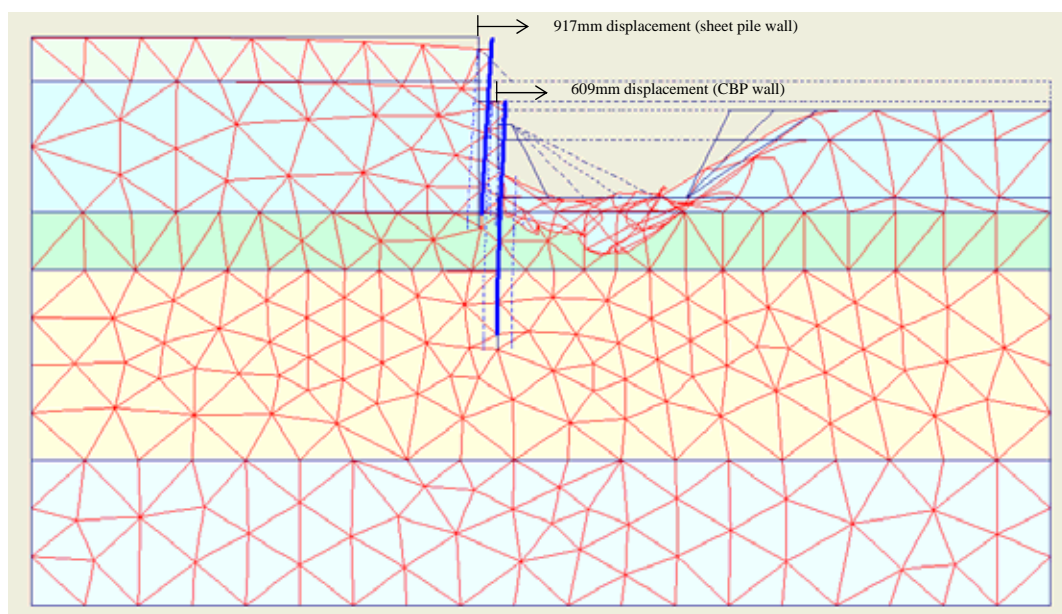


Fig. 41. Results of finite element analysis.

Remedial Design

The immediate remedial measure was to temporarily backfill the excavation adjoining the distressed area to the top of CBP wall with temporary stabilising berm (1V:1H). A variety of permanent remedial options have been explored. Finally, internal strutting against permanent basement structures was adopted to provide a safe and cost effective solution.

The remedial works carried out included installation of additional row of 18m long sheet piles behind the deviated CBP area to stabilise the retained ground. Temporary strutted coffer-dam was constructed to facilitate the localized excavation of lift core. Two layers of temporary horizontal strut were installed propping the sheet pile wall against the permanent basement structures at distant before casting of remaining basement slab as shown in Figure 42. Excavation was only allowed to be carried out in stages after the struts were put in placed. Once the final excavation level (B2) was reached, the CBP wall was cut off at that level and verified with integrity testing (both low and high strain dynamic pile tests) to confirm structural integrity. Fortunately, most of the damages of the deviated CBP wall were well above the B2 level. Cast in-situ reinforced concrete wall was constructed over the starter bars from the intact CBP wall. The finished permanent basement structure is shown in Figure 43.

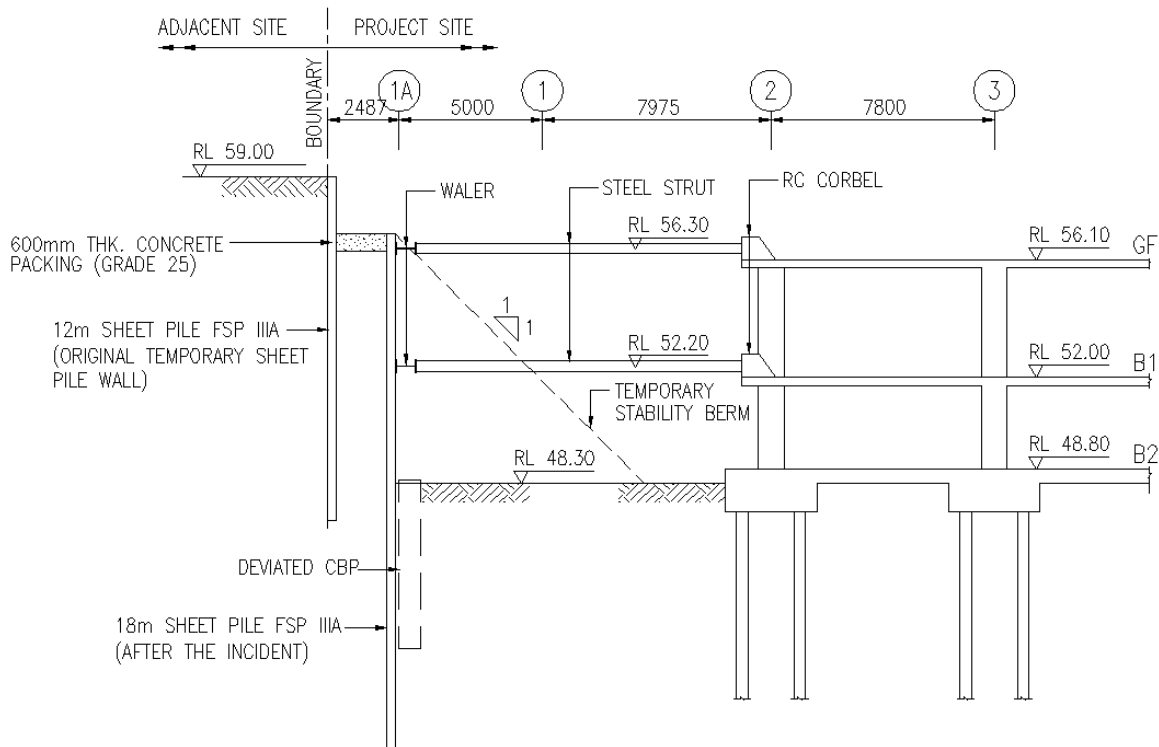


Fig. 42. Cross-section of remedial works.

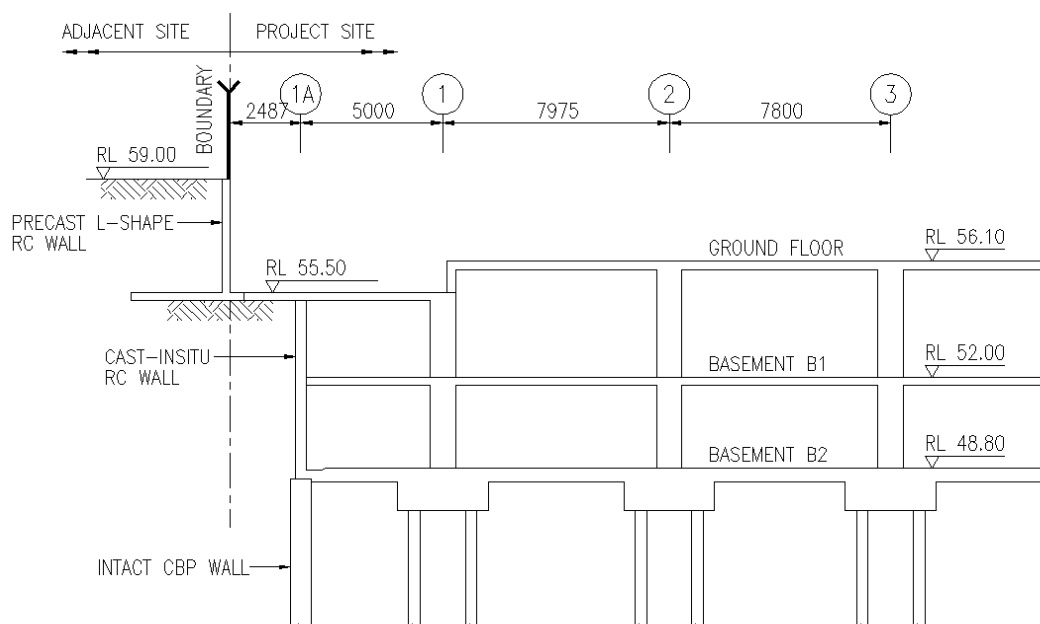


Fig. 43. Cross-section of permanent basement structure.

Summary of Findings

The investigation results from the two case studies collectively revealed the following findings:-

- a. The distressed ground was located over a natural valley where thicker fill was placed over the previous soft deposits without proper engineering treatment to form building platform. The perched groundwater seepage in the original valley was observed and possibly weakened the retained fill.
- b. Occurrence of tension cracks during initial open excavation and installation of sheet piles suggested that the underlying subsoil and at the valley area are inherently vulnerable to ground disturbance and hence are prompted to distressing.
- c. The existence of soft compressible material at the valley area was further confirmed during additional subsurface investigation and localized pile cap excavation when reaching the final excavation level.
- d. Ground loss induced subsidence by soil nailing operation shall be considered in remedial design.
- e. Original topographical features are the important design consideration for excavation stability and remedial strategy. In these case studies, natural valley with soft deposits was not detected during design stage causing cost escalation as a result of remedial work.
- f. Perched groundwater regime can occur in backfilling over natural valley leading to unfavourable behaviour of backfill.

Lessons Learned and Conclusion

By presenting two case histories in this paper, the following lessons learned, in the author's opinion, can be useful mementos for future projects of similar nature:-

- a. Soft deposits at the lower part of the valley and potential concentrated underground seepage are common in hilly terrain and should not be overlooked. Desk study of pre-development ground contours to identify potential geotechnical problems is highly recommended.
- b. Filling over valleys without proper site clearing, removal of unsuitable soft deposits and compaction could result in highly unstable backfill for any open excavation. It is important to thoroughly investigate the subsoil condition beneath the fill. Otherwise, proper treatment to the low-lying ground before the development earthworks shall not always be assumed.
- c. Provision of subsoil drainage schemes for stabilisation works of excavation can be very effective in improving the stability, particularly in filled ground over natural valley area.
- d. Ground subsidence due to ground loss in soil nailing can be minimised by lining protection of casing during drilling through the loose fill though the progress of drilling is generally slower than open-hole drilling.
- e. Comprehensive instrumentation schemes at strategic locations can reveal the associated mechanisms in order to derive effective remedial solution and make necessary design modification to suit actual ground conditions. Generally, these instruments include inclinometers, surface markers, observation wells, etc, are highly recommended.
- f. Back analyses using the ground geometry before the distresses and the sequence of events can provide realistic operative strength parameters for remedial design. Both the limit equilibrium stability assessment approach and FE analysis have been proven to be a useful engineering assessment tool. However, further verification of the adopted soil parameters by field and laboratory tests would be useful and enhance the confidence of the remedial design.
- g. In addition, FE analyses have been successfully utilized to investigate and verify the distresses by revealing the associated mechanism.

4 DISTRESSED RETAINING WALLS

This section will present two case studies of distressed piled retaining structure, in which the pile foundation is structurally distressed when subjecting to significant lateral load from the active earth pressure and water pressure behind the wall as a result of unforeseen coincidence of extreme weather condition during temporary wall construction.

4.1 Case E (Liew, 2007)

A reinforced concrete (RC) retaining wall, with retaining height ranging from about 1.6m to 7m, was built at close proximity to an existing stream to retain a building platform at reduced level of RL48.00m. The project site is underlain by Kenny Hill Formation consisting of Carboniferous to Triassic meta-sediment interbedded between meta-arenite and meta-argillite with some quartzite and phyllite. Due to intense weathering processes in a tropical climate, the upper meta-sediments have been transformed into residual and completely weathered soils (Grades V and VI). The upmost overburden materials are soft compressible alluvial deposits from the stream. Sudden movements and vertical flexural cracking of the 7m high retaining wall were observed when the backfill behind the wall reached the height of about 1m below the wall top. The backfill material was partially removed to reduce the earth pressure on the retaining wall after the wall movement.

A section through the distressed RC retaining wall is shown in Figure 44. The RC retaining wall was supported on five rows of driven vertical precast 200mm RC piles at 2m longitudinal spacing. The vertical compressive working load of the 200mm RC square piles is 450kN. High strain dynamic pile test was previously carried out on seven of the piles. The pile driving records of the test piles indicated that the piles were driven to end-bearing condition and the installed lengths ranged from about 4.5m to 11.7m. The mobilised pile capacity in the six tested piles had achieved a minimum factor of safety of 2.0 and one pile with a

marginally lower factor of safety of 1.8. The test cube results for the wall construction show that the concrete cubes had achieved the designed strength of 30MPa.

Site inspection was carried out immediately after the wall displacement and tilting were reported. During the site inspection, vertical flexural cracks were observed at the front and back of the displaced wall, particularly over the portion in close proximity to the return of the wall. Three (3) levels of weepholes had been installed in the retaining wall at RL42.5m, RL45.0m and RL47.50m. There was water staining from the weephole drains located at the mid height and bottom rows, revealing that groundwater level behind the retaining wall had previously risen above RL45m. The incident occurred after an intensive prolonged antecedent rainfall event. Figure 45 shows the overall site condition after the distress and the water staining at the weephole drains.

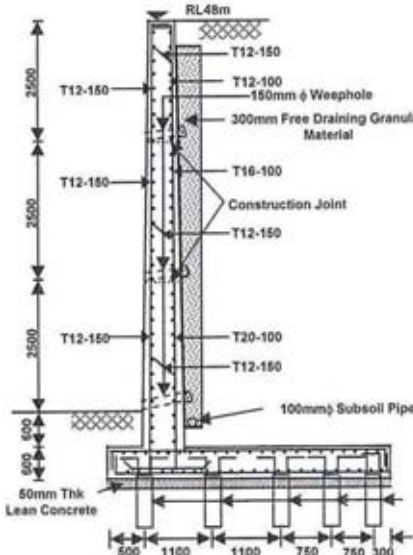


Fig. 44. Typical Section of Piled Reinforced Concrete Retaining Wall



Fig. 45. Overall Site Conditions and Water Staining at Weephole Drains

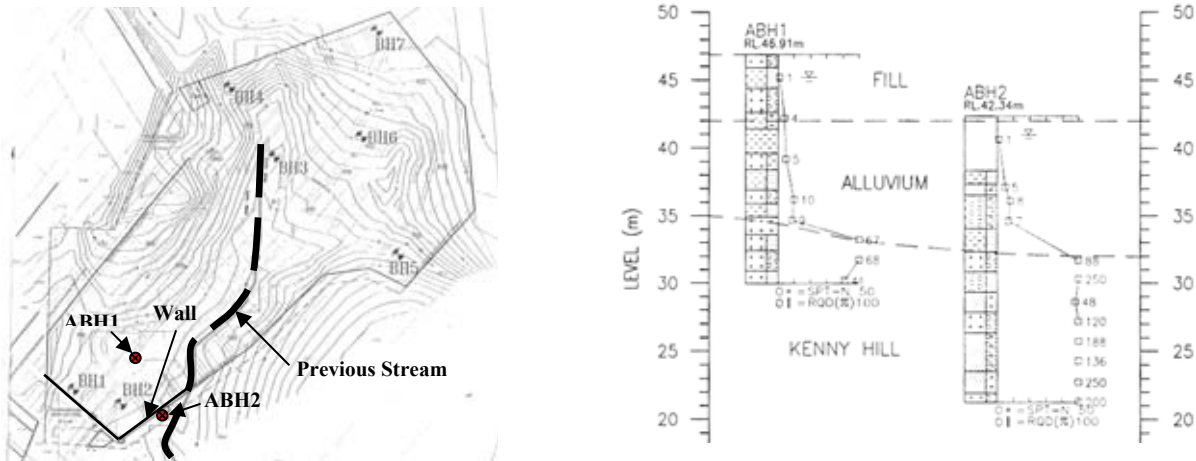


Fig. 46. Site Topography and Subsoil Profile

Subsurface Information

The layout of two previous subsurface investigation (SI) works carried out for the project. The first one consisted of seven boreholes for the entire project site is shown in Figure 46. The second SI works consisted of three boreholes and 25 Mackintosh Probes carried out along the retaining wall alignment. Only basic laboratory tests, such as soil classification had been performed in both SI works. There was no strength testing carried out in either SI.

The particle size distribution tests from boreholes BH-8, BH-9 and BH-10, which are closest to the RC retaining wall, indicate significant percentages of silt and clay materials within the first 3m depth. From interpretation, the overburden materials above RL38m are likely to be alluvial deposits, which are primarily soft compressible fine soils.

Additional SI work, consisting of two boreholes (ABH series in Figure 46), vane shear tests, ten Mackintosh Probes and laboratory testing, was proposed to investigate the shear strength parameters and to reconfirm the subsoil profile. The SI layout plan and borehole profiles are shown in Figure 46. Borehole ABH-1 was carried out about 14m behind the RC retaining wall indicating higher percentage of silt and clay while borehole ABH-2 was carried out near the toe of the RC retaining wall showing high percentage of sand and gravel. Both boreholes ABH-1 and ABH-2 encountered hard material at RL34m and RL32m respectively. The borelog profile of ABH-1 shows the top 5m of fill above RL42m with low SPT-N values. From RL42m down to RL35m for borehole ABH-1 and RL32m for borehole ABH-2, the subsoil materials are considered likely to be alluvial deposits.

Penetrating vane shear tests were carried out next to ABH-2 to determine both the peak and remoulded undrained shear strength profiles of the soil. The vane shear tests indicated a sudden drop in the measurement of peak undrained shear strength ($S_{u,peak}$) at the depth of about 4m below ground, where the measured strength is close to the remoulded strength ($S_{u,remoulded}$), potentially suggesting the existence of a disturbed shearing zone associated with a slip surface at this depth.

Consolidated Isotropically Undrained (CIU) triaxial tests, with pore pressure measurement, were also carried out to confirm the drained shear strength parameters. The interpreted effective shear strength parameters in the material below the wall were $c' = 5\text{kPa}$, $\phi' = 33^\circ$.

Probable cause of wall movement & Remedial solutions

Based on site observations and the analysis results, the assessed cause of the wall movement is primarily due to inadequate lateral resistance of the piled retaining wall when groundwater rises above RL45m after prolonged antecedent rainfall. The lateral resistance of the retaining wall from the wall base friction is insignificant as the stiff pile foundation attracts most vertical wall loading. As a result, the soil beneath the wall base did not experience much increase of vertical effective pressure and hence lowering the effective resistance of the stability slides in the global stability. The assessment shows that the total lateral resistance provided by the retaining wall and foundation system is inadequate to resist the lateral forces when the groundwater level rises above RL45m. Therefore, the increased lateral forces have caused structural failure to the piles, thus leading to excessive wall movement.

Based on the findings from the investigation, the following preliminary suggestions were made when considering the remedial works of the new retaining wall:

- The vane shear results show evidence that the slip surface could have been formed at about 4m below the ground at the RC wall toe. The strength profile indicates that the available strength at the shearing zone has reduced to residual strength. Therefore, it is important that the disturbed material at the shearing zone be removed and replaced with material of higher strength. Otherwise, ground treatment methods such as stone columns can be considered.
- The analyses results reveal that the sliding resistance for the retaining wall is inadequate when groundwater level rises above RL42.5m. Therefore, adequate sliding resistance shall be provided for the new retaining wall system. In addition to this, the global stability with slip surface passing underneath the wall shall be analysed to ensure adequate FOS.
- It is also important to have adequate surface and subsurface drainage for the new retaining wall system during and after construction. This is to minimise the infiltration of surface runoff into the wall backfill as the FOS against instability of the retaining wall system reduces significantly with the rise of groundwater level within the wall backfill.

Conclusions

From this simple forensic investigation, the following conclusions and lesson learnt can be summarised:

- The water level within the wall backfill and the active soil pressure behind the wall would have remarkable influence on the factor of safety against instability. Weepholes on the wall do not necessarily warrant the wall stability. Subsoil drainage system can be incorporated to control the water level within these soil zones to effectively improve the factor of safety.
- When pile foundations are used to support retaining walls, caution must be taken to properly model the vertical effective stress in the soil beneath the wall, which can be model with zero self weight of backfill above the wall base and the wall itself. If this aspect is not properly addressed, errors in factor of safety on the optimistic side can be as high as 105% depending on type of analysis and groundwater condition. If the in-situ soil sliding resistance under the wall self weight is adequate to resist lateral earth pressure and ground water pressure, designing the wall without pile foundation could be safer than the one with all vertical piles, except for necessary assessment on wall settlement to ensure adequate serviceability limit.
- Slender vertical piles are generally not suitable for supporting retaining wall on weak and compressible soils as they offer insignificant lateral resistance to the wall. Raked piles in combination of vertical piles can be a more effective foundation system to support the wall if wall settlement is a concern.

4.2 Case F

This case study presents the collapse of a portion of the reinforced soil (RS) wall. Investigation was carried out to determine possible causes of the collapse. Figure 47 shows the section of the distressed RS wall constructed in front of the monsoon drain.



Fig. 48. Collapsed Reinforced Soil Wall



Fig. 49. Sign of Water Seepage on Top Wall Panels



Fig. 50. Pile Head Connection Exposed after Removal of Collapsed Debris

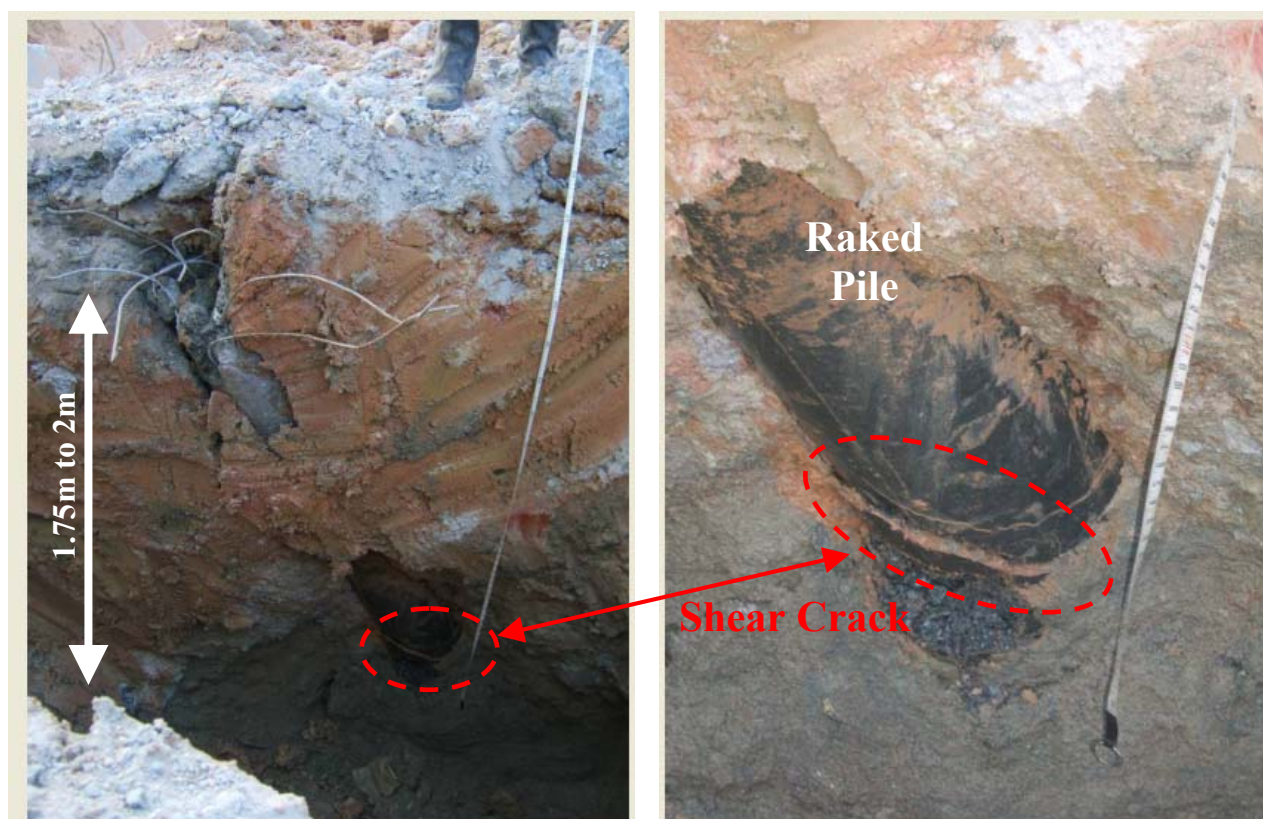


Fig. 51. Pile Damages Exposed after Trial Pit Excavation

Rainfall Data

Figure 52 shows the meteorological daily rainfall data of nearby rain gauge station from October 2006 to 9th January 2007 provided by Malaysian Meteorological Department (MMD). Another rain gauge station of the Department of Irrigation & Drainage (DID), Malaysia is corresponding well to MMD station enhancing the fact that the rainfall event during this period is a large scale regional rainfall event rather than a localised climatic phenomenon. Prolonged rainfall with extremely high intensity was observed at the above two stations from mid to the end of December 2006.

Average Recurrence Interval (ARI) with respect to various precipitation periods was calculated for the rainfall events before the RS wall collapsed. The basis of the ARI calculation has been made to an established Rainfall Intensity-Duration-Curve.

Based on the analysis results, most of the calculated ARI with respective precipitation periods are more than 200 years, which have been far exceeding in normal design life of a local drainage system of development in urban area. The ARI of a temporary drainage of a site is normally the same as the expected construction period with some extra for allowance.

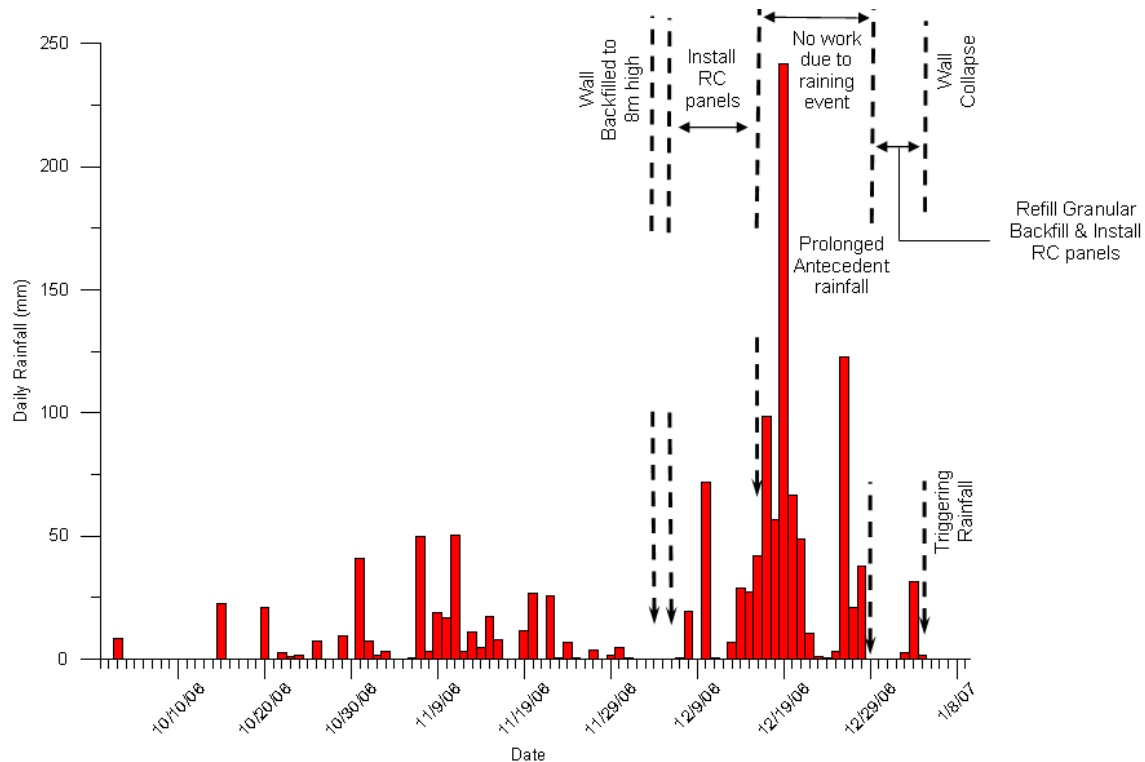


Fig. 52. Daily Rainfall Chart

Analyses

Analyses on ultimate limit state condition of the wall design scenario were carried out to determine possible causes of the wall collapse, including the stability of the RS wall and foundation piles. Sensitivity analyses were carried out to check stability of the wall system with respect to various water levels. Representative design parameters interpreted from the available information were used in the analyses. The safety factors of all the wall failure modes and structural and geotechnical failures of the foundation modes are investigated with respect to the two following scenarios:

- Scenario 1 : Water level within the RS wall block changes together with the water level behind the wall. This represents groundwater regime from the infiltration of prolonged antecedent rainfall.
- Scenario 2 : Water level within the RS wall block changes but the water level behind the wall maintained at the anticipated normal water level. This represents the rapid groundwater table rise in the granular backfill due to concentrated recharge from surface runoff.

Results of the analyses for the two scenarios are summarised in Figure 53. Generally, Scenario 1 is the governing case.

Wall External Stability

The external stability of the reinforced soil block with respect to various water levels behind the wall was checked. As shown in Figure 53, the factor of safety against wall sliding above the piled slab reduces to 1.0 when the water level is at about 7.2m above the wall base in Scenario 1.

Foundation

Checks on lateral pile capacity, bending moment and shear resistances of the pile were carried out. The distributed loads on the foundation piles under combination of wall loadings were analysed using a pile group programme PIGLET developed by Prof. Mark Randolph of University of Western Australia (2004). The maximum loads on the piles obtained from the analyses are compared against the pile capacities. As shown in Figure 53, the lateral pile capacity is more critical when comparing to other failure modes of the wall investigated. The factor of safety against lateral pile capacity reduces to 1.0 when the water level rises to about 7.3m above the wall base in Scenario 1. However, Figures 50 and 51 reveal structural failure of the foundation piles.

Global Stability

Global stability of the RS wall was also checked with respect of different water levels. For more representative modeling, the self weight of RS wall was ignored in the analyses, as the wall is both supported vertically and laterally by piles. The pile resistance against lateral loading was considered in the stability analyses. The results show that the factor of safety is still higher than 1.0 when the water level is about 8m above the wall base.

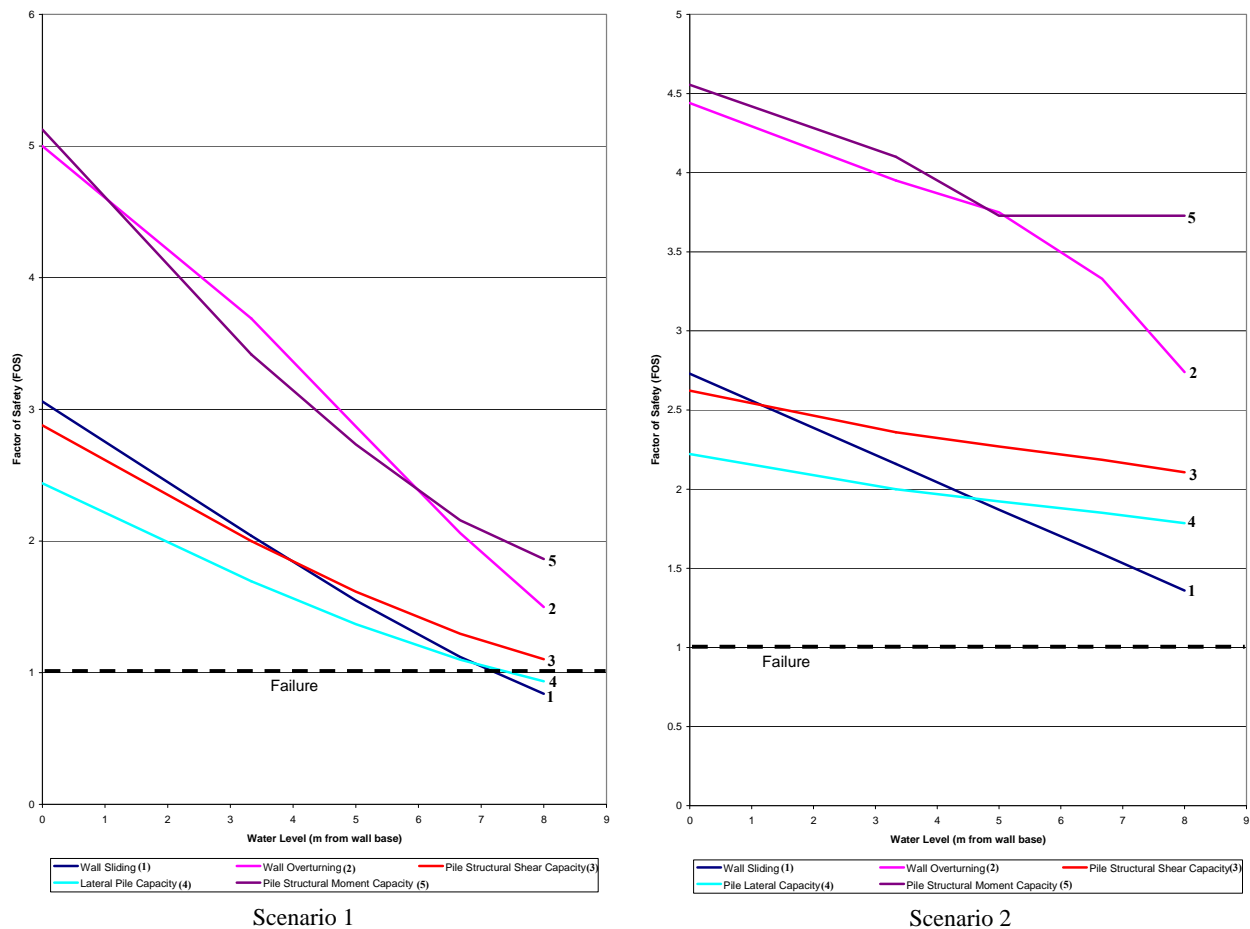


Fig. 53. Safety Factors of Geotechnical Resistance and Achieved Load Factor against Structural Resistance of Wall Elements

CONCLUSIONS

Based on the review on the available information, site observation and analyses, the following conclusions are made:

- The stability of RS wall is adequate under the anticipated groundwater table from normal water seepage condition in the granular backfill in the wall.
- Scenario 1 with “water level within the RS wall block changes together with the water level behind the wall” is more critical with lower factor of safety.
- Since the 2m cohesive in-situ soil backfill had not been carried out at this area before the collapse, and despite the provision of temporary earth drains to cut off surface runoff, the fast water percolation from intense heavy rainfall into granular backfill would cause rapid perched water level in the wall. Although the granular material is of high permeability and the gaps between the RS wall panel can allow discharge of water within the wall, the water level would perch up in the wall when the water inflow due to heavy rainfall is higher than the discharge.
- The probable causes of the wall collapse are elaborated below:
 - Due to the prolonged antecedent rainfall event with extremely high intensity, water level behind the RS wall rise.
 - Following the intense rainfall in the evening on 3 January 2007, the water level behind the wall rises near to the threshold level of about 7.2m above the wall base and triggers the foundation piles failure in flexural bending, as the exceptionally high lateral pressure, particularly high water pressure, exceeding the lateral pile capacity. As a result, the RS wall displaced significantly after failure of the foundation piles.
- The design of piled retaining wall is not as robust as wall supported on treated ground with ground improvement as the pile foundation is usually weak in lateral resistance and exhibits brittle failure when approaching limit state condition. If pile foundation is to be used for settlement control, it is important to have reasonably balance number of raked piles in both directions for robust lateral support.

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