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LESSONS LEARNED FROM TWO INVESTIGATION CASES OF GROUND DISTRESSES DUE TO DEEP EXCAVATION IN FILLED GROUND

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ABSTRACT

Ground distresses such as ground settlement/subsidence, lateral movement, cracking, etc, are usually the main concern for any excavation project. The contributory factors of ground distresses could be from various aspects and sometimes are similar in nature. This paper presents the processes of geotechnical investigation, remedial design, construction monitoring for two case histories of ground distresses occurring on a retained platform due to excavation in filled ground. Desk study, site inspection and subsurface exploration have been deployed to reveal the evidences and identify probable causes of the distresses. Back analyses utilizing finite element computer program "Plaxis" proved useful to reveal the inherent mechanisms of ground distresses. Lessons learned from the investigations are documented as useful mementos for future projects of similar nature.

INTRODUCTION

As deep excavations are normally carried out to utilize underground space in densely populated areas, protection of adjacent buildings and properties is a primary design concern nowadays for underground construction. Decades ago, the task of engineer was to design the peripheral soil support for an excavation to provide an acceptable factor of safety against collapse. Addressing the risk of excessive deformation of walling was frequently not a high priority as there are less sensitive structures at close proximity to the excavation. Now this has changed and the provision of deep basement on urban sites has demanded more stringent serviceability limit state design conditions. The problems of ground settlement/subsidence, heave and horizontal soil movement become top design priorities.

The contributory factors of ground distresses could be from various aspects and sometimes are similar in nature. Experiences indicate that the rectification of buildings and/or structures which have been affected by ground movements is both costly and time consuming. Therefore, careful assessment on the effect of ground movements of retained ground and structures is vital to ensure safety and serviceability. The endeavor would be indeed paid off as damages to adjacent buildings and properties due to ground movements were greatly minimised.

This paper presents two case histories of basement excavation in filled ground investigated by the authors, which caused ground distresses at the retained platform. The processes of geotechnical investigation, remedial design and construction monitoring are discussed. Finite element analyses were carried out to reveal the associated mechanisms of ground distresses. Finally, lessons learned from the investigation are documented as useful mementos for future projects of similar nature.

CASE HISTORY A

Introduction

This case history involves construction of a high-rise mixed development with a five-and-half storey basement car park adjacent to an existing commercial development. Fig. 1 and 2 show the layout and the most severe section (Cross-section A-A) of the development. The entire excavation is about 250m long over an uncontrolled fill to the depths ranging from 7m to maximum of 14.5m.

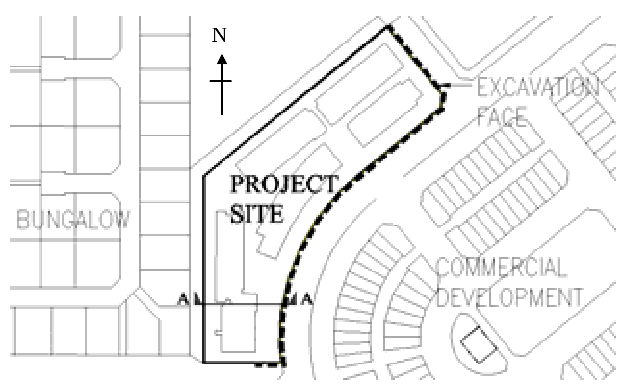

Fig. 1. Development layout.

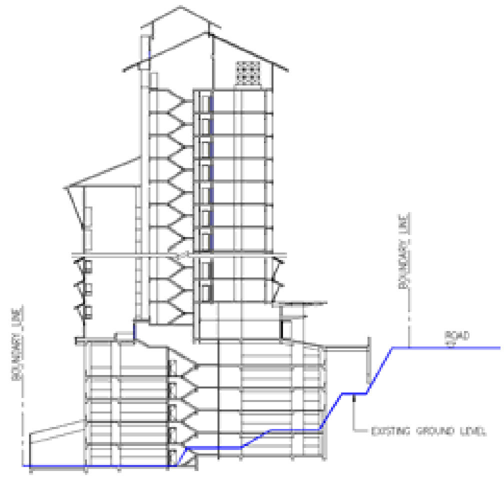

Fig. 2. Cross-section A-A of Fig. 1.

Site Observations

A site 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 Fig. 3. Water seepage at two locations were also observed, indicating high groundwater levels within the filled slope (see Fig. 4). 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.

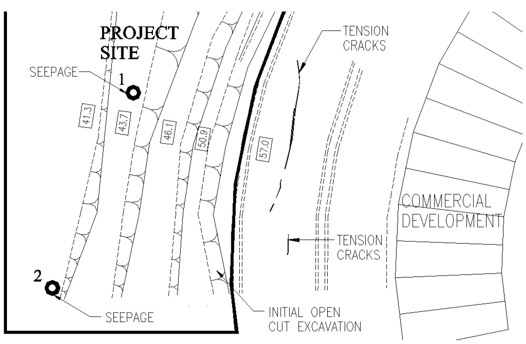

Fig. 3. Tension cracks at earlier stage.

Fig. 4. Seepage (Location 1).

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 Fig. 5. The adjacent commercial development had subsequently levelled 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.

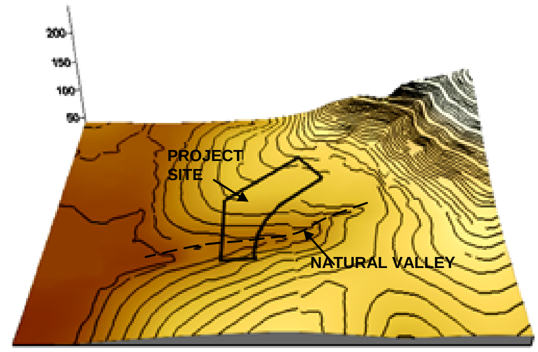

Fig. 5. Three-dimensional original ground contour.

Subsurface investigation (SI) was carried out to establish the subsurface conditions for the geotechnical investigation and remedial design. The SI layout is presented in Fig. 6 together with the instrumentation layout. Figure 7 shows 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.

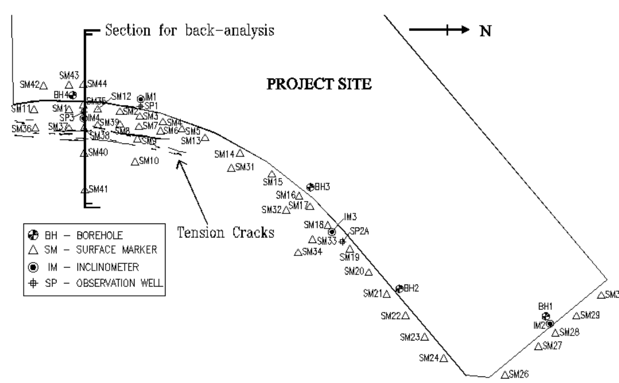

Fig. 6. Subsurface investigation and instrumentation layout.

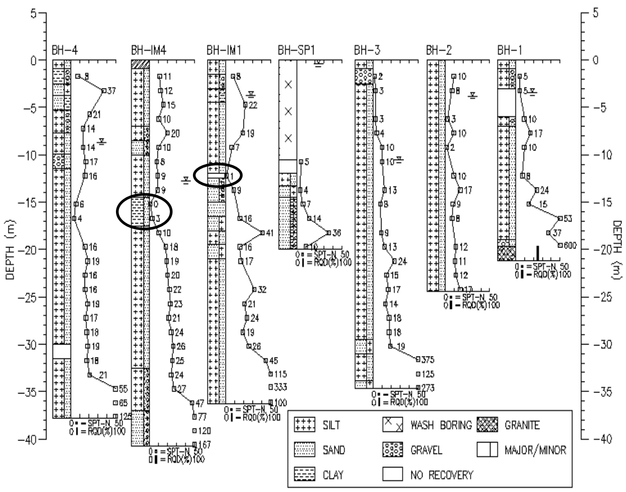

Fig. 7. Borehole logging.

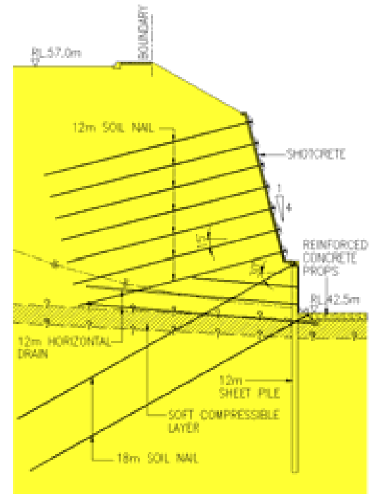

Fig. 8. Cross-section of stabilisation work in the valley area.

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.

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 Fig. 8. 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.

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 9 shows the water continuously discharging from horizontal drain after installation.

Fig. 9. Continuous seepage flow.

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 Fig. 10.

Fig. 10. Soft compressible material detected at localized pile cap excavation.

<u>Instrumentation Monitoring</u>

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 Fig. 6) 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 11 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 Fig. 12. Groundwater levels had generally been monitored from the installed observation wells throughout the construction period. Figure 13 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.

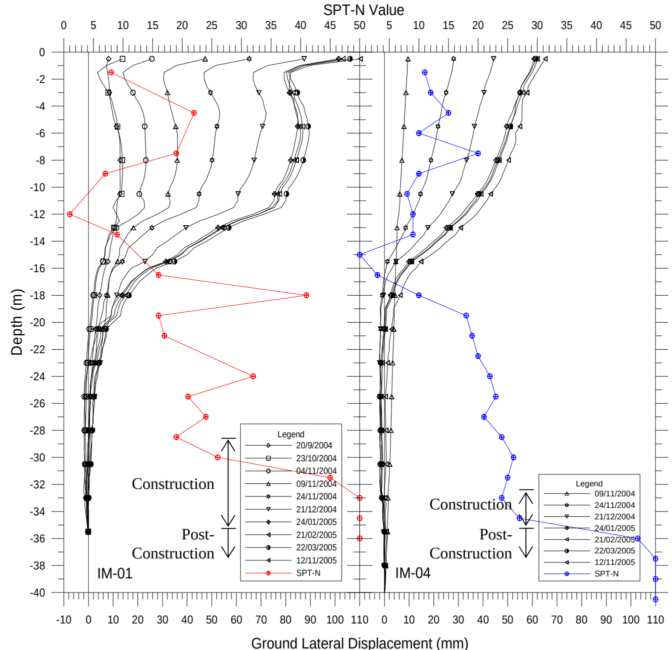

Fig. 11. Measured ground lateral displacements.

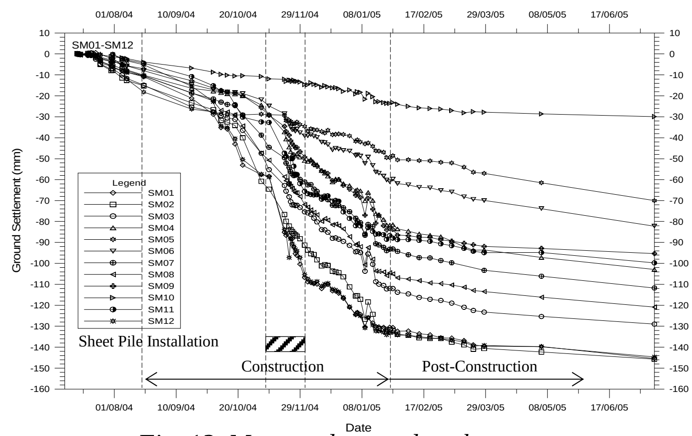

Fig. 12. Measured ground settlements.

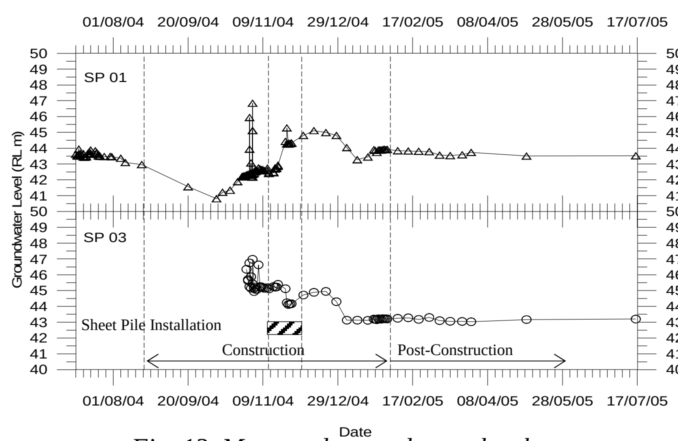

Fig. 13. Measured groundwater levels.

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 Fig. 14). 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. 15. 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.

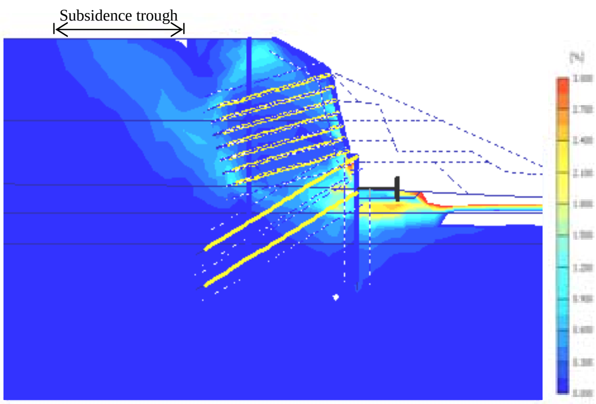

Fig. 14. Shear strain of soil mass within the soil nail reinforcing system.

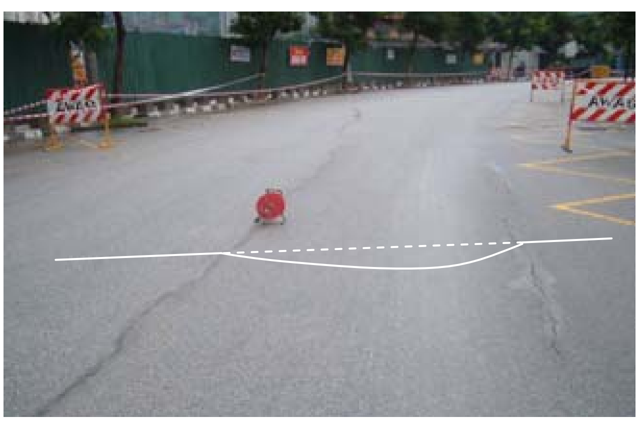

Fig. 15. Subsidence trough developed on the retained ground.

CASE HISTORY B

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 16 shows the location of the project site and the adjacent land lot which had been affected due to ground distresses.

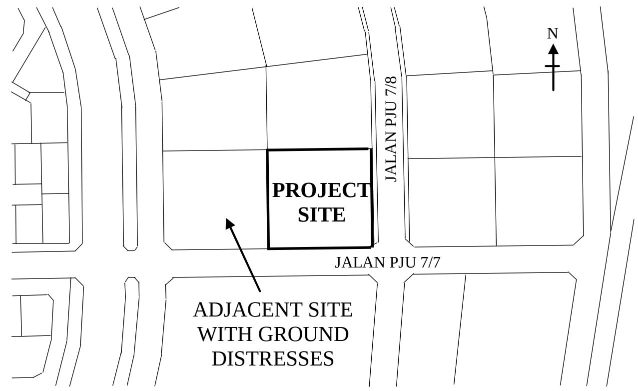

Fig. 16. Site location.

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 17 shows the details and cross-section of the proposed alternative temporary shoring system and permanent retaining wall.

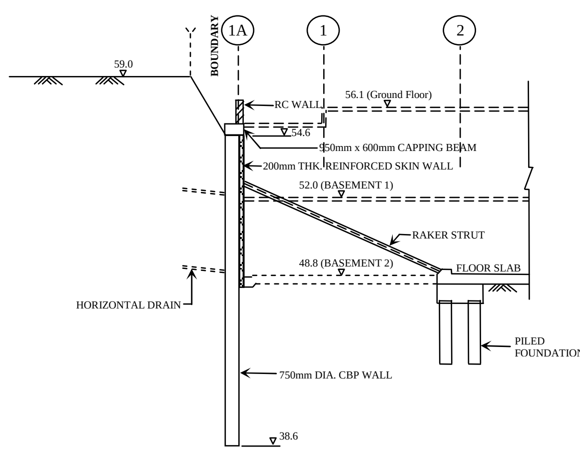

Fig. 17. Cross-section of the proposed alternative retaining walls.

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.

Desk Study

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 Fig. 18.

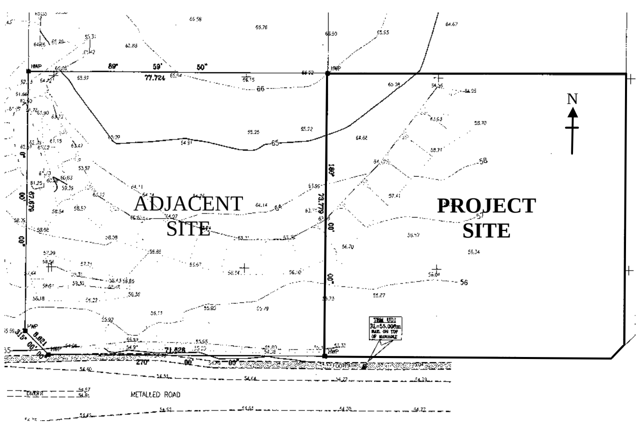

Fig. 18. Topographical survey plan of adjacent lot (subject to disturbance before the finished level).

Based on the original topographical of the pre-development condition as shown in Fig. 19, 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 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.

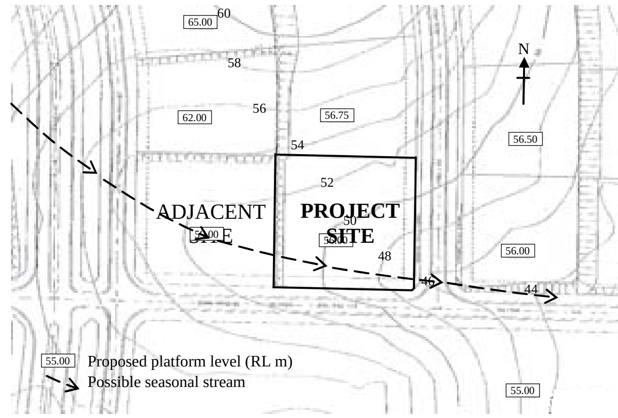

Fig. 19. Regional major earthworks layout with contour lines.

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 Fig. 20. 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.

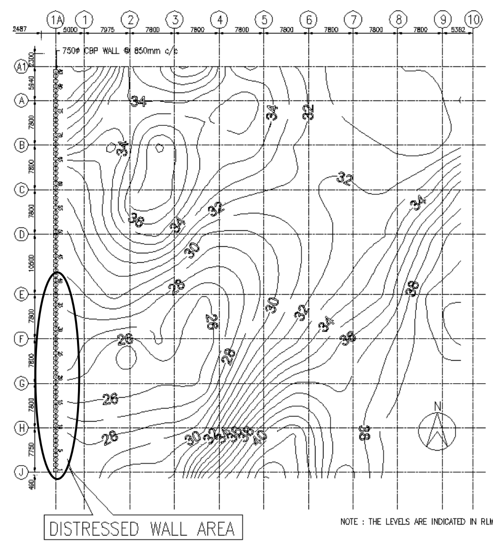

Fig. 20. Interpreted contour of hard stratum from piling information.

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 Fig. 21 and 22.

Fig. 21. Site conditions of adjacent lot after the incidence of wall and ground distresses (Southern view).

Fig. 22. 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 23 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 Fig. 21. 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 Fig. 23.

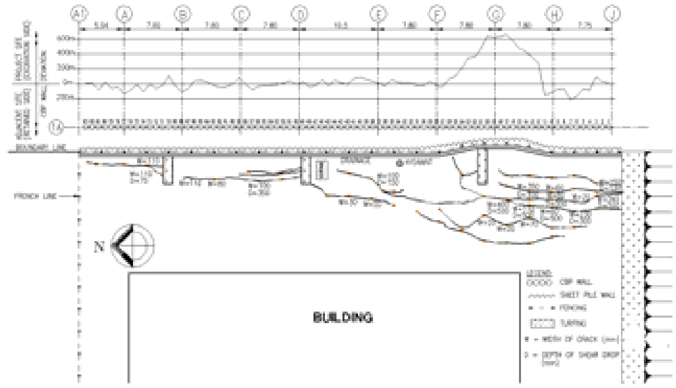

Fig. 23. Crack mapping layout.

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.

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 Fig. 24.

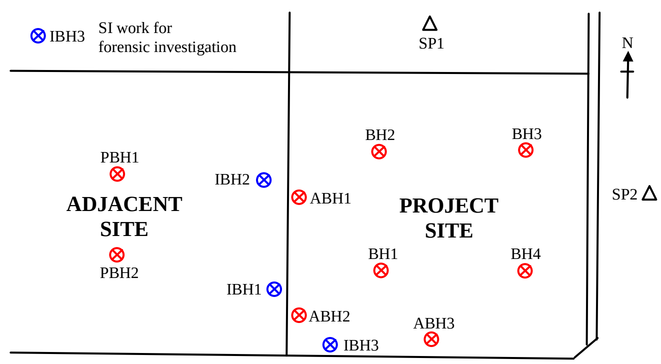

Fig. 24. Subsurface investigation layout.

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 Fig. 25.

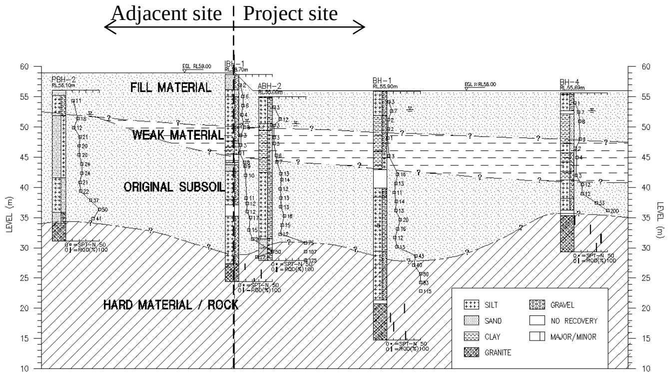

Fig. 25. Typical cross-section of subsurface profiles at the distressed wall area.

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.

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 Fig. 26. 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 Fig. 26) reasonably well agreed with the measured wall movements and ground deformations (e.g. tension cracks, settlement and depression).

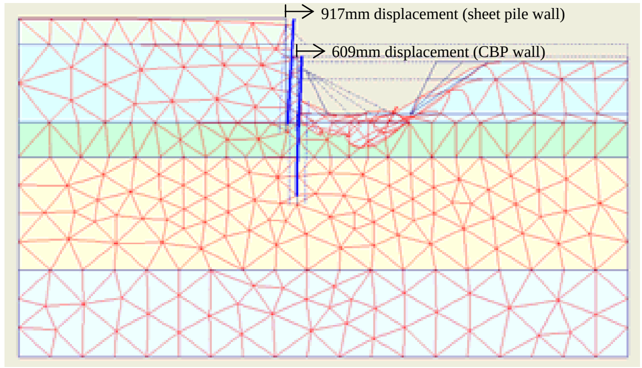

Fig. 26. Results of finite element analysis.

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 Fig. 27. 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 Fig. 28.

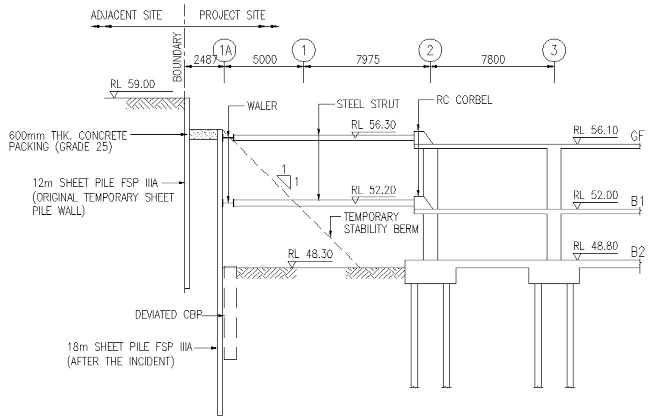

Fig. 27. Cross-section of remedial works.

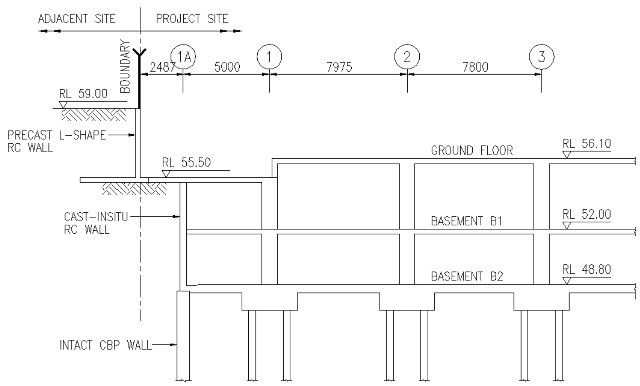

Fig. 28. Cross-section of permanent basement structure.

SUMMARY OF FINDINGS

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

(a) The distressed ground was located over a valley terrain where thicker fill is placed over existing soft deposits without proper engineering treatment to form building platform. The perched groundwater seepage in the original valley was observed and weakens 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 prompt 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 authors' 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 an excavation. It is important to thoroughly investigate the subsoil condition beneath the fill. Otherwise, there would be a remote possibility that a proper treatment to the ground is done before the development earthworks.

(c) Proactive 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.

(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 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 confidence.

(g) In addition, FE analyses have been successfully utilized to investigate and verify the distresses by revealing the associated mechanism.

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