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Design and Construction of Soil Nail Strengthening Work over Uncontrolled Fill for a 14.5m Deep Excavation

Liew, S. S. and Khoo, C. M.
Gue & Partners Sdn Bhd, Malaysia

ABSTRACT

This paper presents the processes of investigation, analysis, design, construction and monitoring of a soil nail strengthening work to facilitate a 14.5m deep excavation at an uncontrolled dump site. Pavement distresses on the retained ground were first observed shortly after initial excavation. Compounding with the extreme weather condition and the uncontrolled dumped materials, the distresses became more apparent. Investigation was carried out to probe the causation of the problem. The investigation concluded that the site was formerly a natural valley with natural stream and without proper land clearing. The development had subsequently leveled the building platform with thick uncontrolled fill, some even over the original soft compressible deposits within the valley. Unfortunately, there was no proper filling and compaction controls over the initial bulk earthworks. The natural drainage, especially the underground seepage, was also not properly addressed. As such, the saturated loose fill had insufficient strength to remain stable after the initial steep excavation. The strengthening strategies were to reinforce the saturated loose fill in-situ with closely spaced soil nails of varying length from 6m to 12m and shotcrete facing, and to freely drain out excessive groundwater from the saturated loose fill with subsoil drains. Generally, the excavation surface was finished with shotcrete facing in 4V:1H gradient. At one location, 12m long sheet pile wall with two rows of 18m long soil nail anchorage was used to enhance the passive resistance of the retained ground, which is seated over the 2m thick original soft deposits almost at the final excavation level. Extensive instrumentation programme had been implemented to monitor the performance of the strengthening solution, validation for the design nail resistance and groundwater variation. From the monitoring results, it was proven that soil nail strengthening technique is suitable for loose saturated fill provided that substantial care is taken during implementation. Construction difficulties, solutions to overcome the problems and lesson learnt will be discussed in the paper.

1. Introduction

Urban development in Kuala Lumpur area of Malaysia has led to space congestion and abrupt rise of land cost in recent years. The use of underground space for car park becomes more common and cost justified. This paper presents a case study involving construction of high-rise mixed development with a five-and-half storey basement car park adjacent to an existing commercial development. The entire excavation face is about 250m long on plan over an uncontrolled dumped backfill primarily made up of loose silty sand and sandy silt to the depths ranging from 7m to maximum of 14.5m. Figures 1 and 2 show the layout plan and a typical cross-section for the development respectively.

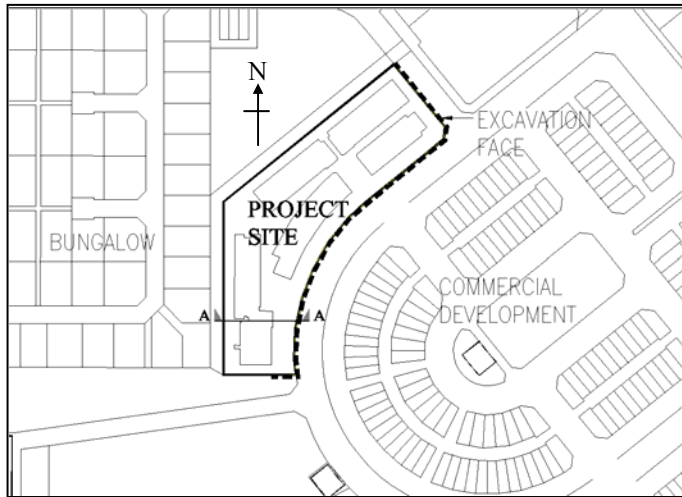


Figure 1: Development layout plan

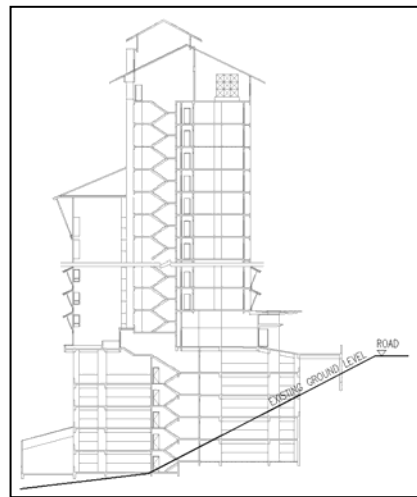


Figure 2: Cross-section A-A

Pavement distresses on the retained ground were first observed during earlier stage of open cut excavation for the basement construction. Compounding with the extreme weather condition of monsoon season and the existence of uncontrolled dumped materials within the excavation, the distresses became more apparent and deteriorating with further excavation. Investigation was carried out to probe the causation of the problem. Soil nail strengthening strategy was then proposed as the remedial solution to replace the initial concept of steep open cut excavation by the design consultant in view of the tight space constraint between the permanent basement structure and the higher level access road on the south-eastern boundary. The proposed remedial solution was successful and had demonstrated good performance throughout the construction stages and post construction. The use of finite element computer programme "Plaxis" to investigate the occurrence of tension cracks on the retained ground and the associated ground movements provides very useful insight of the inherent mechanism within the distressed excavated slope and to explore effective remedial solution.

The processes of investigation, analysis, design, construction and instrumentation monitoring for this well documented and monitored soil nailed wall basement excavation in uncontrolled dumped material are summarized and presented in this paper. Construction difficulties, solutions to overcome the encountered problems and lesson learnt are also discussed here.

2. Site Observations and Investigation

A site inspection was carried out shortly after observing the tension cracks on the road pavement by the project team. The semi-circular pattern cracks were generally observed on the top of the excavated slope as shown in Figure 3. Water seepage at two locations was also observed indicating potential high groundwater level within the filled slope (see Figure 4). At the time of investigation, the excavation at southern portion of the site was in a more advanced stage. It was not surprised that more ground movements were occurring behind the excavation where the tension cracks were first observed.

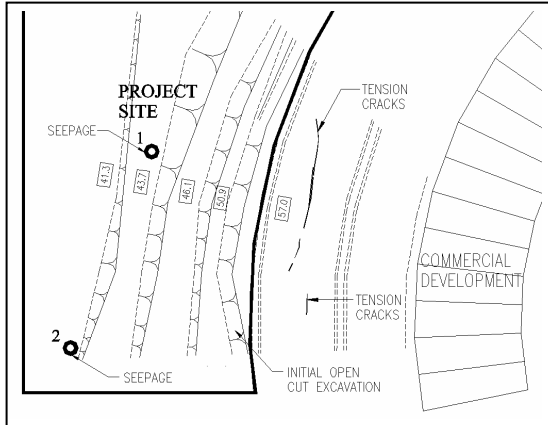


Figure 3: Tension cracks at earlier stage



Figure 4: Seepage (Location 1)

The desk study of the investigation revealed that the distressed area was formerly a natural valley with natural stream flowing from the hilly terrain on the east. Figure 5 shows the three dimensional original ground contour at the development site. The adjacent commercial development had subsequently leveled a wide building platform with uncontrolled fill slopes as thick as 15m, some even overlying the originally deposited soft compressible material within the lower part of the valley. The existence of soft compressible material beneath the filled slope was further confirmed during the additional subsurface investigation when excessive lateral movements were detected in the subsequent staged excavation.

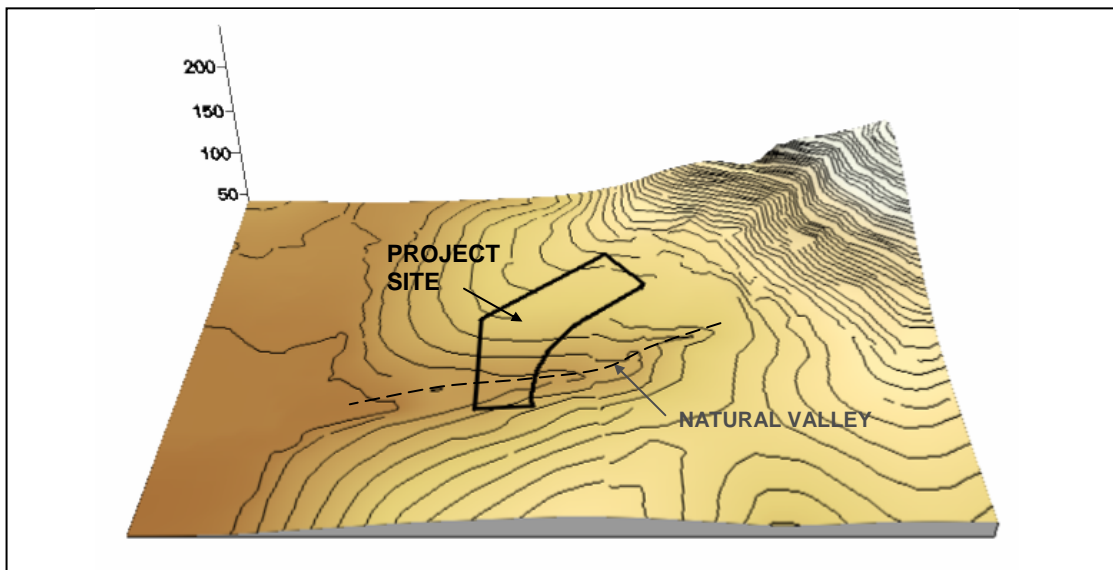


Figure 5: 3-Dimensional original ground contour

This soft compressible material was discovered from the apparent widening of the previous tension cracks despite the soil nail strengthening work had been implemented in the subsequent staged excavation. Additional inclinometer (IM-4) was installed with embedment into hard material for good fixity. The drilling works for inclinometer installation at the access road had revealed the existence of this soft and weak material at the depth of 15m where the fill interfaces with the original ground. The subsequently monitored lateral ground deflection profile confirmed sliding movement of the upper fill over this weak material during excavation.

As such, the underlying soft compressible material coupled with the saturated upper loose fill had insufficient strength to retain the excavated slope in stable state with acceptable safety factor. After the initial steep excavation, the creep movements of the unstable excavated filled slope were believed to have contributed to the widening of these tension cracks. This was evidently demonstrated by the back analysis carried out, in which the distressed slope was of marginal safety factor and the most critical circular slip surface in the back analysis fits very well with the location of actual tension cracks on the retained ground.

3. Subsurface Investigation and Instrumentation Works

Subsurface investigation and instrumentation programme consisting of four boreholes, four inclinometers, three observation wells and forty-four (44) surface markers were planned and implemented to investigate the probable causes, to propose remedial measures of the slope strengthening work and to monitor performance of remedial work during the excavation stage. The layout of the boreholes and instrumentations is shown in Figure 6.

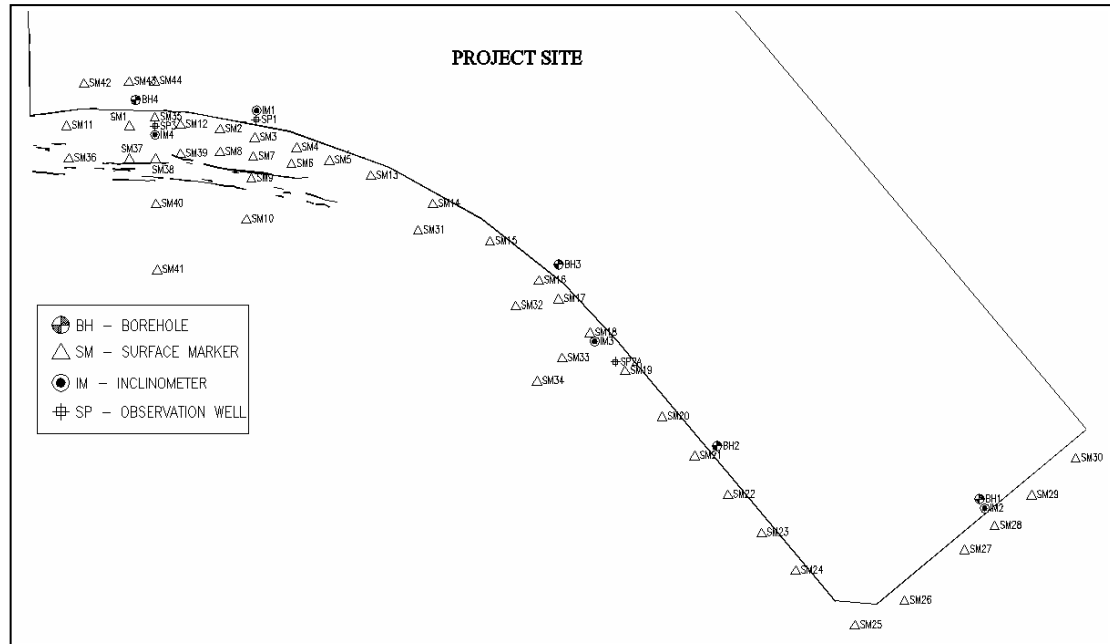


Figure 6: Subsurface investigation and instrumentation layout

Two inclinometers, namely IM-1 and IM-4 were installed in the boreholes sunk within the distressed slope mass upon completion of exploratory boring operation and sample collection. Apart from the inclinometers, observation wells, SP-1 and SP-3 were also installed at 1m away from these inclinometers respectively. Liew & Gue (2001) have presented a similar instrumentation scheme to investigate creep movement of post glacial deposit in East Malaysia, in which the inclinometers have successfully determined the creep movements and slip surface.

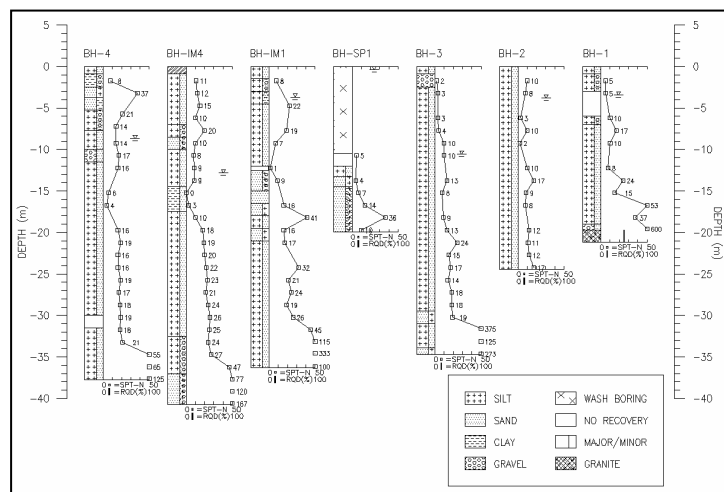


Figure 7: Borehole profiles

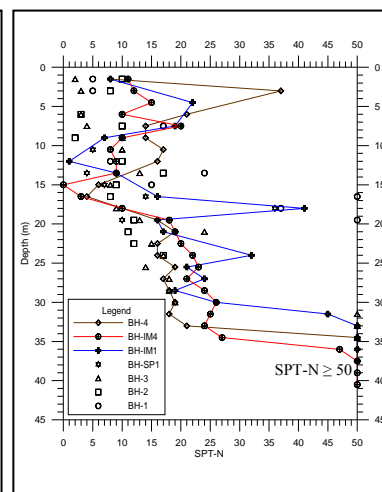


Figure 8: SPT-N plot

Figure 7 shows the interpreted subsoil profiles of the boreholes. The SPT-N values of overburden fill material generally range from 0 to 20 (Figure 8). A layer of soft material of about 2m thick was detected at depth of 12m to 15m below the ground surface in the two

boreholes (BH-IM1 and BH-IM4) within the distressed soil mass, which was overlying the natural valley in the pre-development topographical condition. Generally, the fill material and the underlying subsoil have similar soil consistency with plastic limit (PL) of 30 and liquid limit (LL) from 40 to 60. However, the soft material is of intermediate to high plasticity clayey material with PL of 15 and LL from 40 to 60. The effective shear strength parameters of $c' = 5\text{kPa}$ and $\phi' = 32^\circ$ and undrained shear strength (c_u) of 40kPa were interpreted for the fill material and soft material respectively for geotechnical design of remedial work. Slightly higher undrained shear strength than remolded strength of penetrating vane shear tests was finally adopted for the soft material after noticeable creep movement in this material has been recorded in the inclinometers and substantiated by the back analyses. Peak strength and the remolded strength of penetrating vane shear tests are 65kPa to 80kPa and 36kPa respectively.

As mentioned above, the inclinometers installed within the distressed slope had recorded significant sliding movement at the bottom of fill shortly after installation. The shear-off points of the two inclinometers (IM-1 and IM-4) resemble a well-defined circular slip surface when joining together with the tension cracks, which indicate where the slip surface initiates on the slope profile. Subsequent monitoring discovered that the consistent rate of lateral ground movement of inclinometers had confirmed that the slope was moving towards the excavation side. The monitoring results will be discussed in details in Section 7.

Representative groundwater tables were established from the three observation wells. The valley topographical features suggest that collection and concentration of underground seepage may have occurred within the valley. This is particularly evident in the soggy and saturation conditions of the excavated materials immediately above the valley.

4. Analysis and Design

For remedial and strengthening works, the primary objective is to improve the safety factor of slope to an acceptable design requirement. Soil nailing was adopted to stabilize and control the ground lateral displacements. Soil nailing is in fact a reinforced earth technique using steel reinforcements either driven/jacked into the ground or inserted into a drilled hole with non-shrink cementitious grout in the excavated soil face, to improve the overall strength of soil-reinforcement slope mass. The exposed excavated surface is normally protected by a layer of reinforced shotcrete. Liew (2005) presented a brief overview of the soil nail design philosophy and methodology. For jacked-in anchorage system, Liew et al. (2000 & 2003) had presented two case studies of basement excavation.

The proposed strengthening strategy is to reinforce the in-situ saturated loose fill with closely spaced soil nails of varying lengths from 6m to 12m with structural shotcrete facing and to drain out excessive groundwater freely from the saturated loose fill with subsoil drains. Nail spacing are limited from 1.25m to 1.5m center to center spacing horizontally and vertically. Generally, the excavation surface was finished with shotcrete facing in 4V:1H gradient. At the valley area where excessive creep movement was observed, 12m long sheet pile wall with two rows of 18m long soil nail anchorage and reinforced concrete props against the basement structure were used to enhance the passive resistance of the retained ground, which is seated over the 2m thick original soft deposits almost at the final excavation level (Figure 9).

Due to urgency of the remedial design, back analyses based on the profile of the excavated soil mass at the time of distress were carried out to obtain the necessary soil strength parameters. The shear strength parameters were subsequently verified with the laboratory tests, such as consolidated isotropically undrained (C.I.U.) triaxial tests and multiple reversals direct shear box tests. The soil nail strengthening design was carried out generally in accordance to the design standard by Federal Highway Administration (1998). Slope stability analyses using both classical limit equilibrium method and strength reduction method in finite element (FE) analysis were used to assess the original safety factor and the improvement after strengthening. The FE analyses were performed using "Plaxis" (Brinkgreve, 2002), a finite element method computer program under plain strain condition (Figure 10).

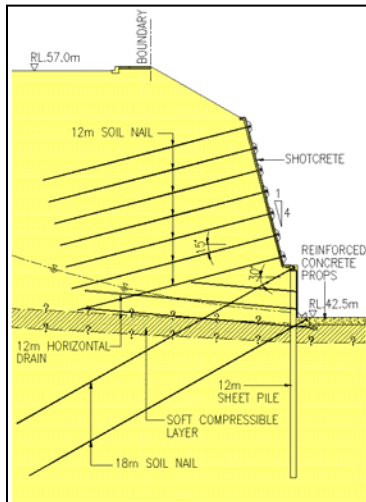


Figure 9: Cross-section of strengthening work at valley area

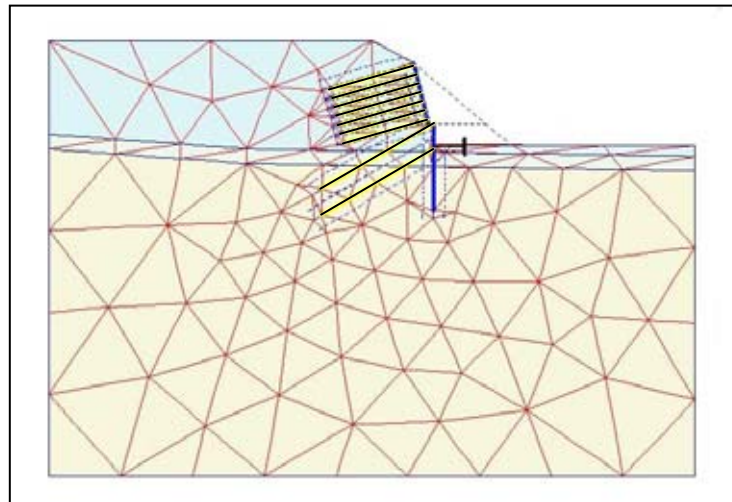


Figure 10: Plane strain FE analysis (Plaxis V8.2)

For effective control of groundwater, horizontal subsoil drains were proposed at certain horizontal and vertical spacings along the slope toe to proactively lower down the groundwater profile and depressurize pore pressure within the reinforced slope mass. Sufficient weephole drains were also provided at grid spacings on the shotcrete facing to prevent build up of water pressure immediately behind the shotcrete facing.

5. Construction

The construction of soil nailed wall was carried out with top down sequence by progressive nailing and staged cutting of slope (not more than 2m in every stage). Sectional excavation was also implemented at areas encountering very loose materials with short self support time in order to minimize the excessive horizontal stress relief before nailing and to prevent collapse.

Drilled hole was formed by rotary air-flushed using portable spider drilling rig. The drilled hole was constructed at inclination of 15° downward from horizontal to contain the grout for the designed 125mm diameter nail. Grouting by normal tremie method from bottom-up was carried out continuously without interruption to ensure good grout-soil bonding and to reduce air entrapment. For the installation of 18m long soil nail at the sheet pile wall, which were below the groundwater table, rotary water wash boring method using micropiling machine was employed to reduce chances of drilled hole collapse.

During drilling the lower rows of nail, unset grout and mud slime were observed flowing out from the freshly grouted nails adjacent to the drilling indicating high groundwater table and high connectivity of the poorly compacted fill (see Figure 11). In connection with this, site instruction was given to drill the holes in alternate sequence with reduced air pressure. The bond resistance of these doubtful nails was further verified by pull-out tests and re-analysis was conducted to re-assess the factor of safety of the overall stability.

Apart from the nominal provision of subsoil drainage, additional subsoil drains were provided at the identified seepage spots on the exposed excavation surface. In one location, groundwater was observed continuously flowing out immediately after excavation. This particular location was believed to be the natural water path of natural stream as revealed in the original ground contour. Sufficient horizontal drains were installed at this location to discharge the groundwater. Figure 12 shows the water continuously discharged from horizontal drains after installation.



Figure 11: Unset grout and mud slime flowing out during drilling at adjacent nails



Figure 12: Continuously seepage flow

When excavation nearly reached the final platform 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 slope stability analyses. The localized excavation for pile cap construction further confirmed the existence of this soft deposited material as shown in Figure 13.



Figure 13: Soft compressible layer detected at localized pile cap excavation

6. Results of Pull-Out Test

A total of 16 pull-out tests including one preliminary pull-out test had been conducted for varying nail lengths and at different locations where different soil conditions were encountered. In addition, strain gauges have been installed in the two representative soil nails at critical areas to monitor the load transfer behavior during the pull-out test and for long term monitoring of nail load development. The strain gauges were installed in pairs to eliminate flexural effect in the interpretation.

Table 1: Results of pull-out tests

No.	Nail Ref.	Nail Length (m)	Creep Movement during Creep Test at 1.5xDTL (mm)	Acceptance
1	Z2/R5/91A	6	0.215	Accepted
2	Z2/R4/84A	6	0.320	Accepted
3	Z2/R8/21B	9	1.210	Not accepted
4	Z2/R2/14C	12	1.785	Not accepted
5	Z2/R5/6C	12	0.240	Accepted
6	Z2/R5/24C	12	0.150	Accepted
7	Z1/R3/35C	12	19.605	Not accepted*
8	Z1/R3/39C	12	18.660	Not accepted*
9	Z1/R4/15C	12	0.950	Accepted
10	Z1/R4/38C	12	0.280	Accepted
11	Z1/R1A/10D	18	0.785	Accepted
12	Z1/R1A/19D	18	0.450	Accepted
13	Z1/R1A/44D	18	0.515	Accepted
14	Z1/R2A/4D	18	4.795	Not accepted
15	Z1/R2A/29D	18	5.125	Not accepted*
16	Z1/R2A/31D	18	0.350	Accepted

Notes: 1. DTL = Design test load
 2. All the unacceptable nails had been downgraded for its working capacity.
 3. * denotes the nail location identified as source of groundwater seepage.

Generally, most of the pull-out test indicated that ultimate shaft resistance can be achieved without much trouble. All the proof pull-out tests had been subjected to creep test with criterion on total creep movement less than 1mm measured between 1 minute and 10 minutes holding time. Table 1 shows the nail head displacement (total creep movement) measured during the creep test. Six pull-out tests have showed unfavorable results with excessive creep movement of the nail during the creep test. In particular, three tests were noted at the localized weak zone where the source of groundwater seepage. Subsequently, the ultimate bond resistance for those specific rows of nail was reviewed and downgraded accordingly.

7. Results of Instrumentation Monitoring

The instrumentation program was set up mainly as an alert system for safety control and to monitor the interaction of the proposed strengthening system to the surroundings. This instrumentation scheme had provided sufficient coverage to monitor the performance of the excavation. Inclinator, ground settlement and groundwater readings were taken weekly. However when critical stages were involved, the frequency of the readings was increased accordingly.

7.1 Ground Lateral Displacement

The lateral movements at the soil nailed walls had been monitored using the inclinometers. Figure 14 shows the monitored soil nailed wall movements during various stages of excavation.

Basically, larger movement was observed during the initial stage of soil nailing for the upper rows as shown in Figure 14. This is likely due to the steep sub-vertical cut compounding with the extreme weather condition. However, the slope movement at the valley area was noted with stabilizing trend upon installation of sheet pile wall anchored with two rows of 18m long soil nails to improve the passive resistance at the toe of the excavated slope.

At the most critical location where the valley is located, the soil nailed wall has moved about 90mm laterally at the final excavation, which is about 0.6% of the depth of excavation and is slightly higher than the typical ratio of maximum wall deflection to excavation depth of the range from 0.2% to 0.5% presented by Ou et al. (1993) for numbers of deep excavation in Taipei basin. When the lateral movement reaches 0.5% of the excavation depth, distress would be expected on the retained side.

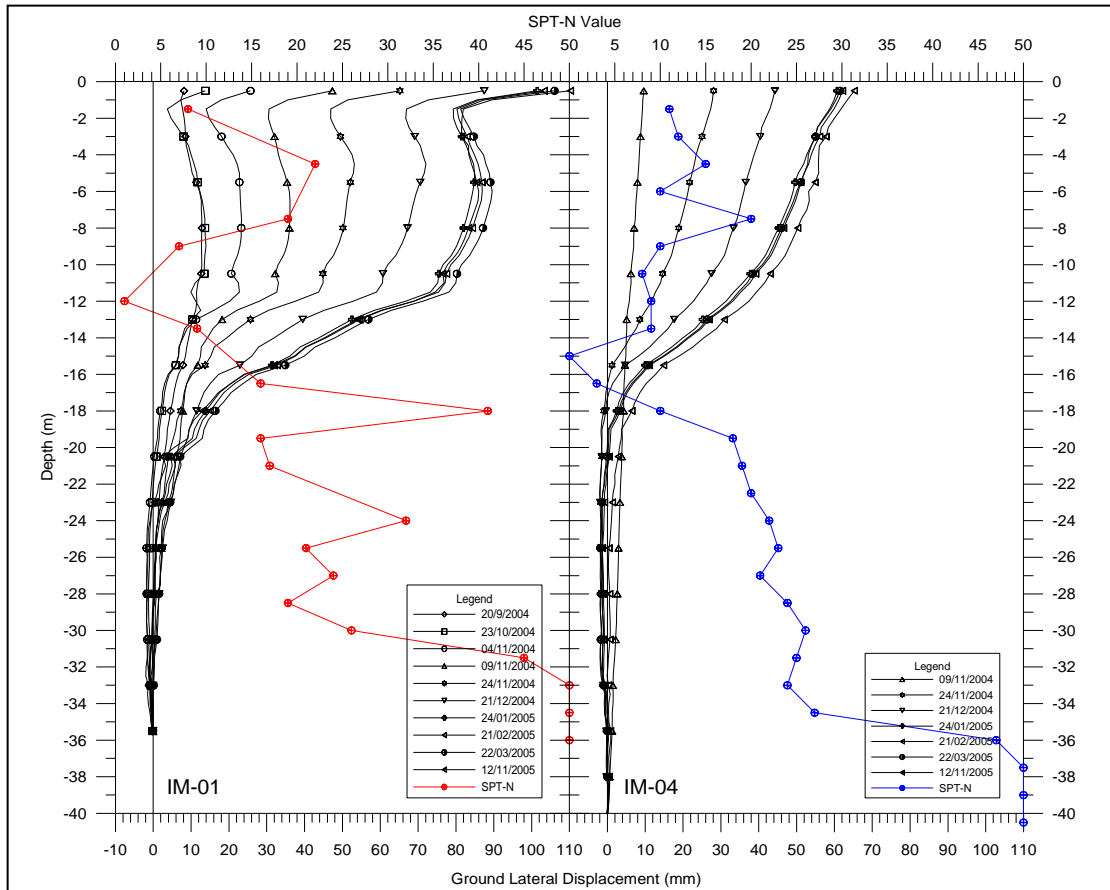


Figure 14: Measured ground lateral displacement

7.2 Ground Settlement

The cumulative ground settlements behind the soil nailed wall at various construction stages are shown in Figure 15. Similarly, larger rate of ground settlement had been observed during the excavation. The ground settlement had stabilized after the completion of slope strengthening works.

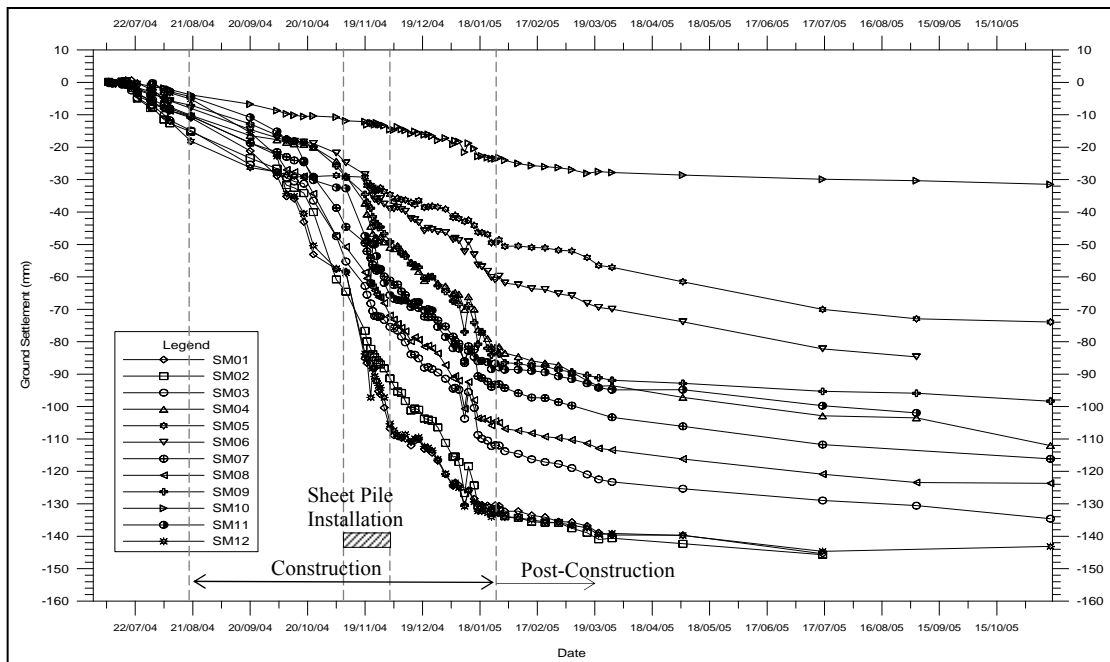


Figure 15: Measured ground settlement

7.3 Groundwater Monitoring

Groundwater had generally been monitored from the installed observation wells throughout the construction period. Figure 16 shows the measured groundwater levels with time.

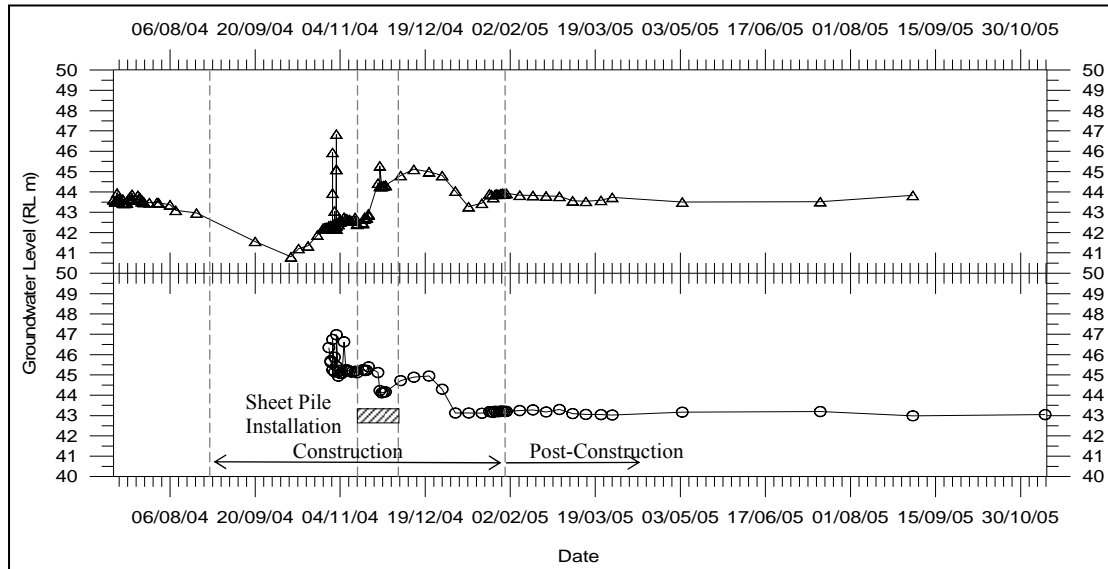


Figure 16: Measured groundwater level

In the initial stage of excavation, the groundwater on the retained side was measured at about RL43.5m at the original ground level. Subsequently, the lowering of groundwater was noted during the localized excavation for pile cap construction at some distance away from the slope toe where the sheet pile wall was not installed cutting off the underground seepage. However, groundwater was gradually raised up to the top of sheet pile wall (RL 45.5m) after the sheet pile wall had been installed. The installation of this sheet pile had cut off the path of underground seepage and caused building up of pore pressure within the soil mass as indicated in Figure 16.

Interestingly, the groundwater was fluctuated as high as 3m during the occasional heavy down pour as measured by SP-01 and SP-03. Once the sheet pile wall was installed section by section, three rows of horizontal subsoil drains at levels of RL 41.975m, RL 43.0m and RL 44.0m were also installed immediately to relieve the build up groundwater behind the sheet pile wall. It was noted that the groundwater had successfully been discharged and lowered to RL 43.5m at the observation wells SP-01 and SP-03 reaching the steady state equilibrium after reaching the final excavation.

8. Conclusions and Recommendations

Based on the discussions in the above sections, soil nailing strengthening technique has proven with successful implementation of 14.5m deep excavation in loose fill slope. Hopefully, by presenting a real example in this paper, the myth and uncertainties of using soil nails in loose fill can be reduced with enhanced confidence. Nevertheless, the following conclusions and recommendations, in the authors' opinion, can be the useful points for future projects of similar nature:

- i. For soil nailing system, particularly in fill materials, larger lateral ground movements as high as 0.6% of excavation depth would be required to mobilize the nail tensile resistance for a stable equilibrium. In such ground lateral movements, the earlier installed nails would be expected to have higher degree of mobilization in nail load.
- ii. Filling over valley without proper site clearing and compaction could result in highly unstable backfill. Soft deposits at the lower part of the valley and potential concentrated underground seepage are common and shall not be overlooked. Desk

- study of pre-development ground contour to identify potential geotechnical problems is highly recommended.
- iii. Back analyses using ground geometry at the onset of the distresses could provide very good operative strength parameters for remedial design. Both 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 is necessary.
 - iv. When creep movements occur and result in significant shearing of material, the operative shear strength parameters are usually very close to remolded strength.
 - v. Drilling with reduced air pressure and alternate locations in loose fill material are strongly recommended to avoid disturbance to the previously completed nails.
 - vi. Proactive subsoil drainage scheme for strengthening work of loose fill can be very effective in improving stability.
 - vii. Comprehensive instrumentation scheme at strategic locations can reveal potential failure mechanism for deriving appropriate remedial solution and modifying the design to suit actual ground condition. Generally, these instruments include inclinometer, surface marker, observation well, etc. and are highly recommended.

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