Performance of Soil Nail Stabilisation Works for a 14.5m Deep Excavation at Uncontrolled Fill Ground

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This paper presents the performance of a soil nail stabilisation technique for a successful basement construction using open excavation to a depth of 14.5m in an uncontrolled fill ground. An extensive instrumentation programme had been planned and implemented to monitor the performance of this stabilisation solution and to provide validation for the designed nail pull-out resistance and groundwater variation. The finite element computer program "Plaxis" was deployed to provide very useful insights into the inherent mechanism within the excavated slope and subsequently verify the occurrence of tension cracks and the associated ground movements as observed. The shear strain distribution within the finite element mesh indicates a zone of high shear strain immediately behind the reinforced soil mass. The 2D modelling and instrumentation scheme have yielded some useful information revealing the actual behaviour of the reinforced soil mass in terms of slope mass movements and ground settlements during excavation in front of the slope.

1 INTRODUCTION

Soil nailing is a soil reinforcing technique that works by either driving/jacking the steel reinforcements into the ground or inserting reinforcements into a drilled hole bonded 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 gunite facing. Of late, it has been extensively used as an economical, effective and simple means of stabilising slopes and retaining excavation.

Perhaps the most popular soil nailing application in engineering practice is for stabilisation of natural and/or cut slopes. However, in some cases of deep excavation, the conventional retaining system may become expensive in fill ground which has led to the search for alternative methods of enhancing the stability of excavated filled slopes using soil nails. In this context, HKIE (2003) had prepared a comprehensive report on soil nails in loose fill slopes.

This paper presents the performance of instrumented soil nail stabilisation works for a 14.5m deep excavation in uncontrolled fill ground in Malaysia. Finite element analyses have been carried out to reveal very useful insights of the mechanisms within this instrumented nailed excavation and subsequently verify the occurrence of tension cracks and the associated ground movements as observed. The behaviour of the reinforced soil mass in terms of slope mass movements, ground settlements and soil shear strain development during excavation in front of the slope are studied.

2 CASE HISTORY

2.1 *Site Location and Ground Condition*

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 shows the location of the site, which was at the toe of a filled slope on the north-west of the commercial development. A typical cross-section of the development is shown in Fig. 2. The entire excavation is about 250m long over an uncontrolled fill to the depths ranging from 7m to maximum of 14.5m. Liew & Khoo (2006) presented the processes of investigation, analysis, design, construction and monitoring of soil nailing stabilisation work to facilitate this deep excavation.

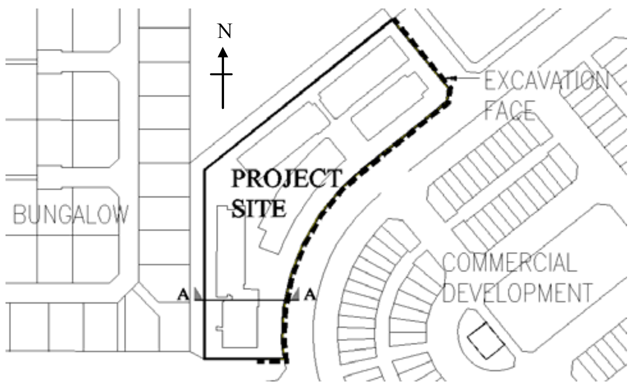

Fig. 1. Development layout.

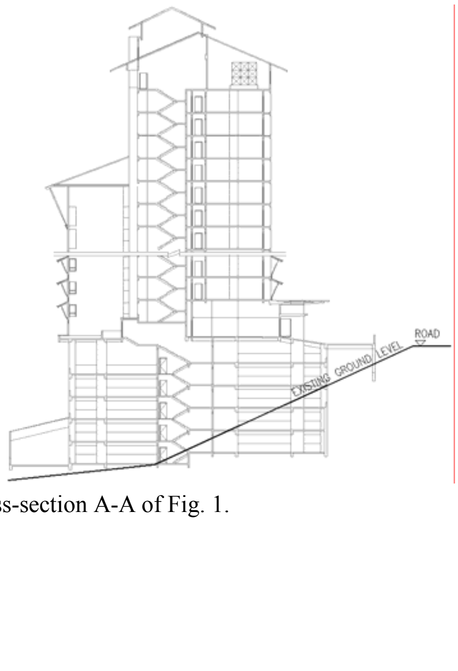

Fig. 2. Cross-section A-A of Fig. 1.

Information obtained from the detailed site investigation revealed that the slope above the basement 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 discovered originating from the eastern hilly terrain as illustrated in Fig. 3. 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.

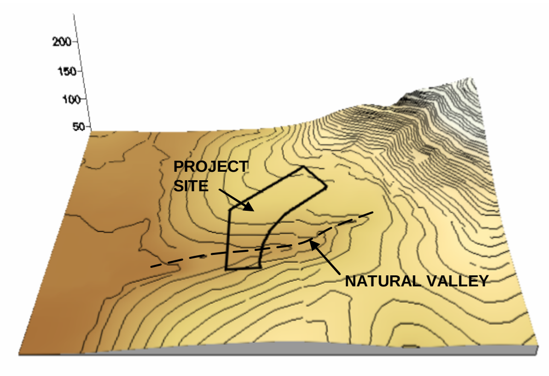

Fig. 3. Three-dimensional original ground contour.

2.2 *Subsurface Investigation*

Subsurface investigation (SI) was carried out to establish the subsurface conditions for the geotechnical investigation and design of slope stabilisation work. The SI layout is presented in Fig. 4 together with the instrumentation layout. Fig. 5 shows the interpreted boreholes logging. The fill 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) near the previous valley.

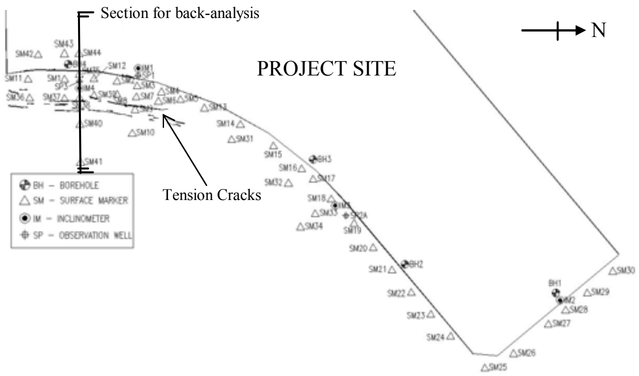

Fig. 4. Subsurface investigation and instrumentation layout.

Generally, the fill material and the underlying subsoil have similar soil consistency with a plastic limit (PL) of 30 and a liquid limit (LL) from 40 to 60. However, the soft material is of intermediate to high plasticity clayey material with a PL of 15 and a LL from 40 to 60. Laboratory triaxial tests showed the effective shear strength parameters of c' = 5kPa and $\phi' = 32^\circ$ for the fill material. Peak strength and the remolded strength for the soft material obtained from penetrating vane shear tests are 65kPa to 80kPa and 36kPa respectively. Nevertheless, a slightly higher undrained shear strength (s_u) of 40kPa than the vane shear remolded strength was finally adopted for the soft material for the design of stabilisation work, after noticeable creep movement in this material had been recorded in the inclinometers and the substantiation by the back analyses as presented by Liew & Khoo (2006).

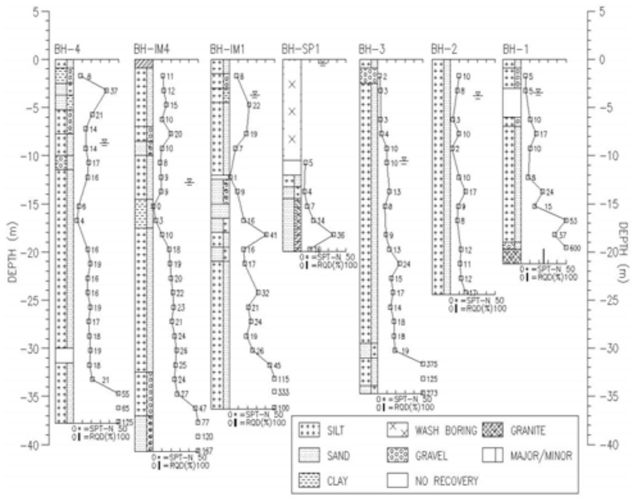

Fig. 5. Borehole logging.

Groundwater levels were fluctuating and exhibiting seasonal storm responses throughout the construction period. The topographical features of 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 the excavation. The continuous monitoring of groundwater had been carried out until achieving steady state equilibrium throughout the monitoring period at post construction.

2.3 *Soil Nailing Design and Construction*

Soil nails with gunite surface was proposed to provide overall stabilisation and lateral support to the excavation of 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.

The soil nail design was carried out generally in accordance with the design standard of the Federal Highway Administration (1998). However, it should be noted that for the above soil nailing configuration, the total number and layout of the nails were obtained from the classical limit equilibrium stability analysis using Spencer's method. The minimum factor of safety obtained for this nail configuration should be 1.4 due to high risk of loss of life and economic loss. In addition, each nail has been designed by considering the various modes of internal stability, such as bond failure between the grout and soil, bond failure between the grout and steel bar, pullout failure between grout and soil, and structural tensile failure of steel reinforcement.

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 enhance the passive resistance of the retained ground in addition to the soil nailed slope on top. The cross-section of stabilisation work at the valley area is shown in Fig. 6, in which strength reduction method in finite element (FE) analysis was used to assess the original safety factor and the improvement after stabilisation for this critical section.

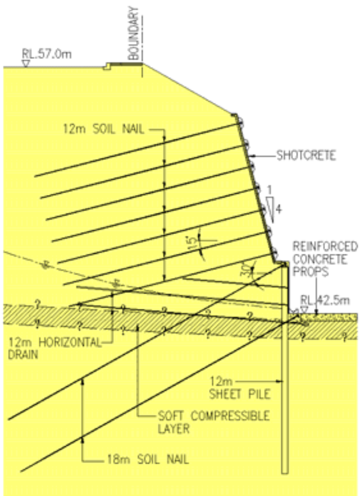

Fig. 6. Cross-section of stabilisation work in the valley area.

The construction of the nailed slope was carried out in a top down sequence by progressive nailing and staged cutting of the slope (not more than 2m in every stage). The nails, which were made of 32mm diameter high yield steel reinforcements and hot-dip galvanized for corrosion protection, were then grouted over the full length in a 125mm diameter pre-drilled hole formed by rotary air-flush using a portable spider drilling rig at an inclination angle of 15˚ downward. Grouting by the normal tremie method from bottom-up was carried out continuously without interruption to ensure good grout-soil bonding and to reduce air entrapment. A 125mm thick gunite facing, reinforced with 2 layers of steel welded wire mesh A8, was finally provided to protect the excavated face. These procedures were repeated for every stage of cutting until the final excavation level.

2.4 *Instrumentation Monitoring*

An extensive instrumentation programme consisting of four inclinometers, three observation wells, forty-four (44) surface displacement markers and strain gauges was planned and implemented to investigate the probable causes of the slope distresses, to monitor the performance of the stabilisation work and to provide validation for the designed nail resistance and potential groundwater variation. The instrumentation layout is shown in Fig. 4.

Two inclinometers, namely IM-1 and IM-4 were installed in the boreholes sunk within the distressed slope mass for monitoring lateral displacements during and after construction. The inclinometer casing was located about 2.0m from the boundary of the road platform (Fig. 6). Apart from these inclinometers, observation wells SP-1 and SP-3 were also installed 1m away from these inclinometers respectively for monitoring of groundwater tables. In addition, two representative nails selected at the critical section were instrumented to monitor the load transfer behaviour during the pull-out test and for long term monitoring of nail load development. Monitoring readings were taken at weekly intervals during construction. However when critical stages were involved, the frequency of the readings was increased accordingly.

From the monitoring results, the ground lateral displacement and settlement had stabilised with no appreciable further deflections were measured after the completion of slope stabilisation works. Most of the detected movements occurred during soil nail installation and excavation. After completion of the stabilisation works, the stabilised slopes had gone through a few monsoon seasons with no noticeable movements despite the groundwater fluctuating with rainfall events.

The instrumentation monitoring data also provided valuable information on ground movements and settlements, which was used to verify the predicted and later back-analysed by numerical modeling.

3 FINITE ELEMENT ANALYSIS

Numerical modeling using the computer program "Plaxis" was used to simulate the excavation sequence and installation of nails. The finite element analyses were aimed at gaining insight into the inherent mechanisms within the excavated slope and subsequently verify the behaviour of the reinforced soil mass in terms of slope movements and ground settlements.

3.1 *2D Modelling*

The finite element mesh, which consisted of about 900 nos. of 6-node elements as shown in Fig. 7, was adopted for the 2D analysis under plain strain conditions (Brinkgreve, 2002). The model boundary conditions were fixed by standard fixities, where side vertical boundaries were fixed in horizontal x-direction but free to move vertically, while the bottom boundary was restrained from any movement in all directions. The initial stresses of the model were calculated by gravity loading to reach its equilibrium.

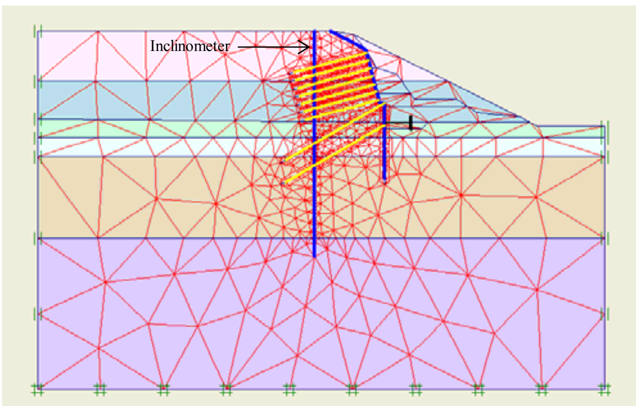

Fig. 7. Two-dimensional finite element mesh.

To achieve a good comparison with the field measurement, the slope geometry, excavation and construction sequences have been simulated as close as possible to the actual conditions including the removal of elements to match the actual stages of excavation. Both the excavation and installation of nails and sheet pile wall were divided into six stages after initializing the initial ground stresses. At each stage, 2m thickness of soil was first excavated and followed by nail installation. A 125mm thick gunite was applied to the excavated face by simply activating the beam element as the gunite properties. The process was repeated until the final row of 12m long soil nails was installed and then followed by a sheet pile wall with the first row of 18m long nails installed. When the final excavation level was reached, a second row of 18m long nails was installed simultaneously. All the analyses were performed until the out-of-balance force was less than 10^{-5} kN.

As the nail was slender steel reinforcement and offered only little bending resistance (Liew, 2005), it was modeled using the one-dimensional geotextile element that was only capable of sustaining uniaxial tension (Brinkgreve, 2002).

3.2 *Constitutive Model and Soil Parameters*

As excavation took place in the loose fill of granitic residual soil, the linear elasto-plastic material with the hardening soil model was selected as the soil model. The associated soil parameters were obtained from a series of laboratory tests and correlations, which are summarized in Table 1. For the value of Young's modulus, design experience in Malaysia revealed that it can be estimated using a correlation with blow count from the standard penetration test (SPT). Thus, the effective Young's modulus (E') was initially assumed as follows:

$$E' = 2500 \times SPT'N' \ (kN/m^2) \qquad (1)$$

4.1 *Lateral Ground Displacement*

Fig. 8 shows the comparison of the lateral slope deflection results obtained from the field measurement and the FE back-analysis results at the critical and final excavation stages. In general, reasonably close agreement in the lateral movement profile of the nailed slope has been achieved in most back-analyses except there is a slight under-estimation of nailed slope displacement at the earlier stage of progressive nailing before installation of the sheet pile wall. This could be due to the load relaxation and creeping of some soil nails with time, particularly for those which have been installed at top levels of loose fill. In addition to this, this was also probably due to drilling disturbances requiring more nail displacement to mobilize stabilizing load compared to the ideal situation in numerical analysis. The relatively large top displacement near the crest of the slope as observed in the field was also seen in back-analyses. This observation could be reasonably explained as the top sloping platform were un-nailed and without gunite in the early stage of construction. The same phenomenon has also been observed in the analysis of nailed slope excavation in Hong Kong (Gui & Ng, 2006). In short, the FE back-analysis results consistently match well with the field measured lateral slope displacement at the stage of post-installation of sheet piles with first row of 18m soil nail and final excavation stage.

Table 1. Engineering properties of the soils.

Layer	Material	Average SPT'N'	γ_b (kN/m^3)	s_u (kN/m^2)	c' (kN/m^2)	ϕ' (°)	E' (kN/m^2)	E'$_{ur}$ (kN/m^2)	Back Analysed E' (kN/m^2)
RL57m – RL49m	Sandy Silt (Fill)	12	18.5	-	5	32	30,000	90,000	18,000
RL49m – RL43m	Sandy Silt (Fill)	9	18.5	-	5	32	22,500	67,500	16,200
RL43m – RL40m	Sandy Clay (Weak Zone)	2	18	40	0.5 (5)#	20	32,500	97,500	32,500
RL40m – RL37m	Sandy Silt	10	18.5	-	5 (10)#	32	25,000	75,000	25,000
RL37m – RL21m	Sandy Silt	20	18.5	-	5 (10)#	32	50,000	150,000	50,000
Below RL21m	Gravelly Sand	50	19.5	-	7	32	125,000	375,000	125,000

Note: ( )# Improved apparent cohesion adopted in FE back analysis at the last few stages.

In the back-analysis, the soil stiffness which has considerable influence on the slope movement has been modified in order to reasonably match the measured ground deformations. The soil strength and the in-situ stress conditions that can also influence the deformation profiles of the nailed slope were generally kept unchanged since the authors considered these parameters and values as more intrinsic in nature as compared to stiffness. However, the strength of the soil layer in front of the sheet pile wall below the building platform level has been improved by a slight increase of the cohesion at the last few stages to account for the densification effect from the installation of large displacement piles. Since the excavation programme was not of a short duration, coupled consolidation undrained analysis was used in the back-analyses.

4 ANALYSIS RESULTS AND DISCUSSIONS

The back-analysis results presented in this paper includes relative lateral displacement of the soil nailed slope and settlement of the retained ground behind the nailed slope due to excavation.

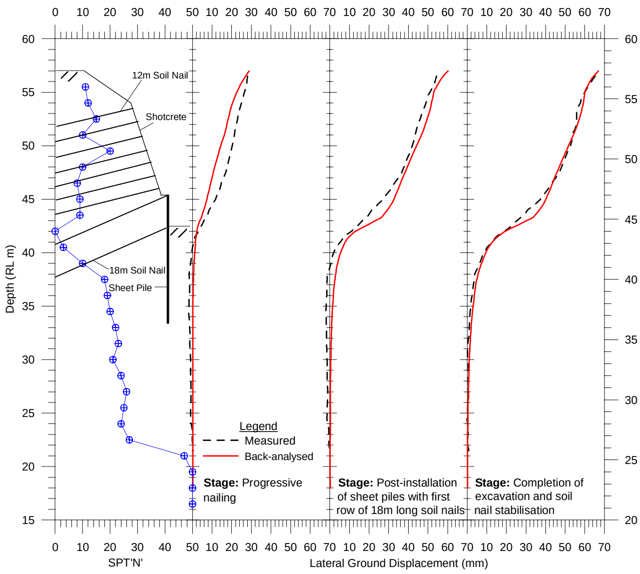

Fig. 8. Comparison of lateral ground displacement.

In this case study, the FE back-analysis results show that effective Young's modulus (E') of the upper loose fill soils are about 1500 to 1800 x SPT'N' while the E' = 2500 x SPT'N' for lower original sandy silt subsoil. The low back-analysed stiffness for loose fill soil is probably due to excessive stress relief caused by the drilling disturbance of the closely spaced nails. In addition, the loose state of this filled slope as reported by Liew & Khoo (2006) also contributed to the reduction of stiffness. On the other hand, the suggested unloading/reloading stiffness (E'_{ur}) used in the 'Hardening Soil' model is about three (3) times that of Young's modulus (E'), which agrees with the outcomes from many researchers.

4.2 *Ground Settlement*

The ground surface settlement measured from settlement markers installed on the retained ground perpendicular to the alignment of excavation and the FE back-analysis results are presented in Fig. 9. Generally, the settlement markers indicate a larger ground settlement magnitude as compared to the FE back-analysis results. The soil nail stabilisation works deployed high compressed air flushing the soil to form a pre-drilled open hole, which is similar to a micro tunneling. The excessive ground loss and stress relief in the soil extraction process have thus contributed further ground settlement in addition to the one caused by excavation. Even after the completion of nailing work, the terrain deformation may continue at a decreasing rate depending on the method of grouting the holes (Dulacska, 1992). In order to improve such conditions, the displacement type of reinforcing elements can be considered to reduce ground loss and stress relief induced settlement. In a case study of deep excavation using jack-in anchors presented by Liew, et al (2003), there was no abnormal ground settlement.

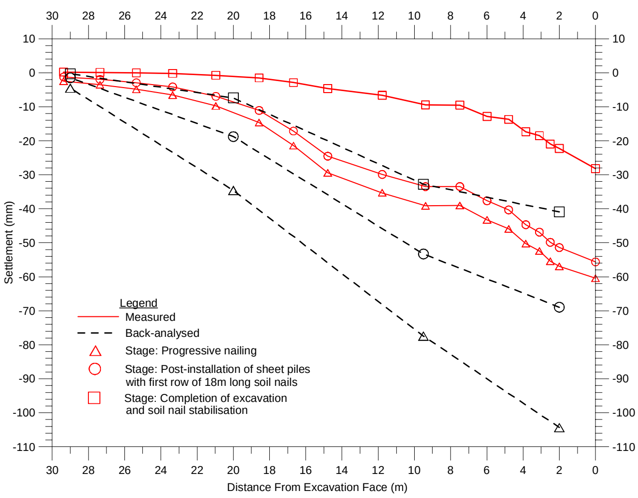

Fig. 9. Comparison of settlement on retained ground.

On the other hand, subsidence troughs developed on the retained ground surface (see Fig. 10) could also be observed from the FE back-analysis results. 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. The location of the trough is just immediately behind the end of the soil nail where high shear strain is generated as explained in the following section.

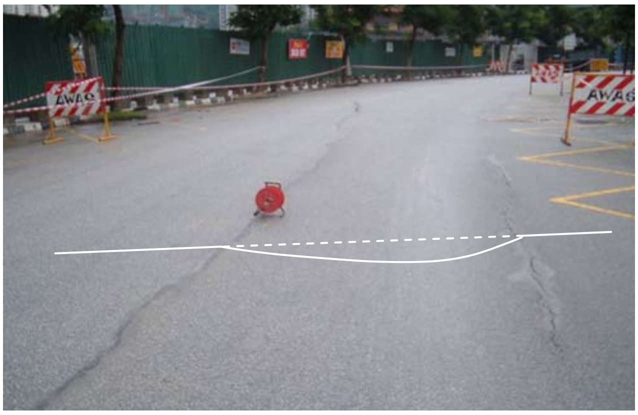

Fig. 10. Subsidence trough developed on the retained ground.

The settlement profile seems to taper off at a distance of 30m from excavation, which is near to the existing commercial buildings. One reason of such settlement profile could be the shielding effect caused by the presence of these commercial buildings supported on deep foundation obstructing the propagation of the ground settlement. Nonetheless, it generally tallies well with the ratio of actual ground settlement.

4.4 *Shear Stresses of Soil During Excavation*

From Fig. 11, the shear strains within the reinforced soil mass as a result of soil nailing are mostly less than 0.5%. However, there is a relatively large shear strain developed along the potential slip surface of the active zone immediately behind the reinforced soil mass. This is fairly close to the formation of an active wedge in the retaining wall design. The settlement trough profile as shown in Fig. 10 is expected at the top of this active wedge. Two major tension cracks signify the extent of the developed active wedge behind the reinforced slope mass.

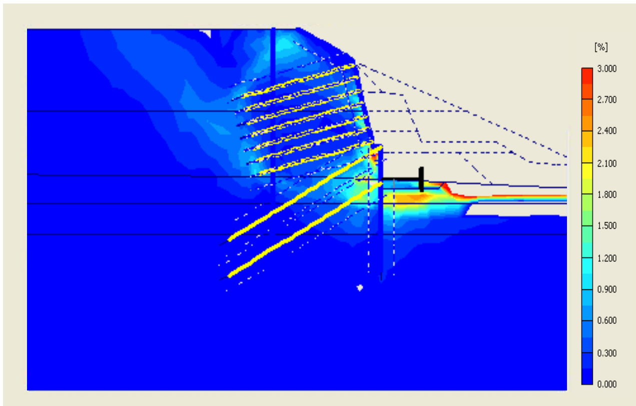

Fig. 11. Soil shear strain within the soil nail reinforcing system.

It is also noticed that there is a band of potential slip surface developed from the active wedge running through the soft clayey deposit at the final excavation level, which appears to suggest that the reinforced slope mass may slide laterally under the huge active earth pressure behind the reinforced slope mass. The inclusion of soil nails has restricted the development of active zones within the reinforced soil mass. To accompany the nailed slope geometry and the potential failure mechanism, it is expected that excessive shear force and flexural stresses would be induced at the embedded sheet pile wall and excessively mobilize the passive resistance leading to catastrophic collapse. In fact, the highest shear strain in the FE analysis is actually located at the passive zone in front of the sheet pile wall embedment. Liew et al. (2003) observed the same mechanism in their analysis of jacked anchor stabilisation works for 17m deep excavation in a cut-and-cover tunnel.

5 CONCLUSIONS

Based on the discussions in this paper, the following conclusions and recommendations can be made:

a. Soil nailing has been proven to be a technically excellent and cost effective solution in the stabilisation of loose fill.
b. It is important to thoroughly investigate the interfacing zone between the fill and the ground before filling. There would be a remote possibility that a proper treatment to the ground is done before tipping the loose fill.
c. Original topographical features can sometimes be the important consideration for stability and remedial strategy. In this case study, a natural valley with soft deposits was not detected in time causing cost escalation due to unexpected remedial work.
d. Excessive ground loss due to open-hole drilling in loose fill with sensitive structure should be carefully considered in order to mitigate against unfavourable ground settlement and stiffness reduction of the stabilized ground. Displacement types of reinforcing elements, such as jack-in anchors, can be considered.
e. FE analyses have been successfully utilized to investigate the distresses by revealing the inherent failure mechanism and back-calculated engineering parameters with validation by laboratory tests, analyses, and optimised the proposed remedial options.

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