The 17th Southeast Asian Geotechnical Conference
Taipei, Taiwan, May 10 ~ 13, 2010

Investigation of Soil Nailed Slope Distress at Fill Ground & Remedial Solutions

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ABSTRACT: This paper presents a case history of a distressed 6m to 10m high soil nailed slope formed for basement excavation at fill ground over a natural valley. Above the soil nailed slope, there is an existing 8m to 11m high reinforced soil wall, located 23m away from the soil nailed slope. During the early stage of excavation for soil nailing works, distresses like ground surface cracking and settlement were observed. Perched groundwater at the natural valley during the heavy monsoon season posed further difficulties to the drilling works and induced instability in the steep excavated face of the fill materials. As such, a well planned instrumentation programme was implemented to monitor the construction safety of the on-going soil nailing works. Remedial solution using permanent steel sheet pile wall with two rows of soil nail anchorages is presented in this paper. The performance of the remedial works through instrumentation results and design verification tests of nailing are presented to illustrate their effectiveness and satisfactory performance even with the adverse rainfall event.

1. INTRODUCTION

A 4V:1H soil nailed slope was adopted for a car park construction after exploring other conventional retaining methods with site constraints. This open excavation was to provide natural ventilation for the car park. The 6m to 10m high slope was strengthened with soil nails with nail lengths ranging from 12m to 15m and with horizontal spacing of 1.3m to 1.5m. The entire soil nailed slope length is approximately 60m long at a previous uncontrolled fill ground over a natural valley (Figure 1). There is an existing 8m to 11 m high reinforced soil wall located 23m away from the soil nailed slope. Localised surface slips happened occasionally during soil nail drilling works (Figure 2). The slope was detected to move laterally as evidenced from inclinometer monitoring results and tension cracks were observed at the crest of the fill slope as shown in Figure 3. Liew & Khoo (2008) presented two case studies of excavation induced ground distresses of similar nature.

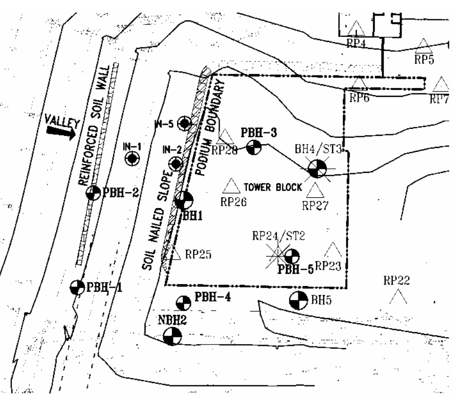

Figure 1 Site layout of soil nailed slope and reinforced soil wall

Figure 2 Localised surface slips occurred during soil nailing work

Figure 3 Tension cracks at the crest of the soil nailed slope

2. SUBSOIL CONDITION

2.1 Borehole Information

Information obtained from the nearest exploratory boreholes to the soil nailed slope area revealed that the subsoils mainly consist of clayey SAND and sandy SILT with SPT-N values ranging from 5 to 33 (denoted by PBH1, PBH2 and BH1 in Figure 4). A hard layer can be found at about 13m below the soil nailed slope toe. The shear strength parameters obtained from the undisturbed samples at the granite residual soil revealed that the average values of c' and ϕ' are 5kPa and 31^o respectively.

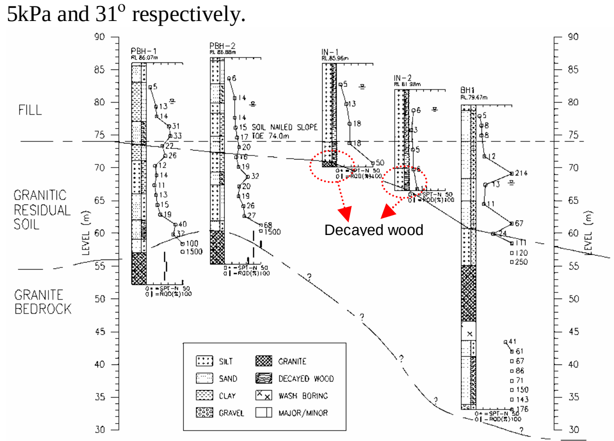

Figure 4 Boreholes profiles

Prior to the construction of the soil nailed slope, two inclinometers (namely IN-1 and IN-2) were installed to monitor the nailed slope performance as a precautionary measure during the construction period. The inclinometer monitoring was carried out on fortnightly basis. At the same time, ten numbers of ground settlement markers were also installed to monitor the ground settlement as shown in Figure 5. An additional inclinometer (IN-5) was installed after the nailed slope suffered distressing. Undisturbed samples were collected during the installation of IN-5 and two numbers of multiple-reverse direct shear box tests were performed on the reconstituted soil samples. The interpreted residual shear strength of the retained soils was approximately c' = 0 kPa and ϕ' = 18°, which were used for remedial design.

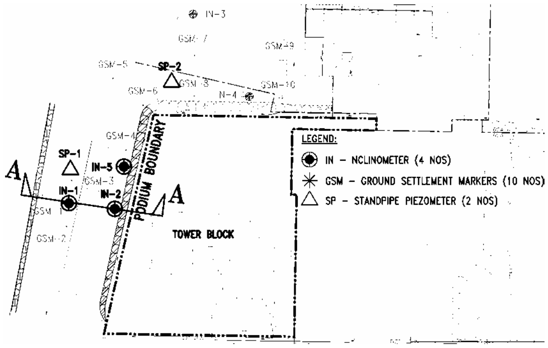

Figure 5 Instrumentation layout

2.2 Groundwater Level

Two observational wells (SP-1 and SP-2) were installed in July 2008 to measure groundwater level on a weekly basis during the period of unexpected prolonged rain storm between March and April 2008 (Figure 6). Monitoring results of SP-1 (Figure 7) located at slope crest showed that groundwater levels changed erratically indicating rain storms during the construction of soil nailed slope especially between August and December 2008. Surface runoff filled up the tension cracks and created perched groundwater. The excavated bare slope was susceptible to shallow surface collapse after saturated by water.

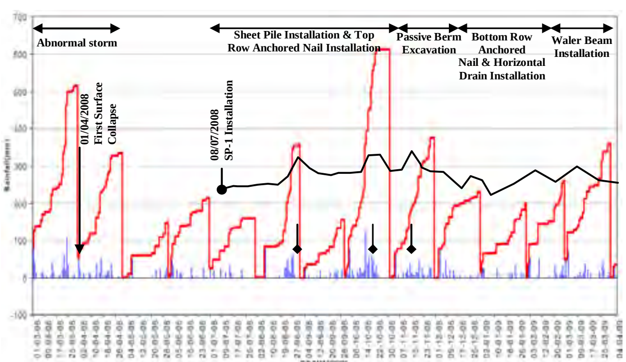

Figure 6 Hyetograph for Kuala Lumpur Rain Gauge Station

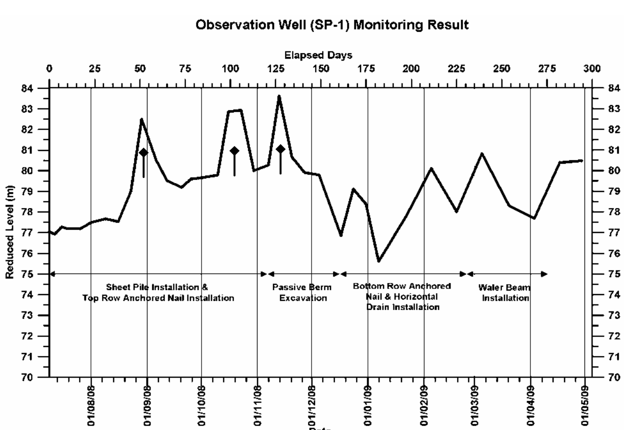

Figure 7 Fluctuation of water level with time

3. CHRONOLOGICAL EVENTS OF SOIL NAILED SLOPE DISTRESS

Slope movement was first detected in April 2008. IN-1 and IN-2 indicated lateral creeping of the slope. Site assessment was immediately carried out to investigate the causation of slope creeping. Preliminary suspicion was attributed to the pocket excavation of pilecaps which were located less than 500mm away from the slope toe. Torrential rain further aggravated the slope movement. Perennial surface collapse and tension cracks at slope crest were noticed as the bare cut slope left unattended by the site team and only focusing on rushing the construction of substructure works. Figure 8 and 9 show inclinometers monitoring results (IN-1, IN-2 and IN-5 respectively) corresponding to the various activities triggering the distressed slope to creep laterally. Tension crack mapping was shown in Figure 10 and displacement markers indicated that the distressed slope formed a bowl-shape unstable slope mass.

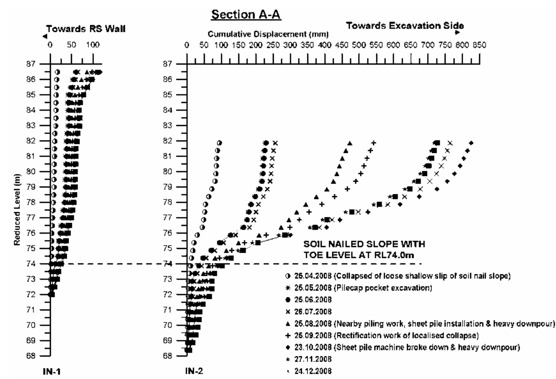

Figure 8 IN-1 and IN-2 monitoring results at Section A-A

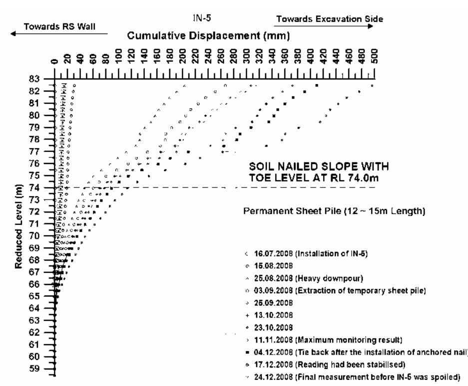

Figure 9 IN-5 Monitoring results

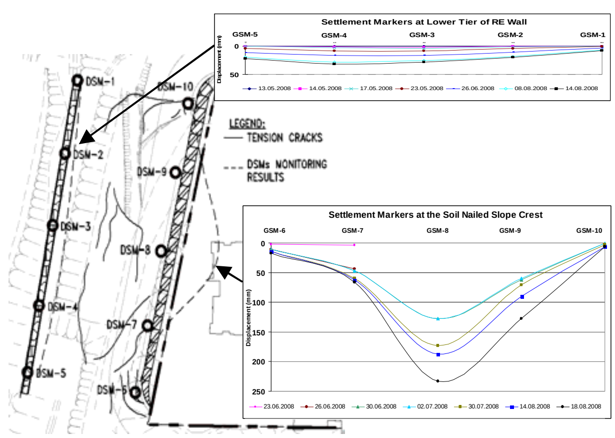

Figure 10 Lateral slope creep movements from DSMs monitoring results and tension cracks

4. REMEDIAL SOLUTIONS

Similar strengthening strategy as presented by Liew & Khoo (2006) was adopted in this case history in view of its similarity.

4.1 Immediate Mitigating Actions at Site

Immediate mitigation action was promptly taken at site by constructing a passive berm in front of the slope and seal up the tension cracks using bentonite-cementituous grout slurry to stop infiltration of the surface runoff when the first surface slip failure happened on 1st April 2008. Inclinometer monitoring frequency had been revised to daily basis to closely monitor the distressed slope behaviour and also monitor the daily site activities which might potentially further endanger the slope. Soil nailing contractor was also instructed to rectify the cave-in collapse with sand bricks and cement grouts as dental filling.

4.2 Instrumentation Monitoring Plan and Results (During and After Slope Distress)

After the distressing incident, the distressed soil nailed slope was fragile when subjected to any disturbance such as rain, nearby piling work, excavation etc. The monitoring results evidenced that the soil nail slope was highly unstable at that time. Additional ten numbers of displacement markers (DSMs) were immediately installed in which five numbers were located near to the crest of the soil nailed slope whilst the rest five numbers were positioned at lower tier of the distant reinforced soil wall from the distressed soil nailed slope. These DSMs were monitored daily as an alert system during the remedial work. The locations of the DSMs are shown in Figure 10.

The slope was further disturbed due to the vibration of sheet pile penetration agitating the loose fill. Air flushing in the open-hole drilling using high compressed air by portable spider drilling rig had also been modified by using water flushing technique with protective casing to minimise the soil disturbance in the anchor nails installation. Due diligence was exercised by the contractor in handling the construction at this highly sensitive distressed slope.

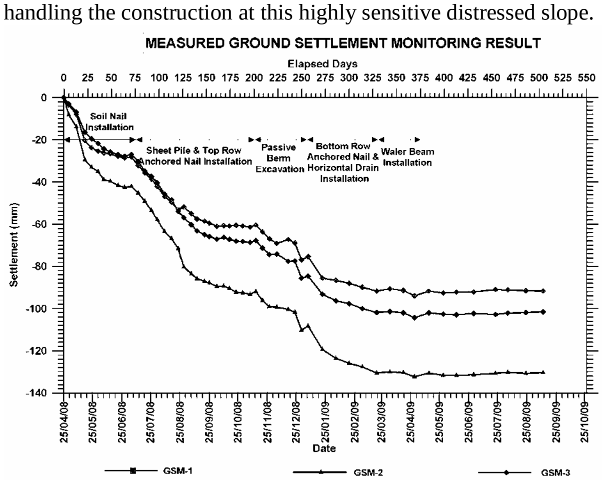

Figure 11 Measured ground settlements

All the monitoring results, including the inclinometers, ground settlement markers, ground displacement markers, tension cracks mapping and groundwater measurement agreed well with the observed pattern of ground movements in which the movement magnitude and directions provide good guidance in postulating the failure mechanisms of the distressed slope. The numerical modelling was calibrated with the monitoring results and offered a useful tool in selecting the best practical remedial solution for the distressed slope in fill ground. During the post remedial works, monitoring results reported stabilised trend with no alerting on-going movement after the completion of the entire slope stabilisation work. The groundwater as indicated by the observational wells was also observed to achieve steady state condition with the completion of horizontal drains.

4.3 Permanent Sheet Piles Remedial Design and Analyses

Permanent sheet piles were proposed as the best practical solution with the primary objective is to arrest the slope from further creeping by intercepting the potential slip surface as identified from the inclinometers. Therefore, permanent sheet pile with 12 to 15m length with embedded depth of 9m and 12m were determined to be required to intercept the potential failure surface. Figure 12 shows the remedial design section of permanent sheet pile wall as follows:

a. Installing one row of 12 to 15m long FSP IIIA sheet piles in front of the toe of the soil nail slope;
b. Two rows of the anchor nails to be installed with rock socket length of minimum 3m into the granite bedrock;
c. Construct capping beam at sheet pile top level to ensure shear transfer between the interlocked sheet pile sections;
d. Waler beam was then propped against the ground level building sub-structural frame as passive resistance toward the residual slope movement;
e. Construct concrete skin wall in front of sheet pile wall for corrosion protection and fire resistance as sheet piles would develop plastic behaviour when subjected to high thermal heat;
f. Perforated horizontal drains (two rows of 18m long & one row of 6m long) to control groundwater level in retained ground.
g. Vibrated wire strain gauges were installed at the one anchor nail located at centre portion of distressed slope for long term performance monitoring purpose.

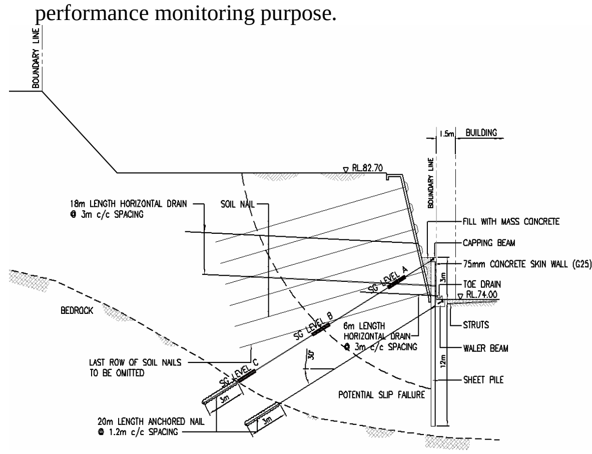

Figure 12 Typical section of soil nailed slope and sheet pile wall

4.3.1 Monitoring Results for Instrumented Anchor Nail

An instrumented anchor nail was installed at the top row of anchored nails to be monitored on fortnightly and monthly basis during the anchored nails installation and further excavation stages. The setting out of vibrating wire strain gauges was shown in Figure 13 and the mobilized force of the instrumented anchored nail was presented in Figure 14. The results indicated the top row nail started to gradually mobilise as a result of earth pressure after progressive removal of passive berm for the installation of bottom row anchored nails. The monitoring results indicated that the anchored nail had mainly mobilized the nail resistance from Level A to B. It is clearly evidenced that the unstable slope mass transferring most of the unbalanced force to the sheet pile wall. The subsequent anchored nail performance had evidenced that the remedial solution had successfully arrested the potential failure.

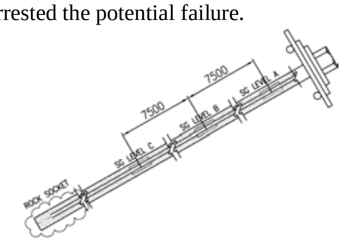

Figure 13 Instrumented anchored nails setting out

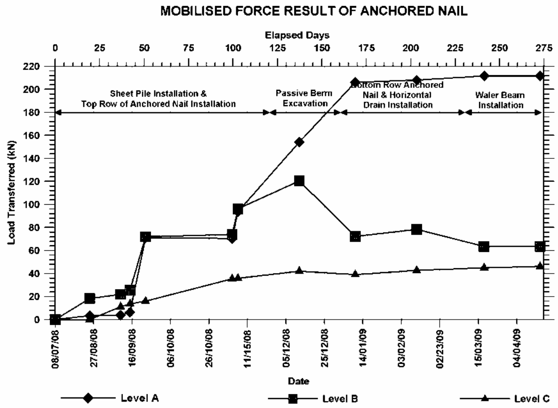

Figure 14 Mobilised force result of instrumented anchored nail

4.3.2 Finite Element Method (FEM) Analyses

Modelling technique with a coupled-consolidation undrained analysis was performed as the construction time was significantly long for potential strain softening. For constitutive soil model, the linear-plastic materials with hardening soil type were selected. The anchor nails with little bending resistance due its intrinsic slenderness and its good ability to take tensile stress were modelled using geotextile element while the sheet pile was modelled using plate element with representative bending stiffness.

Table 1 Soil Material Properties in FEM analyses

Material	Average SPT 'N'	Bulk Density, γ_b (kN/m^3)	Effective Cohesion, c' (kN/m^2)	Effective Friction Angle, ϕ' (^{o})
Loose Fill Material	8	17	0	18
Original Granitic Residual Soils	20	20	5	31
Very Hard Weathered Granite	100	20	0	40

Effective Young modulus, E' for loose fill and original granitic residual soil were estimated using empirical correlation below, which was proven appropriate in the back analyses for similar materials and type of slope distresses presented by Liew & Khoo (2007).

Loose Fill: $E' = 1500{\sim}1800 \times SPT\ 'N'$ (kN/m^2) (1)

Original granitic residual soil: $E' = 2500 \times SPT\ 'N'$ (kN/m^2) (2)

A thin slip shearing band with interpreted effective residual strength of c' = 0 kPa and $\phi' = 18^o$ as identified from the inclinometer result was modelled in the FEM to conjure up the "slip-off" effect on the potential failure mechanism to explore the remedial options.

4.3.3 Back Analysis of Potential Failure Slip

Numerical modelling using computer program – PLAXIS was performed to back analyse and simulate the inherent failure mechanisms whereby the tension cracks become apparent on the retained platform during the pocket excavation of pilecaps. In the back analysis, the soil stiffness which had significant influence on the slope movements had been calibrated to best match the actual slope movement as recorded by inclinometers.

The FEM results confirmed the zoning with high shear strain within the unstable slope mass tally reasonably well with the occurrence of tension cracks location as depicted in Figure 15. A band of potential slip surface was developed in the form of semi circular failure and cutting through the earlier installed nails. The analysis also showed the potential circular slip which was similar to the one detected from the inclinometer results. The c-ϕ reduction method was also performed to confirm the said local potential failure surface. In such cases, the permanent sheet pile with approximately 12 to 15m is essentially required to provide sufficient resistance to maintain the equilibrium and arrest the slope lateral movement.

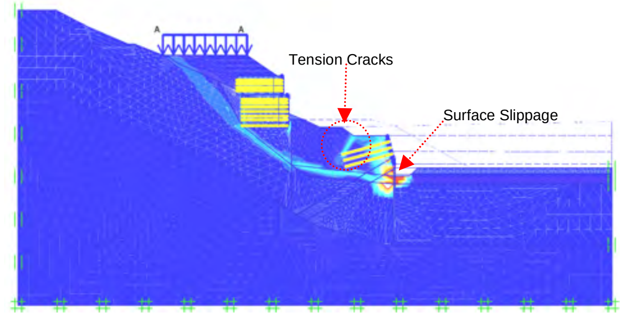

Figure 15 High shear strain area showing potential locations of tension cracks and potential slip surface developed during pilecap pocket excavation

5. CONCLUSIONS

The following inferences could be drawn upon completion of this remedial work:

a. It is imperative to study thoroughly the original topography and identify the possible geo-hazards that may occur. Natural valley will normally be associated with overlying soft deposit. Fail to plan may result in plan to fail may result if due care is not exercised;
b. Drilling method using high compressed air as a flushing medium in the open-hole drilling work shall be carefully assessed in loose fill ground. Failing which can potentially cause uncontrollably slope movements and may jeopardize the sensitive structures in vicinity;
c. Proper and well-planned instrumentation scheme shall be carried out prior to the commencement of construction work. This can closely monitor and detect any anomalies or deterioration in performance arising during the construction work;
d. FEM analysis is a good geotechnical tool in predicting the soil behaviour, but shall be used with well established experience and proper judgement.

6. REFERENCES

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[2] Liew S.S and Khoo C.M., "Performance of Soil Nail Stabilisation Works for a 14.5m Deep Excavation in Uncontrolled Fill Ground, Proceeding of 16th Southeast Asian Geotechnical Conference, Subang Jaya, Malaysia, 8 - 11 May 2007, pp. 827-837.

[3] Liew, S.S. and Khoo, C.M., "Soil Nail Stabilisation for a 14.5m Deep Excavation at Uncontrolled Fill Ground". Proceeding of 10th International Conference on Piling and Deep Foundations, Amsterdam, the Netherlands, 31 May – 2 June 2006, pp. 165-172.