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FAILURE MODE FROM BACK ANALYSIS OF DEFORMED CANTILEVER SHEET PILE WALL

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Abstract: The soil mass movement on failed slope had substantially deformed a sheet pile wall positioned at 6m from the new laboratory of Faculty of Mechanical Engineering (FKM Lab), Universiti Teknologi Malaysia, Skudai, Johor. To facilitate investigating the causes of the wall deformation, a computer modeling of slope stability is applied to simulate slope conditions before and after construction of the study area. The results of simulation analysis establish the fact that global soil mass movement started from the elevation of the newly filled water tank to downhill direction towards the installed sheet pile. Consequently, the mass movement generates a combination of mobilized shear force and lateral pressure larger than strength capacity of the sheet pile. In addition, the finite element analysis shows that the sheet pile might successfully intercept the slip plane of failure slope but the wall moves significantly forward because of inadequate passive resistance and required minimum penetration depth in order to achieve stability.

Photo 2: Physical damages of sheet pile – right side view
(Photo as at March 2003)

A total of five (5) Unconsolidated Undrained (UU) Triaxial Compression Tests have been carried out results in a range of undrained cohesion, c_u from 13kPa to 53kPa. Results from the conducted classification tests classify the soil as CH, sandy fat CLAY (PI_{avg} = 33%, LL_{avg} = 55%). The shear strength parameters for the slope stability are significantly dependant on effective cohesion, c' and effective friction angle, ϕ' for a long-term analysis. Due to in absence of Consolidated Isotropic Undrained (CIU) test data, the back-analysis of the failed slope has been carried out aided by SLOPE/W Version 5. From the interpretation of back-calculated shear strength, the undrained shear strength, c_u at the time of failure is about 50 kPa whereas the $\phi' = 17^o$ (c' is taken as = 0). The ϕ' value at $c' = 0$ can be estimated by using Kenney's chart (Coduto, 1994) giving in value $\phi' = 20^o$. Additional detailed soil investigations are advisable to confirm the above back-calculated strength parameters, the sub-soils stratification and the groundwater table conditions.

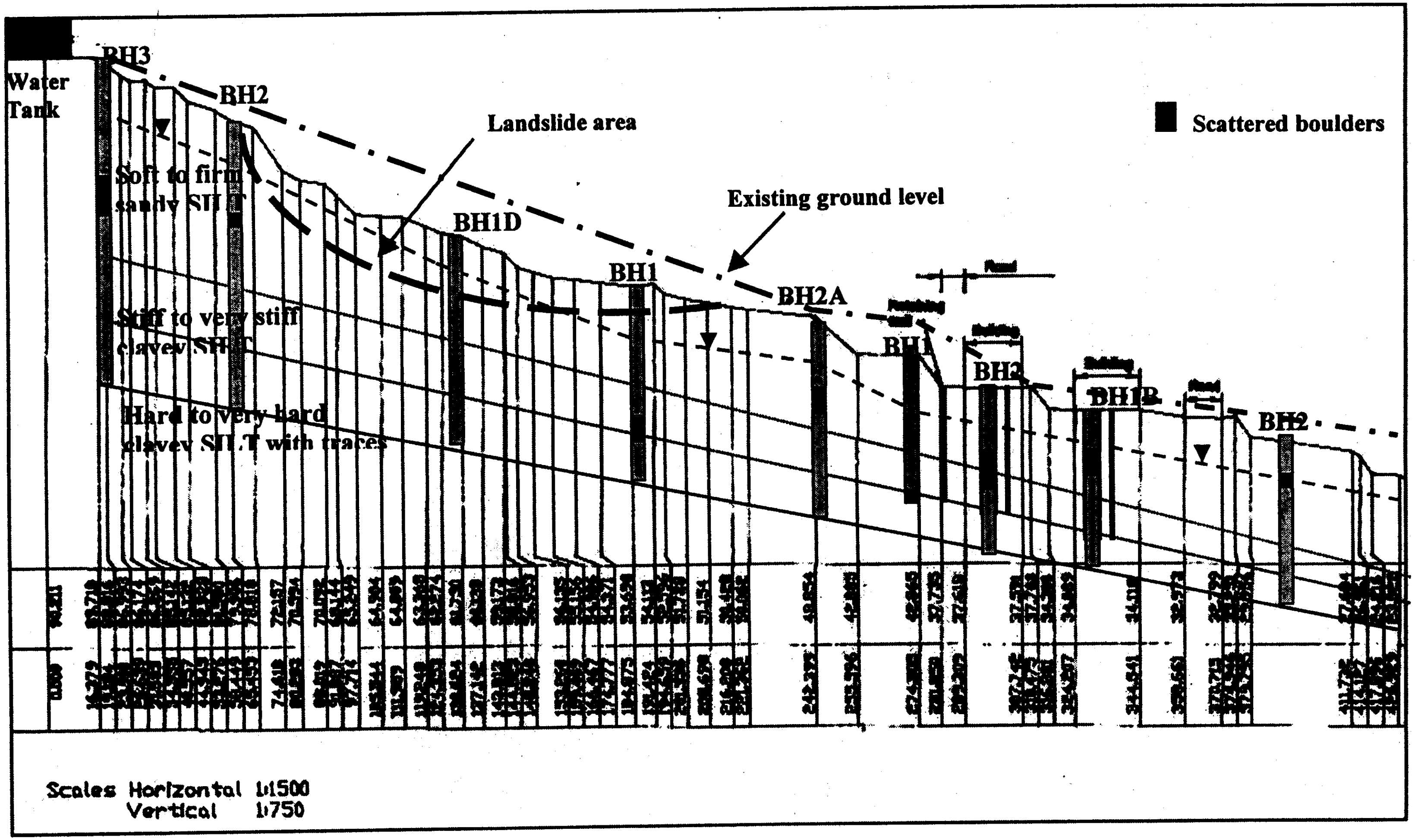

Figure 2: Cross sectional topography and geotechnical properties at the landslide area (after Kassim, 2002)

The earthwork has been extensively carried out on site by cutting the slope into three required platforms at proposed finished levels of 39.5m, 37.5m and 34.0m respectively (**Figure 3**). Platform 39.5m and 36.0m initially intended to sit the new buildings and platform 34.0m was reserved for parking area. While the earthwork progressed, noticeable localized slope failures and cracks at the cut slope appeared some meters behind the slope. Water seepage was seen at some locations of the slopes and the cracks as well as failures progressively expanded to a distance of more than 70 m behind the outermost slope. The ground at failure area subsided to about 3.0 m below the original ground level. Cluster of boulders were found during excavation with concentration at the mid section of the site and it indicates a very porous soil condition and probably closes to the bedrock. Platform at elevation of 39.5m then were raised by heave of 2m high a few months later after the site was subjected to heavy rainfall events. Seepage discharges and soil piping were also noticed at the toe of the upper slope. A decision was then made to shift all the building blocks a platform lower and the parking area moved to the upper platform at elevation of 39.5m.

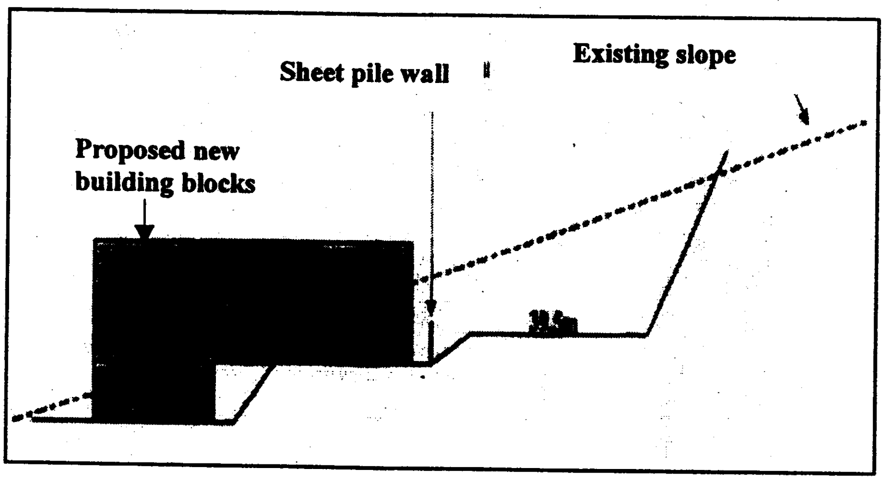

Figure 3: Installation of sheet piles at the toe of the slope between platforms at 37.6m and 39.5m.

Heaving at platform level of 39.5m has extended to platform 37.6m after the completion of the piling works of new building at platform level of 34.0m. Tension cracks have now reached more than 250m beyond the site boundary and earth subsided to about 3.0m closing to the existing water supply reservoir. In an attempt to counter the slope instability or soil movement down the slope towards the proposed new FKM Lab., sheet piles were installed at the toe of the slope between platforms at 36m and 39.5m. However, after three quarter of piles had been installed, lateral movement of the piles was noticed and it indicates movement of the soil mass has yet to stabilize and give concerned over the overall stability and safety of the building structure and also the reservoir.

BACK ANALYSIS OF THE SLOPE FAILURE

A commercially available finite element program, PLAXIS Version 8.2 is used to analyze the slope under drained conditions as to simulate the long term condition of the slope. The mechanism of failure observed on site is simulated analytically, which the slope stability analysis is divided into three different geometries, namely cross section of original/natural condition, cross section after cut to platform level, and cross-section after occurrence of landslides.

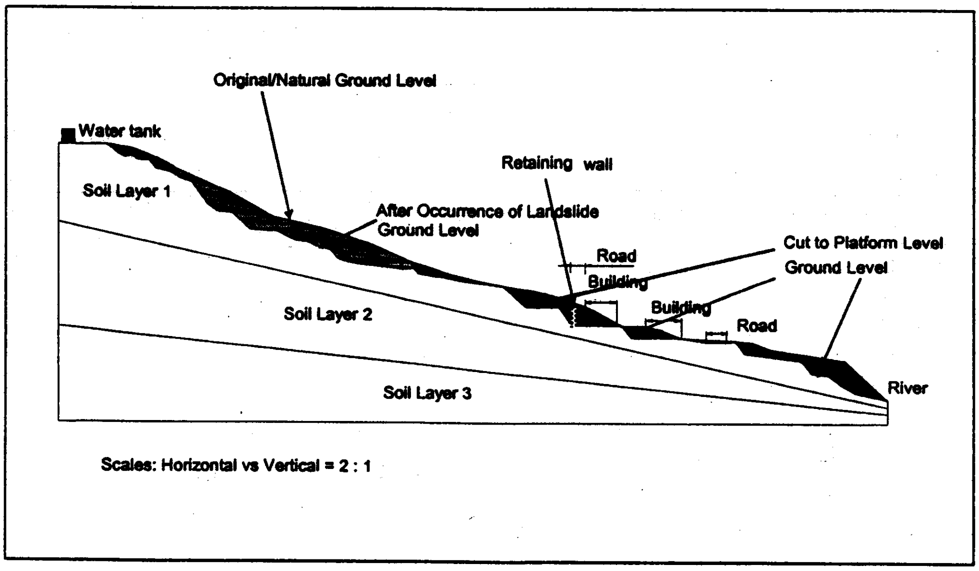

Figure 4: The geometric idealization of the slope

The geometric idealization of the slope at the FKM Lab. is shown in **Figure 4** and it is modeled using the possibilities of the plain strain model. The soil profiles are simplified into three (3) layers while the bedrock is placed as the fourth material, with parameters as given in **Table 1**. For all soil layers, the Mohr-Coulomb model is applied, as it is well known to be used as a first approximation of soil behavior in general. The Mohr-Coulomb model requires a total of five parameters, namely Young's modulus, E, Poisson's ratio, ν, friction angle, ϕ, cohesion, c and dilatancy angle, ψ.

The horizontal boundary is positioned at the side of water tank (crest of slope) and at the side of river (downhill of slope). The lower vertical boundary is placed at 90m below the crest of slope with the assumption that there is hard layer beneath it. Closed flow boundaries are placed at both side of horizontal boundary and also the lower vertical boundary to ensure that flow across the boundary will not occur. A closed consolidation boundary is placed at the lower vertical boundary to simulate the layer of hard stratum, which it means that bottom boundary is located in an impermeable layer.

Table 1: Material properties (after Kassim et al., 2004)

Properties	Soil Layer 1	Soil Layer 2	Soil Layer 3
Material model	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb
Elastic modulus	6.5 MPa	7 MPa	7 MPa
Poisson's ratio	0.35	0.35	0.3
Dry unit weight	15	15.8	15.8
Wet unit weight	18	19	19
Effective friction angle	24°	27°	30°
Effective cohesion	2 kN/m^2	5 kN/m^2	10 kN/m^2
Dilation angle	0°	0°	0°

The slope geometry is divided into finite elements in order to perform finite element calculations, as shown in **Figure 5**. The type of element used in this study is 15-node triangular elements, which provides a fourth order interpolation for displacements and the numerical integration involves twelve Gauss points. The finite elements meshes are generated automatically, which is a built-in feature of PLAXIS V8.2®. The mesh generator is a special version of the Triangle mesh generator developed by Sepra (Brinkgreve & Vermeer, 2002).

The results of the analysis are categorized into two main groups that are potential failure mechanism and associated minimum factor of safety. Potential failure mechanism varies from one condition to the other condition. **Table 2, Table 3, Table 4** and **Table 5** report the description of failure mechanism, whereas the calculated minimum factor of safety for each condition is summarized in **Figure 6**.

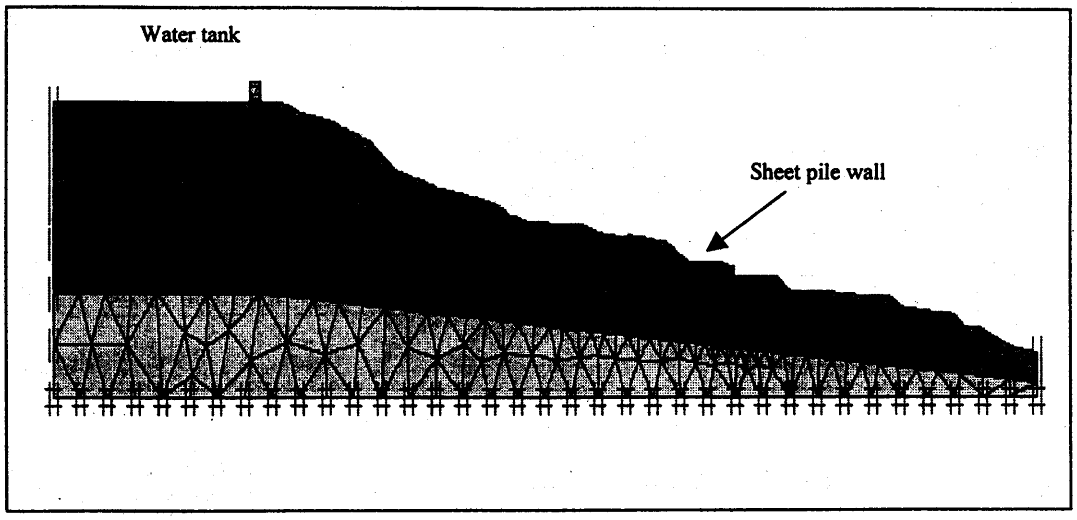

Figure 5: Mesh generation of geometric idealized slope

Table 2: Description of failure mechanism for dry condition

Geometry	Safety factor	Extend of failure (x elevation from water tank)	Mode of failure
1	1.3944	427m to 457m	Shallow rotational slides
2	1.6522	72m to 109m	Shallow rotational slides
3	1.4007	52m to 77m	Shallow rotational slides

Table 3: Description of failure mechanism for groundwater table at 25m below ground level

Geometry	Safety factor	Extend of failure (x elevation from water tank)	Mode of failure
1	1.3701	426m to 460m	Noncircular rotational slides
2	1.1345	275m to 285m	Circular rotational slides
3	1.1327	275m to 286m	Circular rotational slides

Table 4: Description of failure mechanism for groundwater table at 10m below ground level

Geometry	Safety factor	Extend of failure (x elevation from water tank)	Mode of Failure
1	1.1425	-25m to 49m	Localized failure
2	1.1358	-	*-
3	1.1291	-30m to 100m	Localized failure

* No failure mechanism noticed

Table 5: Description of failure mechanism for groundwater table at 5m below ground level

Geometry	Safety factor	Extend of failure (x elevation from water tank)	Mode of Failure
1	1.3536	426m to 459m	Circular rotational slides
2	1.5017	9m to 125m	Circular rotational slides
3	1.0264	14m to 90m	Circular rotational slides

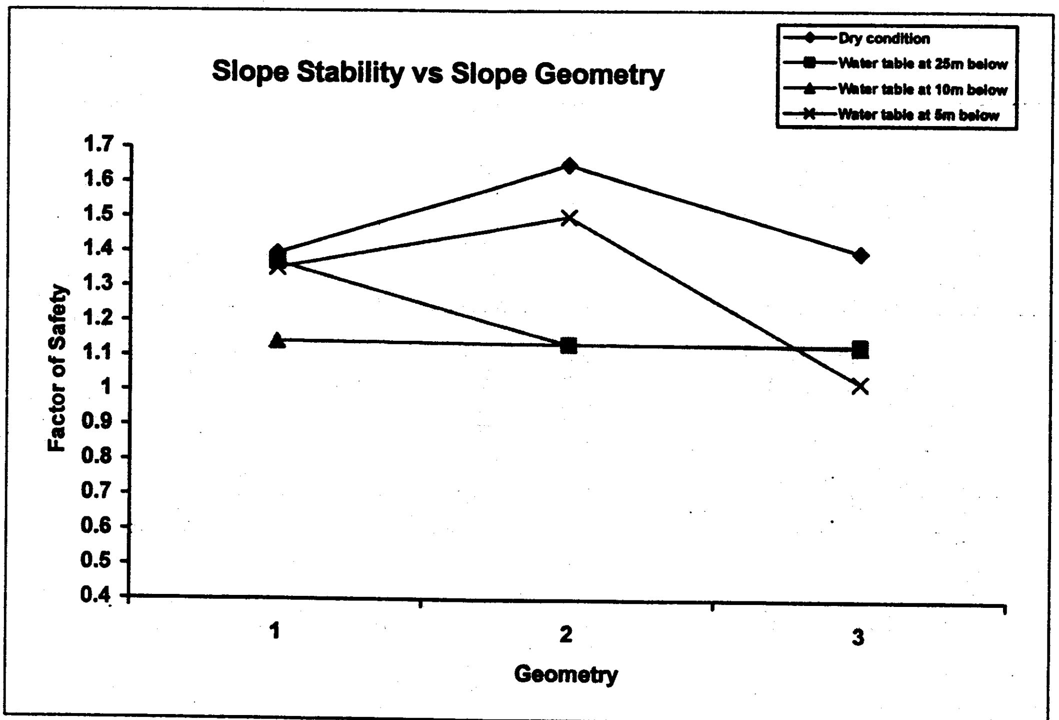

Figure 6: Slope stability for various conditions

Note:
Geometry 1 - Original/natural slope
Geometry 2 - Slope after cut to platform level
Geometry 3 - Slope after occurrence of landslide

THE ANALYSIS OF DESIGNED SHEET PILE WALL ON SITE

The sheet piles were driven at an average of 18m depths to form 2m wall above ground level by employing Section ESC-B8, Grade S275 (section properties with allowable section modulus, pressure strength and allowable moment capacity of 2,518 cm^3/m, 145 $kN/m^2/m$ and 630 kNm, respectively). The basic principles of Blum's Simplification Model are used during analysis for the estimation of net lateral pressure distribution on the cantilever sheet pile wall (**Table 6**).

Table 6: Design parameters for estimation of net lateral pressure distribution

Effective angle of friction, ϕ'	17^o
Coefficient of active pressure, K_a	0.66
Coefficient of active pressure, K_p	1.8
Bulk density, γ_b	17 kN/m^3
Saturated density, γ_{sat}	19 kN/m^3
Factor of safety, F.S.	2.0

Although many theories have been put forward to demonstrate the form and magnitude of active and passive earth pressures on sheet pile wall, they are generally influenced by the desire to obtain pressure diagrams which simplify the determination of maximum bending moment and minimum necessary depth of penetration (Clayton et. al, 1993; Graham & Peter, 2000). To begin with, it is unduly to analyze the designed sheet pile wall as if it were gravity wall or cantilever retaining wall. Sheet pile walls differ from any retaining wall in several ways that there are flexural and are designed structurally as simple or propped cantilevers with their weight are ignored in stability analysis. The principle of design of cantilever sheet pile wall depends for support upon net passive resistance, R mobilized at the toe of the sheet, but behind the wall that required minimum penetration depth to achieve stability (Lee, 1961).

Figure 7 shows the results of the analyzed pressure distribution of the cantilever sheet pile wall. In summary, the depth of embedded, D_e is found to be 37 m and minimum depth of penetration, D' = 26.1 m with net passive resistance value, R = 271.82 kN. The net total pressure, shear and moment calculated from pressure distribution diagrams in **Figure 7** yield the magnitude of maximum moment = 430.25 kNm. The allowable section modulus is found to be 1,564 cm^3/m which it is not exceeding 2,518 cm^3/m of section modulus of driven sheet pile on site. The results confirm that the developed pressures do not exceed the section properties. A conclusion can be derived from the analysis is that the sheet pile can be considered a good and safe material to perform as a retaining structure if it is driven at minimum depth of penetration, D' = 26.1 m.

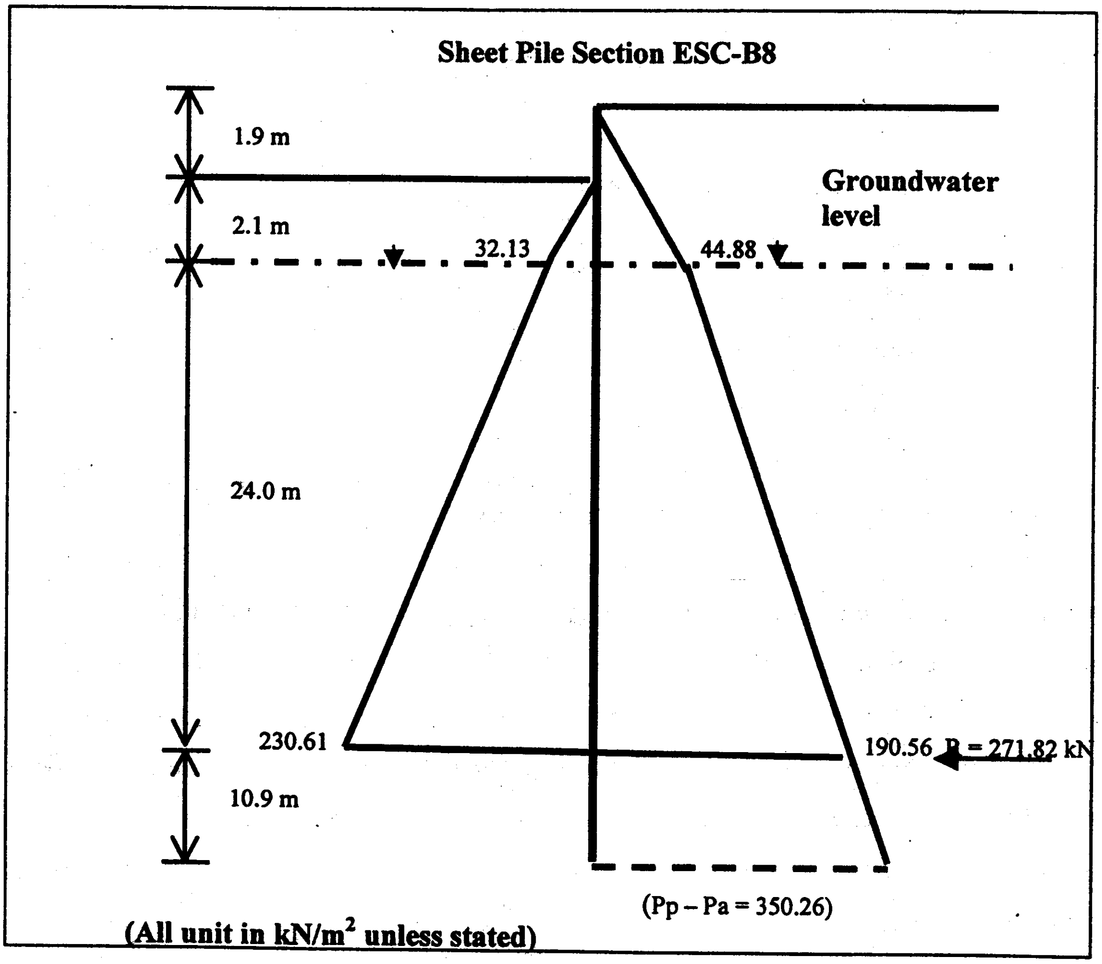

Figure 7: Analyzed pressure distribution at depth of penetration, D'

DISCUSSION

The analysis of the slope shows locations of the critical failure plane on slope depending on the geometry and situation being analyzed. The significant changes of critical slip surface are experienced for the original/natural slope, and the slope after cut to platform level. Initially, the location of the failure plane emerges at the toe of the slope which is near river at x-elevation of 400m to 460m. However, after the slope is cut to platform level, the failure plane is shifted towards the crest (at x-elevation of 10 to 100m) of the slope adjacent to the newly filled water tank.

Generally, the slip surfaces go deeper with the increase of groundwater table. Initially, the slip surfaces are considered shallow and parallel to the slope surface, but do not go beyond the vicinity of soil layer 1. However, the mode of failure changes to circular rotational slides as the groundwater table increases, particularly when groundwater table is assumed 5m below ground level. The results show that the stability of the cut slopes is directly governed by the existence of water. This observation is generally accepted as landslides on hill-site in Malaysia are always triggered by water.

In a number of situations, it is necessary to assess potential danger of slope failure based on each stage of slope deformation from stable to unstable states. The stability analysis shows that the deformation increases after slope is cut to platform level as opposed to natural condition. This observation is supported by the observations of Nishida who found that elastic and plastic deformation normally occurs to cut slope, by releasing of the stress in the soil due to excavation and resulting in the decrease of strength (Kassim & Mohd Amin, 1999). Generally, after cutting, the strength of the soil in the surface layer of the new slope area decreases rather rapidly under the action of the external forces produced afterwards, such as repeated seasonal drying and wetting. The phenomenon is normally known as secondary weathering.

The conclusion can be derived from the analysis of the designed sheet pile wall is that developed pressures due to active and passive conditions do not exceed the sheet pile section capacity. However, the back analysis of the failure slope importantly proves that the global soil mass movement started from the elevation of the filled water tank to downhill direction towards the installed sheet piles. The results show that a net pressure of about 1200kN/m^2/m is generated during soil movement downhill adjacent to installed sheet pile at about ten times the capacity or strength of the pile of only 145 kN/m^2/m. The mass movement generates a combination of mobilized shear force and lateral pressure larger than the capacity or strength of the whole sheet piles irresponsive to the relative satisfactory factor of safety in general.

CONCLUSION

Back analysis clearly shows that the effect of disturbance on slope caused by the development is not locally but globally. Installation of sheet pile wall at x-elevation of 280 m from the water tank proves to be successful in cutting the block failure occurred at 280m but not in cutting the failure plane that exists at the crest of slope after excessive earthwork being carried out. Existence of water tank at the crest of the hill slope seems not to add any instability to the slope. In most condition, fluctuation of groundwater table assumed in the study appears to slightly decreased the stability of slope, which means that the increasing of pore water pressure due to increasing groundwater table leads to the deeper failure plane.

The results of the analysis of sheet pile establish the fact that though the sheet pile on site might successfully intercepted any potential slip plane of the failure slope, the minimum penetration depth, D' of 26.1 m however has never been achieved. The embedded pile of 18m depths on site is not sufficient to resist the forward movement due to insufficient of net passive resistance behind the pile. Although the sheet pile on site might successfully intercept the slip plane of failure slope, but the sheet pile wall on site has failed as it moved forward due to inadequate passive resistance at the bottom of sheet pile that required minimum penetration depth in order to achieve stability as illustrated in **Figure 8.**

An additional remote possibility that the sheet pile could have bulged during the driving since the pile might have encountered bedrock stratum as shown in **Figure 2.** **Figure 2** shows granite or very hard layer (N > 50) was encountered at depth of 9m to 20m which required the minimum wall modulus of 3000cm^3/m (N > 50) of sheet pile to be driven in ground that exceeding the wall modulus of Section ESC-B8, Grade S275.

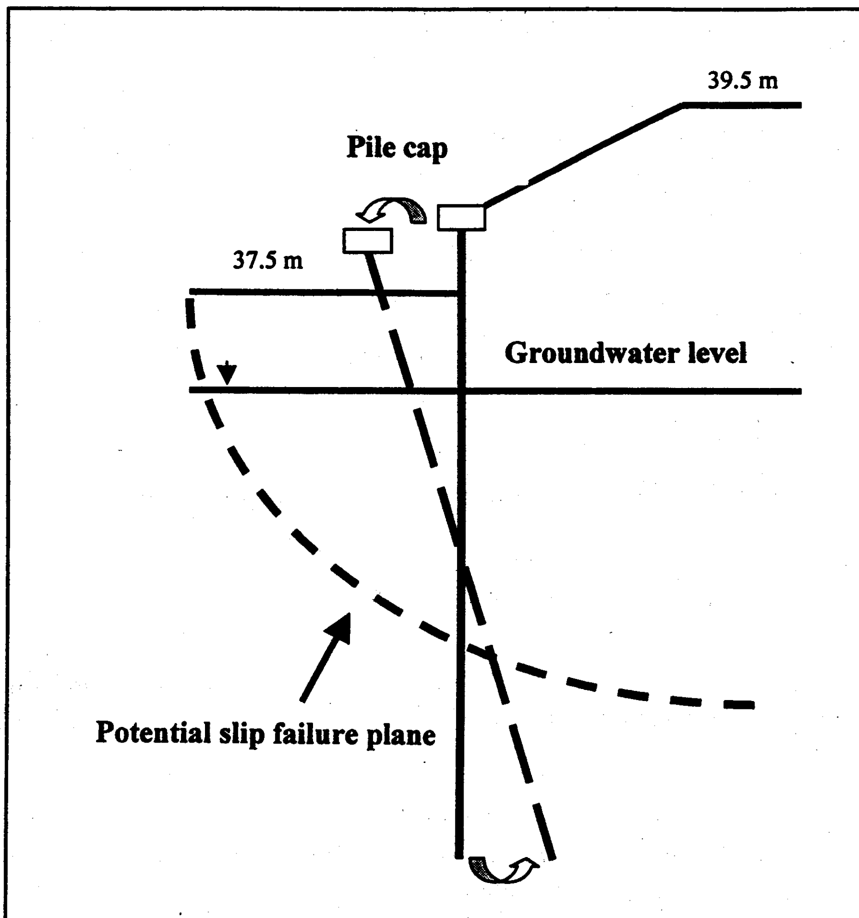

Figure 8: Probable mode of failure of the sheet pile wall

Furthermore, non-uniform filling or non-uniform consolidation and ineffective drainage of the soil behind the wall may result in the line of capping becoming irregular and the sheet pile to bulge. It is recommended that ample penetration (e.g. at minimum penetration depth, D') and good drainage of the backfill are better investments than conservative design of sheet piles for bending stresses. If the pressures on sheet piles are such that an excessively thick pile or the depth of required penetration is required, the provision of a tie at the level of capping beam can be used. With the provision of one or more ties or props, the required depth is reduced as is the lateral deflection and bending moment in the pile.

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