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A SIMULATION STUDY OF SLOPE STABILITY AFFECTED BY CONSTRUCTION OF NEW BUILDINGS AT UNIVERSITI TEKNOLOGI MALAYSIA, SKUDAI.

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Abstract: Landslide occurrences had caused failure of sheet pile wall located about 6 m from Bangunan Tambahan, Fakulti Kejuruteraan Mekanikal (FKM), Universiti Teknologi Malaysia in Skudai, Johor. With the aim of investigating cause(s) of the failure, the slope stability conditions prior and after construction were simulated using SLOPE/W Version 3.03. The results showed that effects of load of a newly filled water tank sited on the top of slope contributed to subsident of the slope surface within the vicinity by creating tension cracks near its raft footing. The slope became more unstable as the soil moisture or pore-water pressure increased due to infiltration of rainwater. This has reduced the shear strength, in particular the cohesion value, *c*. The simulation also confirmed the hyphothesis that the global soil mass movement started from the hill top where the water tank was located toward the installed sheet pile. The combined mobilized shear force and lateral pressure of the global slope was about 8 times the strength of the sheet pile. The associated stresses are found to be related to the formation of the heave pushing up the soil at the toe of slope and soil under the road pavement adjacent to the new laboratory buildings.

Keywords*: Landslide; Slope Stability; Computer Simulation*

Abstrak: Kejadian tanah runtuh telah mengakibatkan kegagalan dinding cerucuk keping yang terletak 6 m dari Bangunan Tambahan, Fakulti Kejuruteraan Mekanikal (FKM), Universiti Teknologi Malaysia Skudai, Johor. Penyiasatan bagi mencari punca-punca kegagalan dilakukan dengan kaedah simulasi komputer menggunakan SLOPE/W Versi 3.03 untuk keadaan tapak sebelum dan selepas pembinaan siap. Hasil simulasi menunjukkan bahawa pengaruh beban tangki air baru yang terletak di puncak cerun adalah penyumbang kepada kejadian enapan di sekitar permukaan cerun dengan kewujudan retak tegangan berdekatan asas rakit tangki air tersebut. Ketidakstabilan cerun didapati bertambah dengan pertambahan kandungan lembapan atau tekanan air liang akibat penyusupan air hujan menyebabkan pengurangan kekuatan riceh tanah terutamanya pada tegasan jelekitan, *c*. Simulasi juga menunjukkan pergerakan jisim tanah secara global bermula dari kedudukan tangki air di puncak cerun menghala dinding cerucuk keping di kaki cerun menghasilkan paduan daya riceh dan tegasan melintang yang bergerak pada keupayaan 8 kali ganda daripada kekuatan sebenar cerucuk keping. Paduan daya ini juga didapati berkaitan dengan kejadian lambung yang menolak ke atas tanah di kaki cerun dan di bawah permukaan jalan berdekatan bangunan-bangunan makmal yang baru.

Katakunci: *Tanah Runtuh; Kestabilan Cerun; Simulasi Komputer*

1. Introduction

The construction of the Bangunan Tambahan Fakulti Kejuruteraan Mekanikal, Universiti Teknologi Malaysia began in October 1999 covering a total development area of 20,000 m^2. In the middle of December 1999, localized slope instability and tension cracks were observed in the upper slope with respect to initial proposed platform level at the end of the earthwork. Platforms at elevation of 39.5 m were then raised by heave of 2 m high a few months later after the proposed 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 to a lower platform and the parking area was moved to the upper platform at an elevation of 39.5 m.

By June 2000, the tension cracks and slope movement had reached more than 250 m beyond the boundary of the proposed site and approached the existing water supply tank (at elevation of about 90 m). In an attempt to counter the slope instability or soil movement down the slope towards the proposed buildings, a row of sheet piles were installed at the toe of the slope between platforms at 37.6 m and 39.5 m (Figure 1). However, after three quarter of the piles had been installed, lateral movement of the piles was observed. The downhill force of global soil mass was so big that caused the pile wall shifted towards the road with end of pile as a center of tilt.

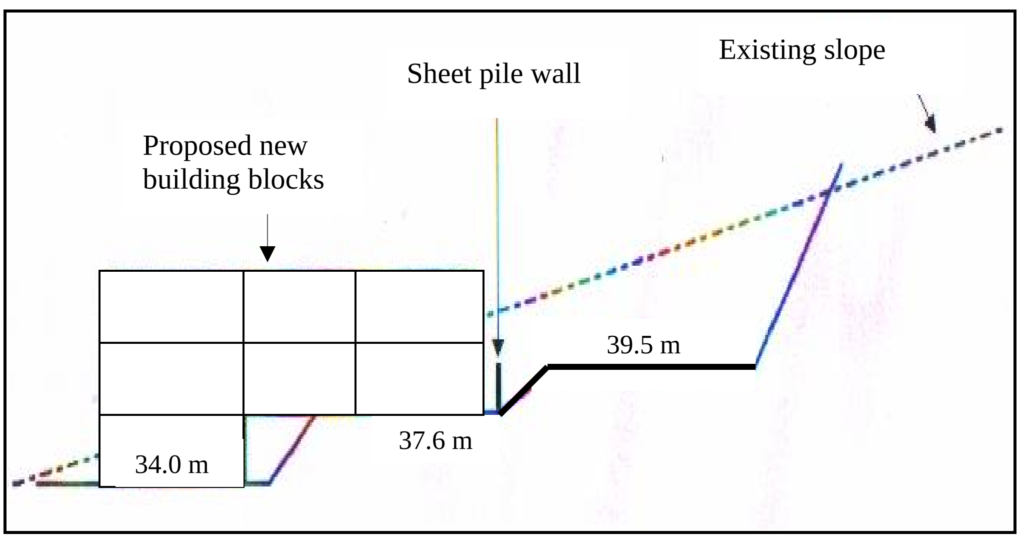

Figure 1: Installation of sheet piles between platforms at 37.6 m and 39.5 m.

2. Modelling of Slope Stability

The mechanism of failure observed on site was simulated analytically with respect to deep-seated slip movement initially with combination of lateral pressure of adjacent slice in soil mass and shear mobilized along the slip surface and later followed by the movement of soil mass along the slope surface due to localised tension crack slip failures. The slope stability analysis was divided into two categories, i.e., global slope stability and localized slope stability. There were four cross sections considered, namely CS1, CS2, CS3 and CS4 (Figure 2).

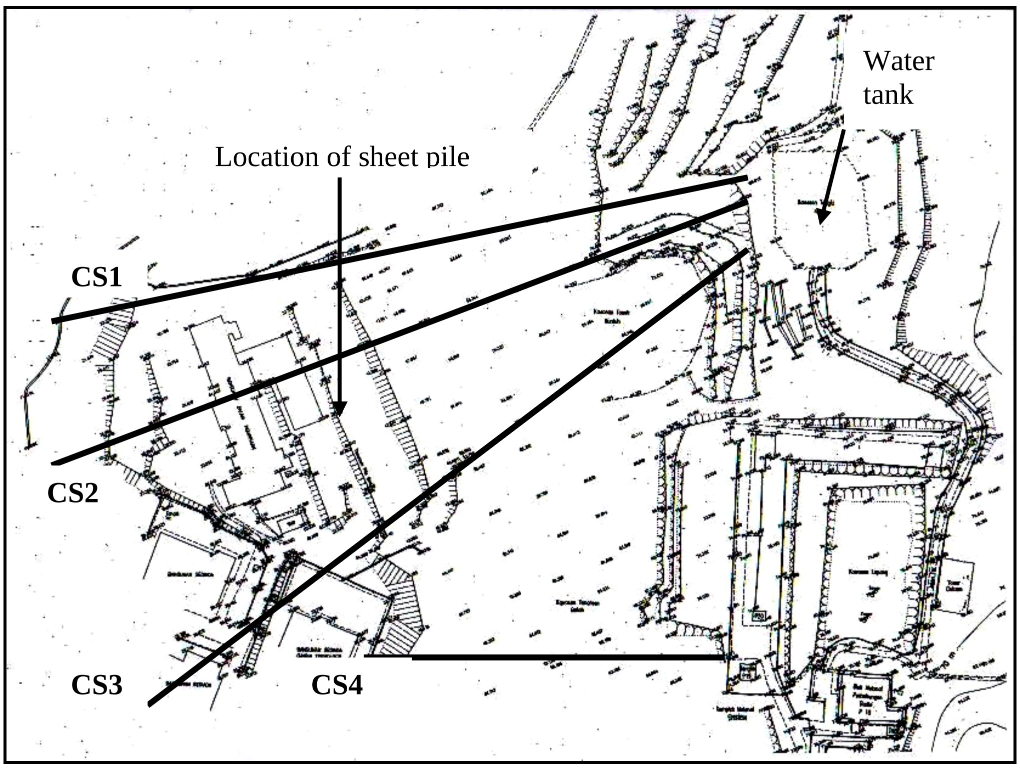

Figure 2: Layout of cross sections CS1, CS2, CS3 and CS4.

Figure 3 shows the general geotechnical profile interpreted from the site investigation carried out earlier (Kassim, 2002). Based on available report (Universiti Teknologi Malaysia, 2001) on the landslide occurrence between 1999 and 2001, the summary of the simulation inputs for four cross sections are shown in Tables 1 and 2.

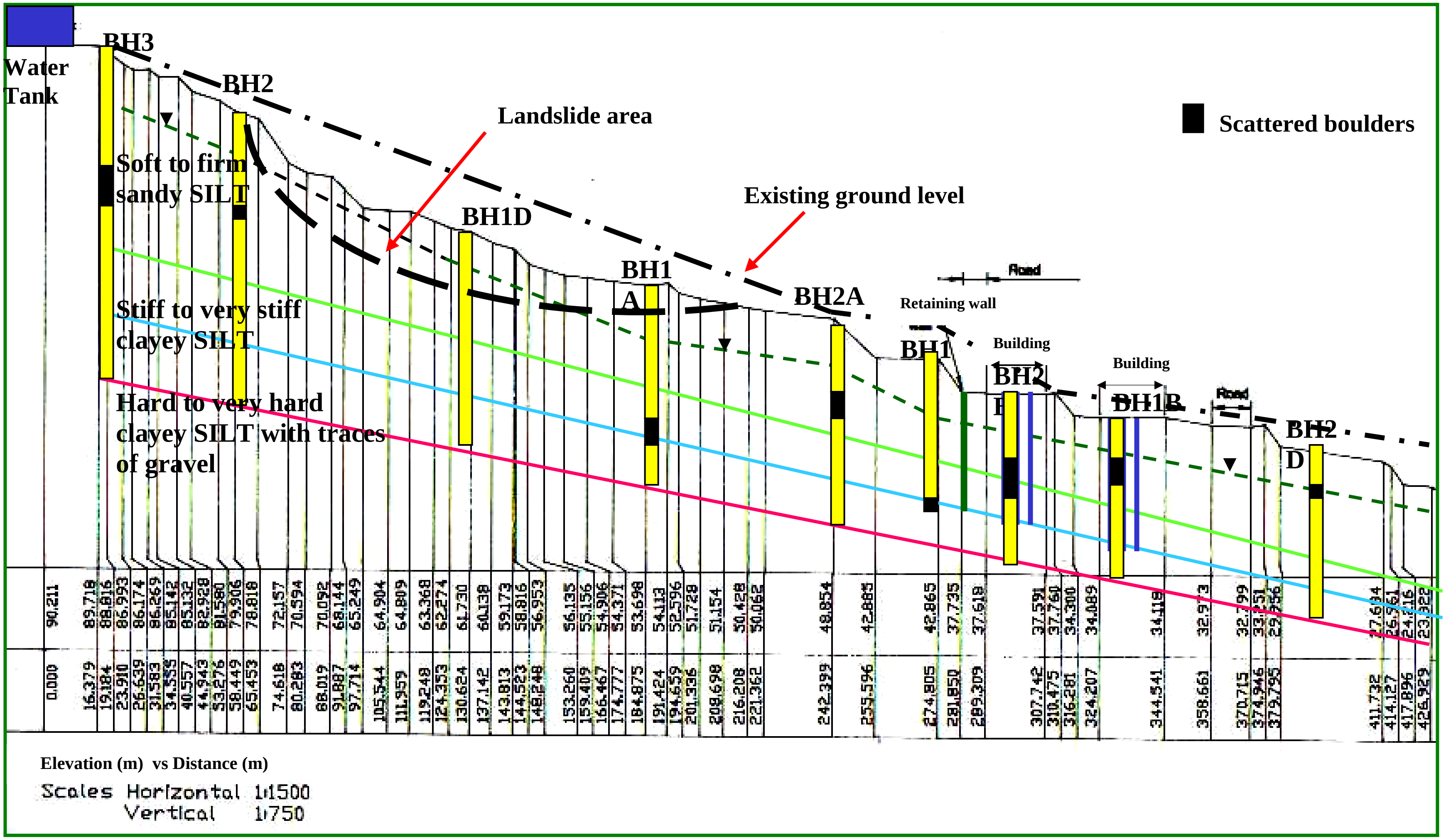

Figure 3: Cross sectional topography and geotechnical properties at the landslide area.

The soil profiles for CS1, CS3 and CS4 were considered as two-layer soils for the simulation works because detailed soil properties for the mentioned cross sections were not available. The slope stability analysis for CS1, CS3, and CS4 was based on global analysis only. On the other hand the soil profiles for CS2 that consist of 4 layers were used in both global and local slope stability analyses (Tables 3 and 4). The bedrock was found at an elevation 0 m as the third and fifth material for CS1 & CS3 and CS4 and CS2, respectively. Effects of external loads and pore-water pressures on the slope stability were simulated using SLOPE/W Version 3.03 of GEO-SLOPE International Limited, Calgary, Canada. The Modified Bishop method of slices (McCarthy, 2002) was chosen for the determination of slope stability factor for its formulation was based on a moment equilibrium. The method is independent of changes in ratio of interslice shear and horizontal forces.

Table 1: Soil properties used for the slope stability analysis.

Cross-sections	Soil Layer	Cohesion, c (kNm^{-2})	Angle of friction, ϕ (°)	Unit weight, γ (kNm^{-3})
CS1, CS3 and CS4	1	30	10	18
(Two-layer soil profile)	2	50	10	19
	1	30	10	18
CS2	2	40	10	18
(Four-layer soil profile)	3	30	10	19
	4	50	10	19

Table 2: Descriptions and assumptions for simulating the global slope stability of CS1, CS3, and CS4 cross-sections.

Cross-sections	Simulation scheme	Descriptions & Assumptions
CS1	a	▪ Original/natural slope prior water tank ▪ Dry slope
	b	▪ Original/natural slope prior water tank ▪ Water table at about 25 m below the crest of the slope
	c	▪ Post-trimming with water tank ▪ The water tank was assumed safely supported by the raft footing ▪ Water table at about 10 m below the crest of the slope
CS3	a	▪ Post construction of the water tank ▪ Dry slope
	b	▪ Post construction of the water tank ▪ Water table at about 10 m below the crest of the slope
CS4	a	▪ Slope between Einstein Laboratory (Institut Ibnu Sina) and Gabions at the toe of the slope, adjacent to a road ▪ Water table at 7 m and 4 m below the crest and toe of the slope, respectively

Table 3: Descriptions and assumptions for simulating the global stability of CS2 cross-section.

Cross-section	Simulation scheme	Descriptions & Assumptions
CS2 (for global analysis)	a	▪ Original/natural slope prior to construction of water tank or other subsequent developments ▪ Dry slope condition
	b	▪ Original/natural slope prior to construction of water tank or other subsequent developments ▪ Water table at about 25 m from slope surface
	c	▪ Original/natural slope prior to construction of water tank or other subsequent developments ▪ Water table at about 10 m from slope surface
	d	▪ Original/natural slope prior to construction of water tank or other subsequent developments ▪ Water table at about 10 m from slope surface ▪ Back-analysis for Fs ≈ 1.0 for determining values of *c* and ϕ at failure via trial-an-error method ▪ The *c* value was not necessarily zero as usually assumed in manual-calculated back-analysis method
	e	▪ Post-trimming and subsided slope surface due to construction and/or weight of water tank ▪ The water tank was assumed safely supported by the raft footing ▪ Water table at about 25 m below the crest of the slope ▪ Prior tension cracks
	f	▪ Post-trimming and post-subsidence of slope surface due to the construction and/or weight of water tank ▪ The water tank was assumed safely supported by the raft footing ▪ Water table at about 10 m below the crest of the slope ▪ Prior tension cracks
	g	▪ Post-trimming and post-subsidence of slope surface due to construction and/or weight of water tank ▪ A pressure line of 1520 $kNm^{-1}m^{-1}$ (i.e., from the weight of the filled water tank) was considered in the analysis ▪ Water table at about 25 m below the crest of the slope ▪ Prior tension cracks
	h	▪ Post-trimming and subsided slope surface due to construction and/or weight of water tank ▪ A pressure line of 1520 $kNm^{-1}m^{-1}$ (i.e., from the weight of the filled water tank) was considered in the analysis ▪ Water table at about 10 m below the crest of the slope ▪ Prior tension cracks

Table 4: Descriptions and assumptions for simulating the local stability of CS2 cross-section.

Cross-section	Simulation scheme	Descriptions & Assumptions
CS2 (Local analysis)	a	▪ Empty/dry tension cracks ▪ Used soil properties similar to global analysis for soil layers 2, 3, and 4 ▪ Water table at about 10 m from the slope surface ▪ Water tank was adequately supported by the raft footing ▪ Sheet pile and RC piles were not considered as reinforcement but as retaining structure to lateral force/pressure or sliding soil mass ▪ Selected shear strength parameters for soil layer 1 ❑ Similar soil properties as for the global analysis (c = 30 kPa and ϕ = 10°) ❑ c = 20 kPa and ϕ = 10° ❑ c = 10 kPa and ϕ = 10°
	b	▪ Half-filled tension cracks ▪ Use soil properties similar to the global analysis for soil layers 2, 3, and 4 ▪ Water table at about 10 m from the slope surface ▪ Water tank was adequately supported by the raft footing ▪ Sheet pile and RC piles were not considered as reinforcement but as retaining structure to lateral force/pressure or sliding soil mass ▪ Selected shear strength parameters for soil layer 1 ❑ Similar soil properties as for the global analysis (c = 30 kPa and ϕ = 10°) ❑ c = 20 kPa and ϕ = 10° ❑ c = 10 kPa and ϕ = 10°
	c	▪ 100 % filled tension cracks ▪ Used similar soil properties of the global analysis for soil layers 2, 3, and 4 ▪ Water table at about 10 m from slope surface ▪ Water tank was adequately supported by the raft footing ▪ Sheet pile and RC piles are not considered as reinforcement but as retaining structure to lateral force/pressure or sliding soil mass ▪ Selected shear strength parameters for soil layer 1 ❑ Similar soil properties as for the global analysis (c = 30 kPa and ϕ = 10°) ❑ c = 20 kPa and ϕ = 10° ❑ c = 17 kPa and ϕ = 10°

Assumption: c was reduced due to saturation of slope via infiltration or temporary perched water table

The assumptions made for various schemes of simulations were based on the following factors:

a) Pore-water changes via changes in location of water table or perched water table
b) Load of the water tank as pressure line input
c) Reduction of shear strength parameters (cohesion, *c* and angle of internal friction, ϕ) for back-analysis simulation for F_s=1.0 (Wesley and Leelaratnam, 2001) and for local slope stability analysis for tension crack problems

The most critical cross-section in this study was CS2, where the sheet pile wall at the toe of the slope had been pushed by a combination of lateral pressure of soil mass adjacent to the sheet pile and mobilized shear force along the slip surface. In order to simulate the occurrence of soil mass movement globally and the failure of the sheet pile wall, both global movement and localized tension crack failures were performed. The generated geometry of slope for the local slope stability analysis extends from the coordinates (84, 70.092) and (289.31, 37.618) in the simulation. The coordinate (84, 70.092) was the location of tension cracks and slump after the completion of earthwork in December 1999. A heave had occurred between the sheet pile wall and the FKM's new laboratory (coordinate (289.31, 37.618)).

For the slope stability analysis using SLOPE/W software, the uniform pressure (kNm^{-2}) has to be converted to a pressure line of unit of $kNm^{-1}m^{-1}$. For the simulation works, the uniform pressure of the filled water tank was approximated at 250 kNm^{-2}. With the tank's height of 6.096 m, the pressure line used in the slope stability analysis was 1520 $kNm^{-1}m^{-1}$. The pressure line for the weight of the filled water tank was applied in the analysis if the raft footing is assumed to be inadequate to support the filled water tank. For these simulation works, the variations in coefficient of permeability on moisture or pore-water pressure conditions was not considered. In other words, the coefficient of permeability of the soil in slopes was assumed constant throughout the analysis.

3. Results

The results of the factor of safety for slope stability in each simulation are presented in similar sequence as tabulated in the previous section. The results on overall slope stability for cross-sections CS1, CS3 and CS4 are shown in Table 5 and the results of cross-section CS2 are shown in Tables 6, 7 and 8.

In overall, the slope profiles of cross-sections CS1, CS3 and CS4 are safe as the factor of safety values found are between 1.189 and 1.676 for moisture conditions ranging from dry to near saturation. However, slope-inspection activities should be planned for CS1, CS3, and CS4 especially during the monsoon season (October to December) as the saturation of slope surface may cause reduction of shear strength which will result in a decrease of factor of safety of slope stability. For cross-section CS4, for $F_s = 1.242$, the gabions at the toe of the slope is about 1 m away from the slip surface. The gabions are expected to be able to retain soil mass or force of 257.17 kNm^{-1} in cases of rising of the water table or the toe of the slope is saturated via infiltration.

The results of cross-section CS2 for global slope stability analysis show the importance of efficiency of the raft footing foundation for the water tank. In a situation where the tank is unable to support the load of filled water tank, the slope adjacent to the tank will no longer stable at both water table 25 m and 10 m below the crest of the slope. The results also show the importance of maintenance program for monitoring of settlement and leakage of the water tank for example, the leakage of the water tank could have been due to the differential settlement of raft foundation as a result of differential subsidence of underlain soil or adjacent soil mass.

Table 5: Factor of safety for global slope stability analysis of CS1, CS3 and CS4.

Cross-sections	Simulation scheme	Descriptions & Assumptions	Factor of safety
CS1	a	▪ Original/natural slope prior water tank ▪ Dry slope	1.235
	b	▪ Original/natural slope prior water tank ▪ Water table at about 25 m below the crest of the slope	1.189
	c	▪ Post-trimming with water tank ▪ The water tank was assumed safely supported by the raft footing ▪ Water table at about 10 m below the crest of the slope	1.153
CS3	a	▪ Post construction of the water tank ▪ Dry slope	1.676
	b	▪ Post construction of the water tank ▪ Water table at about 10 m below the crest of the slope	1.328
CS4	a	▪ Slope between Einstein Laboratory (Institut Ibnu Sina) and gabions at the toe of the slope, adjacent to a road ▪ Water table at 7 m and 4 m below the crest and toe of the slope, respectively	1.242

The results at cross-section CS2 for global and local slope stability analysis show that a net pressure of about 1200kNm^{-2}m^{-1} was generated during soil movement downhill adjacent to the installed sheet pile. This value is about 8 times higher than the capacity or strength of the pile of 145kNm^{-2}m^{-1} (OMK, 2000; Clayton et al., 1993). These results justify the reason for the sheet pile failure. It is recommended to install retaining structures at the top and the toe of the slope as remedial measures to safeguard the water tank and the FKM's new laboratory buildings. The strength of new retaining structures must be able to resist the net lateral pressure as shown in Table 8.

Table 6: Results of possible combination of *c* and ϕ values at failure (i.e., $F_s \approx 1.0$) for global slope stability analysis of CS2 prior to the construction of water tank for simulation scheme CS2-d.

Trial No.	Cohesion, *c* (kPa)	Friction angle, ϕ (°)	Calculated F_s	Slip surface location with respect to x-distance of generated geometry in simulation
1	20	5	1.132	x = - 21 to 224 m (deep-seated slip starts from location of proposed water tank)
2	15	5	1.125	x = - 21 to 223 m (deep-seated slip as above trial no.1)
3	10	5	1.113	x = - 21 to 223 m (deep-seated slip as above trials)
4	10	4	1.108	x = - 21 to 223 m (deep-seated slip as above trials)
5	8	6	1.077	x = - 233 to 319 m (localized slip starts from upper slope at the proposed parking area to road adjacent to FKM's new laboratory)
6	5	7	1.039	x = 232 to 322 m (localized slip similar to trial no. 5)
7	3	8	1.039	x = 232 to 322 m (localized slip)
8	1	9	1.045	x = 231 to 322 m (localized slip)
9	0	10	0.483	x = 230 to 321 m (localized slip)

Table 7: Factor of safety for global slope stability analysis of CS2.

Cross-section	Simulation scheme	Descriptions & Assumptions	Factor of safety
CS2 (for global analysis)	a	▪ Original/natural slope prior to the construction of water tank or other subsequent developments ▪ Dry slope condition	$F_s = 1.509$
	b	▪ Original/natural slope prior to the construction of water tank or other subsequent developments ▪ Water table at about 25 m from slope surface	$F_s = 1.407$
	c	▪ Original/natural slope prior to the construction of water tank or other subsequent developments ▪ Water table at about 10 m from slope surface	$F_s = 1.176$
	d	▪ Original/natural slope prior to the construction of water tank or other subsequent developments ▪ Water table at about 10 m from slope surface ▪ Back-analysis for Fs ≈ 1.0 for determination of values of *c* and ϕ at failure via trial-an-error method ▪ The *c* value is not necessarily zero as usually assumed in manual-calculated back-analysis method	Please see Table 7 for a summary on possible combination of c and ϕ values at failure (i.e., at $F_s \approx 1.0$)
	e	▪ Post-trimming and subsided slope surface due to the construction and/or weight of water tank ▪ The water tank was assumed safely supported by the raft footing ▪ Water table at about 25 m below crest of the slope ▪ Prior tension cracks	$F_s = 1.411$
	f	▪ Post-trimming and post-subsidence of slope surface due to the construction and/or weight of water tank ▪ The water tank was assumed safely supported by the raft footing ▪ Water table at about 10 m below crest of the slope ▪ Prior tension cracks	$F_s = 1.206$
	g	▪ Post-trimming and post-subsidence of slope surface due to the construction and/or weight of water tank ▪ A pressure line of 1520 $kNm^{-1}m^{-1}$ (i.e., from the weight of the filled water tank) is considered in the analysis ▪ Water table at about 25 m below crest of the slope ▪ Prior tension cracks	$F_s = 0.402$ (Slope fails)
	h	▪ Post-trimming and subsided slope surface due to the construction and/or weight of water tank ▪ A pressure line of 1520 $kNm^{-1}m^{-1}$ (i.e., from the weight of the filled water tank) was considered in the analysis ▪ Water table at about 10 m below crest of the slope ▪ Prior tension cracks	$F_s = 0.364$ (Slope fails)

Table 8: Results of safety factor (F_s) for local slope stability analysis of CS2 after the construction of new laboratory and sheet pile wall (assume no changes in ϕ of the first layer soi1).

Simulation scheme	Descriptions & Assumptions	Factor of safety (F_s) & Remarks
a	Empty/dry tension cracks ▪ Used similar soil properties as of the global analysis for soil layers 2, 3, and 4 ▪ Water table at about 10 m from slope surface ▪ Water tank was adequately supported by the raft footing ▪ Sheet pile and RC piles were not considered as reinforcement but as retaining structure to lateral force/pressure or sliding soil mass ▪ Selected shear strength parameters for soil layer 1 ❑ Similar soil properties as of global analysis (c = 30 kPa and ϕ = 10°) ❑ c = 20 kPa and ϕ = 10° ❑ c = 10 kPa and ϕ = 10°	i) F_s = 1.803 Deep-seated slip surface The slice no.35 adjacent to sheet pile has a net lateral pressure of 1137.38 $kNm^{-2}m^{-1}$ compared to the sheet pile capacity of 145 $kNm^{-2}m^{-1}$ only. The slice no.36 which is between the sheet pile and road adjacent to FKM's new laboratory shows a net pressure of 200 $kNm^{-2}m^{-1}$, causes heaving of soil under the road pavement. ii) F_s = 1.641 Localized tension crack slip at x of 143 m to 147 m iii) F_s = 1.023 Approaching tension cracks slip failure at x of 143 m to 147 m
b	Half-filled tension cracks ▪ Same assumptions as scheme a ▪ Selected shear strength parameters for soil layer 1 ❑ Soil properties same as of global analysis (c = 30 kPa and ϕ = 10°) ❑ c = 20 kPa and ϕ = 10° ❑ c = 10 kPa and ϕ = 10°	i) F_s = 1.803 Deep seated slip surface with a net lateral pressure of about 1200 $kNm^{-2}m^{-1}$ versus capacity of sheetpule of 145 $kNm^{-2}m^{-1}$ only. The slice no.36 which is between the sheet pile and road adjacent to FKM's new laboratory shows a net pressure of 235 $kNm^{-2}m^{-1}$, causes heaving of soil under the road pavement. ii) F_s = 1.528 Localized tension crack slip at x of 87 m to 94 m iii) F_s = 0.951 Tension cracks slip failure at x of 87 m to 94 m
c	**100 % filled tension cracks** ▪ Same assumptions as schemes a and b ▪ Selected shear strength parameters for soil layer 1 ❑ Similar soil properties as of global analysis (c = 30 kPa and ϕ = 10°) ❑ c = 20 kPa and ϕ = 10° ❑ c = 17 kPa and ϕ = 10°	i) F_s = 1.586 Localized tension crack slip at x of 274 m to 279 m ii) F_s = 1.152 Localized tension crack slip at x of 274 m to 279 m iii) F_s = 0.973 Localized tension crack slip failure at x of 276 m to 278 m

4. Conclusions

The main contributing factor for the slope instability surrounding the FKM's new laboratory was the effects of load of the filled water tank. Initially, the applied pressure of the filled water tank had caused the occurrence of subsidence of the slope surface within the vicinity. It also created tension cracks in the upper slope/terrace near the raft footing of the water tank. The simulation showed that the global soil mass movement started from the hill top where the water tank was located to downhill direction toward the installed sheet pile wall at the toe of the final slope adjacent to the parking area of the FKM's new laboratory.

The combined mobilized shear force and the lateral pressure of the global slope as a result of soil mass movement was 8 times higher than the strength of the sheet pile wall. Therefore, it can be concluded that the offset of the sheet pile wall was due to the global soil mass movement along the slip surface. The result also confirmed the occurrence of heave pushing up the soil at the toe of slope and soil under the road pavement adjacent to the new laboratory. The slope became more unstable as moisture or pore-water pressures in the slope increased due to heavy rainfall events prior to and during earthwork. The infiltration of rainwater into the slope has reduced the shear strength particularly the cohesion, c value. It is important to determine effects of pore-water pressure in both global and localized slope stability as a requirement in approving any proposal for development.

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