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ABSTRACT: Rainfall-induced slope failures often occur in steep residual soil slopes with a deep groundwater table. An understanding of the slope failure conditions and effective remedial measures can be achieved by comprehensive numerical modelling and application of unsaturated soil mechanics principles. This paper presents two case studies of slope failures in Singapore in which analysis of pre and post failure conditions of the slopes were performed using a finite element model based on unsaturated soil mechanics principles. The analyses of pre-failure conditions was performed to confirm the observation made of the failure conditions through a back analysis on the slope and the analyses of post-failure conditions was performed to investigate the effectiveness of hypothetical remedial works for the slope after the landslide. The analysis suggests an understanding of the failure conditions in a slope and effective post-failure slope stabilization strategies could be achieved through the application of unsaturated soil mechanics principles.

1 INTRODUCTION

Singapore is located in a tropical region where heavy rainfall and high temperature provide conditions for rapid and thorough in-situ chemical and mechanical weathering that results in the development of deep residual soil profile. Lying in a tropical belt where residual soil is a dominated feature of the geology, minor landslides are a common occurrence in many regions of Singapore (Pitts, 1985). The landslides in Singapore are mainly shallow slips and take place during periods of heavy rainfall.

Toll *et al.* (1999) examined the association between slope failures, slope height and slope angle using data from Pitts (1985), Tan *et al.* (1987; 1988), Lo *et al.* (1988), and Wei *et al.* (1991) on 35 slope failures in the Bukit Timah and the Jurong formation in Singapore. Toll *et al* (1999) indicated (Figure 1) that although there were no apparent relationship between slope height and slope angle of the failed slopes the majority of the failures (97%) occurred in slopes with slope angle greater than or equal to 27^o (Figure 1).

Negative pore-water pressure or matric suction plays a crucial role in the stability of earthworks. Although its importance has been identified, it is not often understood and therefore ignored in much geotechnical design. Common engineering practice usually involves the evaluation of the saturated component of soil under steady state conditions in a simple hydrogeologic setting (Fredlund and Rahardjo, 1994). In doing so, a significant contribution from the unsaturated zone is ignored and therefore application of conventional methods often lead to unrealistic calculations that indicate failure conditions while in reality the earthworks remain safe. Rulon *et al.* (1985) explained how heterogeneous slopes are characterised by complex interactions between saturated and unsaturated systems, perched water tables, and multiple seepage faces. These are characteristics of the residual soils in tropical regions.

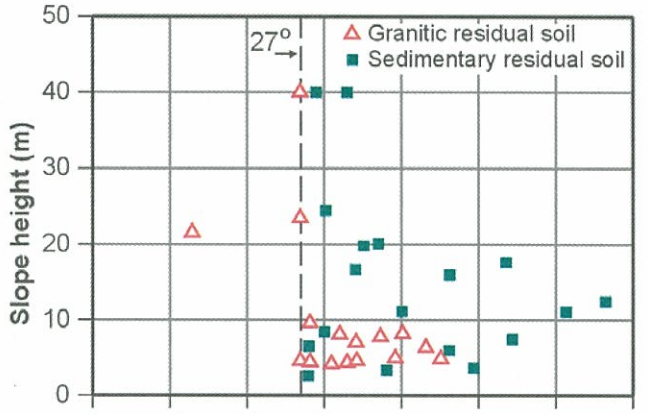


Figure 1. Slope height against slope angle for landslides in Singapore (from Toll *et al.*, 1999)

Avoiding principles of unsaturated soil mechanics can be attributed to two main reasons. The first reason is the development of unsaturated soil mechanics has lagged in comparison to saturated soil mechanics. Secondly, those who are aware of the strength contribution attainable from unsaturated soil will often question whether or not the negative pore-water pressure can be maintained throughout the lifetime of a design. Since this is often not guaranteed, the risk of incorporating unsaturated soil mechanics principles into design is not warranted.

A slope failure due to a rising water table should not always be the only concern because the development of a wetting front can be equally destructive. During rainfall, even though the water table may be significantly deep and unaffected, the destruction of negative pore-water pressures in the vadose zone may be enough to produce a failure condition (Yong *et al.*, 1985; Tan *et al.*, 1987; Pitts, 1985; Fredlund and Rahardjo, 1994). The result is a rainfall-induced slope failure. With proper drainage, it may be possible to reduce much of the infiltration. Thus it is expected that the negative pore-water pressure or matric suction will not be destroyed and can therefore be included in design.

The objective of this paper is (i) to draw attention to the considerations for unsaturated soil mechanics principles in slope stability analysis and (ii) to show through case studies how the principles of unsaturated soil mechanics can be applied effectively in arriving at an understanding of the critical conditions for a slope and how numerical analysis can be used in an effective design of earthworks in residual soils for stability.

2 CASE STUDY OF BUKIT BATOK SLOPE FAILURE

A landslide occurred at the Bukit Batok Sports Complex in Singapore which had been deemed one of the largest slides in Singapore (Wei *et al.* 1991). Failure occurred at midnight of 2 December 1989, approximately 24 hours after an extremely heavy period of rain (151 mm in three days). This period was in the midst of the monsoon season and the preceding (19 days earlier) rainfall had also been an exceptionally heavy day of rain (112 mm).

2.1 *Site Observation*

A detailed post failure site investigation revealed that the failed slope, located in the Gombak Norite outcrop, was a residual soil of highly decomposed granite rising about 50m above the stadium ground level. The slope was cut at approximately 1 vertical to 2 horizontal leaving about 14m of residual soil at the crest which tapered to only a thin layer near the toe. Below this layer was the rock formation sloping at a lesser angle of approximately 15 degrees.

After the slide, debris spanned 100m across the football field. The slide extended approximately 56m from the toe to a 4m to 6m deep backscar. The slip surface was approximately 0.5m to 1.0m above the underlying rock. Five days after the slide, water was still seeping out of the slope at elevation 164.0m indicating that the water table was about 2.6m below the slope surface. The appearance of water at this location meant that a lot of water still remained in the slope but more importantly, the water table had likely risen to the slope surface at the time of failure.

A summary of soil properties for the residual soil layer is listed in Table 1. Effective strength parameters, c′ and ϕ', were obtained from consolidated undrained (CU) triaxial tests on saturated samples with pore-water pressure measurements.

Table 1. Soil properties of slope at Bukit Batok Sports Complex (from Wei *et al.*, 1991)

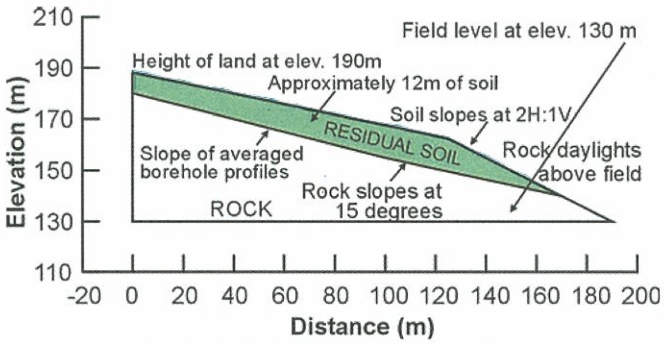
Properties	Max.	Min.	Avg.	Std.
Effective cohesion, c'(kPa)	33.00	3.00	10.59	4.84
Angle of friction, ϕ'(^{o})	44.50	25.00	36.41	2.93
In-situ water content, w (%)	64.00	17.00	43.92	9.59
In-situ total density, ρ_t (Mg/m^3)	2.00	1.22	1.73	0.11
Liquid limit, *LL* (%)	100.00	29.00	61.18	13.31
Plastic limit, *PL* (%)	50.00	17.00	35.64	6.00
Degree of saturation, S (%)	103.10	53.10	94.34	7.80
In-situ void ratio, e	2.283	0.690	1.245	0.253

Max. = Maximum; Min. = Minimum; Avg. = Average;
Std. = Standard deviation.

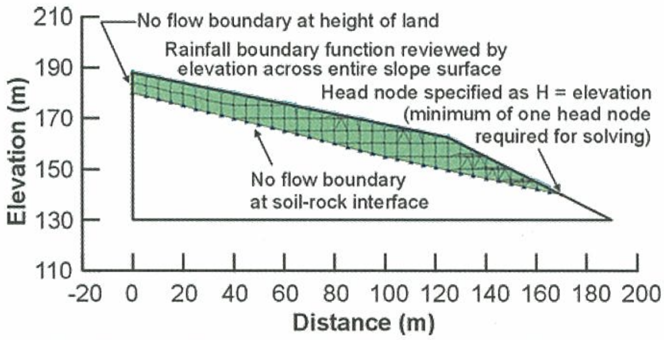
2.2 *Analysis of pre-failure conditions*

The analysis of pre-failure conditions (Hritzuk, 1997) was performed using a finite element seepage program Seep/W (Geoslope International Ltd, 1992) and a slope stability program Slope/W (Geoslope International Ltd, 1991). The slope profile is shown in Figure 2a. The finite element mesh for the slope model (Figure 2b) was generated for the soil only. The rock that existed below the soil layer was assumed to be impermeable and therefore the interface between the rock and soil was also specified as a no flow boundary. The leftmost edge of the model was specified as a "no flow" boundary. A previous site investigation has shown that the properties of this soil layer are rather consistent throughout the slope therefore the same soil properties could be applied to the entire soil layer. Upon the soil surface, a flux boundary function with respect to time was applied to represent the annual rainfall events. This boundary condition was reviewed by elevation which is a feature in the Seep/W program to prevent calculated nodal head values from being larger than their respective elevation which would be a ponding condition.

The analyses included the unsaturated part of the slope; therefore, information on the unsaturated properties of the soil was required. Average values of soil properties shown in Table 1 were used in the analysis together with test results from block samples taken from the residual soil layer of the slope at the crest.



(a) Bukit Batok slope configuration



(b) Finite element mesh configuration for Bukit Batok slope model

Figure 2. Seepage model for the Bukit Batok slope (a) Slope configuration; (b) Finite element mesh configuration.

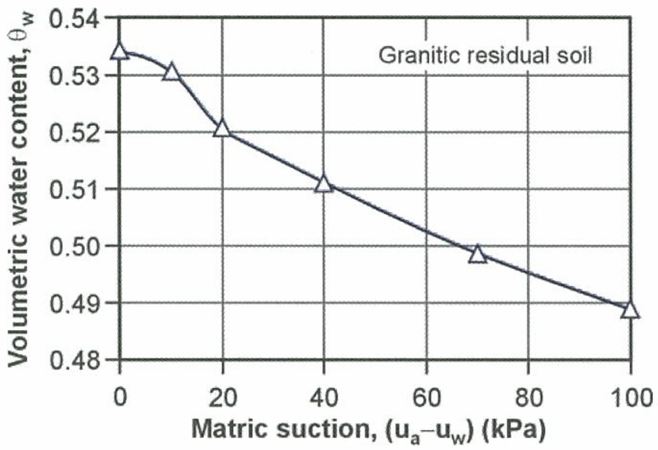


Figure 3. Soil-water characteristic curve for soil at Bukit Batok slope

A soil-water characteristic curve (Figure 3) was established from laboratory testing of the block samples by using Tempe Cells. Block samples were obtained from the residual soil layer near the crest of the slope at a depth of approximately 1.0m. Using the saturated permeability, k_s, of 1×10^{-6} m/s reported by Wei *et al.* (1991) and the soil-water characteristic curve (Figure 3), the permeability function of the soil (Figure 4) was derived from the model originally proposed by Childs and Collis-George (1950) as described in Fredlund and Rahardjo (1993).

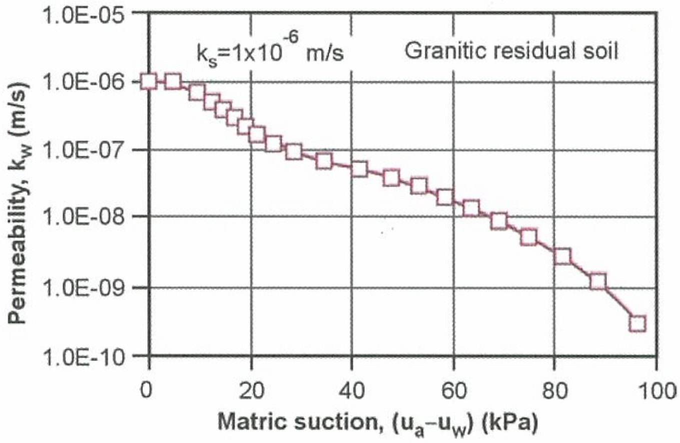


Figure 4. Permeability function of soil at Bukit Batok slope

The value of k_s reported by Wei *et al.* (1991) may be considered high for an intact residual soil sample but in this case, k_s as a field value is reasonable as it reflects the bulk permeability of the slope. It has been shown by Karrupiah and Taranam (1996) and Lim (1994) that bulk k_s in the field can be 2 to 3 orders of magnitude greater than that of an intact laboratory sample due to the presence of roots and other macro-features such as root holes and sand seams. Figure 5 and Table 2 show the monthly rainfall variation at Bukit Batok.

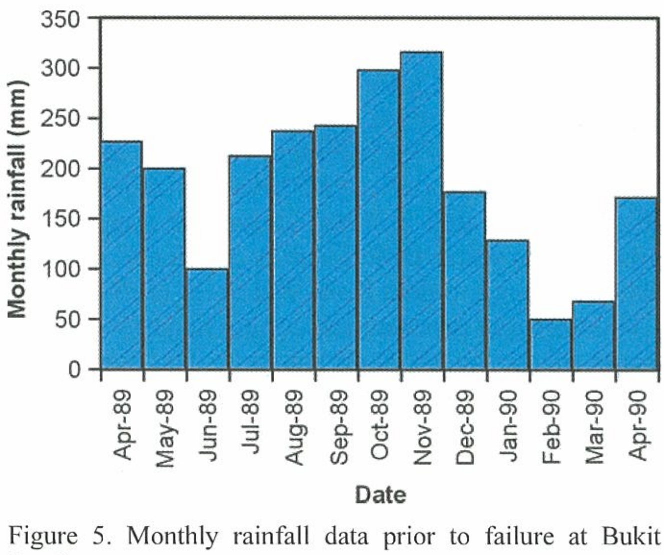


Figure 5. Monthly rainfall data prior to failure at Bukit Batok

In order to establish the initial steady state in the model, the catchment area around the site was first studied. It was assumed that the highest elevation of land in the catchment area could be deemed as the water divide. As such, all rainfall on a particular side of the height of land would move downslope and become part of this particular water regime. Rainfall occurring in any other areas outside the catchment area would move away from the slope and thus be of no influence.

Table 2. Monthly rainfall and ground water levels at Bukit Batok (from Wei *et al.*, 1991)

Period. End of	AMR (mm)	AMCRR (m/s)	AGWL (m)
Mar. 31-89	226	8.72E-08	6.9
Apr. 30-89	226	8.72E-08	7.6
May 31-89	200	7.72E-08	8.1
Jun. 30-89	100	3.86E-08	7.6
Jul. 31-89	213	8.22E-08	8.2
Aug. 31-89	238	9.18E-08	8.9
Sep. 30-89	242	9.34E-08	9.5
Oct. 31-89	298	1.15E-07	10.6
Nov. 30-89	316	1.22E-07	11.9
Dec. 31-89	176	6.79E-08	11.1

AMR = Actual monthly rainfall; AMCRR= Average monthly continuous rainfall rate; AGWL = Calculated ground water level above rock with respect to ground surface elevation of 164 m

Since it is unreasonable to analyse every rainfall event, a total of five time instances were used to best represent the actual rainfall data. Beginning at the end of March, rainfall data on time instances at 1 day, 15 days, 91 days, 122 days, and 246 days were considered. These time instances on a calendar time scale represent April 1, April 15, June 30, and July 31, respectively with day 246 representing the failure date 2 December 1989.

Wei *et al.* (1991) provided a derivation for the fluctuation of the groundwater table in this slope from 1986 to 1990 based on a monthly water mass balance. The results were found to compare well with the observed groundwater levels as measured with respect to the surface elevation of 164m. This groundwater level information served as a point of reference for this model study. A steady state solution was achieved by applying a constant rate of rainfall or water flux of 1.4×10^{-8}m/s causing the water table to be located at 6.9m above the rock surface. This was the water table location at the end of March 1989 according to data from Wei *et al.* (1991).

The transient process was commenced by using the results of the steady state analysis. The steady state rainfall of 1.4×10^{-8}m/s was replaced with an equivalent continuous rainfall function based on monthly totals for the time period up until the date of failure. For example, the total monthly rainfall for April 1989 was 226 mm over 30 days. As a continuous rate, this is equivalent to 8.72×10^{-8}m/s. Each month on record is of a different total rainfall (see Figure 5 and Table 2) and therefore the resulting flux boundary is a dynamic function. It was found that using the average monthly continuous rainfall rate as the flux boundary function would cause the water table to rise too quickly over the modelled time frame as compared to the reported movement. This flux boundary function was comprised of flux rates in the order of 10^{-8}m/s to 10^{-7}m/s. According to the permeability function shown in Figure 4, the soil could accept such a rate as long as matric suction was less than approximately 60 kPa. At the steady state condition, the matric suction was only slightly greater than 60 kPa and this only occurred at the surface of the slope. Once rainfall was introduced into the transient analysis, the matric suction was quickly reduced and the soil easily accepted all the rainfall that was applied.

However, this was not the case in the actual situation because the rainfall did not occur at a continuous rate over several months prior to failure. It occurred in short irregular intervals, perhaps only in a few hours, at rates which were significantly higher than the saturated permeability. Thus the majority of rainfall left the slope as runoff. It was not possible to model such diversity in the boundary function. Therefore, the rainfall boundary function was averaged over an extended period and the rainfall rates were reduced proportionately until the modelled water table movement over the months up to the date of failure was best matched with the reported movement by Wei *et al.* (1991). The best match occurred when each total monthly rainfall used for the flux boundary condition was reduced to 30% of the actual total. Seep/W has a "flux line" feature used to measure flow across any cross section drawn in the model prior to the analysis. When these flux lines were drawn across the slope surface, the modelled infiltration was found to be even less than the applied rainfall rate of 30%. This reflects the significance of the permeability function shown in Figure 4 which controls the amount of infiltration according to the water content at any given time.

The modelled transient process in Figure 6 shows the water table fluctuation over the eight month period until the failure date. The modelled slope shows a nearly full saturated condition on 2 December 1998 as was reported by Wei *et al.* (1991).

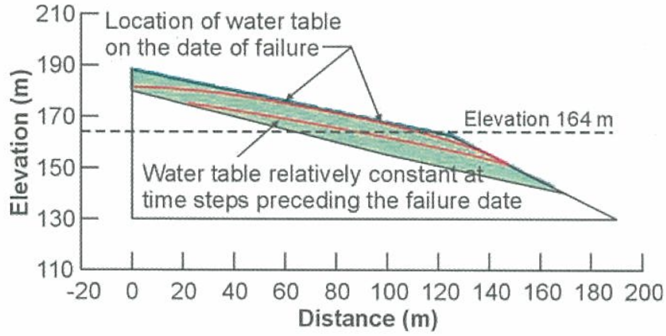


Figure 6. Transient processes for the water table

Table 3 lists the results of the water table movement and infiltration values obtained from the model. The results are based on the assumption that rainfall can be converted into a continuous monthly rate, the method of which was explained earlier. Since Seep/W offers graphical output in scaled dimensions, the water table location could be measured for each time step. Flux lines, which are a feature of the

Seep/W program, were drawn covering a surface distance of 50m along the slope over the slip area. Net movement of water volume across this zone into the slope was thus calculated by the program for each time step. This allowed for the determination of the average infiltration since the area through which the infiltration passed was known (i.e. length of flux line multiplied by the unit width of slope).

Table 3. Results of finite element model simulation

Time Step	Date	MCRR (m/s)	MGWL (m)	INF (%)	MINF (m/s)
1	Apr. 1-89	2.62E-08	6.9	19%	1.67E-08
2	Apr. 15-89	2.62E-08	7.4	24%	2.12E-08
3	Jun. 30-89	1.16E-08	6.9	29%	1.11E-08
4	Jul. 31-89	2.47E-08	7.6	25%	2.02E-08
5	Dec. 2-89	3.66E-08	11.8	23%	2.83E-08

MCRR=Modeled continuous rainfall rate as 30% of actual continuous rainfall; MGWL=Modeled GWL above rock with respect to ground surface elevation of 164m.; INF=Infiltration as % of average actual monthly continuous rainfall (Table 2); MINF=Modeled infiltration

The derivation used by Wei *et al.* (1991) to calculate the water table movement, incorporated a runoff coefficient of 0.75. In other words 25% of rainfall was assumed to infiltrate the slope. Table 3 lists the modeled infiltration for each time step (i.e., INF in Table 3) ranging from 19% to 29% with an average of 24%. This agrees very well with the infiltration rate reported by Wei *et al.* The results found in this table are combined with those of Table 2 and presented graphically in Figure 7.

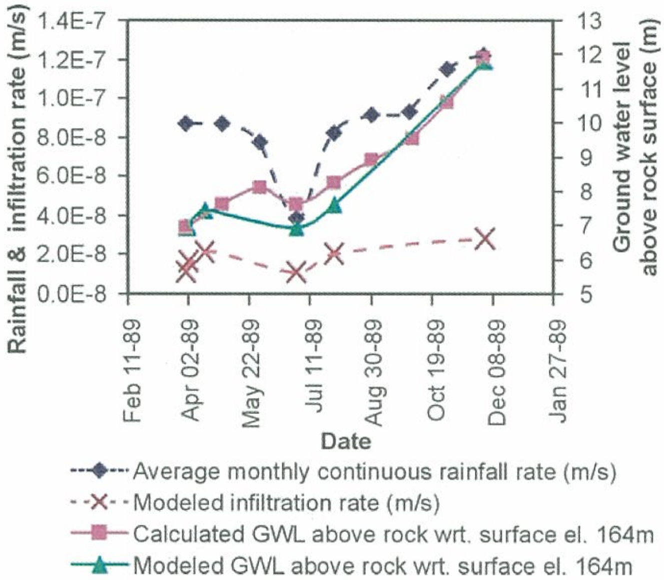


Figure 7. Dynamics of groundwater level with respect to rainfall at elevation 164m

Hydraulic conditions on the failure date were extracted from the Seep/W data file in the form of nodal head values and transferred into the slope stability program Slope/W. Soil data required for the analyses were obtained from the average values listed in Table 1. In addition, the ϕ^b value was estimated at $2/3\phi'$ (Fredlund and Rahardjo, 1993). Bishop's analysis method, where interslice normal forces are considered but interslice shear forces are assumed to be zero, was used in the analysis. Potential slip surfaces were set parallel to the slope of the rock surface near the toe. The critical slip surface is shown in Figure 8. The calculated factor of safety for Bishop's analysis fell from 1.35 prior to the rising of the water table to 1.05 which basically indicates an unstable condition.

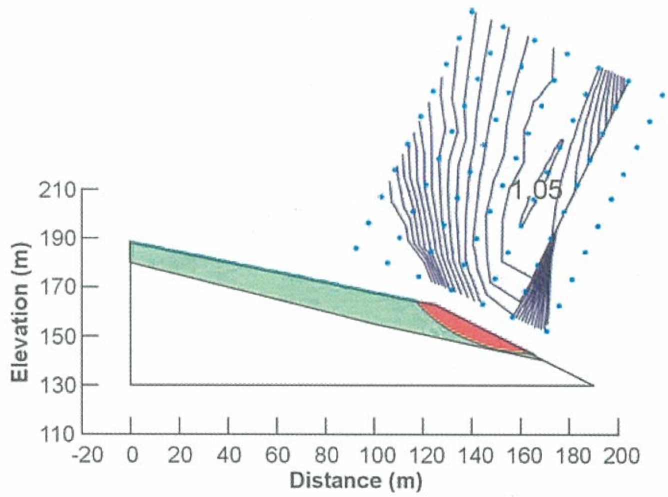


Figure 8. Result of slope stability analysis for the final time step condition

2.3 *Analysis of post-failure conditions*

In order to isolate the effectiveness of different re-post-failure remedial measures, three more numerical analyses were conducted (Hritzuk, 1997) in addition to the analysis of pre-failure conditions (Case 1). These were;

- Case 2: Analyses of a hypothetical situation of the original slope with horizontal drainage only.
- Case 3: Analyses of a hypothetical situation of the slope wit slope flattening only.
- Case 4: Analyses of a hypothetical situation of the slope with two remedial measures adopted simultaneously i.e. slope flattening with horizontal drainage.

The hypothetical horizontal drainage modelling (Case 2) comprised of 15 rows of 80mm diameter perforated polyvinyl chloride (PVC) pipes with lengths ranging from 6m to 10m and spaced 2.5m centre to centre that ran longitudinally through the slope. Between section AA and CC (Figure 9, Case 1) the original slope had two gradients (the upper slope portion was at 12^o and the lower slope portion was at 26^o). The hypothetical slope flattening

(Case 3) was modelled by regrading the original slope surface to a slope angle of about 13°.

A visual representation of the 4 cases is shown in Figure 9. For each of the additional cases, the identical soil properties, initial steady state condition and time steps for the transient analysis used in Case 1 were applied. Horizontal drains were specified internally along the edges of elements as a line of zero flux and reviewed by maximum pressure. This meant that during the iterative process, drain nodes were checked for instances where pressures were calculated to be greater than zero. If such is the case, the nodal value was adjusted back down to zero pressure or in other words total head was equated to the elevation head. In the instance where pressures of the drain nodes were less than the specified zero value, the nodal values remain as such and were treated like any other node. This was representative of the actual case where one would expect drains to only be active in a saturated part of a slope. If drains were in the unsaturated part of the slope, the groundwater would move throughout the slope as if the drains were not there at all. That is, at the same elevation, the water would not flow from a zone of negative pressure (the unsaturated zone) to a zone of zero or atmospheric pressure (the drain hole). Figure 10 shows the movement of the water table for each transient seepage analysis. The stability analysis for each time step is summarised in Table 4 and also presented graphically in Figure 11.

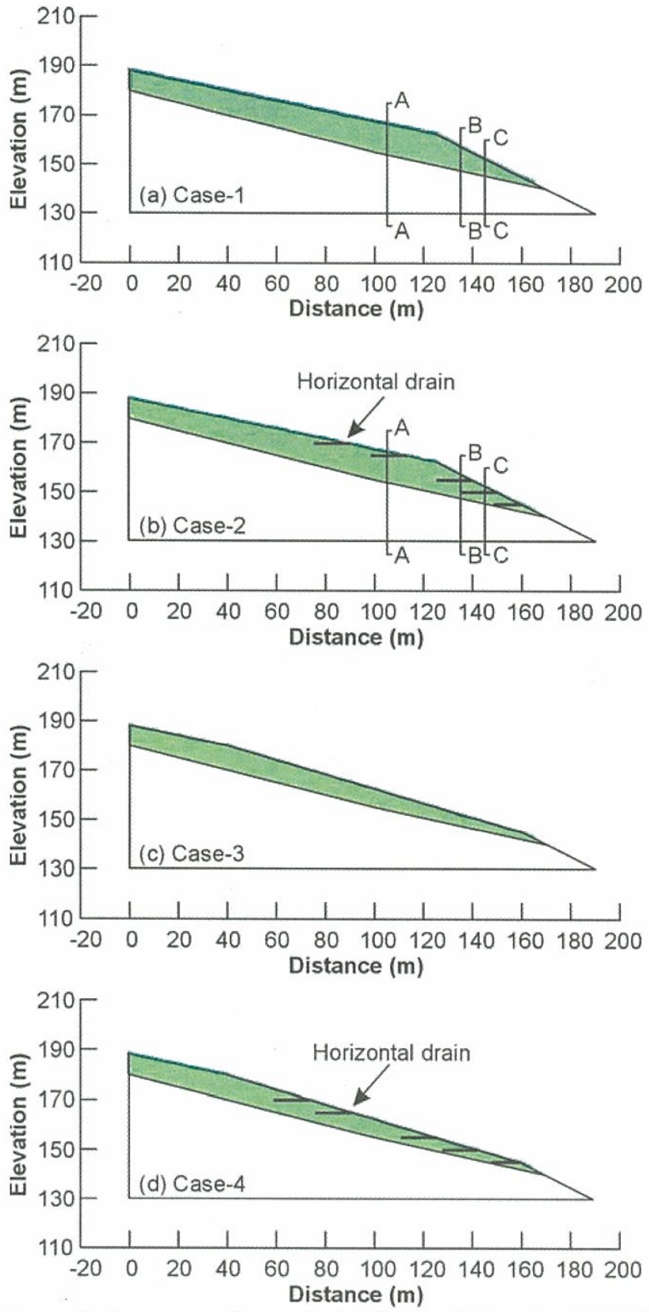


Figure 9. Four cases for analysis of remedial works at Bukit Batok

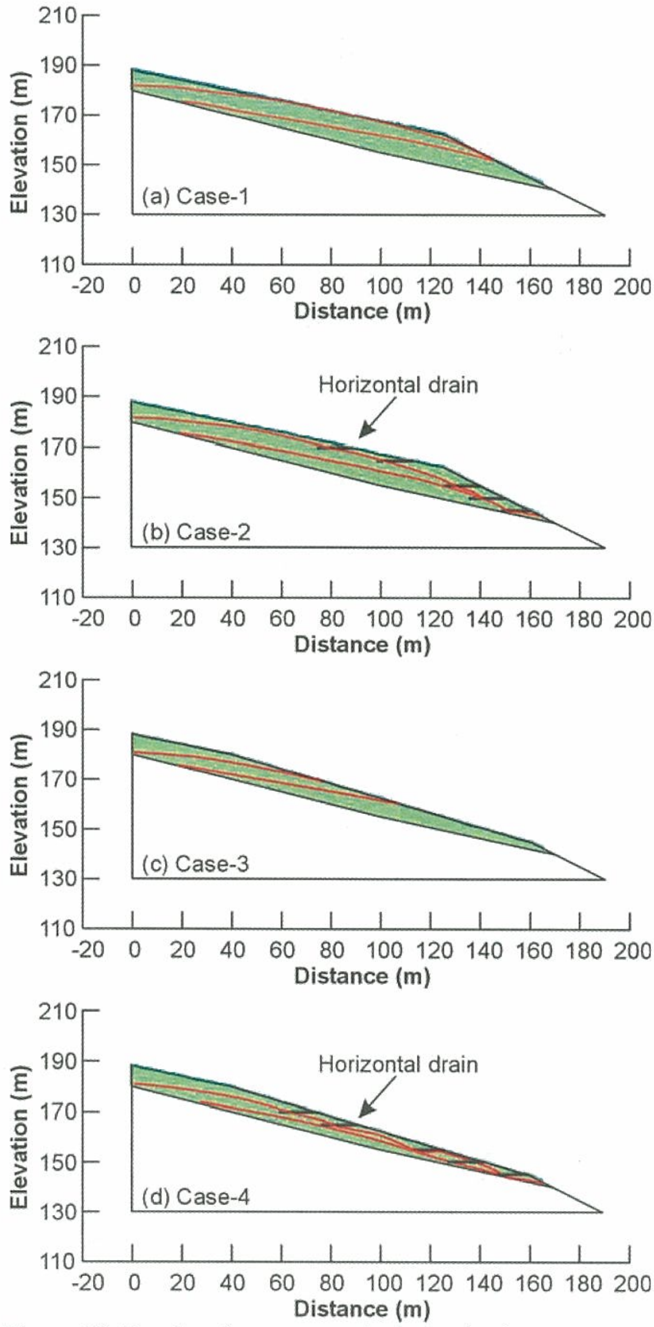


Figure 10. Results of seepage analysis for the four cases

Figure 10 shows the movement of the water table for each transient seepage analysis. The stability analysis for each time step is summarised in Table 4 and also presented graphically in Figure 11.

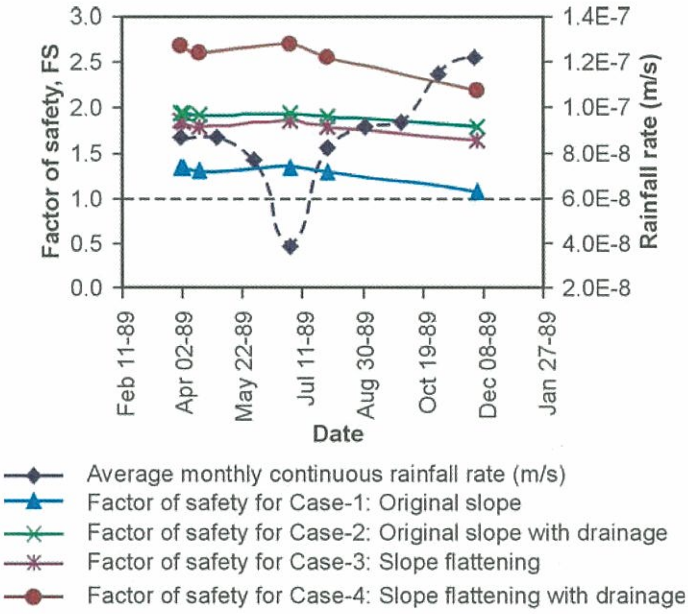


Figure 11. Dynamics of factor of safety for Cases 1 to 4

Table 4. Factor of safety for the four cases of remedial works at Bukit Batok

Time Step	Date	Case 1	Case 2	Case 3	Case 4
Steady State	Mar. 31-89	1.35	1.94	1.85	2.69
1	Apr. 2-89	1.34	1.94	1.83	2.68
2	Apr. 15-89	1.31	1.92	1.80	2.60
3	Jun. 30-89	1.35	1.95	1.85	2.70
4	Jul. 31-89	1.30	1.91	1.79	2.56
5	Dec. 2-89	1.08	1.78	1.64	2.19

2.4 *Discussion on Bukit Batok case study*

The movement of the water table and the shape and location of the slip surface obtained from the modelling process indicated that the assumptions made for the model were correct. One such assumption was that rainfall could be averaged to a constant rate of application based on monthly totals, which ranged from approximately 100 to 300 mm per month (Figure 5). The best match was achieved between the modelled and measured results when the applied rainfall rate was reduced to 30% of the original monthly total in order to match the reported water table movement. The average amount of infiltration was slightly less at 24% of the monthly total. This compared well with the 25% infiltration rate that Wei *et al.* (1991) used in their prediction of the water table movement.

The results of the seepage analysis indicate that at the time of failure the entire slope was nearly saturated. Wei *et al.* (1991) also reported the saturated condition at failure. A second assumption was that the test results could be used to represent soil conditions across the entire slope so that the residual soil could be analysed as a single layer overlying the rock. Figure 8 shows the resulting slip surface originated at elevation 165 m and extended approximately 50m to the toe. The calculated factor of safety for this model was 1.08. Wei *et al.* (1991) reported field measurements of the slip starting at elevation 170m and extending 56m to the toe. The size and dimensions of the slip surface compared well with that reported from site observations and the factor of safety indicated a nearly unstable condition. Thus the development of the soil-water characteristic curve and the governing equations for the water flow and slope stability were valid and applicable. It was concluded that the soil properties and estimated slope profile were accurate and could therefore be used for subsequent post failure analyses of the repaired slope.

It should be pointed out that the horizontal drainage system is a three dimensional (3-D) problem. The software used for the numerical modelling was limited to two dimensional (2-D) analyses. Horizontal drains were specified as a line of connected nodes along the edges of elements with a zero flux type boundary condition and were reviewed by maximum pressure. This meant that a computed pressure value for a drain boundary was prevented from being greater than zero. The 2-D analysis limitation was that the boundary condition for the horizontal drain was more representative of a drainage blanket. In the field situation, lateral spacing of the horizontal drains would be reflected in the third dimension.

The analysis was then extended to study the remediation case. As was mentioned, the remediation involved flattening of the slope angle and the addition of drains. To isolate the effects of each, separate studies were done for drainage only (Case 2), slope flattening only (Case 3) and then the actual case of both conditions (Case 4). As Figure 11 shows, either drainage or regrading alone would have sufficed as a stabilisation measure providing factors of safety of 1.78 and 1.64, respectively. With both effects combined in the model (Case 4), the factor of safety was calculated to be 2.19; a very stable situation. Recalling the original factor of safety, 1.08 (see Table 4), the improvement in factor of safety for Case 2 was (1.78−1.08) = 0.7. Similarly the improvement for Case 3 was 0.56. The benefits of these two cases combined numerically gave a net increase in factor of safety of (0.70+0.56) = 1.26. This showed a close resemblance to the increase in factor of safety for Case 4 at (2.19–1.08) = 1.11. However, this may also suggest that it was not exactly additive.

3 CASE STUDY OF NTU-ANX SLOPE FAILURE

On 26 February 1995, a heavy storm occurred on the Nanyang Technological University (NTU) Campus. The rain started at 17:02h and lasted about 2.5 hours. According to rainfall readings recorded by the School of Civil and Structural Engineering, NTU, the daily rainfall on that day was 94.6 mm. Since all the rainfall on that day took place during a 2.5 hour period, the average hourly rainfall was 37.8 mm/h. More than twenty slope failures were found on the NTU campus the following day (Rahardjo *et al.*, 2001). A field study was carried out in the morning of 27 February 1995 when the geometries of the failed slopes were measured. The morphological features of landslides were also recorded. None of these landslides could be attributed to changes in geometry or additional loading applied to the slopes.

In this section the case study of one slope failure (out of 20) at the NTU campus, hereinafter called the NTU-ANX slope is presented.

3.1 *Site observation*

The Nanyang Technological University (NTU) campus is located in the south-western part of Singapore. Compared with many other parts of the country, there is a relatively high relief around the campus. The topography comprises a series of hills and valleys ranging in height from about 21 m to 56 m above sea level and is classified as Ridge and Valley (steep, high relief). Many of the slopes were terraced during the landscaping of the campus in the 1950s. The geological formation at the NTU campus is very complicated since it has been subjected to intrusions, isoclinical overfolds and faults in the past (Moh and Associates, 1994). According to the study by Pitts (1985), the ridges consist of a series of folded silty mudrocks, sandstone and conglomerates of the Tengah and Queenstown facies of the sedimentary Jurong formation. These are covered by variable thickness of residual soil or in some cases, slope wash or colluvium. The valleys are deeply infilled with sandy and clayey soils representing the alluvial member of the Kallang formation. Limited observations have indicated that the groundwater tables on the NTU campus are at a generally low level between 6 and 9 m below the ground surface (Pitts, 1985).

The NTU-ANX was formed by fill placement in the upper part and was a natural slope with minor cuts in the lower part at the time of the construction of the Administration Annex building. The slope had been trimmed into three terraces. The angle of the upper slope was about 29° while that of the lower slope was around 25°. The slope was covered with grass.

The volume of material involved in the landslide was 45m^3. The length, width and depths of the landslide were 5m, 6m and 1.5m respectively. The slope failure was rotational in type and the slip surface position was within grade six materials. The morphological feature of the slope failure was characterised as deep backscar, convex and intact accumulation zone at the toe.

Based on site investigation four soil layers were identified in NTU-ANX slope (i) Soft to medium sandy silty clay with building debris (fill); (ii) Medium to stiff silty clay / clayey silt with sand; (iii) Stiff to hard silty clay / clayey silt with sand; (iv) Hard sandy silt to very dense silty sand. Since the first three layers of soil had similar characteristics in terms of physical properties and shear strengths, these three soils were simply considered as one layer (hereinafter called Layer-1) of soil in the failure analyses. There were no measurements of piezometric level at this location. According to Pitts (1985) the groundwater tables at the ridges of the NTU campus are typically low. In this study, the groundwater table in the area of interest was assumed to be 6m below the ground surface.

3.2 *Analysis of pre-failure conditions*

For the numerical analysis, a soil water characteristic curve (Figure 12) was determined by performing pressure plate tests. The saturated coefficient of permeability (k_s =1.193×10^{-9}m/s) was obtained from a triaxial permeameter test and the non linear relationship between the unsaturated coefficient of permeability and matric suction (Figure 13) was established using the soil-water characteristic curve and the saturated coefficient of permeability as explained earlier.

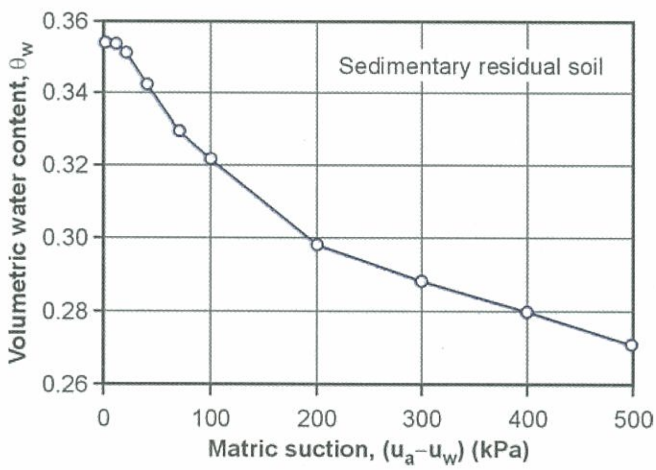


Figure 12. Soil water characteristic curve for a residual soil from the Jurong Sedimentary formation (from Rahardjo *et al.*, 2001)

As discussed earlier, two types of residual soils were involved in the landslide, namely, the Grade VI silty clay (Layer 1) and the Grade V sandy silt (Layer 2). The results of shear strength tests conducted at NTU (Lim, 1995, Rahardjo et al, 1995) were used to characterise the soils for slope stability analyses. The following values of the shear strength parameters and total unit weight were used for these two soil types. (i) Layer 1 Silty clay (Grade 6) with γ= 19.6 kN/m^3, c' =2 kPa, ϕ'=26° and ϕ^b =13°. (ii) Layer 2: Sandy silt (Grade 5): with γ=21.5 kN/m^3, c' =20 kPa, ϕ'=35° and ϕ^b =17°

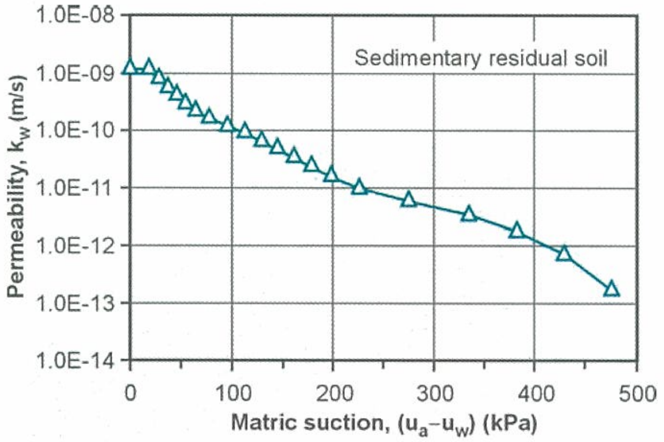


Figure 13. Permeability function for a residual soil from the Jurong Sedimentary formation (from Rahardjo *et al.*, 2001)

The rainfall patterns for the 5 days (120 hours) preceding the 26 February 1995 major rainfall event (after which the NTU-ANX slope failed) were expressed as equivalent hourly rainfall intensities and these are plotted in Figure 14.

The finite element computer program SEEP/W was used to compute the changes in pore-water pressure when the slope was subjected to different rainfall patterns. The nonlinear relationship between the coefficient of permeability and matric suction is incorporated into the program. The mesh used in the analysis comprised 418 nodes and 424 triangular or quadrilateral elements. The bottom boundary was set to zero flux. The left and right boundaries were specified as constant total head boundaries below the groundwater table and as zero total flux boundaries above the groundwater table. The total head applied corresponds to a hydrostatic condition. The slope surface was treated as a flux boundary condition. It was assumed that there would be no ponding on the surface of the slope. Therefore, a pore-water pressure greater than zero was not allowed on the slope surface.

Prior to carrying out a transient seepage analysis for the storm event it was necessary to establish some initial conditions. Firstly a steady state analysis was used to produce a hydrostatic condition. Then a transient analysis was used to establish an initial pore-water pressure (suction) profile. The initial insitu condition of pore-water pressure before a storm is governed by seasonal infiltration and evaporation. In this analysis, the pre-storm initial condition was achieved by performing a transient analysis using a low unit flux applied to the slope surface for a long duration. This low unit flux corresponds to the net influx of water into the slope over the year. The value of the applied net flux on the slope surface was adjusted by trial and error until the required initial pore-water pressure condition was achieved. According to Lim et al (1996) the average matric suction in the residual soils on the NTU campus is around 20 kPa. In this case, the initial matric suction profile in the slope was adjusted to be close to this magnitude.

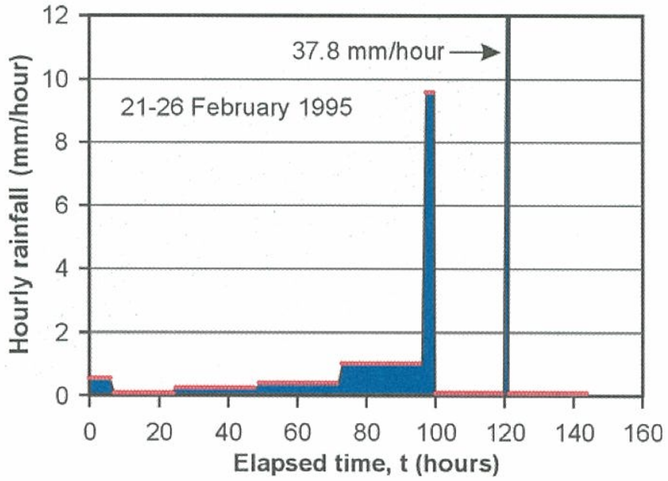


Figure 14. Rainfall intensities with elapsed time used in the NTU-ANX slope failure analysis (from Rahardjo *et al.*, 2001)

For the seepage analysis, the same soil-water characteristic curve and permeability function were used for both soil types (Layer 1 and Layer 2). Only the shear strength parameters were taken to be different for the slope stability analysis.

The computer program SLOPE/W was used to perform a limit equilibrium analysis on the landslide investigated in this study. Bishop's simplified method was used in this study. The pore-water pressure distribution computed in the program SEEP/W was imported into SLOPE/W for the slope stability analysis. The time-dependent pore-water pressure distribution could therefore be used directly to compute the factor of safety with time.

3.3 *Discussion on NTU-ANX case study*

The simulated pore-water pressure distributions within the NTU-ANX slope are shown in Figure 15 for four different elapsed times. The results show a rise in the water table that reached a maximum elevation at the end of the storm (i.e., 122.5 hours).

The factor of safety of the slope decreased from a value greater than 1.6 at 5 days before the storm to the lowest factor of safety of 1.05 at the end of the storm (Figure 16). The lowest factor of safety of 1.05 was very close to unity indicating that limiting equilibrium was reached and slope failure would be expected to occur at the end of the storm of 26 February 1995. The critical slip surface corresponding to the lowest factor of safety is shown in Figure 17.

4 CONCLUSIONS

An examination of the case studies presented suggests that numerical analysis with unsaturated soil mechanics principles could reveal the failure conditions of a slope when the initial condition, flux boundary conditions and soil properties are adequately available. Both case studies indicated that the failure conditions in the slopes were reached when the factor of safety dropped close to 1. The numerical simulation of a hypothetical situation of Bukit Batok slope with horizontal drainage indicated the effectiveness of horizontal drains in maintaining negative pore-water pressures in a slope and demonstrated how unsaturated soil mechanics principles can be used to adopt reasonable slope stabilization methods.

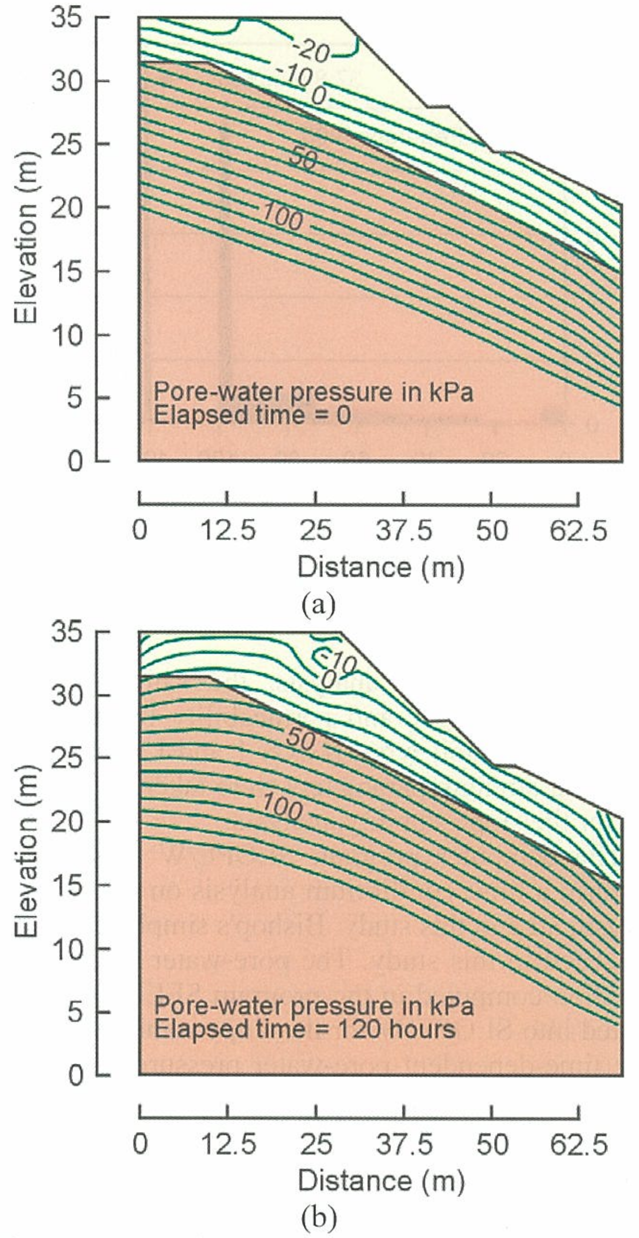


Figure 15. Simulated pore-water pressure contours at elapsed times of (a) 0h; (b) 120h (from Rahardjo *et al.*, 2001)

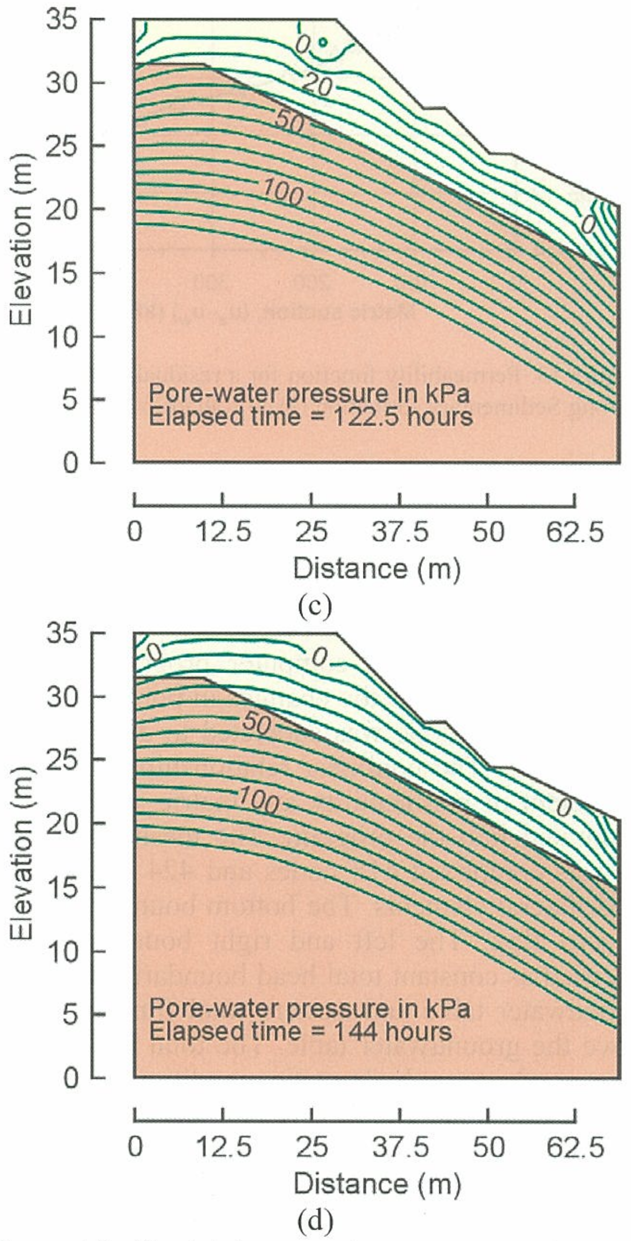


Figure 15. Simulated pore-water pressure contours at elapsed times of (c) 122.5h and (d) 144h (from Rahardjo *et al.*, 2001)

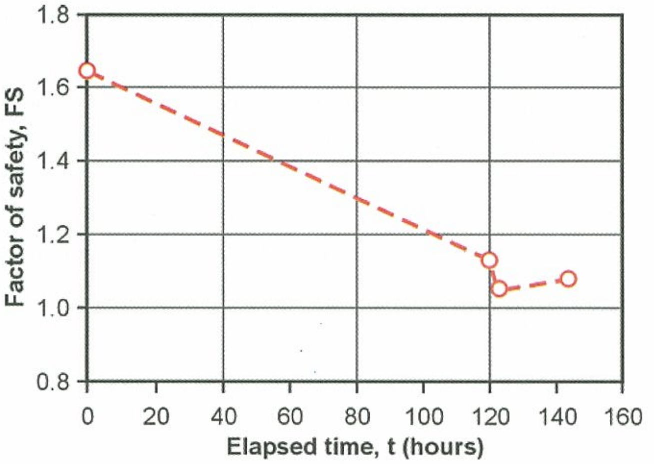


Figure 16. Factor of safety variation with elapsed time for NTU-ANX slope (from Rahardjo *et al.*, 2001)

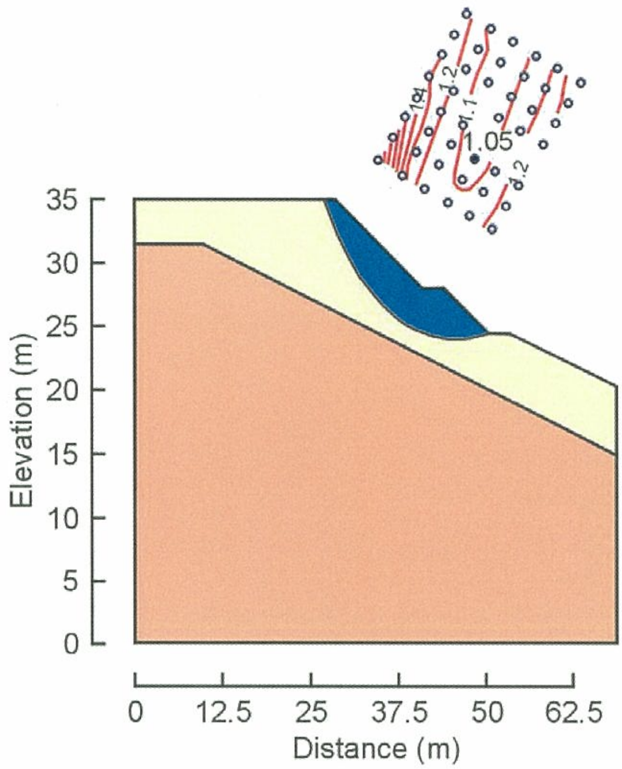


Figure 17. The critical slip surface at elapsed time of 122.5h (from Rahardjo *et al.*, 2001)

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