The role of curved-surface envelope Mohr-Coulomb model in governing shallow infiltration induced slope failure

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ABSTRACT

Shallow rainfall infiltration induced slope failure is a difficult soil behavior to model. It occurs under low stress level. There are two unique soil shear strength characteristics under this stress level which has a strong influence on the failure. They are the steep drop in shear strength when the soil is wetted and when the effective stress approaches zero. Their incorporation is inevitable in order to back analyze the actual failure plane. These soil shear strength characteristics are excellently replicated in curved-surface envelope Mohr-Coulomb soil shear strength model. Its application in a new slope stability equation is aimed to improve the reliability of the output stability factor. Furthermore, the new slope stability equation is able to replicate the actual mechanism of rainfall induced failure which involves the reduction in shear strength and the increase in the disturbing weight when the soil is wetted. Its applicability is tested by conducting back analysis on the 6th December 2008 a slope failure in granitic residual soil at Bukit Antarabangsa, Malaysia. The soil parameters applied in the analysis was deduced from the common parameters reported and basic information obtained from a visit to the site. The progression of infiltration into the soil is incorporated in the analysis by applying the changes in the moisture content and the increasing depth of wetting front. The influence of the microscopic change in the clay content of the soils is substantiated and incorporated in the analysis. The actual failure surfaces have been back analyzed. The result support the applicability of the method for rainfall infiltration induced failure and the technique would be very appropriate to be applied in slope monitoring system for detection of the shallow type of failure.

KEYWORDS: Slope Failure; Infiltration; curved-surface envelope Mohr-Coulomb model; Unsaturated soil; Rainfall.

INTRODUCTION

Shallow mode of failure is a typical attribute for rainfall infiltration-induced failure. Some definitions for shallow landslide is like the ratio of depth to the length of the convex failure shape less than 0.1 according to Santacana *et al.* (2003) or less than 1.0 according to Giannecchini (2006). Another definition of shallow landslides is referred to slides less than 5 meters thick referring to the failure being above the toe according to Fell *et al.* (2000). However the best likely definition that would comply with the characteristic of shallow landslides that have been occurring is "the failure to be above the toe irrespective of the thickness". This type of failure is very difficult to analyze. It involves the interaction between the mechanics of saturated and partially saturated soils. The mechanism of this type of failure involves surface water infiltration that wetted the soil. The effect is the reduction in the shear strength and the increase in the disturbing effect as the soil becomes heavier when wet. Most of failures in cut slopes in tropical countries are triggered by this kind of mechanism rather than the rise in the groundwater table. The latter is commonly applied in back analysis of failure in those slopes despite its isolation from the failure zone. A specific infiltration-induced slope stability method that actually replicates the above mechanism is described in Section 3.

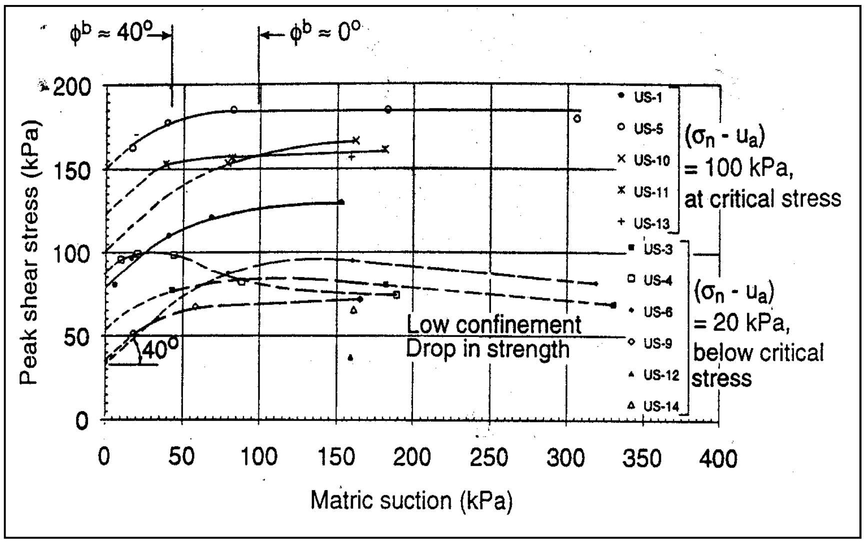
Conventional slope stability methods always apply saturated shear strength despite the actual soil conditions above the ground water table being partially saturated. This is because of the standard practice in the triaxial laboratory shear strength test where the specimen is always subjected to saturation stage before it is consolidated and sheared. The shear strength at saturation is the lowest and this approach is thought to be very conservative. However the method cannot replicate the actual failure profile of the shallow rainfall induced slope failure. This discrepancy would also jeopardize the reliability of the factor of safety produced. The application of the saturated shear strength would produce low stability factor and may even be lower than unity whereas the slope may still be standing (Othman, 1993). This is the problem when the actual shear strength of the soil is not being applied.

There is nothing wrong with the engineering mechanics like the principal of moment equilibrium applied in slope stability analysis. The problem that makes this subject very difficult to understand is the soil shear strength behavior itself which is very complex. They are not just a mere linear behavior relative to effective stress and suction like the model of Terzaghi (1936) and Fredlund *et al.* (1978). Researchers in landslide behavior should have a greater concern on the shear strength behavior at low stress level since that is the stress range relevant for shallow landslide.

Initially shear strength was characterized purely base on effective stress as in the shear strength equation of Terzaghi (1936). However it was later realized that soil structure become stronger when it is partially saturated. Therefore there is a need for a shear strength model that can characterize strength base on the effect of moisture content. This aspect of shear strength can be applied in design when Fredlund *et al.* (1978) introduced plane envelope shear strength model which encompass the strength in partially saturated and fully saturated conditions.

Nevertheless, there has been many reports for tropical residual soil that shear strength behavior relative to net or effective stress and suction are non-linear especially at low stress levels. Gan and Fredlund (1996) and Toll *et al.* (2000) have reported that the soils in Hong Kong and Singapore have this non-linear attribute of shear strength with respect to suction as shown in Figures 1(*a*) and (*b*) respectively.

Fredlund *et al.* (1995) has proposed a shear strength equation to replicate this unique behavior and this is followed by two more equations by Vanapalli *et al.* (1996). However, those equations could not produce a good fit to the experimental data. With regards to shear strength behavior relative to effective or net stress there has been many reports for the non-linear behavior for granular soils. These are like the works of Bishop (1966), Charles and Watts (1980) and Indraratna *et al.* (1993). This kind of behavior can also be seen for sedimentary residual soil from Jurong formation in Singapore. The reinterpretation of the data reported by Rahardjo *et al.* (1995) for selected samples having standard penetration test value N between 10 and 12 is shown in Figure 2. Evidently the behavior shows a steep drop in shear strength as effective stress drop from 100 kPa to zero.



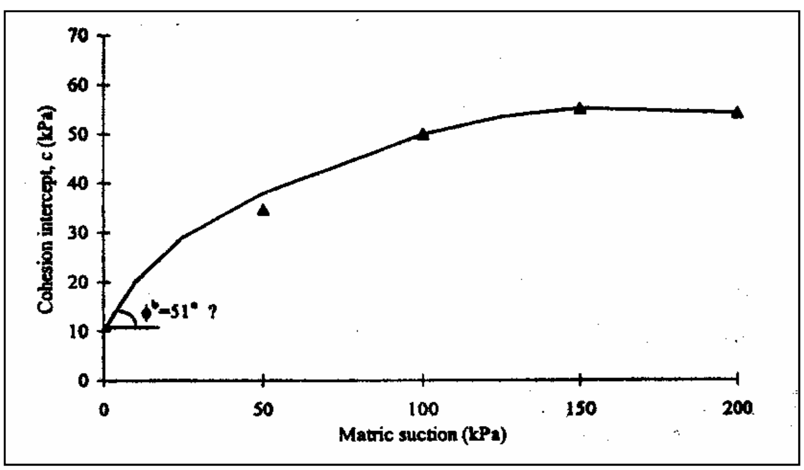


Figure 1: Shear strength behaviour of tropical residual soil relative to suction (*a*) Hong Kong residual soil (Gan and Fredlund, 1996) (*b*) Sedimentary residual soil, Jurong formation, Singapore (Toll *et al.*, 2000)

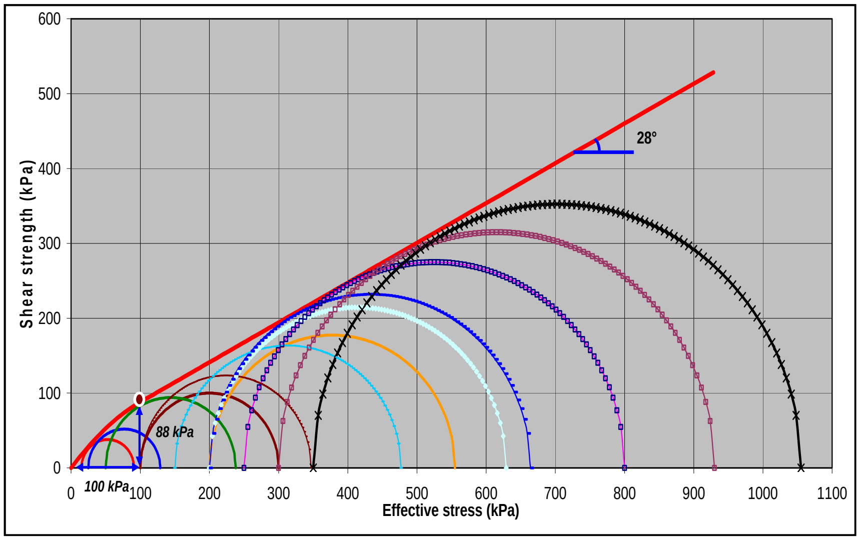


Figure 2: Curvi-linear shear strength envelope of sandy silty Clay from Jurong formation, Singapore reinterpreted from Rahardjo *et al.* (1995).

Consolidated drained triaxial tests have been conducted on saturated remolded specimens of granitic residual grade VI from Rawang, Malaysia. The soil has 30% clay proportion. The curvi-linear shear strength envelope for the soil is shown in Figure 3. Evidently the soil also exhibits steep drop in shear strength as effective stress approaches zero.

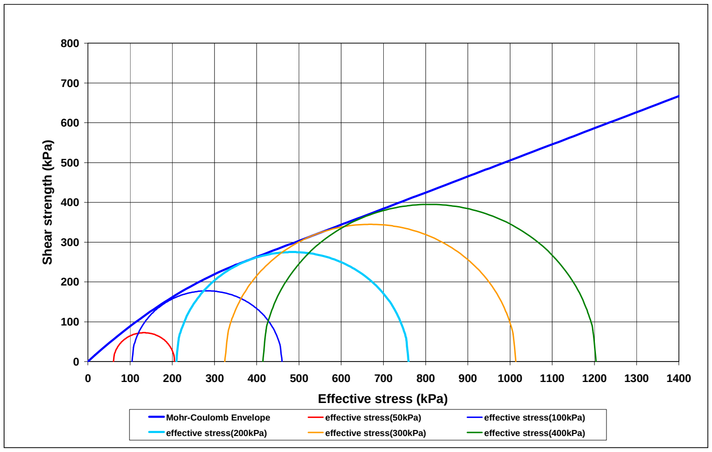


Figure 3: Curvi-linear shear strength envelope of granitic residual soil grade VI from Rawang, Malaysia (Md.Noor *et al.*, 2008).

These two unique soil shear strength characteristics must have a strong influence on the shallow type of rainfall infiltration induced failure. The steep drop in shear strength as suction approaches zero is relevant when rain water infiltrates into the slope. There will be a steep and drastic drop in the shear strength as wetting front propagates into the slope and the soil becomes wet. This is anticipated to have the prime influence on the infiltration induced slope failure. Secondly, is the steep drop in shear strength when effective or net stress decreases from about 200 kPa to zero as seen in Figures 2 and 3. This is implicating that at shallow depth which is about 10.0m and less from the slope face the drop in shear strength is steep as the depth gets closer to the surface. In another words, the change in shear strength relative to depth within this shallow zone is very drastic. Undoubtedly this attribute is very much relevant for the shallow slope failure and believe to have a strong influence on the behavior.

In order to improve the slope stability analysis for rainfall induced failure it requires the application of a shear strength model with its analytical form that can replicates those unique shear strength behavior at low stress level. A shear strength model that has these characteristics will be described in the following section.

CURVED-SURFACE ENVELOPE EXTENDED MOHR-COULOMB MODEL

Curved-surface envelope extended Mohr Coulomb shear strength model was introduced by Md.Noor and Anderson (2006) as shown in Figure 4. The uniqueness of the model is that it is able to exhibit both the steep non-linear drop in shear when suction and depth approaches zero. It makes use of seven shear strength parameters and produce excellent replication to the shear strength behavior obtained from laboratory tests of many soils like those reported by Charles and Watts (1980), Escario and Juca (1989), Indraratna *et al.* (1993), Gan and Fredlund (1995), and Toll *et al.* (2000). The curved-surface envelope shear strength model can represent the shear strength behavior of many soils by choosing the right shear strength parameters. The shear strength parameters are:-

1. $\phi'_{\min_f}$ is minimum friction angle at failure
2. τ_t is transition shear strength
3. $(\sigma - u_w)_t$ is the transition effective stress
4. $(u_a - u_w)_r$ is the residual suction
5. $(u_a - u_w)_u^{\sigma'=0}$ is the ultimate suction when the net stress is zero
6. $c_s^{\max}$ is the maximum apparent cohesion
7. ζ is the rate of change of ultimate suction with respect to net stress

The minimum friction angle, $\phi'_{\min_f}$ is used for the definition of the envelope since the internal friction angle varies on the low stress range. At any value of suction the variation of shear strength relative to net stress is curvi-linear. It is non-linear on the low net stress and becomes linear at higher net stress. The transition shear strength and the transition effective stress are representing a point (i.e. C) that differentiate the linear and the non-linear section of the envelope for saturated soil condition. The line CEF is the demarcation line between the non-linear and the linear shear strength behavior relative to net stress. Residual suction is the value of suction that corresponds to the maximum apparent shear strength, $c_s^{\max}$. The ultimate suction is the value of suction of which it does not produce any apparent shear strength. Ultimate suction increases with the net stress as represented by the line BF and ζ is the rate of change of ultimate suction with respect to net stress. Thence $(u_a - u_w)_u^{\sigma'=0}$ is just the value of ultimate suction when net stress is zero.

Notice that the shear strength behavior along AO and CO in Figure 4 represent the steep drop in shear strength when suction and net stress approach zero respectively. These are the parts that would have strong influence on the shallow infiltration induced landslide. The application of the model in slope stability analysis can help towards achieving the realistic slope stability factor. This aspect of steep drop in shear strength relative to net stress and suction is also important in the design of retaining wall. Using conventional type of shear strength would over estimate the soil strength near the ground surface even though the shear strength at saturation has been applied. This may lead to failure when the zone is infiltrated. And in hazard risk slope monitoring system its application can lead to a reliable threshold parameters.

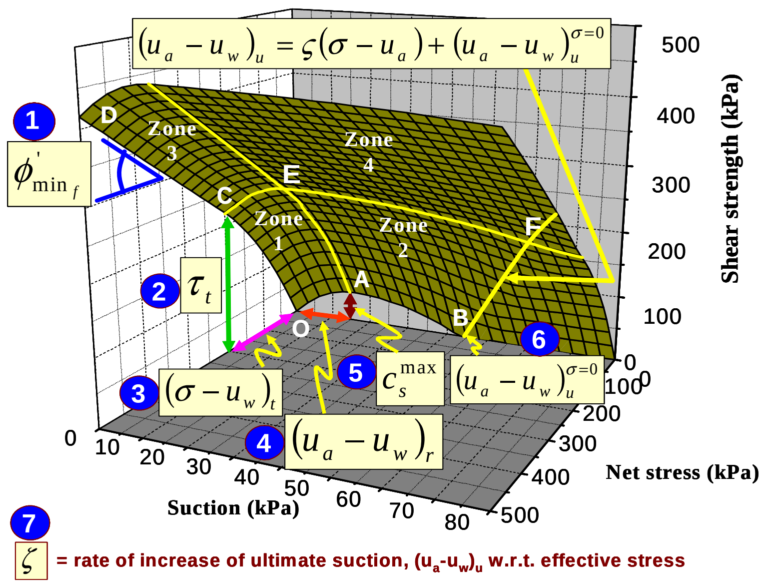


Figure 4: Curved-surface envelope Mohr-Coulomb model of Md.Noor and Anderson (2006) and the seven shear strength parameters.

In fully saturated conditions only three shear strength parameters are involved. They are the first three parameters in the list. In most shallow slope failures the shear strength variation is confined in Zone 1 and Zone 3 only where the variation of the field suction is within approximately 100 kPa and this is less than the residual suction of many tropical residual soils.

The equation for the non-linear apparent shear strength (c_s) behavior produced by suction as represented by curve OA is as in Equation 1. It is valid for suction from zero up to residual suction irrespective of the net stress.

$$c_s = \frac{(u_a - u_w)}{(u_a - u_w)_r}\left[1 + \frac{(u_a - u_w)_r - (u_a - u_w)}{(u_a - u_w)_r}\right] c_s^{\max} \qquad (1)$$

The equation for the non-linear shear strength behavior at saturation is represented by curve OC and is valid for effective stresses from zero up to the transition effective stress irrespective of the net stress. This represented by Equation 2.

$$\tau_f = \frac{(\sigma - u_w)}{(\sigma - u_w)_t}\left[1 + \frac{(\sigma - u_w)_t - (\sigma - u_w)}{N(\sigma - u_w)_t}\right]\tau_t \qquad (2)$$

Where
$$N = \frac{1}{1 - \left[(\sigma - u_w)_t \dfrac{\tan\phi'_{\min_f}}{\tau_t}\right]}$$

The equation for the straight line CD is representing shear strength at saturation and represented by Equation 3. It is valid for any net stress greater or equal to transition effective stress irrespective to the value of suction.

$$\tau_f = (\sigma - u_w)\tan\phi'_{\min_f} + \left\lfloor \tau_t - (\sigma - u_w)_t \tan\phi'_{\min_f} \right\rfloor \qquad (3)$$

The equation for apparent shear strength represented by the line AB is as in Equation 4. It is valid for suction between residual suction and ultimate suction irrespective of the net stress. It is to be noted that the ultimate suction increases with the value of net stress as represented by the line BF in Figure 4.

$$c_s = c_s^{\max}\left[\frac{(u_a - u_w)_u - (u_a - u_w)}{(u_a - u_w)_u - (u_a - u_w)_r}\right] \times \left[1 - \frac{(u_a - u_w)_r - (u_a - u_w)}{(u_a - u_w)_u - (u_a - u_w)_r}\right] \qquad (4)$$

The shear strength equation representing each zone can be obtained by adding the saturated and the partially saturated equations that represent the area. This is summarized in Table 1.

Table 1: Shear strength equation for the curved-surface envelope

Zone	Representing lines	Saturated + partially shear strength
1	OC and OA	Eqn. 2 + Eqn. 1
2	OC and AB	Eqn. 2 + Eqn. 4
3	CD and OA	Eqn. 3 + Eqn. 1
4	CD and AB	Eqn. 3 + Eqn. 4

The shear strength at any point on the envelope depends on the value of net stress and suction. When the point falls on the specific zone then the shear strength can be determine by adding the respective equations as defined in the third column in Table 1. In this manner the shear strength at any depth in a soil mass can be determined provided the net stress and suction at that point are known. Thence in the slope stability slice method, when the magnitudes of the net stress and suction at the slice base are known then the shear strength there can be calculated.

The curved-surface shear strength envelope or the curvi-linear lines in Figures 2 and 3 are drawn using the combination of Equations 2 and 3 which represent the line OC and CD in Figure 4. The experimental data points in Figure 1(b) are plot again in Figure 5. The curve lines JK and

KL are formed using Equations 1 and 4 respectively by applying residual suction of 150 kPa, maximum apparent shear strength of 45 kPa and ultimate suction of 425 kPa. Evidently the shear strength equations (i.e. Equations 1, 2, 3 and 4) are able to produce a good representation of the actual shear strength envelope of the soils.

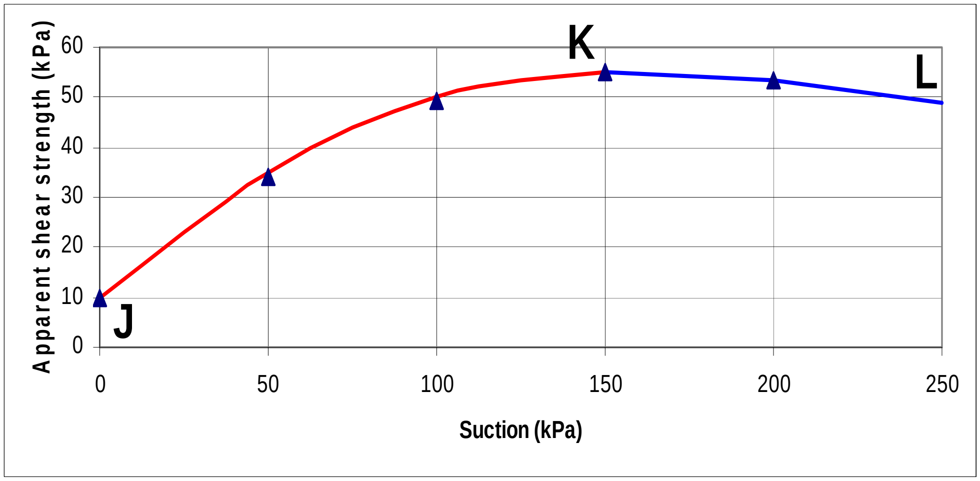


Figure 5: Fitting experimental data points reported by Toll *et al.* (2000) for sedimentary residual soil from Jurong formation, Singapore using Equations 1 and 4 for the curve JK and KL respectively.

SLOPE STABILITY EQUATION AND SIMPLIFIED MODELING OF INFILTRATION INDUCED FAILURE

A slope stability equation that makes use of the curved-surface envelope extended Mohr-Coulomb model of Md.Noor and Anderson (2006) is introduced as in Equation 5.

$$FOS=\frac{\sum s_i \times R}{\sum W_i \times x_i}$$

$$=\frac{\sum\{\tau_i \times \beta\} \times R}{\sum\left[\gamma_{dry} \times V_i + \theta_{wet} \times 9.81 \times V_{i_{TOP}} + \theta_{bulk} \times 9.81 \times V_{i_{BOT}}\right] \times x_i} \qquad (5)$$

where

s = shear force at the base of slice

R = radius of the potential slip circle

Wi = weight of slice

xi = perpendicular distance of the line of the slice weight from the center of rotation

τi = shear strength mobilized at the slice base

β = length of the slice base

γ_{dry} = soil dry unit weight

Vi = volume of typical slice

$V_{i_{TOP}}$ = volume of top wetted portion of slice

$V_{i_{BOT}}$ = volume of bottom portion of slice

θ_{wet} = wet volumetric moisture content

θ_{bulk} = bulk volumetric moisture content

The numerator is the failure resisting variable while the denominator is the disturbing variable. The value of the numerator changes when the shear strength, τ_i is affected by the reduction in suction due to infiltration. The variation of the shear strength relative to net stress and suction is taken according to the curved-surface envelope soil shear strength model. The relevant equation for τ_i is taken according to Table 1. In this way the steep drop in shear strength relative to net stress and suction are incorporated in the analysis. The value of the denominator increases when the wetting front propagates deeper into the slope. W_{wet} and W_{bulk} in Figure 5 are the weight of the wetted and the unaffected sections of the slice respectively. In this manner the slope stability equation is actually replicating both the effect of infiltration on the shear strength and the weight of the soil. Thence the actual mechanism of the shallow rainfall induced failure has been incorporated.

Figure 6 shows the typical slice involved in the application of Equation 5. When water infiltrates into the soil the wetting front will propagate deeper into the slope. The wetting front is assumed to be parallel with the slope face. The zone above the wetting front can still be partially saturated but it would be very close to saturation if there is a continuous supply of water at the surface. Therefore there would be a lot of simplification in the analysis if the zone above the wetting front is assumed to be fully saturated. This assumption would be a conservative approach.

The critical slip failure line is taken to be confined within the wetted zone or the zone of wetting front. If the slip failure plane cuts through the partially saturated zone below the wetting front obviously the stability factor will increase. And thus that wouldn't be the critical failure plane. The partially saturated zone is acting like a hard layer which pushes the critical failure plane to be confined within the overlying wetted zone.

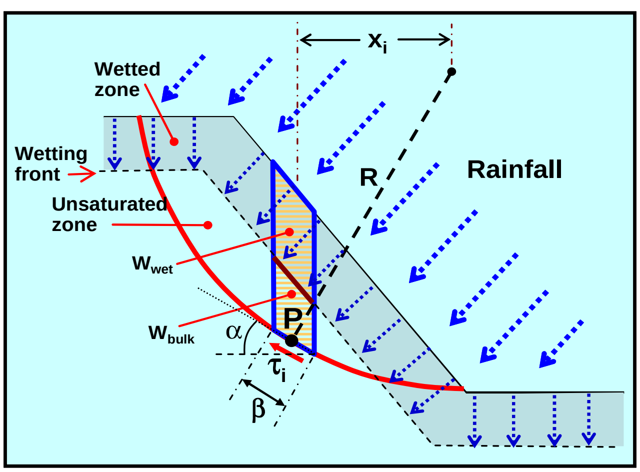


Figure 6: Typical slice in slope stability analysis incorporating the reduction in shear strength and increase of soil weight in effect of infiltration proposed by Md.Noor (2007).

Thence the analysis for shallow rainfall infiltration can be simplified by the following assumptions:-

a. The wetting front is parallel to the slope face.
b. Critical failure plane is confined within the wetted zone.
c. The wetted zone is fully saturated.

In this manner the analysis for the critical shallow failure plane will only involve the three soil shear strength parameters for saturated condition. This method of slope stability analysis for shallow rainfall infiltration induced failure will be applied in the slope failure investigation in the following section.

INFLUENCE OF CLAY MICROSCOPIC STRUCTURE CHANGE ON ITS SHEAR STRENGTH

Clay is the content of many soils. In soil that has only a small portion of clay, it forms the cementing agent between the granular particles in the soil and has a major influence on the soil shear strength. The change in the properties of clay would affect the overall properties of the soil. This is like granitic residual soil in Malaysia which may contain clay up to 52% (Pushparajah and Amin, 1977). The clay in this soil is derived from the weathering of feldspar in the parent rock. This section will demonstrate that the shear strength of remolded kaolin clay will decrease when it absorbs water.

Kaolin slurry was prepared and placed in a Rowe cell with drainage layers provided at top and bottom. It was then consolidated under 250 kPa consolidation pressure until 100% consolidation is achieved. Then triaxial specimens of 38mm diameter and 76mm height were extracted and consolidated drained triaxial test were conducted on saturated specimens. There were three triaxial test series conducted with 4 specimens in each series. The targeted effective stresses in each test series were 50, 100, 200 and 300 kPa. In the first test series the specimens were not soaked. But in the second and third test series the specimens were soaked for 1 day and 2 days respectively. The moisture content of the specimens was taken immediately after the shearing stage. The average moisture obtained from the three test series were 21.43%, 27.68% and 31.89% respectively.

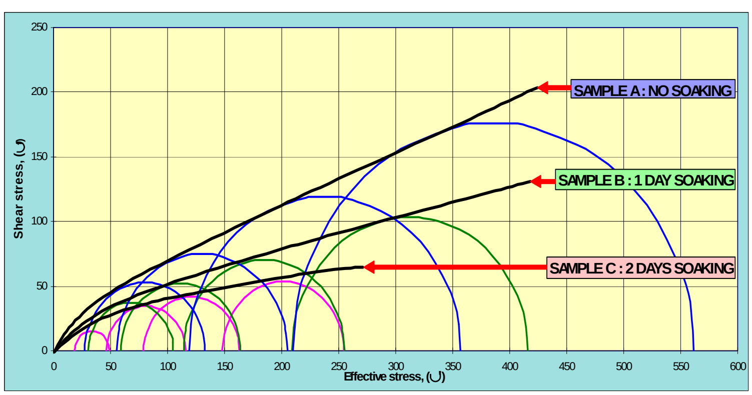


Figure 7: The reduction in the shear strength of kaolin clay when it absorb water.

Evidently the shear strength envelope obtained from the three test series are curvi-linear which is in compliance with the curved-surface envelope Mohr-Coulomb shear strength model of Md.Noor and Anderson (2006). The shear strength envelopes are shown in Figure 7. The envelopes show that there is a reduction in shear strength when kaolin absorbs water.

This aspect of shear strength reduction is very important when investigating slope failure in soil that has a portion of clay content like granitic residual soil.

BACK ANALYSIS OF BUKIT ANTARABANGSA SLOPE FAILURE ON 6TH DECEMBER 2008

Bukit Antarabangsa is a hillside residential area in Ulu Klang, Selangor, near Kuala Lumpur, Malaysia. The area is dubbed as the Beverly Hill of Malaysia because many rich and famous locals live there. A landslide which occurred on the 6th December 2008 has damaged 14 bungalows and claimed 5 lives. The soil there is granitic residual soil where some granite boulders and outcrops can still be seen in the surrounding area. Back analysis on the failure is conducted in order to investigate the cause of the failure.

The main body of the failed slope is the mess up slope face overlooking the rows of houses at the toe as shown in Figure 8. The picture shows that the areas at the flanks of the failure zone are heavily vegetated to indicate the similar vegetation used to be on the failed slope. Among the information obtained from a visit to the site on the 12th April 2009 are as follows;

1. The slope was 32m high and at 38° inclination from the horizontal.

2. There was no load imposed on the failed slope.

3. The depleted mass was very wet and the momentum of the mass has laterally moved the houses at the toe for approximately 120m.

4. The toe of the slope was very wet with plenty of caladium plants grown along the toe drain before failure. The presence of this plant indicates that the soil there was under prolong soaking. This also indicates that there is a continuous supply of water that seeps down the slope. This cannot be the groundwater water table since there was no water table noticed in a 1m deep hole dug for the caisson piles for the repair work during the visit.

5. The depth of the rupture surface is 10m below the original ground surface.

6. There used to be a water pipe line running longitudinally at the top of slope. The pipe line is suspected to be broken from the previous slope movement and cause internal wetting from top to bottom that is barely noticed.

7. The slope was claimed to be as part of the previous un-engineered back-filling.

8. The soil there is granitic residual soil and the friction angle is similar to the medium dense sand.

There are two types of the soil shear strength behavior that will be considered in the analysis. They are the linear shear strength envelope of Terzaghi (1936) to represent the conventional interpretation of shear strength and the curved-surface envelope soil shear strength model of Md.Noor and Anderson (2006) to represent the concept of zero cohesion and non-linear shear strength behavior on the low stress level. The influence of the groundwater table on the failure is ruled out base on the information number 4 obtained from the site visit. The failure is mainly related to the surface water infiltration or a continuous seepage of water from an unknown source.

Soil properties and its shear strength parameters will be deduced from the reported typical data for the soil in the literature and some information gathered from the visit to the site and these will be applied in the back analysis. The possibility whether the insitu soil was an un-engineered fill or granitic residual soil will be tested analytically. The aspect of soaking on reducing the shear strength will also be incorporated in the back analysis. Even though it is commonly understood that when clay absorbs water it becomes softer or weaker as its microscopic structure changes but this aspect of strength change is never been quantified by any researcher and thus it never being incorporated in slope failure analysis. This absorption behavior of clay minerals is often referred as clay expansion without any incorporation in term of strength. In Malaysian granitic residual soil the range of clay content varies between 28% to 52% (Pushparajah and Amin, 1977; Ting *et al.*, 1982; Taha *et al.*, 1999). Therefore this aspect of strength reduction must have a significant role for slope failure in this type of soil.

The first indication by the picture in Figure 8 is that there must be some localize condition that must have occurred at the failed slope since there is not much difference between slope at the flanks. The specific condition that has taken place needs to be identified from the back analysis.

Figure 8: The 6th December 2008 Bukit Antarabangsa, Ulu Klang, Malaysia landslide.

The first step is to consider the chances that the soil is of the un-engineered backfill. According to conventional interpretation of shear strength the soil of this type would have the internal friction angle of 26° and cohesion of 5 kN/m^2. This is considered as the kind of strength for a loose granitic residual soil as reported by Ting *et al.* (1972). The shape of the shear strength envelope base on conventional interpretation for this soil in comparison to the curved-surface envelope is shown in Figure 9. Note that the latter envelope has minimum internal friction angle at failure, $\phi'_{\min_f}$ of 26°. Their only difference is that at low stress level the curved-surface envelope has a non-linear drop in shear strength towards zero cohesion at zero effective stress. In other words there is a slight over estimation of shear strength at this stress range by the conventional interpretation of the shear strength. This is an assumption base on zero cohesion at zero effective stress like those reported by Bishop (1966) even for clay soils and supported by the shear strength behavior relative to effective stress for tropical residual soils discussed in Section 1.

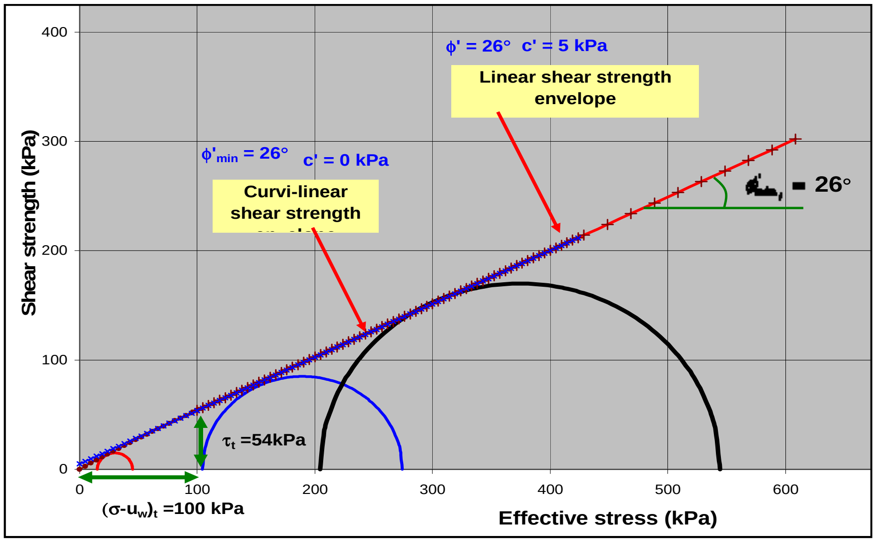


Figure 9: Shear strength envelope of the un-engineered backfill according to shear strength model of Terzaghi (1936) and Md.Noor and Anderson (2006).

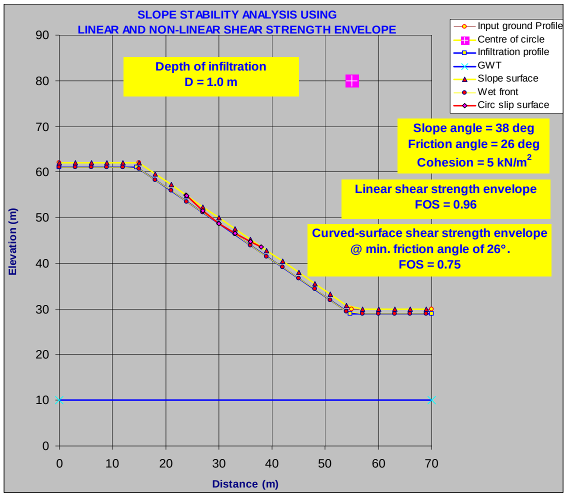
The analysis will also consider the profile of the wetting front to be parallel to the slope face. This type of wetting profile considers a uniform supply of water over the slope surface in order to simplify the field condition. This is base on the negligible influence of gravity on the infiltration process (Philip, 1957). The analysis considered failure to be confined within the wetted zone. This characteristic of rainfall-induced slope failure has been substantiated from a parametric study by Md.Noor *et al.* (2006). The wetted zone is considered to be fully saturated with wetted volumetric moisture content, θ_{wet} equals to the saturated volumetric moisture content of 0.3 while the infiltration unaffected zone underneath is having field volumetric moisture, θ_{field} of 0.1. When the depth of failure was reported to be 10m this indicates that the wetting front has reached to this depth. This extend of infiltration is considered very deep and abnormal. The typical extend of infiltration reported by Faisal and Rahardjo (2004) in similar soil type (i.e. granitic residual) for two days continuous rainfall barely reached the 3m depth. Thence it is anticipated that there is a peculiar source of water that cause the 10m infiltration. The soil properties applied in the analysis are shown in Table 2.

Table 2: Shear strength parameters for the un-engineered backfill applied in back analysis.

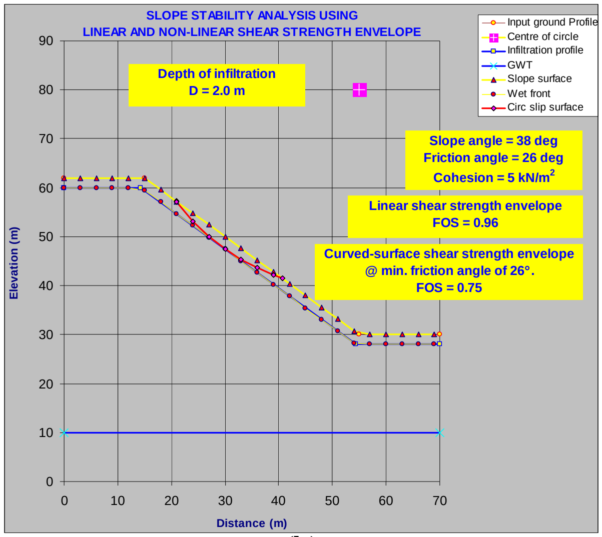
Shear strength model	Internal friction angle, φ'	Cohesion (kN/m^2)	Soil properties
Terzaghi (1936)	26°	5	Bulk unit wt.= 19kN/m^3
Curved-surface extended Mohr-Coulomb model of Md.Noor and Anderson (2006)	26° $(u_a - u_w)_r = 120kN/m^2$ $\tau_{app}^{max} = 65kN/m^2$ $(\sigma - u_a)_t = 100kN/m$ $\tau_t = 54kN/m^2$	0	Dry unit wt.= 16.5kN/m^3 θ_{wet}= 0.3 θ_{field}= 0.1

*Note: θ = volumetric moisture content.

Considering the soil is a loose un-engineered backfill, the result of the slope stability analysis for 1m, 2m and 4m infiltration is shown in Figure 10(a) (b) and (c) respectively. The result shows that in all cases the stability factors obtained are less than unity to indicate failure irrespective whether the shear strength is the conventional linear type or the non-linear type without cohesion. This is indicating that it is impossible for the soil to have the loose type of strength because failure would have occurred when the depth of infiltration has reached 1m, 2m or 4m. Despite of the reinforcement by the trees root this type of shallow localizes failure would occur between the trees. The recurring of this type of failure would have brought down the whole slope even before the 10m depth of failure. Thence the soil there must not mainly make up of the un-engineered fill. It could be just the top thin layer. The underlying soil must be the granitic residual.



(a)



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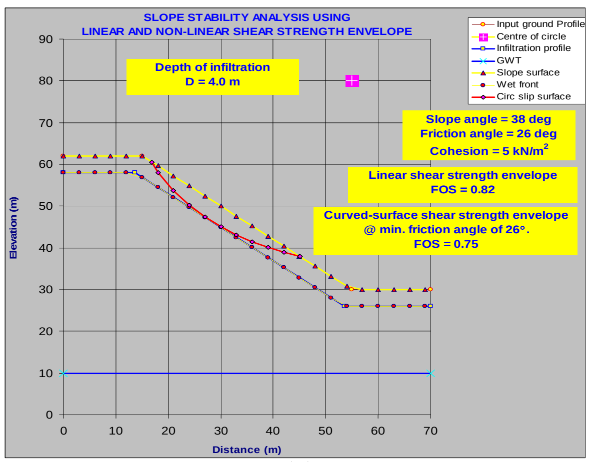


(c)

Figure 10: Stability factor for (a) 2 and (b) 4m infiltration using conventional linear shear strength envelope and curved-surface envelope at saturation.

The next step in this analytical investigation is to consider the soil as granitic residual grade VI. The friction angle for this type of soil varies between 23° to 40° as reported by Ting *et al.* (1972) and Komo (1985). Since the information obtained on the conducted laboratory strength tests indicates that the friction angle of the soil is similar to medium dense sand then the friction angle of this soil is deduced as 33°. Due to the presence of clay content in the soil, conventionally the soil is assumed to have a low value of cohesion of approximately 10 kN/m^2 according to the reports by Ting *et al.* (1972) and Komo (1985). However base on the curvi-linear shear strength envelope there would be zero cohesion. The described shear strength envelopes are illustrated in Figure 11. The characteristic of the curvi-linear envelope is believed to be the best representation of the in-situ soil. The shear strength behavior described in Figure 11 will be applied in the following back analysis.

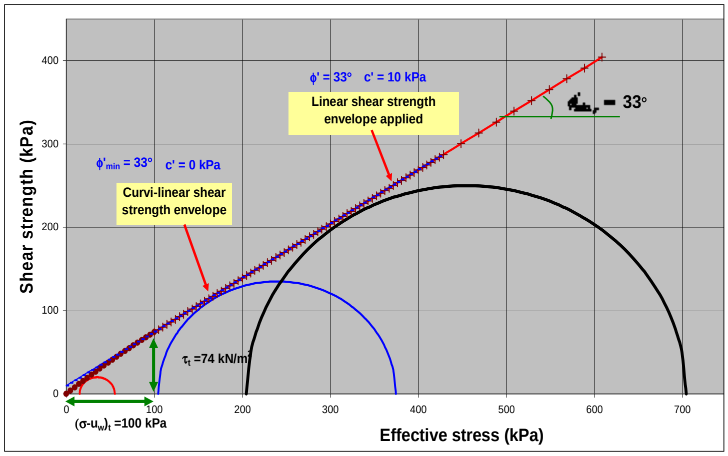
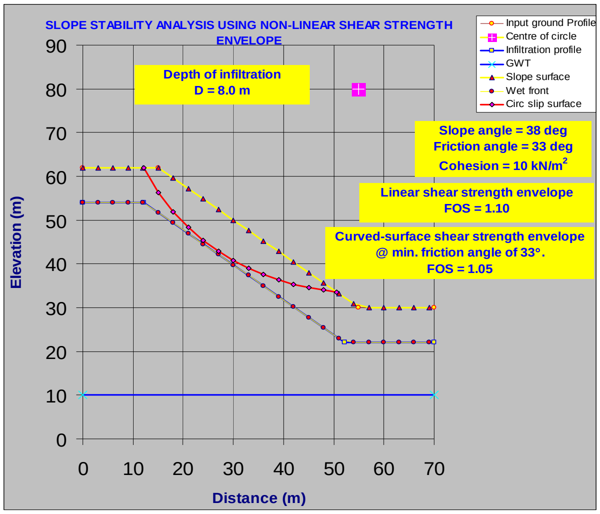


Figure 11: Shear strength envelope deduced for granitic residual soil grade VI according to shear strength model of Terzaghi (1936) and Md.Noor and Anderson (2006).

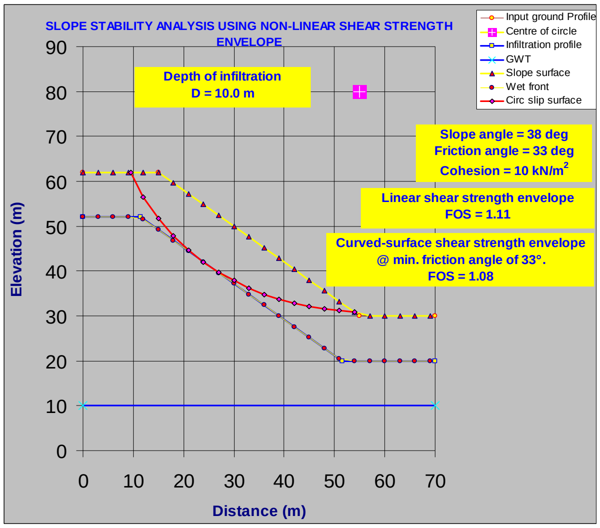
Figure 12 shows the critical slip circles and the stability factors for the 8m and 10m depth of infiltration in the granitic residual soil grade VI are 1.05 and 1.08 respectively for the curved-surface envelope shear strength model and they are 1.11 and 1.10 respectively for the linear type shear strength model. The result shows that the stability factors are above unity irrespective of the shear strength type. Evidently the analysis using the curvi-linear shear strength envelope produced a lower stability factor compared to the conventional linear shear strength envelope. This is indicating that the application of the curvi-linear envelope is more conservative than the conventional linear envelope. Furthermore the stability factor obtained from the curvi-linear shear strength envelope should be reliable since it applies the true shear strength behavior.

The actual slip failure surface determined from the failure investigation conducted by the authorities is as shown in Figure 13. Essentially the slope failed at approximately 10m deep. When the back analysis indicates that the stability factor is 1.08 using the curved-surface shear strength envelope for the 10m depth of infiltration, this is indicating that the slope is still stable even when being infiltrated that deep. If this is the case then what makes the slope failed?

The next step of this analytical investigation is to assume a slight deterioration in strength when the soil there is soaked which changes the clay microscopic structure due to the absorption of water. The slight reduction in the strength is described by the curvi-linear shear strength envelope shown in Figure 14. The soaked shear strength envelope of the granitic residual soil grade VI is assumed to have minimum internal friction angle at failure, $\phi'_{\min_f}$ of 32° which has decreased from 33°. The transition shear strength has reduced from 74kN/m^2 to 67kN/m^2 while the transition effective stress remained unchanged at 100 kN/m^2. Essentially the strength envelope for the soaked soil is slightly lower compared to the initial curvi-linear envelope before being soaked. The result of the slope stability analysis using this reduced strength envelope is shown in Figure 15.



(a)



(b)

Figure 12: Critical slip circles and the stability factor obtained for (a) 8m and (b) 10m depth of infiltration in granitic residual soil grade VI.

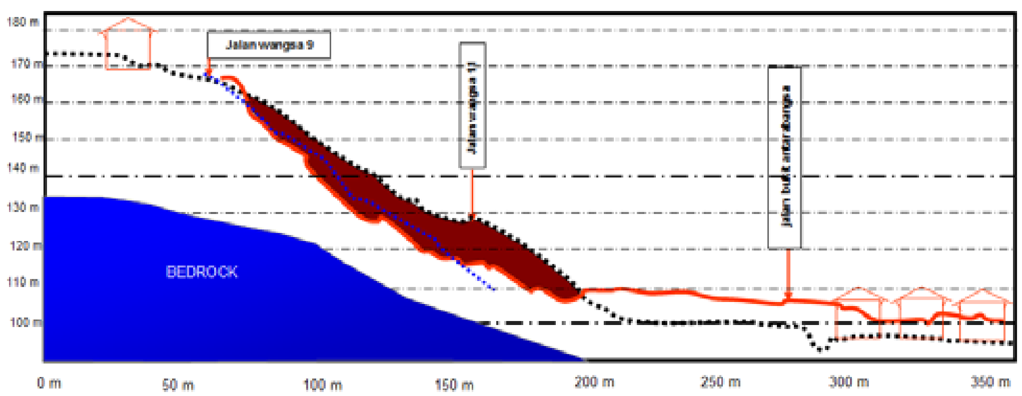


Figure 13: Actual failure surface determined from the failure investigation.

Evidently the effect of soaking has produced the stability factor of 0.99 which indicates failure. The result has substantiated that the effect of soaking has triggered the failure. The source of water could be from the broken water supply pipe that runs longitudinally at the crest. If the soil undergo alternate wetting and drying through the process of infiltration during rainfall and subsequently followed by evaporation when the rain stopped, the infiltration wouldn't reach to the 10m depth. And thus the slope would not fail. It only fails when there is prolong continuous supply of water which really pushed the wetting front up to 10m deep. Upon reaching the 10m depth there is still a continuous supply of water and this cause the soaking of the soil. This hypothesis of the failure is supported by the observation number 4 from the site visit.

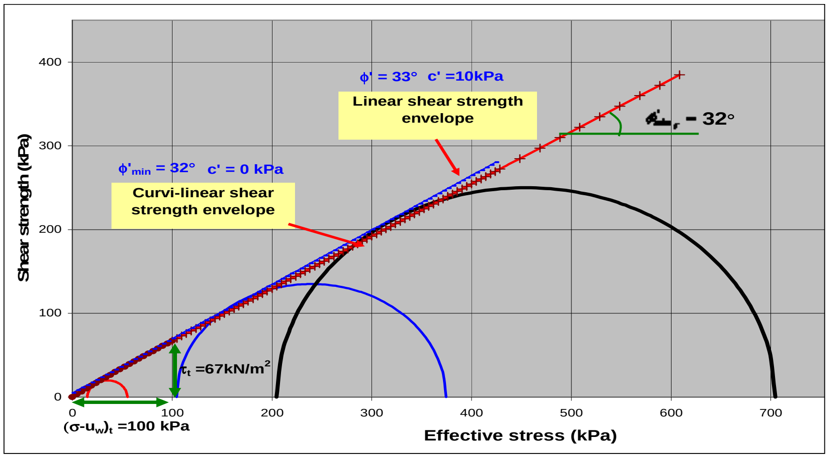


Figure 14: Reduction in shear strength due to the microscopic change of the clay in the soil described by the slightly lowered curvi-linear shear strength envelope.

The analysis using the conventional shear strength behavior could not model the failure as indicated by the result shown in Figures 12(b) and 15 which gives the stability factor of 1.11. Besides, it is to be noted that the slope stability equation applied in this exercise is the Equation 5 which is base on the effect of infiltration irrespective of the considered shear strength types of the soil. Conventional slope stability method would elevate the groundwater table in order to achieve failure condition. However this is not the actual mechanism of the rainfall induced failure.

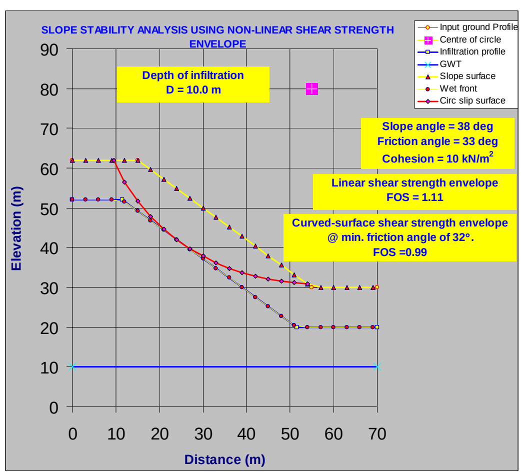


Figure 15: Critical slip circle at depth of 10m and the stability factor of 0.99 when the granitic residual soil is soaked.

ADVANTAGE OF THE CURVED-SURFACE EXTENDED MOHR COULOMB ENVELOPE AND THE INFILTRATION TYPE SLOPE STABILTY METHOD

The main advantage of the curved-surface shear strength envelope is that it is able to replicate the actual non-linear soil shear strength behavior when water infiltrates into the slope and when the depth under consideration approaches the ground surface. The problem with the common practice in laboratory shear strength test is that the test at low stress level (< 200 kPa) is seldom being carried out and the shear strength envelope there is just being extrapolated from the strength at higher stress level. It is always being assumed that the envelope is linear and there would be a cohesion intercept on the shear strength axis. This assumption would not be very wise when there are still many soil complex behaviors that need to be explained like the shallow mode of infiltration induced landslide and the wetting induced settlement. There is a need to change this practice so that the true shear strength can be deployed and a true soil mechanical behavior can be understood. Upon the application of the true shear strength behavior the calculated stability factor is of higher reliability compared when using the conventional linear shear strength envelope.

The main advantage of the infiltration type slope stability method (i.e. Equation 5) is that it replicates the actual mechanism of the rainfall infiltration induced failure from the aspects of shear strength reduction and the increase in soil weight. The applications of the curved-surface shear strength envelope and the infiltration type slope stability method have finally able to back analyze the actual shallow mode of the failure presented in Section 5. Even though failure still cannot be achieved after considering the worst scenario where the depth of infiltration has reached 10m as what actually being discovered from the failure investigation and the wetted zone has been assumed to be fully saturated, but this does not jeopardize the reliability of the resulted stability factor. Instead another explanation is sought due to the confidence in the stability factor obtained. This subsequently allowed for the real understanding for the cause of failure which involved the reduction in the shear strength when the soil underwent a prolong soaking.

The application of the curved-surface shear strength envelope also helped to realize that the application of the Terzaghi (1936) shear strength model is not the most conservative approach as thought especially when the behavior involved low stress levels. Evidently the curved-surface envelope shear strength model produced a lower stability factor compared to the linear shear strength model as shown by the result of stability analysis on the Bukit Antarabangsa 6th December 2008 failure in Figures 10, 12 and 15.

CONCLUSIONS

The conclusions that can be drawn from this study are:

The application of the curved-surface envelope soil shear strength model defined by Equations 1, 2, 3 and 4 is able to replicate the actual soil shear strength behavior of tropical residual soils with respect to net stress and suction. A good verification of the model using selected tropical residual soils has been achieved (refer last paragraph in Section 2).

The new slope stability equation as in Equation 5 which applies the curved-surface envelope soil shear strength model of Md.Noor and Anderson (2006) is able to model the actual mechanism of rainfall infiltration induced failure. It is able to replicates the steep drop in shear strength when water infiltrates into the slope and the increase in the disturbing weight as soil becomes wetter.

The shear strength of the remolded kaolin clay at saturation is essentially curvi-linear when the strength tests at low stress level were conducted. The shear strength decreases when the clay is allowed to absorbed water. This is indicated in Figure 7 where the curvi-linear envelope is lowered as the moisture content increases.

Failure of the 6th December Bukit Antarabangsa failure in Malaysia still could not be achieved from back analysis applying the actual 10m deep circular failure surface as determined from the failure investigation and considering full saturation of the soil up to the 10m depth of infiltration. This is despite the consideration of the steep drop in shear strength relative to net stress and suction and the soil become heavier when wetted. Failure can only be back analyzed by considering the reduction in the shear strength which was brought about by the softening of the clay content in the soil due to the absorption of water. Thence cause of the failure is deduced as the prolong wetting of the soil up to 10m deep and not the rise of the groundwater table.

Nevertheless the result of this exercise is base on the assumptions and anticipations made by the authors. A detail failure investigation of this approach would be recommended for a more convincing result.

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