Infiltration type slope stability method using non-linear shear strength behaviour

M J Md Noor¹, M A Muda¹,² and M M Ramle³

¹ Faculty of Civil Engineering, Universiti Teknologi Mara, Shah Alam, Selangor, Malaysia
² Public Works Department, Kuala Lumpur, Malaysia
³ Project Management Office, FGV Applied Technologies Sdn. Bhd.

Abstract. Shallow slope failure in hilly area triggered by rainfall infiltration is very common in the tropics and this is very difficult to quantify using the conventional slope stability methods. Those methods assess the failure by elevating the groundwater table (GWT) and apply the Terzaghi’s linear shear strength envelope with cohesion intercept. This is despite knowing that the location of the GWT is far below to have any influence on the failure at the slope face and the true soil shear strength behaviour is perhaps non-linear at the low stress levels with zero cohesion intercept. As a result such failure cannot be pre-detected and this pose risk to the public safety if the slope is close to human activity. In order to replicate the true mechanics of failure an Infiltration Type Slope Stability Method using the non-linear failure envelope with zero cohesion intercept has been applied to assess a slope failure in Bandar Baru Selayang, Selangor, Malaysia. Consolidated Undrained Triaxial tests have been conducted on undisturbed soil specimens to substantiate that the failure envelope is perhaps non-linear at low stress range and has zero cohesion intercept. The slope stability analyses back calculate the failure by considering various depths of infiltration. This approach managed to prove that the factor of safety (FOS) is less than unity when the wetting front reached the depth of the actual slip failure surface.

1. Introduction
A shallow mode of landslide of about 4 m deep has occurred in August 2017 at Jalan Persiaran 1, Bandar Baru Selayang, Mukim Batu, Daerah Gombak, Selangor. This is a typical rainfall induced slope failure which is triggered by rain water infiltration. The failure site is in the vicinity of the Majlis Peperiksaan Malaysia as shown in Google Map in Figure 1. Base on the observation during a site visit after the failure incidence the slope has no berm and there is no proper drainage system. This inadequacy can lead to excessive infiltration during rainfall. The overall inclination of the slope is about 60° and the slope height is 40 m. The failed area is 30 m width and 35 m height as shown in Figure 2.
Figure 1. Location of the failed slope at Jalan Persiaran 1, Bandar Baru Selayang, Mukim Batu, Daerah Gombak, Selangor.

Figure 2. Size of the failed section is 30m width and 35m height.

2. Site investigation works and slope characterisation
Two borehole tests were conducted after the failure incidence to investigate the cause of failure and their locations are shown in site plan in Figure 3. The bedrock is granite and the overlaying soil is granitic residual soil. Borehole BH2 is at the crest and the bedrock is located at 6.5 m deep and at borehole BH1 which is at the toe the bedrock is deeper than 15.5 m and the actual location is unknown since this borehole was terminated at depth 15.5 m after reaching SPT-N greater than 50 blows per 300 mm five consecutive times. The characterization of the soil layers was done by considering a hard layer when the SPT N is equal or greater than 30 blows/300 mm. At BH1 the hard layer is located at 3 m depth and at BH2 the hard layer is located at 6 m depth. The stability analysis was carried out for cross section at chainage CH50 as shown in Figure 3. This chainage runs aligned with the failure run-out direction compared to the initial survey cross-sectional lines which are slightly skew from the direction of failure. The interpreted soil layers from the two borehole tests for the cross section at chainage CH50 and the profile of the sliding failure surface is as shown in Figure 4. This failure surface will be applied in the
back analysis to seek the soil condition during failure. There is no groundwater table (GWT) detected in the two boreholes. Particle size distribution curve shows that the top soil layer of about 8 to 10 m deep is well graded clayey SAND.

Figure 3. Site plan.

Figure 4. Slope cross section at CH50 interpreted from borehole tests.

3. Determination of non-linear soil shear strength behaviour using Consolidated Undrained Triaxial Test

The shear strength of the top soil layer where the failure has occurred is very important and thus was determined by carrying out CIU triaxial tests. Four undisturbed soil specimens of 50 mm diameter and 100 mm height were obtained using steel split spoon samplers for the triaxial tests. The samplers were dipped...
into the ground by carefully knocking using a hammer so that the overall soil structure is maintained. This was followed by digging the soil around the sampler up to the bottom end of the split spoon sampler and carefully cut the soil along the bottom face of the sampler. Figure 5 shows the dipping of sampler into the ground and Figure 6 shows the resulted undisturbed soil samples contained in the split spoon samplers and wrapped with plastic. Figure 7 shows the cylindrical specimen taken out from the split spoon and ready to be assembled in CIU triaxial test machine. Figure 8 shows the specimen mounted in the CIU triaxial machine. However, the shear strength of the underlying hard layer was deduced from the correlation between the SPT N values and the friction angle.

![Figure 5. The dipping of the split spoon sampler into the ground to obtain undisturbed soil specimen for CIU triaxial tests.](image1)

![Figure 6. The undisturbed soil samples contained in the split spoon samplers and wrapped with plastic.](image2)
Figure 7. The cylindrical soil specimen obtained after opening the split spoon sampler.

Figure 8. Conducting CIU triaxial tests.

The stress-strain curves obtained from Consolidated Undrained Triaxial Tests are shown in Figure 9 and the deviator stresses, pore-water pressures and cell pressures at failure for the four CIU tests are shown in Table 1. The interpreted Mohr circles at failure and the deduced soil shear strength linear [14] and non-linear failure envelopes [12] are shown in Figure 10 and the corresponding soil shear strength parameters are shown in Table 2. This shows that when the failure envelope is considered linear and being extrapolated to the low stress range then the cohesion intercept would be 8 kN/m$^2$ whereas in effective stress analysis the cohesion intercept should be zero even though it is a clay soil.
Figure 9. Stress-strain curves obtained from Consolidated Undrained Triaxial tests.

Table 1. Deviator stresses, pore-water pressures and cell pressures at failure.

| Soil Sample and targeted effective stress (kPa) | Cell Pressure at Failure, $\sigma_3$ (kPa) | Pore Pressure at Failure, $u_w$ (kPa) | Deviator Stress at Failure, $\Delta \sigma$ (kPa) | Effective Stress at Failure, $\sigma'$ (kPa) |
|-----------------------------------------------|------------------------------------------|------------------------------------|------------------------------------------|-----------------------------------------|
| A Eff. stress 20                              | 453.4                                    | 431.9                              | 48.9                                     | 21.5                                    |
| B Eff. stress 35                              | 500                                       | 463.8                              | 64                                       | 36.2                                    |
| C Eff. Stress 50                              | 550                                       | 495.7                              | 87.2                                     | 54.3                                    |
| D Eff. Stress 70                              | 600                                       | 527.6                              | 110.2                                    | 72.4                                    |

Table 2. Interpreted linear and non-linear soil shear strength parameters.

| Shear Strength Model                  | Parameters | Top soil layer | Bottom soil layer |
|---------------------------------------|------------|----------------|-------------------|
| Curved surface shear strength envelope| $\phi_{min}$ (deg) | 22             | 36                |
|                                        | $\tau_c$ (kN/m²) | 27             | 95                |
|                                        | $(\sigma - u_w)_{min}$ (kPa) | 46             | 120               |
|                                        | Cementation, $c'$ | 0              | 0                 |
| Linear shear strength envelope        | $\phi'$ (deg) | 22             | 36                |
|                                        | Cohesion, $c'$ (kN/m²) | 8              | 8                 |
Figure 10. The interpreted Mohr circles at failure and the deduced soil shear strength linear and non-linear failure envelopes.

4. Slope stability analysis applying the state-of-the-art infiltration type slope stability method

Slope failure due to rainfall is triggered by the advancement of the wetting front into the slope which is termed as infiltration [2,5,13]. The infiltration type slope stability method replicates exactly the mechanics of slope failure due to rainfall. This is in contrary to the conventional method which assumes the failure due to rainfall is triggered by the rise of the GWT as resulted from the continuous rainfall despite that it is located far down [1]. Most importantly, this state-of-the-art slope stability method applies the curved-surface soil shear strength model which is the shear strength model of Md Noor and Anderson [12] which can replicate very well the non-linear shear strength behavior with respect to effective stress and suction for tropical residual soil. The shear strength of the soil layers as in Figure 10 is interpreted based on curved-surface soil shear strength model of Md Noor and Anderson [12] applying zero cohesion intercept and linear envelope with cohesion intercept. According to Lade [4] there is no cohesion intercept for soils according to effective stress analysis and the application cohesion intercept would produce a higher and fake value of FOS. The application of the curved-surface soil shear strength model would take into account the steep drop in the apparent shear strength as suction approaches zero when surface water infiltrated into the slope and this is a very important influence of soil shear strength on the slope stability that needs to be considered in analysis [12].

In Infiltration Type Slope Stability Method introduced by Md Noor [10,11] and Md Noor et al [13] the slope is considered to be infiltrated and the bottom portion of the wetted band is bounded by a line called wetting front. Zone above the wetting front is considered having zero suction and the underlying zone is considered to be in partially saturated condition. The critical slip surface is essentially when the slip circular failure surface is within the wetted zone [6]. The typical slice considered under this Infiltration Type Slope Stability Method is as shown in Figure 11. The slope stability equation and the applied shear strength equations are shown in Figure 12. When the slip circle is within the wetted zone then the suction at the base of all the slices is taken as zero and this will eliminate the second term in the four shear strength equations in Figure 12. Thence the mobilized shear strength at the base of the slices depend on the stress, $\sigma$ only since the pore air pressure, $u_a$ is zero or the pore water pressure, $u_w$ is also zero when the soil is fully wetted. In other words the effective stress at the base of the slice becomes;
And this simplified the calculation for the shear strength at the slice base.

\[
(\sigma - u_d) = (\sigma - u_w) = \sigma' = \frac{W \cos \alpha}{\beta} \tag{1}
\]

Figure 11. Typical slice considered in the infiltration type slope stability method.

In this slope stability investigation of a failed slope, the slip failure surface is already a known variable. The pre-determined failure surface needs to be applied in the stability analysis and the critical depth of wetting front needs to be investigated. The stability of the slope before being infiltrated is 1.94 as shown in Figure 13. The FOS will decrease when water starts to infiltrate into the slope. Figure 14 shows the stability of the slope is 1.76 when being infiltrated for 1m deep. When the slope is being infiltrated for 2, 3 and 4 m deep the FOS are 1.66, 1.50 and 0.92 as shown in Figures 15, 16 and 17 respectively. Essentially the slope failed at infiltration depth of 4m and this is in agreement with the depth of the actual failure surface encountered at the failed slope.

Figure 12. The algorithm for the calculation of the FOS in the Infiltration Type Slope Stability Method.
Figure 13. Stability of slope before being infiltrated.

Figure 14. Stability of slope when infiltrated for 1m deep.

Figure 15. Stability of slope when infiltrated for 2 m deep.
5. Conclusions
The conclusions that can be drawn from this slope failure investigation are :-

[1] The Infiltration Type Slope Stability Method is very appropriate to check stability of rainfall induced shallow slope failure since it is in compliance with the mechanism of such failure.

[2] It is very important to apply the true non-linear soil shear strength behaviour with zero cohesion intercept so that the shear strength within the shallow sliding mass is not over-estimated in order to achieve a realistic FOS of the slope.

[3] The slope has factor of safety greater than unity before being infiltrated and when the depth of infiltration are 1, 2 and 3 m to indicate that the slope is stable if it is not affected by surface water.

[4] The FOS of safety decreases when depth of infiltration increases.

[5] Essentially the investigated slope failed when the infiltration reached 4m deep and this is in agreement with the actual depth of failure encountered in the field. This demonstrated the accuracy of applying the Infiltration Type Slope Stability Method to check for potential rainfall induced failure.
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