

Landslides : Abuses of the Prescriptive Method

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Abstract

Based on the Authors' investigations of 49 cases of landslides over the last six years, 60% of the failures are due to design alone, and the rest are either due to construction errors, a combination of design and construction errors, geological features or maintenance. The majority of these design errors were contributed by abuses of the prescriptive method. This paper highlights the common abuses of the prescriptive method in designing slopes in residual soils, and the lessons learned from this expensive damage and loss. This paper is intended for civil engineers who are involved in slope analysis and design rather than geotechnical specialists who are experts in this topic.

Keywords: Landslide; Abuses; Prescriptive Method; Residual Soils; Slopes; Lessons Learned

1.0 INTRODUCTION

Every year during the monsoon seasons, the occurrence of landslides is common in Malaysia. These landslides cause the closure of roads, affect buildings or worse they sometimes cause casualties. The potential economic loss and loss of life could escalate if the causes of landslides in Malaysia are not identified and addressed properly by all parties especially engineers who are directly involved in the design and supervision of slopes and retaining walls. This paper presents statistics on the causes of landslides in Malaysia. It is based on 49 cases of landslides investigated by the Authors over the last six years. The common causes of landslides in residual soils, and lessons learned from them are presented in this paper. The abuse of the prescriptive method in designing slopes and retaining walls is also highlighted.

2.0 FACTORS ATTRIBUTED TO LANDSLIDES

49 cases investigated by the Authors over the last six years are primarily large landslides on residual soil slopes of weathering grade IV to VI. This was part of forensic engineering engagements. Large landslides are landslides which involve more than 5,000 cubic metres. Table 1 shows the percentage of landslides caused by different factors. The results of the investigations indicate that 60% of the failures are due to inadequacy in design alone. The inadequacy in design is generally the result of a lack of understanding and appreciation of the subsoil conditions and geotechnical issues. Failures due to construction errors alone either of workmanship, materials and/or lack of supervision contributed to 8% of the total cases of landslides. About 20% of the landslides investigated are caused by a combination of design and construction errors. For landslides in residual soil slopes, the landslides caused by geological features only account for 6% which is same as the percentage contributed by lack of maintenance.

Table 1 : Causes of Landslides

Causes of Landslides	Number of Cases	Percentage (%)
Design Errors	29	60
Construction Errors	4	8
Design and Construction Errors	10	20
Geological Features	3	6
Maintenance	3	6
Total	49	100

The results clearly reveal that the majority of these failures were avoidable if extra care was taken and input from engineers with relevant experience in geotechnical engineering was sought from planning to construction. Many of the landslides which were caused by design errors reported above were due to the following:-

- 1) The abuse of the prescriptive method on the slope gradient (slope angle) to be adopted for cut or fill slopes without proper geotechnical analyses and calculations. It is very common in Malaysia to find many cut slopes formed for residual soils that are 1V:1H (which means one vertical: one horizontal = 45 degrees angle). Based on literatures published on residual soils and the authors' own experience of residual soils, it is very unlikely to have an effective angle of friction (ϕ') of the residual soils of 45° (degrees) or near to this value. The authors' own experiences indicates that the ϕ' values of residual soils generally ranges from 29° to 36° and mainly depend on the particle size distribution of the materials. Therefore, if proper analysis of the slopes' stability was carried out with correct soil parameters, most of these 45° gradient slopes would not have a sufficient Factor of Safety (FOS) recommended against slip failure in the long term, even with some effective cohesion. In summary, engineers should not only follow the slope gradients (e.g. 1V: 1H) that have been done previously, without proper geotechnical analysis and design.
- 2) Subsurface investigation (S.I.) and laboratory tests were not carried out to obtain representative soil parameters, subsoil and groundwater profiles for design and analysis of slopes. Therefore, the analysis and design carried out are not representative of the actual site conditions, and are thus unsafe.
- 3) A lack of good understanding of fundamental soil mechanics so that the most critical condition of cut slopes is in the long term (in the "Drained Condition"). Therefore, it is necessary to adopt effective shear strength parameters for the "Drained Analysis" of the cut slopes in residual soils instead of undrained shear strength (s_u or c_u).

For landslides that were caused by construction errors alone or combined with design, the common construction errors are as follows:-

- 1) Tipping or dumping of loose fill down the slopes to form a filled platform or filled slope. This is the most rampant construction error for earthworks construction in Malaysia. Contractors carrying out the filling works on slopes will find it most "convenient" and "easy" to dump or tip soil down the slopes to form the fill. The condition is worsened by not removing the vegetation on the slopes causing the bio-degradable materials to be trapped beneath the dumped fill, forming a potential slip plane with a very low friction angle of the bio-degradable materials (vegetation). The uncompacted fill slopes having a very low Factor of Safety will likely fail in the long term.
- 2) Errors of the construction method such as forming cut slopes by excavating slopes from the bottom (undermining) instead of the correct practice of cutting from the top downwards. This wrong practice will trigger landslides or potential shear planes extending beyond the proposed cut slope profile.

- 3) Over-excavation of cut slopes. Contractors unintentionally over-excavate cut slopes and then try to fill back the excavated materials to reform the slope to the required gradient. The uncompacted loose materials will slip down.

The way to prevent these bad construction practices is to have proper full-time supervision by members of the design consultant together with reliable and responsible earthworks contractors having clear method statements for construction. Failure of slopes and retaining walls can also take place if the temporary works (e.g. temporary excavation) are not properly designed and constructed.

Landslides due to geological features contributed to about 6% of the total failures investigated. However, it should be recognised that these geological features such as discontinuities in residual soils, especially sedimentary formations, are not usually detectable during the design stage even with extensive subsurface investigation (boreholes, geo-physical method), by an experienced engineering geologist who carries out geological mapping at the site prior to cutting. Most of these geological features can only be detected after exposing the slopes during excavation. In view of this, it is best to carry out confirmatory geological slope mapping of the exposed slopes (after excavation), by an experienced engineering geologist or geotechnical engineer to detect any geological discontinuities that may contribute to potential failure mechanisms, namely planar sliding, anticline sliding, active-passive wedges, etc.

By understanding that geological discontinuities could not be fully addressed during the design stage, design engineers should make moderately conservative assumptions about the soil/rock parameters and also the groundwater profile to ensure adequacy in design and should only carry out adjustments on site after geological slope re-mapping and re-analysis of the slopes. On the contrary, when optimistic assumptions are made and the results obtained during construction on site are less favourable then expensive options such as retaining walls or slope strengthening using soil nails are required due to space and boundary constraints. Thus the safety of slopes is often compromised.

The common problems of landslides caused by a lack of maintenance are blockage of drains for surface run-off, and erosion. Blockage of drains will cause large volumes of water to gush down a slope causing erosion to the slope and the formation of gullies. These gullies will further deteriorate into a big scar on the slope and finally lead to a landslide. The blockage of drains could also be due to debris accumulated on cracked drains, the collapse of drains, etc. If proper maintenance is carried out, then all these small defects would have been rectified and landslides caused by erosion would be prevented.

3.0 SELECTION OF SOIL STRENGTH PARAMETERS FOR RESIDUAL SOILS

Subsurface investigations (S.I.) should be properly planned to obtain the representative subsurface conditions and necessary soil parameters of the whole site for a development or along the alignment of a proposed road. For details on planning of subsurface investigation and interpretation of test results for geotechnical design, reference can be made to Gue & Tan (2006) and Gue & Tan (2000).

The two most important parameters needed to analyse and design cut slopes in residual soils are the effective stress strength parameters (c' & ϕ') and the groundwater level. This key information should be properly obtained when carrying out a subsurface investigation. The groundwater profile can be obtained from standpipes installed in boreholes to allow monitoring of groundwater in the boreholes to be carried out even after S.I. field works and over a period of at least one monsoon. In order to obtain effective stress strength parameters (c' & ϕ') for residual soils, undisturbed soil samples should be collected from boreholes using a Mazier Sampler (Retractable triple-tube Core-barrel) with a sample diameter of about 70mm. Foam drilling can improve the recovery of Mazier sampling. In situations where Mazier sampling recovery is

bad/insufficient and foam drilling is not feasible, another method of obtaining “undisturbed” soil samples from stiff residual soils is the use of the Thick Wall sample (sample diameter of 70mm) which is hammered into the hard soil. Although the sampling process using thick walls will cause some disturbance, the effect is not significant for stiff residual soils and the samples collected can still be used for laboratory strength tests. Undisturbed soil samples can also be collected at shallow depth using block sampling which is very useful for collecting high quality undisturbed soil samples during the excavation of slopes. These undisturbed residual soil samples shall be sent to the laboratory for a series of classification and strength tests.

For cut slopes, effective stress conditions (drained or long term conditions) are normally more critical than total stress (undrained) conditions. Therefore, the effective stress strength parameters c' and ϕ' , determined from testing representative samples of matrix materials, are used in the analysis. In Malaysia, Isotropic Consolidated Undrained Triaxial Tests (CIU) are commonly carried out on large diameter undisturbed soil samples (from a Mazier sampler without trimming and side drains). It is important that soil samples are tested at stresses comparable to those in the field, and should be saturated. It is appropriate to measure strength parameters on saturated soil samples because residual soils are usually of high permeability (usually 10^{-4} to 10^{-6} m/sec). Prolonged and high intensity rainfall, especially during the two monsoon periods every year, allows rainwater to infiltrate into it with ease and it is likely that saturation conditions will be approached at shallow depths in the field during the service life of a slope.

The shear strength of soil may be represented graphically on a Mohr diagram. For simplicity of analysis, it is conventional to use a c' - ϕ' soil strength model for saturated and unsaturated soil as expressed in the equations below respectively:

$$\tau_f = c' + \sigma_{nf}' \tan \phi' \quad \text{(for Saturated Soils)} \quad \text{----- (Eq. 1)}$$

$$\tau_f = c' + (u_a - u_w) \tan \phi^b + (\sigma_{nf}' - u_a) \tan \phi' \quad \text{(for Unsaturated Soils)} \quad \text{----- (Eq. 2)}$$

- where
- τ_f = shear strength of soil.
 - σ_{nf}' = effective normal stress at failure.
 - ϕ' = effective angle of friction (degree).
 - c' = apparent cohesion (kPa).
 - u_a = pore-air pressure
 - u_w = pore-water pressure
 - ϕ^b = Angle of shear strength change with a change in matric suction.(degree).

The shear strength of the soil for unsaturated soils as in Eq. 2 above has included suction in the calculated soil strength which will give higher shear strength compared to saturated soils. However, to date it is not advisable to include soil suction (negative pore pressure) in the design of long term slopes in view of many factors that can cause the loss of suction e.g. prolonged and high intensity rainfall, etc. Most steep cut slopes have not yet collapsed because of the presence of soil suction, but if the suction is lost, these slopes will collapse. The most prominent example is that a slope can stand at a very steep angle (even near vertical) immediately after excavation but with time or after rain, the slope will collapse.

In view of the great uncertainty of relying on the stability of slopes with soil suction, the following section will only concentrate on saturated soil shear strength, which is commonly used in analysing and designing slopes. Figure 1 shows the typical bonding and dilatant characteristics of the residual soil at a low stress range (low confining and consolidation pressure) which exhibits a peak shear strength envelope in terms of effective stress which has an apparent cohesion intercept (c') if the Mohr-Coulomb c' - ϕ' failure line is used. As the

consolidation pressure in laboratory tests prior to shearing increases beyond its

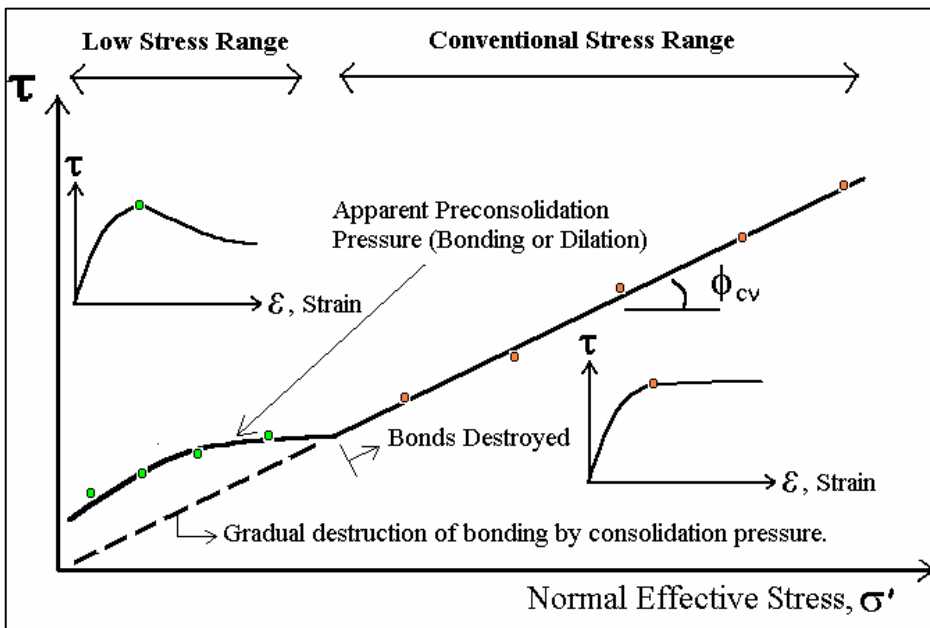


Figure 1 : Effect of Bonding on the Apparent Cohesion Intercept of a Drained Strength (Effective Stress) Failure Envelope.

yield stress, the bonds are destroyed and residual soil will likely behave like normally consolidated or slightly overconsolidated transported soil. The critical state friction angle is represented as ϕ_{cv} .

It is important to be aware that c' and ϕ' parameters are not intrinsic soil properties, but are merely coefficients in the simplified design model and should only be assumed to be constant within the range of stresses for which they are evaluated as shown in Figure 2.

Brand (1995) states that most of the critical slip surfaces in residual soil slopes are commonly shallow with effective stress of typically of about 30 to 200kPa. He also reported that there is some evidence suggesting that the strength envelopes for some residual soils are curved at low effective stresses, and that the straight-line projection of strengths measured at high stresses under-estimates the shear strengths in the low stress range. Therefore, for different stress ranges, c' and ϕ' values could be adopted using the method shown in Figure 2.

Figure 2 illustrates a typical stress-strain curve for residual soil. A sample is isotropically consolidated (Point A) then sheared to reach the peak strength (Point B) at a low stress range and continued shearing until the critical state strength (Point C) is reached. Normally the peak strength is obtained at a relatively small strain and after continued shearing, the critical state strength (ϕ_{cv}) is obtained at a larger strain. The critical state usually occurs in the 10% to 30% strain range where a soil sample continues to shear at constant volume and constant effective stress. The critical state strength is also called the ultimate strength (Atkinson & Bransby, 1978) or the fully softened strength (Skempton, 1970). The critical state strength is also different from the residual strength (Skempton, 1964) which is lower and occurs after very large movement on the slip/failure surface. The residual strength is also associated with highly polished slip surfaces in which soil particles have become aligned in a direction parallel with the direction of sliding, and is relevant only after displacements of the order of several meters (Crabb and Atkinson, 1991).

As shown in Figure 2, the critical state strength falls on a straight line through the origin. The conventional interpretation of peak failure strength is the Mohr-Coulomb shear strength envelope ($c' - \phi'$) at the stress range using the tangent method. It should be noted that ϕ' is different from ϕ_{cr} (critical state); and c' is simply the intercept of the peak failure envelope on the shear stress axis, τ' . It is important to note that c' does not imply that at zero effective stress the strength is of that value. It is just a parameter in the Mohr-Coulomb shear strength envelope. Therefore, at very low effective confining stress (outside the representative stress range),

the Mohr-Coulomb failure envelope ($c'-\phi'$) may overestimate the strength of a soil. On the other hand, if critical state strength is used in the normal stress range of a slope, the strength value will be underestimated, giving an unrealistically low Factor of Safety (FOS). Therefore, when the in-situ stress range and the stress path during shearing is correctly determined, the $c'-\phi'$ peak shear strength envelope will be representative of the field conditions.

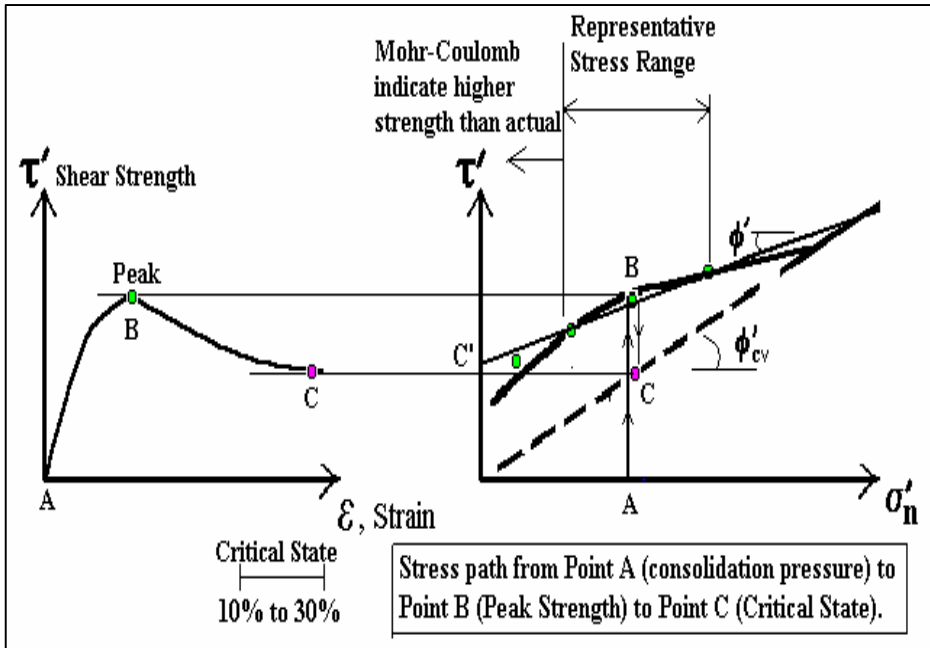


Figure 2 : Typical Shearing Characteristics of Residual Soil and the Tangent Method in Selection of Shear Strength Envelope.

For residual soils of grades IV to V that are very stiff, as indicated by indirect values of actual and extrapolated Standard Penetration Tests (SPT) blow counts of high value, these materials usually have bonds between soil particles. These bonds are a component of strength that can be reflected as apparent cohesion, c' and stiffness that is independent of effective stress and void ratio/density. For the strength and stiffness of the soil as a large mass in-situ, the bond actually has a significant influence. The bonding also contributed to the 'apparent'

overconsolidated behaviour of the soils. Vaughan (1988) highlighted some of the possible causes of the development of bonds as :-

- Cementation through the deposit of carbonates, hydroxides, organic matter, etc.
- Pressure solution and re-precipitation of cementing agents, such as silicates.
- Cold welding at particle contacts subject to high pressure.
- Growth of bonds during chemical alteration of minerals.

Figure 3 shows the relationship between the peak effective angle of friction (ϕ'_{peak}) and the percentage of fines (silt & clay) in the residual soils obtained from thirteen (13) different sites. It is observed that the value of ϕ'_{peak} generally falls between 26° to 36° and there is a trend showing reduction of ϕ'_{peak} with increasing fines content. Therefore, during selection of ϕ' for design, it is important to aware of the common range of values for the type of soils.

Generally, c' is taken at 0 (zero) unless there are sufficient test results to obtain c' values or from back analyses of similar residual soils (in terms of strength, stress range, etc). Sometimes, unrealistically high c' value could be wrongly obtained from laboratory tests due to the rate of strain or time of shearing to failure being too fast. The rate of strain should be estimated from the results during consolidation. Side drains should not be used as this has been shown to produce inconsistency in the sample (Tschebotarioff, 1950 and GCO, 1991). Multistage tests should also not be used as the second test will be significantly affected by the failure surface formed in the first test (GCO, 1991). Further details of the laboratory tests can be obtained from Head

(1986). Figure 4 shows c' obtained from thirteen (13) different sites. It is obvious that the c' value is generally less than 10kPa.

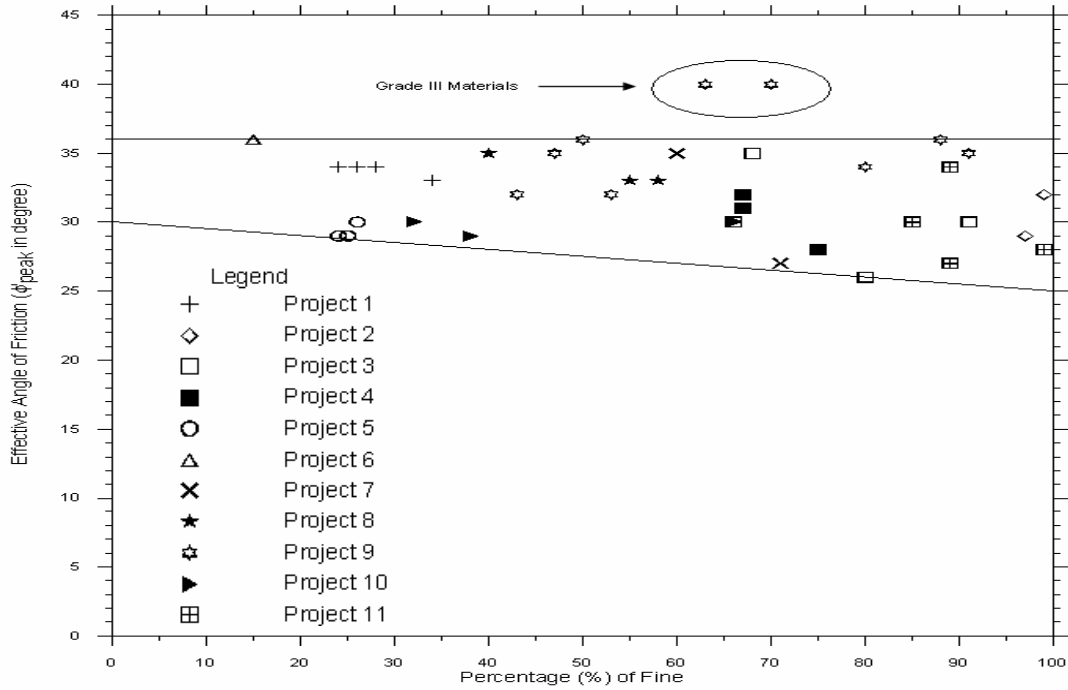


Figure 3 : ϕ'_{peak} versus Percentage of Fines in Residual Soils

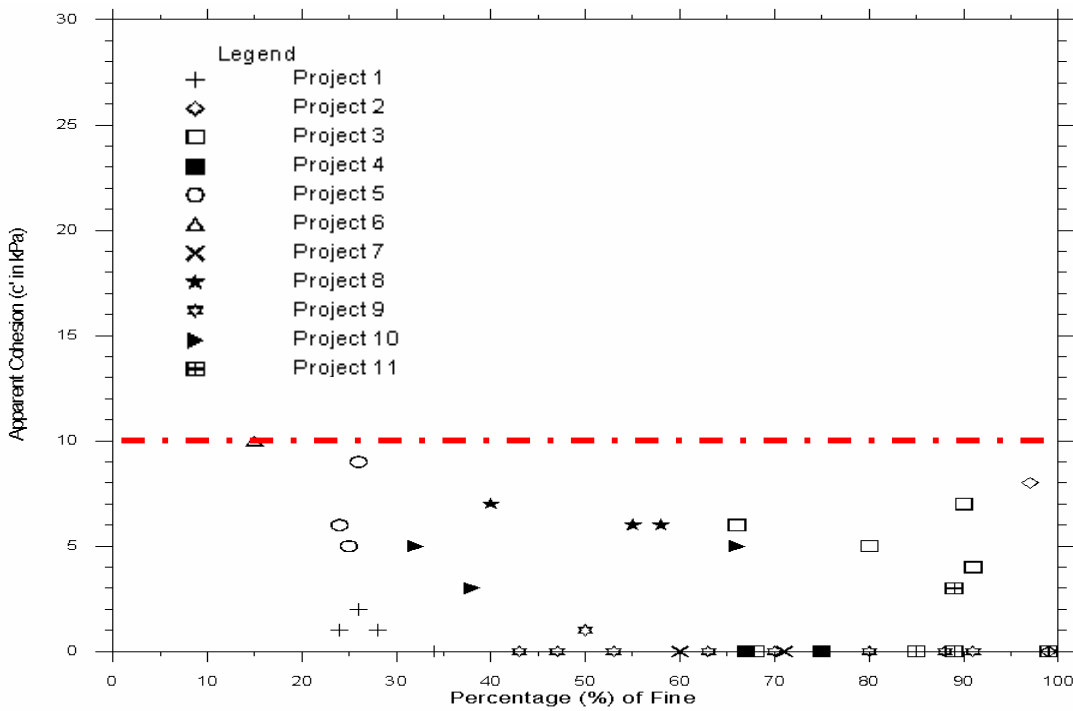


Figure 4 : c' versus Percentage of Fines in Residual Soils

4.0 ABUSES OF THE PRESCRIPTIVE METHOD

Abuses of the prescriptive method by engineers on the selection of slope gradient (slope angle) to be adopted for cut or fill slopes have been quite rampant in Malaysia. The slope gradient was specified without any proper geotechnical analyses and calculations. It is common in Malaysia to find many residual soil cut slopes that are 1V:1H (which means one vertical: one horizontal = 45 degrees angle).

The proposed gradient was borrowed from obsolete methods for slopes along roads adopted in the early sixties and seventies in Malaysia. This obsolete method does not warrant an adequate Factor of Safety (FOS) against failure required for slopes with high risks to life (to take care of public safety) and high risks of economic loss. The FOS against slope failure recommended for high risk-to-life and high economic risk is 1.4. In addition, previous slopes along roads are commonly either of one to two berms height (less than 10m) and therefore the impact of landslide is less severe. There are signs indicating that the abuses of the prescriptive method had been extended to retaining walls and soil nailed slopes designs. This is of major concern for the safety of the public as the consequences of failure of high walls or high soil nailed slopes will be much more severe.

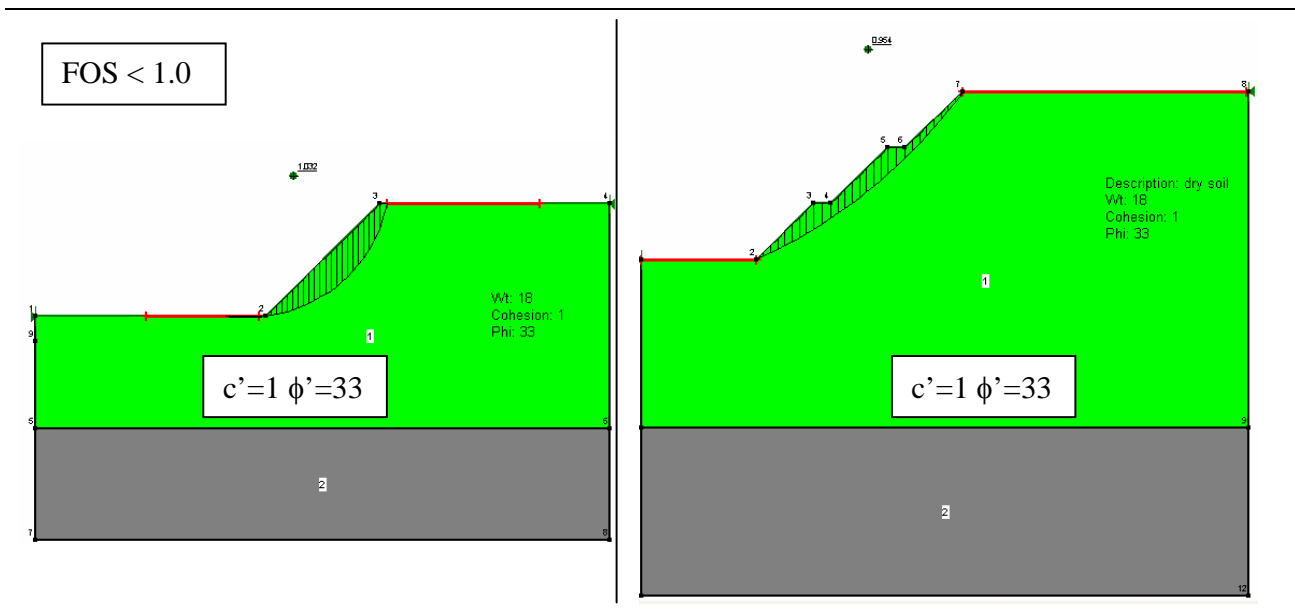


Figure 5 : Insufficient Factor of Safety (FOS) for 1v:1h (45 degree) slope based on moderately conservative soil parameters and ground water profile.

In order to show the seriousness of specifying unsafe slope gradients, we show a simple stability analysis for typical cut slopes and also soil nailed slopes using soil strength parameters that are common for residual soils. Figure 6 shows the typical cut slopes in residual soil with slope gradient of 1V:1H (45 degree), based on a soil strength of $c' = 1 \text{ kPa}$ and $\phi' = 33^\circ$ which is a typical value for residual soils in Malaysia. The Factor of Safety (FOS) obtained is 1.0 (imminent failure) even without groundwater. However, if the soil has higher shear strength ($c' - \phi'$), then the FOS will increase and vice versa if the shear strength is lower. Design engineers must be aware that every slope should be designed based on the representative soil strength parameters, ground water levels, unit weight and correct geometry instead of relying on bad past experiences using the prescriptive method. Although there are many cut slopes in Malaysia that are 45 degrees and have not failed, but these slopes are either assisted by suction (which is not reliable in long term) or they are weathered rocks

(class II to III) with high effective shear strength. Understanding fundamental soil mechanics is necessary when designing slopes to prevent failure.

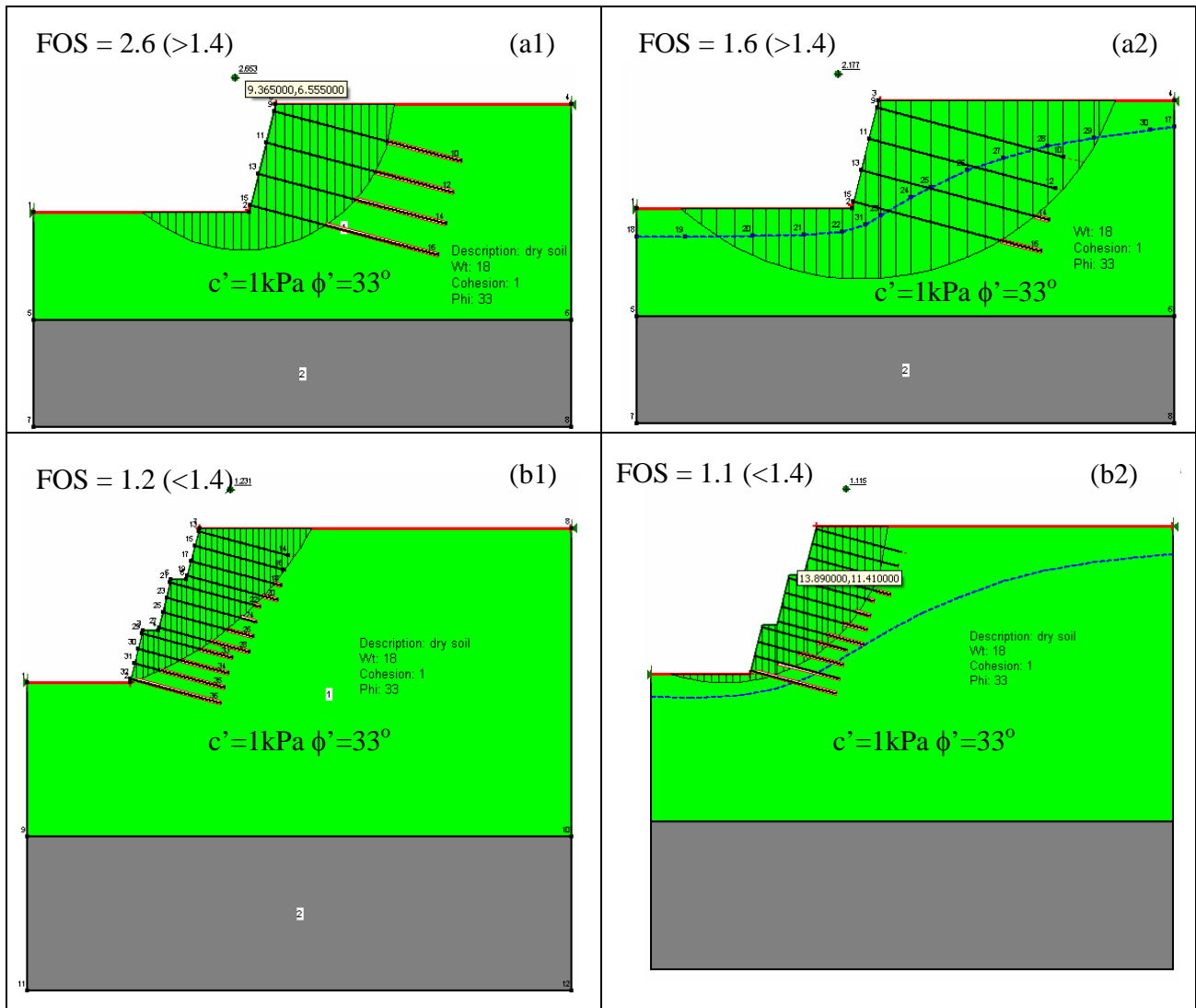


Figure 6 : FOS Obtained for Soil Nailed Slopes with 9m Length Nails.

Figure 6 shows the typical soil nails reinforced slopes in residual soil with a slope gradient of 4V:1H, based on a soil strength of $c' = 1\text{kPa}$ and $\phi' = 33^\circ$. For all four analyses, the soil nails adopted are 9m in length. The Factor of Safety (FOS) of a single berm reinforced slope (5m high) with and without groundwater is higher than required at 1.4. However as the height of the slopes increases to three (3) berms, the FOS reduces to 1.2 and 1.1 respectively for dry slopes and slopes with groundwater. It is obvious and simple that as the height of the slope increases, the length of the nails should be increased and should be at closer spacing to improve the FOS. However, it is still common in Malaysia to find the opposite. Liew & Liong (2006) highlighted two cases of soil nailed slopes failures and Chow & Tan (2006) presented design methodologies for proper design

of soil nails. In summary, it is dangerous and irresponsible to abuse the prescriptive method and not carry out proper design for slopes, either reinforced or not reinforced.

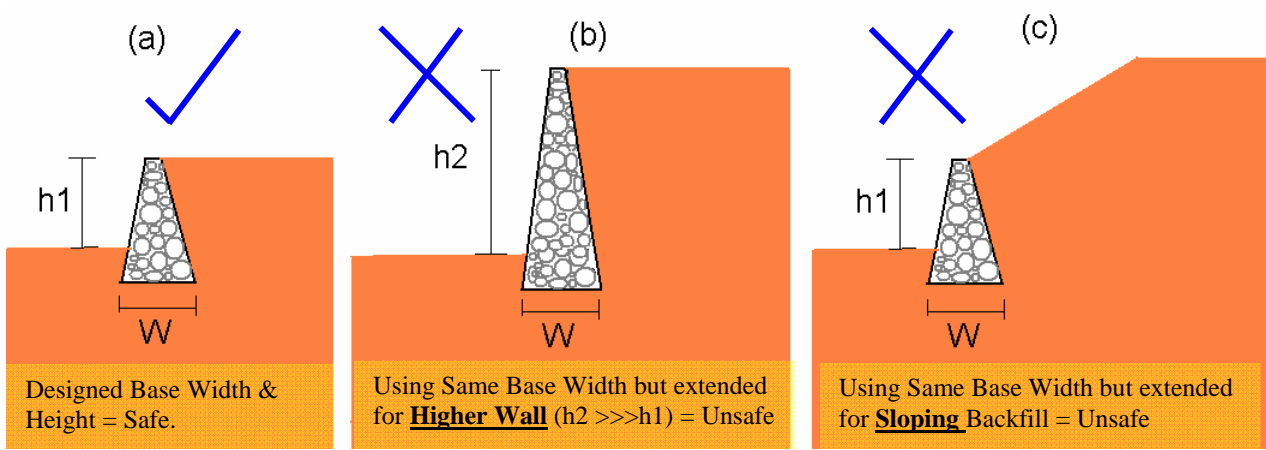


Figure 7 : Example of design errors for rubble walls.

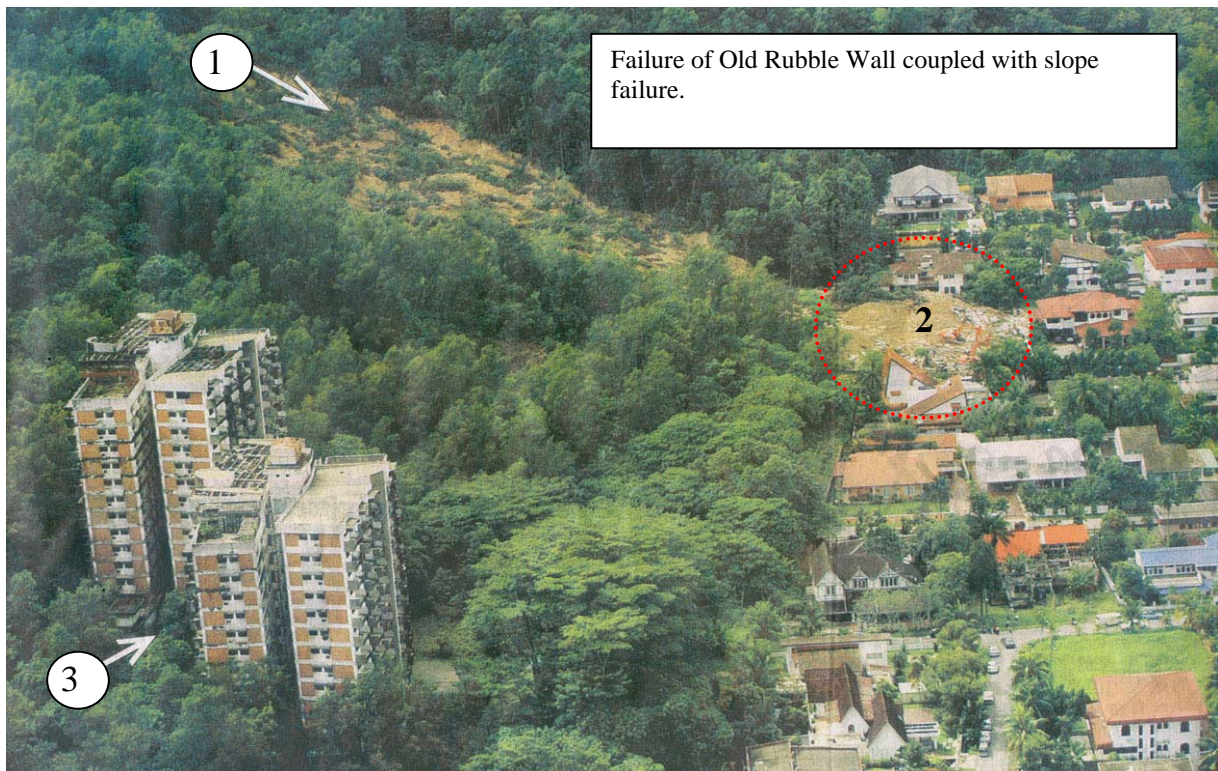


Figure 8 : Wall and Slope Failures at Taman Hillview (2002). (1) Location of unsafe rubble wall that failed (2) Location of Collapsed Bungalow (3) Remaining Two Blocks of Highland Tower that Collapsed in 1993.



Figure 9 : Close up view of Collapsed Rubble Wall at Taman Hillview failure site.

Another common error of design or construction is adopting the same design as one used earlier for different configurations without further analysis. Figure 7 shows common errors that occurred in Malaysia. Figure 7a shows a rubble wall which is properly designed with adequate FOS against bearing capacity, sliding, overturning and also slip failure. However sometimes due to site conditions, the retained height has to be increased on site. The wall was constructed wrongly following the original drawings without changing the base width (W) despite the height (h_2) having increased. Therefore, the wall is not safe and will fail. The Authors' believe that this is the one of the main causes of wall failure at Taman Hillview that claimed eight lives. Figure 8 shows a picture of the Taman Hillview failure site and Figure 9 shows the close up view of the unsafe rubble wall after failure.

The prescriptive method can be useful if applied properly and complying with all the following conditions :

- a) Proper analysis and design previously carried out for similar slope configurations and geometry, and
- b) Same Geological Formation, and
- c) Same subsoil conditions, and
- d) Same groundwater conditions.

5.0 CONCLUSION

It is time for all engineers in Malaysia to stop abusing the prescriptive method, to protect the public. With the present development of geotechnical engineering in Malaysia, we have the experience and knowledge to carry out proper geotechnical design of slopes to ensure safety and mitigate the risk of landslides.

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