

THE DETERMINATION OF SHEAR STRENGTH IN RESIDUAL SOILS FOR SLOPE STABILITY ANALYSIS

Ir. Tan Yean Chin & Ir. Dr. Gue See Sew
Gue & Partners Sdn. Bhd.

1.0 INTRODUCTION

Selection of strength parameters for design of cut slopes in residual soils in Malaysia has mostly followed standard procedure of employing triaxial and direct shear box tests in the laboratory and penetration tests in the field (e.g. Standard Penetration Tests and Piezocone Tests). It is pertinent to have a clear understanding of the different characteristics between transported soils (e.g. alluvium, etc.) and residual soils.

This paper presents a brief definition of residual soils together with a brief guideline on the relevant laboratory testing and selection of strength parameters of the residual soils for stability analysis of cut slopes. The importance of effective stress path in influencing the shear strength and factor of safety (FOS) of the slope are discussed. Finally, the paper also highlights that many slope failures due to an increase in pore water pressure which has not been accounted for in the design.

2.0 PLANNING OF SUBSURFACE INVESTIGATION

Prior to the planning of the subsurface investigation, the designer should carry out desk study which includes reviewing of geological maps and memoirs to understand the geological formation. Topographic map, laser topography (if available), aerial photographs of the site or main alignment and adjacent areas should be examined for the geomorphology features, previous and present land use, current development, construction activities, problematic areas like previous slope failure, etc. The knowledge of the site histories particularly previous landslides is a very important factor for the design of the cut and fill slopes in mountainous terrains.

Site reconnaissance is required to confirm the information acquired from the desk study and also to obtain additional latest information from the site. It is also very important to locate and study the outcrops to identify previous landslides or collapse that can act as an indicator of the stability of the existing slopes.

Subsurface investigation (S.I.) for the design of slopes in mountainous terrains should be carried out in two stages or more. Preliminary S.I. usually consists of boreholes and sometimes also include geophysical survey. The locations of the field tests should be carried out with the intention to obtain the overall subsurface condition of the site or main alignment like general depth of soft soil, groundwater conditions, hard stratum and most important, the bedrock profile at the proposed cut areas.

The boreholes in Preliminary S.I. are usually spread out to cover the whole site or main alignment in typical cluster and placed at areas of potential major cuts and fills. Disturbed and undisturbed soil samples should also be collected from the boreholes to carry out laboratory tests to obtain the necessary soil and rock parameters for preliminary geotechnical design of the slopes, foundations and retaining walls. In addition, the ground water profile should also be established. In sensitive and critical areas, long term monitoring of water table is also needed.

The general information on the subsurface profile and properties will be useful when planning the cut and fill because the depths of hard stratum and bedrock will have major influence on the cost and time for earthworks. Once the areas of uncertainties in the subsoil have been identified and the cut and fill layout have been confirmed, the detailed S.I. should be carried out to obtain the necessary information for detailed geotechnical designs. In the detailed S.I., field tests can be carried out at the following locations :

- Critical areas of uncertainties.
- Areas of major cut and fill.
- Areas of soft ground.
- Retaining walls.
- Structures with Heavy Loading (Bridges)

For details on the planning of subsurface investigation and interpretation of test results for geotechnical design, reference can be made to Gue & Tan (2000) and Gue (1995).

3.0 RESIDUAL SOILS

3.1 General

Generally residual soils can be defined as 'a soil-like material derived from the in-situ weathering (both physical and/or chemical weathering) and de-composition of rock which has not been transported from its original location' (Blight, 1997). The amount of weathering and the balance between physical, chemical and biological depends principally on the climate and parent rock material as well as localized influences such as drainage, topology and vegetation.

Residual soils can have characteristics that are quite different from those of transported soils. Particles of residual soil usually consist of aggregates or crystals of weathered mineral matter that break down and become progressively finer if the soil is manipulated. The permeability of residual soils may not be related to its granulometry (e.g. by the well-known Hazen formula) like transported soils, instead it is usually governed by its micro and macro fabric, jointing and by superimposed features such as slickensiding, termite and other bio-channels.

Descriptive Term	Grade	General Characteristics
Residual Soils	VI	- Original rock texture completely destroyed. - Can be crumbled by hand and finger pressure.
Completely Decomposed	V	- Rock wholly decomposed but rock texture preserved. - No rebound from N Schmidt hammer - Can be crumbled by hand and finger. - Easily indented by point of geological pick. - Slakes when immersed in water. - Completely discoloured compared with fresh rock.
Highly Decomposed	IV	- Rock weakened and can be broken by hand into pieces. - Positive N Schmidt rebound value up to 25. - Makes dull sound when struck by hammer. - Geological pick cannot be pushed into surface. - Does not slake readily in water. - Hand penetrometer strength index greater than 250kPa. - Individual grains may be plucked from surface. - Completely discoloured compared with fresh rock.
Moderately Decomposed	III	- Usually cannot be broken by hand but easily broken by geological hammer. - N Schmidt rebound value 25 to 45. - Makes dull or slight ringing sound when struck by hammer. - Rock material not friable. - Completely stained throughout.
Slightly Decomposed	II	- Not broken easily by geological hammer. - N Schmidt rebound value greater than 45. - Makes ringing sound when struck by hammer. - Strength approaches that of fresh rock. - Fresh rock colours generally retained but stained near joint surfaces.
Fresh Rock	I	- No visible signs of weathering, not discoloured. - Not broken easily by geological hammer. - Makes ringing sound when struck by hammer.

Table 1 : Material Grade Classification System (modified from GEO, 1988)

In brief, the main characteristics of residual soils are :

- Very heterogeneous, which makes sampling and testing for representative parameters difficult.
- Usually high permeability. Therefore susceptible to rapid changes in material properties when subjected to changes of external hydraulic condition.

In general, the process of formation of a residual soil profile is complex and very difficult to model and generalise. Therefore, a simplified weathering profiles which contain material of different 'grades' are usually used to describe the degree of weathering and the extent to which the original structure of the rock mass is destroyed varying with depth from the ground surface. The weathering profile is an important information for slope stability analysis because it usually controls :

- The potential failure surface and mode of failure.
- The groundwater hydrology, and therefore the critical pore pressure distribution in the slope.
- The erosion characteristics of the materials.

The Geotechnical Engineering Office (GEO) (formerly Geotechnical Control Office) of Hong Kong has adopted a system for granites in which a profile is logged according to the six rock 'material grades' given by GCO (1988). Table 1 presents the modified grades classification based on the above reference for ease of classification. For geotechnical design of the slopes, materials of Grades I to III are usually treated as 'rock' and materials of Grades IV to VI as 'soil'.

3.2 Stress-Strain and Shear Strength Characteristics of Residual Soils

The selection of appropriate shear strength parameters is very crucial for the design of the cut slopes. The special features encountered with residual soils that are mainly responsible for the difference in stress-strain and strength behaviour in comparison with transported soils are listed in Table 2.

One of the significant characteristics of a residual soil is the existence of bonds between particles and these bonds is a component of strength (can be reflected as apparent cohesion, c') and stiffness that is independent of effective stress and void ratio/density. The bonding also contributed to 'apparent' overconsolidated behaviour of the soils. Vaughan (1988) highlight some of the possible causes of the development of bonds as :

- Cementation through the deposition 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.

In engineering applications, these bonds are purposely omitted because it is easily destroyed and not reliable for design. However, for the strength and stiffness of the soil as a large mass in-situ, the bond actually has a significant influence.

Factors Affecting Strength	Effect on Residual Soils	Effect on Transported Soils
Stress History	Usually not important.	Very important, modifies initial grain packing, causes overconsolidation effect.
Grain / Particle Strength	Very variable, varying mineralogy and many weak grains are possible.	More uniform; few weak grains because weak particles become eliminated during transport.
Bonding	Important Component of strength mostly due to residual bonds or cementing; causes cohesion intercept and results in a yield stress; can be destroyed by disturbance.	Occurs with geologically aged deposits, produces cohesion intercept and yield stress, can be destroyed by disturbance.
Relict structure and discontinuities	Develop from pre-existing structure or structural features in parent rock, include bedding, flow structures, joints, slickensides, etc.	Develop from deposition cycles and from stress history, formation of slickensided surfaces possible.
Anisotropy	Usually derived from relict rock fabric, e.g. bedding.	Derived from deposition and stress history of soil.
Void ratio / density	Depends on state reached in weathering process, independent of stress history.	Depends directly on stress history.

Table 2 : Comparison of Residual Soils and Transported Soils with Respect to Various Special Features that Affect Strength (from Brenner et. al., 1997)

4.0 MEASUREMENT OF SHEAR STRENGTH IN RESIDUAL SOILS

For cut slope, effective stress (drained or long term condition) is normally more critical than total stress (undrained) condition. Therefore, the commonly used effective stress strength parameters c' and ϕ' , determined from testing of representative samples of matrix materials are used in analysis. In Malaysia, the most common approach to measure shear strength of residual soils is through a large number of small scale in-situ (field) and laboratory tests. In-situ tests could include the standard penetration tests (SPT), cone penetrometer tests (CPT or CPTU), vane shear tests and pressuremeter tests. Laboratory tests commonly used are shear box tests, consolidated undrained triaxial compression tests with pore water pressure measurements (CIU), consolidated drained triaxial compression tests (CID) carried out on undisturbed soil samples (from Mazier sampler without trimming and side drains). Shear box tests with the direction of shearing

in specified orientation are sometimes carried out to explore the effects of anisotropy. In this paper only laboratory tests will be discussed.

Direct Shear Box Test	Triaxial Test
Advantages : - Relatively simple and quick to perform. - Enables relatively large strains to be applied and thus the determination of the residual strength. - Less time is required for specimen drainage (especially for clayey soils), because drainage path length is small. - Enables shearing along a predetermined direction (e.g. plane of weakness, such as relict bedding)	Advantages : - Enables the control of drainage and the measurement of pore pressures. - Stress conditions in the sample remain more or less constant and are more uniform than in direct shear test. They are controllable during the test and their magnitude is known with fair accuracy. - Volume changes during shearing can be determined.
Disadvantages / Limitations : - Drainage conditions during test, especially for less pervious soils, are difficult to control. - Pore pressures cannot be measured. - Stress conditions during the test are indeterminate and a stress path cannot be established, the stresses within the soil specimen are non-uniform. Only one point can be plotted in a diagram of shear stress τ versus normal stress σ, representing the average shear stress on the horizontal failure plane. Mohr's stress circle can only be drawn by assuming that the horizontal plane through the shear box is the theoretical failure plane. During straining the direction of principal stresses rotates. - Shear stress over failure surface is not uniform and progressive failure may develop. - Saturation of fine-grained specimens (e.g. by back-pressuring) is not possible. - The area of the shearing surfaces changes continuously.	Disadvantages / Limitations : - Influence of value of intermediate principal stress, σ_2, cannot be evaluated independently. In certain practical problems which approximate the conditions of plane strain, σ_2, may be higher than σ_3. This will influence c' and ϕ'. - Principal stress directions remain fixed, conditions where the principal stresses change continuously cannot be simulated easily. - Influence of end restraint (end caps) causes non-uniform stresses, pore pressures and strains in the test specimens and barrel shape deformation.

Table 3 : Comparison of Direct Shear Box Test and Triaxial Test (from Brenner et al., 1997)

Figures 1 and 2 show the systems of stresses applied in the direct shear box tests and the triaxial tests respectively. Both laboratory tests have their advantages and disadvantages, but certain field conditions may be simulated better by one type than by the other. The main features of these two types of tests are summarised in Table 3.

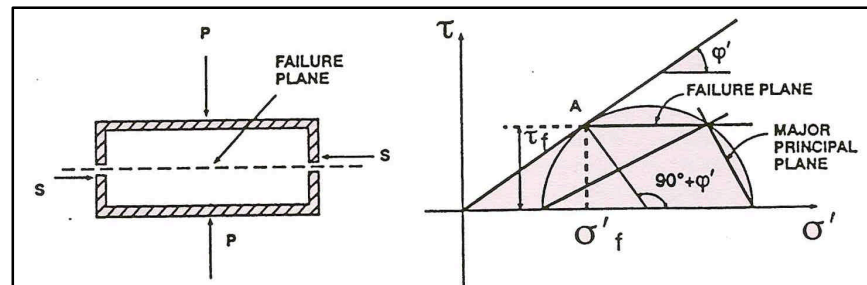


Figure 1 : Stress System Applied in Direct Shear and Failure Envelope through Point A and Mohr's Circle.

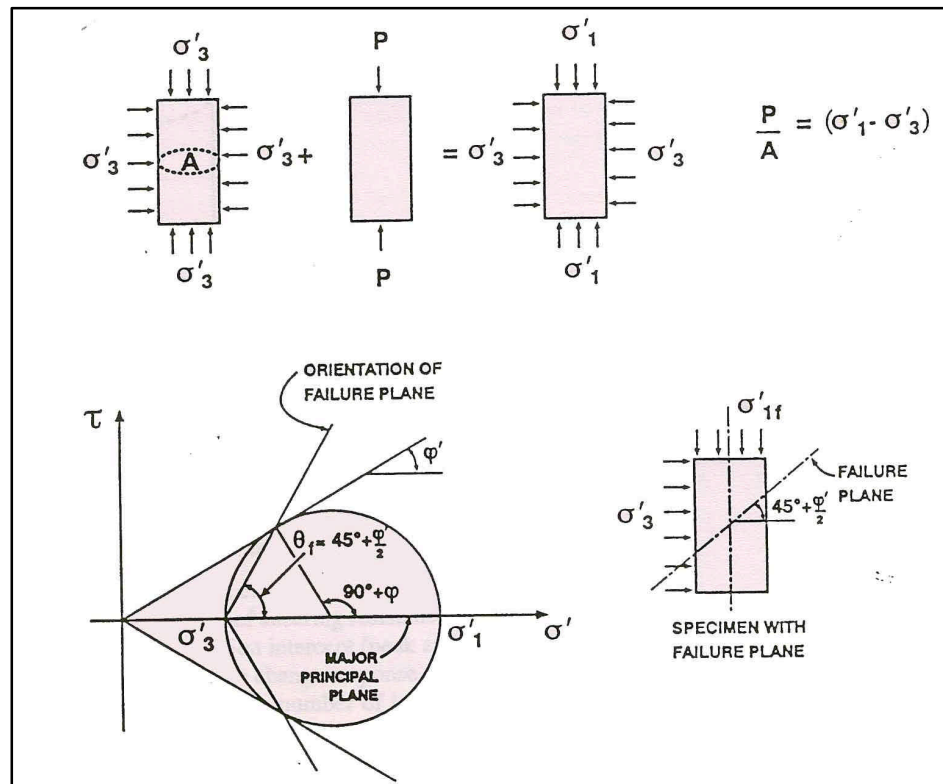


Figure 2 : Stress System Applied in Triaxial Compression Test, Mohr's Circle and Orientation of Failure Plane

For the laboratory tests, the soil samples should be 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 the residual soils are usually of high permeability (usually 10^{-4} to 10^{-6} m/sec), rainwater can infiltrates with ease into it and likely that saturation conditions will be approached at shallow depths in the field during the life of a slope. To date it is not advisable to include soil suction (negative pore pressure) in the design of the long term slopes in view of many factors that can cause the loss of the suction.

It is also important to acknowledge that stiff materials like residual soil usually contains discontinuities which the small scale strength tests may miss the discontinuities in the sampling process and over-estimate soil shear strength. On the other hand, if there are corestones and other large sized particles present in the residual soil mass, the effect of this materials cannot be quantitatively determined and the small scale laboratory strength tests carried out on the 'matrix' material of residual soils will usually underestimate the overall shear strength of the in-situ material mass. Therefore, special care shall be taken in the selection of representative soil strength for stability analysis.

4.1 Direct Shear Box Test

The soil parameters that can be obtained from a direct shear box tests are :

- The angle of friction (peak and residual)
- The cohesion intercept (peak and residual) (NOTE : to use with care)
- The volume change response of the soil due to shearing which can be either dilatant or contractive.

In the direct shear box test, the following variables have to be determined first before starting of a test :

- Minimum size of shear box
- Thickness of soil specimen
- Drainage condition
- Consolidation and saturation status
- Shearing rate and displacement allocated
- Stress level (normal stress)

Shear boxes can either be square (60mm, 100mm, 300mm and more (rarely)) or circular (50mm and 75mm diameter) with the thickness of sample not more than half the size. The direct shear box tests can be carried out by via Stress-Controlled (increasing the shear stress in increments and measuring the displacement) or Strain-Controlled (shear by a certain displacement rate and measuring the resulting stress. Usually Strain-Controlled test is used because it is easier to perform and allows both ultimate and residual shear strength of the soil to be determined.

The following direct shear tests categories are possible : unconsolidated undrained (UU), consolidated undrained (CU) and consolidated drain (CD). For the UU and CU tests, the shearing rate has to be as rapid as possible to maintain the "undrained" condition and total stress strength parameters can be obtained.

The shearing rate has to be extremely slow especially for soil with low permeability in the CD type of direct shear tests. Usually tests for which drainage is allowed should be performed with the soil specimen fully immersed in water to eliminate the effects of capillary moisture stresses. Gibson & Henkel (1954) and Head (1982) recommend a time to failure, t_f , for drained direct shear tests to be $12.7t_{100}$. Where t_{100} is the time to 100%

primary consolidation and can be obtained by extrapolating the linear portion of the square root of time plot of the consolidation phase of the test. The maximum permissible rate shearing in a drained direct shear test can be estimated to be less than δ_f / t_f . Where δ_f is the horizontal displacement of the shear box at peak strength and this value is not known prior and has to be estimated.

In deciding on the normal pressures to be applied, usually the soil samples should be tested at stresses comparable to those in the field. With coarse grained soils (cohesionless soils), the test results usually passes through the origin but for soils with bonded structure, there will usually be a apparent cohesion intercept. For details of carrying out direct shear box test, reference can be made to Head (1982).

4.2 Triaxial Test

The soil parameters that can be obtained from a triaxial tests are :

- The angle of friction (peak and residual)
- The cohesion intercept (peak and residual) (NOTE : to use with care)
- The pore water pressure response due to shearing (in undrained tests)
- Initial tangent and secant moduli (unloading and reloading)
- Consolidation characteristics and permeability

In normal practice, the following triaxial tests should be routinely carried out where practical :

- (a) Isotropically Consolidated Undrained Compression Test (CIU) with pore pressure measurement. In this test, drainage is permitted during the isotropic consolidation under consolidation stress, σ_3 . After the soil sample is fully consolidated, the sample is sheared through application of the deviator stress ($\sigma_1 - \sigma_3$) without permitting drainage (undrained). CIU tests are the most commonly used laboratory tests in Malaysia to obtain effective stress strength parameters, $c' - \phi'$ for analysis of cut slope.

Note : In the triaxial test, $\sigma_2 = \sigma_3$.

- (b) Isotropically Consolidated Drained Compression Test (CID) with pore pressure measurement. Similar to CIU, drainage is permitted during the isotropic consolidation under consolidation stress, σ_3 . Full drainage during shearing so that no excess pore water pressure is generated. CID tests although more superior are the not often used due to its long duration required during shearing to obtain the effective stress strength parameters, $c' - \phi'$ for analysis of cut slope.

For triaxial testing of residual soils, the specimen diameter should not be less than 70mm. Therefore, the use of mazier samples without trimming is suitable. Specimens with smaller diameters are not considered representative because of the scale effect relating to fissures and joints in the soil. The ratio of specimen length to diameter must be at least 2 to 1.

In the triaxial tests, multi-stage tests should not be used as these tests will usually produce misleading high apparent cohesion, c' . The multi-stage test will also give misleading results as the second test will be significantly affected by the failure surface formed in the first test (GCO, 1991). Sometimes high c' obtained from testing is often due to the rate of strain or time of shearing to failure is too short. The rate of strain should be estimated from the results during consolidation. Side drains should not be used as this has shown to produce inconsistency in the sample (Tschebotarioff, 1950 and GCO, 1991). Further details on the laboratory triaxial tests can be obtained from Head (1982).

5.0 INTERPRETATION OF EFFECTIVE SHEAR STRENGTH FROM LABOLATORY TESTS

The shear strength of the soil in normal practice is usually represented graphically on a Mohr diagram. The 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.

For simplicity of analysis, it is conventional to use a linear Mohr-Coulomb failure envelope (c' - ϕ' soil strength model) for the concerned stress range as expressed in the equation below :

$$\tau_f = c' + \sigma_{nf}' \tan \phi'$$

where τ_f = shear strength of soil.
 σ_{nf}' = effective normal stress at failure.
 ϕ' = effective angle of friction (degree).
 c' = apparent cohesion (kPa)

Figure 3 shows the typical bonding and dilatant characteristic of the residual soil at low stress range (low confining and consolidation pressure) which exhibits a peak shear strength envelope in terms of effective stress which has a apparent cohesion intercept (c') if the Mohr-Coulomb c' - ϕ' failure line is used. As the consolidation pressure in the laboratory test prior to shearing increases, the bonds are destroyed and the residual soil will likely to behave like normally consolidated or slightly overconsolidated transported soil. The critical state friction angle is represented as ϕ_{cr} .

Brand (1995) also states that most of the critical slip surfaces in residual soils 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 sols are curved at low effective stresses, and that the straightline projection of strengths measured at high stresses underestimates that shear strengths in the low stress range. Therefore, for different stress range, different shear strength envelopes (c' and ϕ' values) can be adopted using either of the two different method shown in Figures 4 and 5 respectively.

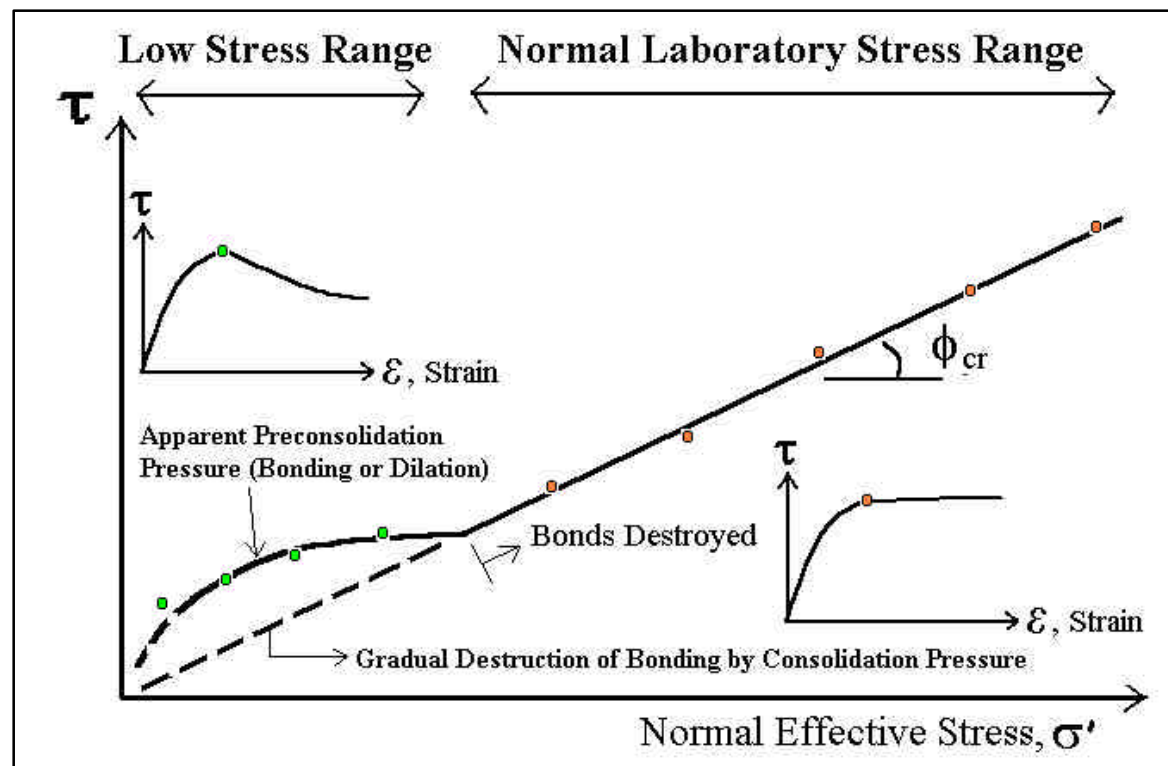


Figure 3 : Effect of Bonding on the Apparent Cohesion Intercept of a Drained Strength (Effective Stress) Failure Envelope.

Figure 4 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 low stress range and continue shearing until the critical state strength (Point C). Normally the peak strength is obtained at a relatively small strain and after continue shearing, the critical state strength (ϕ_{cr}) is obtained at a larger strain. The critical state usually occurs in 10% to 30% strain range where the soil sample continue 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 different from residual strength (Skempton, 1964) which is lower and it occurs after very large movement on the slip/failure surface. The residual strength is also associated with highly polished slip surfaces in which the soil particles have become aligned in directions 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 4, the critical state strength fall on a straight line through the origin. The conventional interpretation of peak failure strength is the Mohr-Coulomb envelope ($c'-\phi'$) at the stress range concern 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 know that c' does not imply that at zero effective confining stress, the strength is c' (kPa). Therefore, at low effective confining stress (outside representative stress range), Mohr-Coulomb failure envelope ($c'-\phi'$) might overestimate the strength of the soil. On the other hand, if critical state strength is used,

the strength value will be underestimated. Therefore, if the in-situ stress range and the stress path followed (see details in the Section 6) during shearing is correctly determined, the $c'-\phi'$ shear strength envelope will be representative of the field condition.

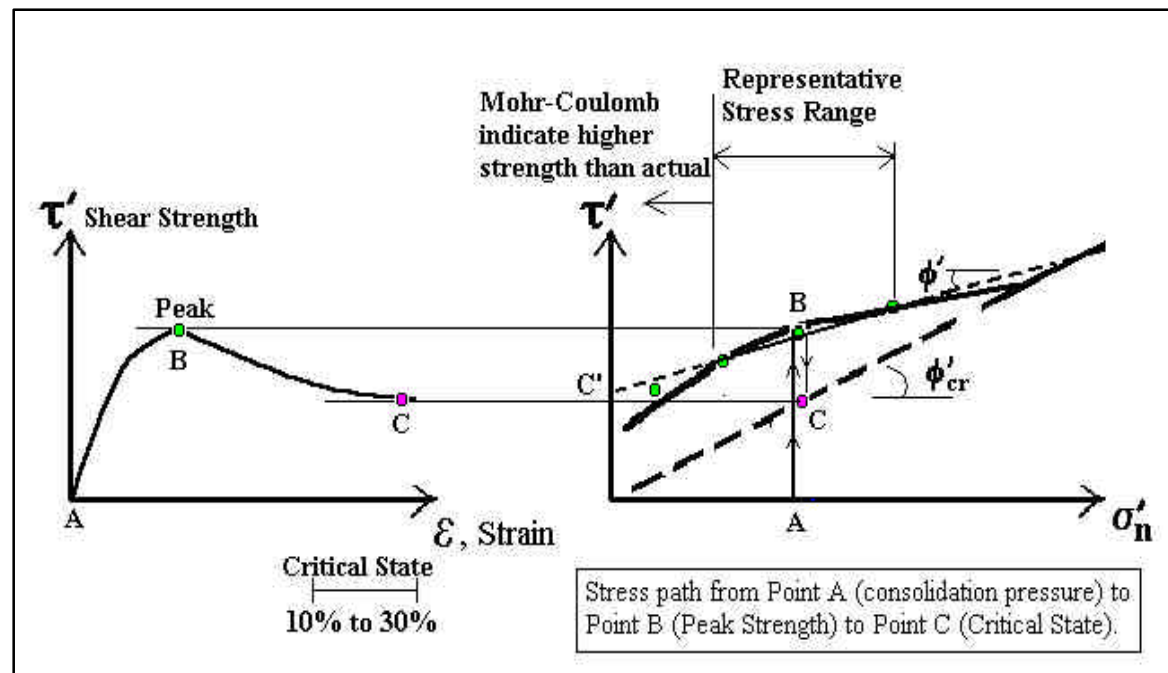


Figure 4 : Typical Shearing Characteristic of Residual Soil during Drained Shear Tests and the Tangent Method in Selection of Shear Strength Envelope.

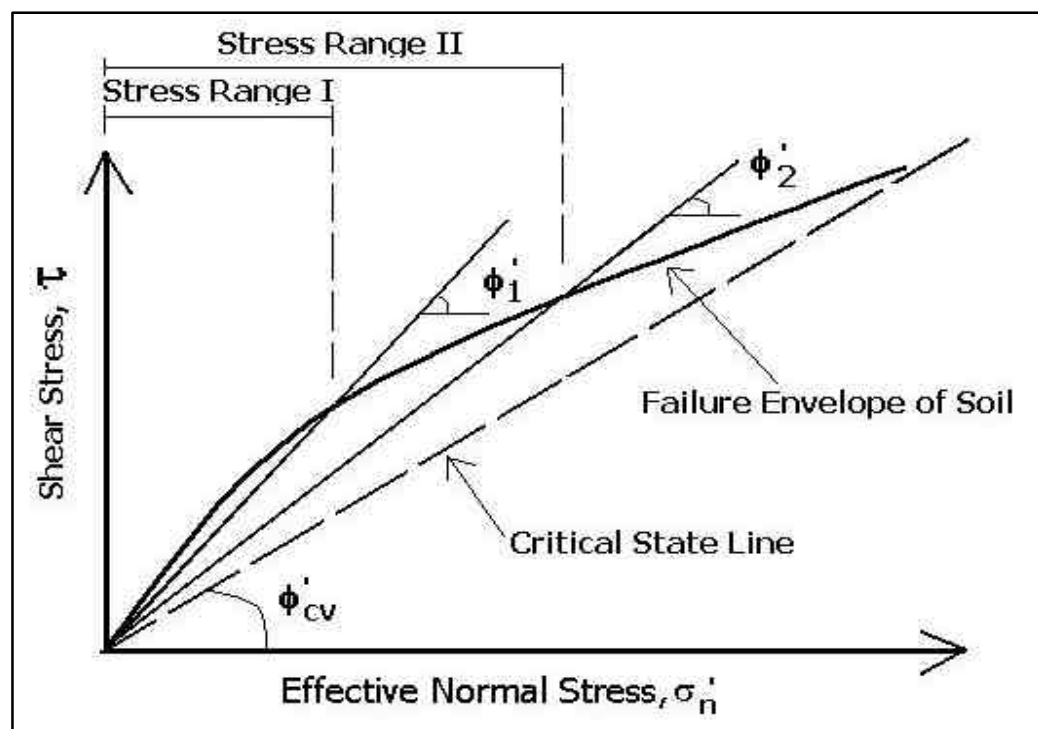


Figure 5 : Secant Method in Selection of Shear Strength Envelope.

Another method of determining the shear strength envelope is through the secant method for the stress range concerned as shown in Figure 5. In this method, generally, the c' is taken at 0 (zero) unless there are sufficient test results to obtain the representative c' . Usually the c' should not exceed 10kPa. This method will yield a more conservative (lower) peak strength value compared to tangent method at the low stress range and both will yield same results at high stress range. Therefore, if the stress range at site during design cannot be confirmed, then secant method shall be used instead of tangent method.

6.0 INFLUENCE OF STRESS PATH ON THE SHEAR STRENGTH

6.1 General

In analysis of geotechnical problems including slope stability, Stress Path Method is the most rational and useful method because it uses the laboratory and field data to obtain the stress path of average stresses in the a field situation for a past, present and future conditions (Lambe & Silva, 1998). In general, the effective stress path for the field situation must be determined first then performs tests (usually laboratory tests) along the field effective stress path with the specimens at the field conditions; water content, degree of saturation, stress, pore pressure, geometry, etc.

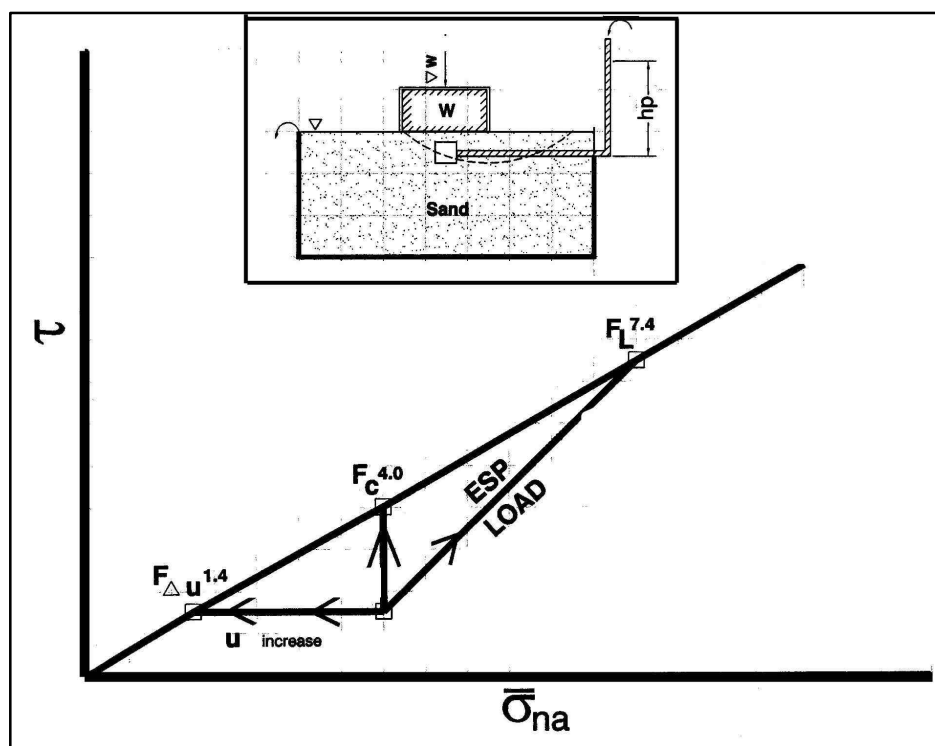


Figure 6 : Stress Path to Failure (from Lambe & Silva, 1998)

Figure 6 demonstrates the stress path to failure due to three different conditions (Lambe & Silva, 1998). Increasing the weight (W) will give the loading effective stress path with a strength of F_L . On the other hand, building up the pore water pressure in the

subsoil will result in failure along the horizontal stress path to failure $F_{\Delta U}$. In assessing the stability of the weight (W), a limit equilibrium computer program (e.g. slope stability program) will use a stress path corresponding to a constant σ'_n to failure along the vertical effective stress path to failure at F_C . This simple demonstration indicates that different shear strengths are obtained for different stress paths followed ($F_L > F_C > F_{\Delta U}$) and thus yield different factor of safety (FOS). It is very important to note that the vertical stress path (usually used in limit equilibrium computer, F_C) does not properly model the other two mechanism that could occur in the field.

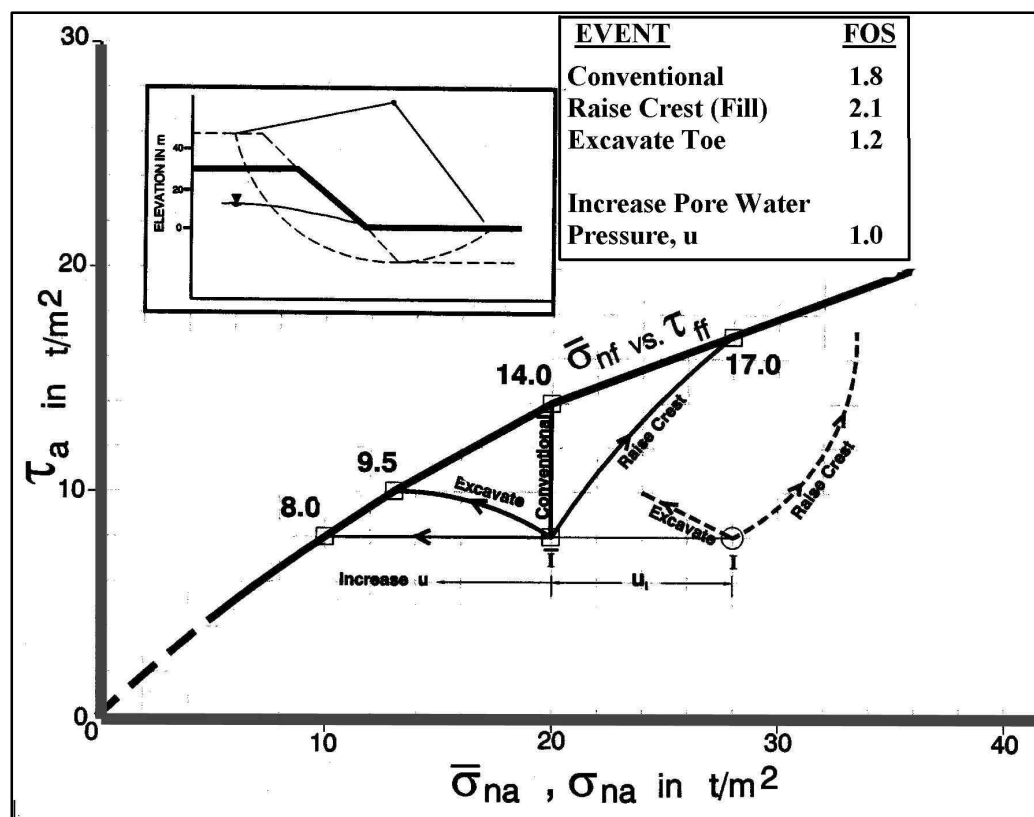


Figure 7 : Example on Effect of Stress Path on Factor of Safety from Stability Analysis (from Lambe & Silva, 1998)

Figure 7 shows another example of three different stress paths to failure (for one element of soil):

- Raising the crest through filling on top of the slope will yield FOS of 2.1.
- Excavate the toe of the slope will yield FOS of 1.2.
- Conventional vertical stress path (limit equilibrium analysis assumption) will yield FOS of 1.8.
- Increase of pore water pressure will yield the lowest FOS of 1.0 (Most Critical).

From the two examples shown (Figures 6 & 7) have illustrated three important points :

- The factor of safety (FOS) for slopes depends on the stress path to failure.
- The most dangerous situation occurs when pore water pressure builds up (e.g. rising of groundwater). Therefore it is very important and critical in slope

analysis to have a representative groundwater and pore water pressure in the subsoil.

- (c) Conventional procedure of assuming a vertical stress path to failure does not represent the actual stress path to failure of slope.

In brief, it is important to recognise the significance of the stress path effect in the stability analysis to yield the correct factor of safety.

6.2 Groundwater and Pore Water Pressure

The hydrological effects of rainfall on a permeable slopes are shown in Figure 8. Some of the rain water runs off the slope and may cause surface erosion if there is inadequate surface protection. In view of the high soil permeability, majority of the water will infiltrates into the subsoil. This causes the water level in the slope to rise or it may cause perched water table to be formed at some less permeable boundary, usually dictated by the weathering profile. Above the water table, the degree of saturation of the soil increases thus reduces the soil suction (i.e. negative pore pressure).

Failures in residual soils cut slopes might be caused by 'wetting-up' process by which the decrease in soil suction and hence the decrease in soil strength due to the suction. There is also evidence suggesting that transient rises in groundwater table are responsible for some rain-induced landslides (Premchitt et al, 1985). Lambe & Silva (1998) have also reported that over the 60 slope failures they have investigated, three-quarters of these failures are resulted from an increase in pore water pressure.

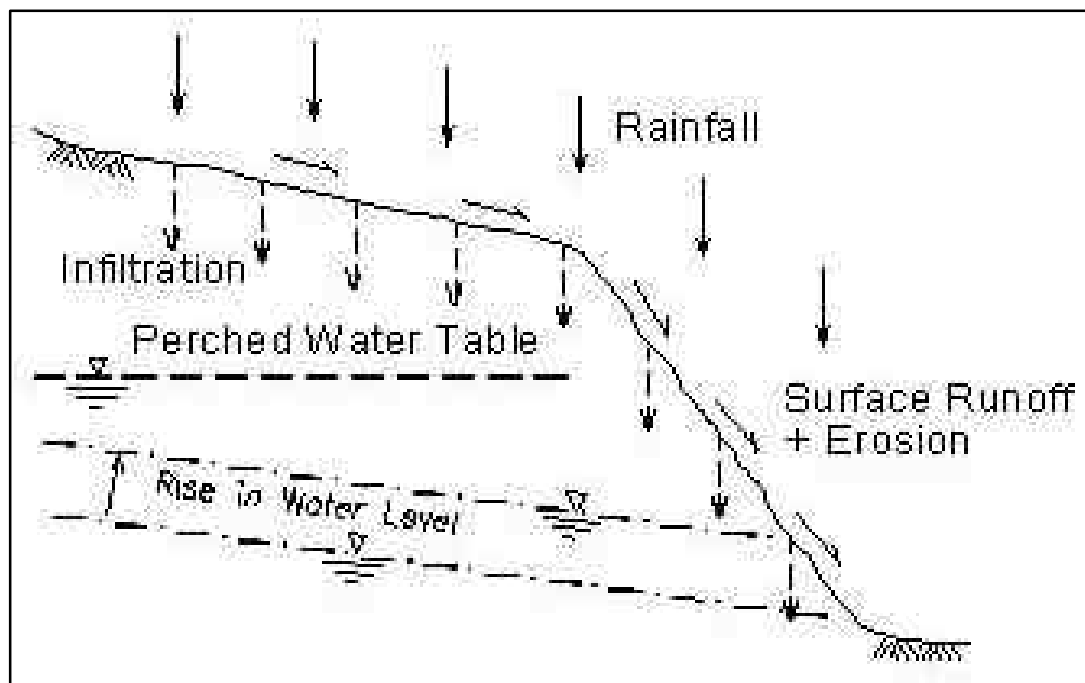


Figure 8 : Effects of rainfall on high permeable slope (from Brand, 1995)

Slopes should be designed for the groundwater conditions resulting from a ten-year return period rainfall or representative groundwater level through observation and estimation. However, to predict the pore water pressure or ground water level in the cut slope is one of the most difficult task because there are many unknown variables that require long term monitoring of completed cut slopes (especially high cut slopes and with catchment behind them) and are usually not available during design stage. Therefore, slopes in the high risk-to-life and high economic risk category should be checked to determine the sensitivity of the water levels to the stability of the slopes and this required prediction of the worst groundwater conditions.

Transient perched water tables might be formed at the interface of layers of differing permeability. Therefore examination of the material profiles within a slope and the catchment above the slope must be carried out. Sometimes leakage from services, such as sewers, drains or water mains can cause rising of groundwater level. Services on hill-site should be properly protected from leakage to prevent contributing to the failure of the slopes. In some cases, subsurface drainage (e.g. horizontal drains, vertical wells, etc.) can be used to reduce the groundwater levels thus increase the Factor of Safety against failure on any potential slip surface which passes below the water table. If subsurface drainage system is employed, regular maintenance is required to prevent reduction of efficiency caused by siltation, deterioration of seals or growth of vegetation blocking the outlet.

7.0 CONCLUSIONS

Residual soils can have characteristics that are quite different from those of transported soils, as follows:

- Very heterogeneous, which makes sampling and testing for representative parameters difficult.
- Usually high permeability. Therefore susceptible to rapid changes in material properties when subjected to changes of external hydraulic condition.

The weathering profile of residual soil is an important information for slope stability analysis because it usually controls :

- The potential failure surface and mode of failure.
- The groundwater hydrology, and therefore the critical pore pressure distribution in the slope.
- The erosion characteristics of the materials.

For cut slope, effective stress (drained or long term condition) is normally more critical than total stress (undrained) condition. Therefore, the commonly used effective stress strength parameters c' and ϕ' , determined from testing of representative samples of matrix materials are used in analysis. Direct shear box and triaxial tests are commonly carried in laboratory to obtain the representative c' - ϕ' for slope analysis. For the laboratory tests, the soil samples should be tested at stresses comparable to those in the field, and should be saturated. Generally, the two commonly used methods to select c' - ϕ' soil strength model are the tangent and secant method as illustrated in details in Section

5.0. If the stress range at site during design cannot be confirmed, secant method shall be used instead of tangent method. To date it is not advisable to include soil suction (negative pore pressure) in the design of the long term slopes in view of many factors that can cause the loss of the suction in our weather conditions with heavy prolong rain during monsoon season.

Finally, when designing slopes, the engineer must aware that the most dangerous situation occurs when pore water pressure builds up (e.g. rising of groundwater). Therefore it is very important and critical in slope analysis to predict a representative groundwater and pore water pressure in the subsoil. Sometimes, active measures such as provision of internal drainage can be used to control groundwater in the slopes.

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REFERENCES

- Atkinson, J.H. & Bransby, P.L. (1978). The Mechanics of Soils. McGraw Hill, London.
- Blight, G.E. (1997) "Origin and Formation of Residual Soils". Mechanics of Residual Soils, ed. G.E. Blight, Chapter 1. A.A. Balkema, Rotterdam.
- Brand, E.W. (1995) "Slope Stability and Landslides under Tropical Conditions – A Review". Symposium on Hillside Development, Kuala Lumpur, Malaysia.
- Brenner, R.P., Garga, V.K. & Blight, G.E. (1997) "Shear Strength Behaviour and the Measurement of Shear Strength in Residual Soils". Mechanics of Residual Soils, ed. G.E. Blight, Chapter 9. A.A. Balkema, Rotterdam.
- Crabb, G.I. & Atkinson, J.H. (1991) "Determination of Soil Strength Parameters for the Analysis of Highway Slope Failures." Slope Stability Engineering. Thomas Telford, London.
- GCO (1988). Guide to Rock and Soil Descriptions : Geoguide 3. Geotechnical Control Office, Hong Kong, 295p.
- GCO (1991). Geotechnical Manual for Slopes (2nd ed.). Geotechnical Control Office, 301p.
- Gibson, R.E. & Henkel, D.J. (1954) "Influence of Duration of Tests at Constant Rate of Strain on Measured 'Drained' Strength." Geotechnique, Vol.4(1), pp.6-15.
- Gue, See Sew & Tan, Yean Chin (2000). "Subsurface Investigation and Interpretation of Test Results for Geotechnical Design". Seminar on Geotechnical Engineering, The Institution of Engineers, Malaysia, Penang, Malaysia.
- Gue, See Sew & Tan, Yean Chin (2000). "Hill-Site Development – Planning, Design, Construction & Maintenance Considerations". Seminar on Geotechnical Engineering, The Institution of Engineers, Malaysia, Penang, Malaysia.
- Gue, See Sew (1999) (Interview) "Face to Face with the Man Who Knows Landslides". The Edge Capital, November, 1999.
- Gue, See Sew (1995). "Geotechnical Assessment for Hillside Development". Proc. of Dialogue Session on Geological and Geotechnical Considerations in Civil Works, Geological Survey Department Malaysia, Kuala Lumpur, pp. 27-34.
- Head, K.H. (1982) Manual of Soil Laboratory Testing, Volume 2 : Permeability, Shear Strength and Compressibility Tests, ELE International Ltd. Pentech Press, London.

- Head, K.H. (1982) Manual of Laboratory Testing. Volume 3 : Effective Stress Tests, ELE International Ltd. Pentech Press, London.
- Lambe, T.W. & Silva, Francisco (1998) Special Paper : "The Determination of Soil Strength for a Stability Analysis". Proc. 13th Southeast Asian Geot. Conf., Taipei, Vol 1.
- Premchitt, J., Brand, E.W. & Phillipson, H. B. (1985) "Landslides Caused by Rapid Groundwater Changes." Groundwater in Engineering Geology, ed. J.C. Cripps et al, 87-97. Geol. Soc., London.
- Skempton, A.W. (1970) "First Time Slides in Overconsolidated Clay". Geotechnique, Vol.20, pp.320-324.
- Skempton, A.W. (1964) "Long Term Stability of Clay Slopes". Geotechnique, Vol.14, pp.77-101.
- Tschebotarioff, G. P. (1950) Discussion on "Effect of Driving Piles into Soft Clays" by Cummings, A.E., Kerkhoff, G.O. and Peck, R.B.. Transactions, ASCE, Vol. 115, pp 296-301.
- Vaughan, P.R. (1988) Keynote paper : "Characterising the Mechanical Properties of In Situ Residual Soil." Proc. 2nd Int. Conf. Geomechanics in Tropical Soils, Singapore, Vol. 2, pp.469-487.