Tropical residual soils are derived from in situ weathering and decomposition of rock and have characteristics that are quite different from those of transported soils. Some methods commonly adopted in Malaysia for slope stabilization work are discussed, highlighting good practices and relevant design guidelines. Some aspects on measurement of strength parameters and groundwater profile assessment together with a discussion on stability analysis are also presented.

10.1 CHARACTERISTICS OF TROPICAL RESIDUAL SOIL SLOPE

Residual soils are derived from in situ weathering and decomposition of rock and remain at their original location. The stratigraphy may be a continuous gradation from the fresh, sound unweathered parent rock through weathered soft rock and hard soil to highly weathered material containing secondary deposits of iron, alumina, silica or calcium salts with its original rock texture completely destroyed (Blight 1997a). Based on the above definition, it is clear that residual soils would have characteristics that are quite different from those of transported soils. The same also applies with respect to the behavior of a residual soil slope as compared to a transported soil slope. Figure 10.1 illustrates the typical weathering profile for a residual soils showing gradual changes of gradation from sound unweathered rock (Grade I) to highly weathered (decomposed) material (Grade VI).

The weathering process leading to the formation of residual soils is highly complex and weathering sequence such as those proposed by van der Merwe (1965) cannot be used as a general guide to relate the properties of residual soils to its parent rock. The weathering process is influenced by climate and drainage conditions and any changes to the two factors may arrest or reverse certain stages of weathering. It has also been demonstrated extensively in the literature that behavior of residual soils in different regions of the world (or even within the same region) shows distinct differences in characteristics. However, Blight (1997a), based on the works of Vargas & Pichler (1957), Ruxton & Berry (1957) and Little (1969) summarised that a residual soils will typically consist of three zones (Figure 10.1):

a) Upper zone
This zone consists of highly weathered (or decomposed) and leached soils often reworked by burrowing animals and insects or cultivation, and intersected by root channels. This zone is also possibly subjected to some transport processes.

b) Intermediate zone
This zone consists of highly weathered to moderately weathered material (50% to 90% rock) but exhibits some features of the structure of the parent rock and may contain some core stones (or boulders).

c) Lower zone
This zone consists of fresh rock and slightly weathered material.

Sometimes, behavior of a residual soil slope is governed more by its structural features such as relict jointing, bedding or slickensiding which it has inherited from the parent rock. The influence of such structural features cannot be ignored in the analysis of slope stability and stabilization involving residual soils. For example, a slope with pre-existing shear plane which dips in the direction of the slope (‘daylighting’) as shown in Figures 10.2 and 10.3 will definitely have a lower factor of safety compared to a slope without such structural feature or one in which the direction of dip is in the opposite direction. In addition, Blight (1997a) also highlighted the difficulties of determining the relevant engineering properties of residual soils. This is due to the weathering characteristics of residual soils which tend to form soil which consists of aggregates or crystals bonded together by a combination of capillary forces and bonding. These aggregates of soil particles can be easily broken down and become progressively finer if the soil is manipulated.
In brief, the main characteristics of residual soils are:

a) Very heterogeneous; this makes sampling and testing for relevant engineering parameters difficult.

b) Usually high permeability, therefore susceptible to rapid changes in material properties when subjected to changes of external hydraulic condition.

In general, the formation process of residual soils is complex and is very difficult to model and generalize. Therefore, a simplified weathering profile which differentiates the material into different ‘grades’ is 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 important for slope stability analysis in residual soils because it usually controls:

a) The potential failure surface and mode of failure.
b) The groundwater hydrology, and therefore the critical pore pressure distribution in the slope.
c) The erosion characteristics of the soil materials.

The Geotechnical Engineering Office, GEO (formerly Geotechnical Control Office, GCO) of Hong Kong, has adopted a system for granites in which a profile is logged according to six rock material ‘grades’ given by GCO (1988). Table 10.1 presents the modified grades classification based on the above reference for ease of classification. For geotechnical design of slopes, materials of Grade I to III are usually treated as ‘rock’ and materials of Grades IV to VI (upper zone) as ‘soil’.

It is also important to be aware of the contribution of capillary forces, bonding and structural features on the shear strength of residual soils. Therefore, proper determination of relevant engineering
properties for residual soil slope is essential in order to produce design that is safe but not over-conservative.

10.2 MEASUREMENT OF STRENGTH PARAMETERS AND GROUNDWATER PROFILE FOR ASSESSMENT OF SLOPE STABILITY

Stress-Strain and Shear Strength Characteristics of Residual Soils

The special features encountered in residual soils that are mainly responsible for the difference in stress-strain and strength behavior in comparison with transported soils are listed in Table 10.2.

One of the significant characteristics of residual soils is the existence of bonds between particles. These bonds are a component of strength (can be reflected as apparent cohesion, \( c' \)) and initial stiffness that is independent of effective stress and void ratio/density. The bonding also contributes to 'apparent' overconsolidated behavior of the soils.

Vaughan (1988) highlighted some of the possible causes of the development of bonds as:

a) Cementation through the deposition of carbonates, hydroxides, organic matter, etc.

b) Solution and re-precipitation of cementing agents, such as silicates.

c) Cold welding at particle contacts subjected to high pressure.

d) Growth of bonds during chemical alteration of minerals.

In engineering applications, these bonds are purposely omitted (on the conservative side) because it is easily destroyed and not reliable for design. In addition, these bonds also contribute to the peak strength of the soil. These bonds affect residual soils in a way which is similar to dense sand. In dense sand which tends to dilate during shearing, Bolton (1986) shows that in plane strain, the contribution of dilation to peak strength is closely represented by the expression:

\[
\phi'_{\text{peak}} = \phi'_{\text{cr}} + 0.8 \psi_{\text{max}}
\]

where \( \phi'_{\text{peak}} \) = peak strength; \( \phi'_{\text{cr}} \) = critical state strength; and \( \psi_{\text{max}} \) = maximum angle of dilation.

Table 10.1. Material grade classification system modified from GCO (1988).

<table>
<thead>
<tr>
<th>Descriptive term</th>
<th>Grade</th>
<th>General characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Residual soils</td>
<td>VI</td>
<td>Original rock texture completely destroyed</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Can be crumbled by hand and finger pressure</td>
</tr>
<tr>
<td>Completely</td>
<td>V</td>
<td>Rock wholly decomposed but rock texture preserved</td>
</tr>
<tr>
<td>decomposed</td>
<td></td>
<td>No rebound from N Schmidt Hammer</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Can be crumbled by hand and finger easily indented by point of geological pick</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Slakes when immersed in water</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Completely discoloured compared with fresh rock</td>
</tr>
<tr>
<td>Highly</td>
<td>IV</td>
<td>Rock weakened and can be broken by hand into pieces</td>
</tr>
<tr>
<td>decomposed</td>
<td></td>
<td>Positive N Schmidt rebound value up to 25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Makes dull sound when struck by hammer</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Geological pick cannot be pushed into surface</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Does not slake readily in water</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Hand penetrometer strength index greater than 250kPa</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Individual grains may be plucked from surface</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Completely discoloured compared with fresh rock</td>
</tr>
</tbody>
</table>

Table 10.2. Comparison of residual soils and transported soils with respect to various special features that affect strength (from Brenner et al. 1997).

<table>
<thead>
<tr>
<th>Factors affecting strength</th>
<th>Effect on residual soils</th>
<th>Effect on transported soils</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stress History</td>
<td>Usually not important</td>
<td>Very important, modifies initial grain packing, causes overconsolidation effect.</td>
</tr>
<tr>
<td>Grain / Particle Strength</td>
<td>Very variable, varying mineralogy and many weak grains are possible.</td>
<td>More uniform; few weak grains because weak particles become eliminated during transport.</td>
</tr>
<tr>
<td>Bonding</td>
<td>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.</td>
<td>Occurs with geologically aged deposits, produces cohesion intercept and yield stress, can be destroyed by disturbance.</td>
</tr>
</tbody>
</table>
Such findings have led to recommendations of using critical state strength instead of peak strength in the design of cut slopes as summarized by Skempton (1977):

a) The shear strength parameters of clay relevant to first-time slides is the ‘fully-softened’ value (critical state) and also by the lower limit of strength measured on structural discontinuities (joints and fissures).

b) The peak strength is considerably higher; so some progressive failure mechanism appears to be involved.

c) The residual strength is much smaller than the ‘fully-softened’ value (critical state) and corresponds to the strength mobilized after a slip has occurred, with large displacements of the order of 1 or 2 m.

d) It is a characteristic of first-time slide in clay slopes to occur many years after a cutting has been excavated. The principal reason for this delay is the very slow rate of pore-pressure equilibration; a process which in typical cuttings is not completed, for practical purposes until 40 or 50 years after excavation.

In addition, Powrie (1997) also highlighted the potential danger of using peak strength:

a) It can lead to the overestimation of the actual peak strength at either low or high effective stresses, depending on where the ‘best fit’ straight line is drawn.

b) The designer cannot guarantee that the peak strength will be uniformly mobilized everywhere it is needed at the same time. It is much more likely that only some soil elements will reach their peak strength first. If any extra strain is imposed on these elements, they will fail in a brittle manner as their strength falls towards the critical value. In doing so, they will shed load to their neighbors, which will then also become overstressed and fail in a brittle manner. In this way, a progressive collapse can occur, which – like the propagation of crack through glass – is sudden and catastrophic. Experience also suggests that progressive failure is particularly important with slopes.

c) Many of the design procedures used in geotechnical engineering assume that the soil can be relied on to behave in a ductile manner. When a ductile material fails, it will undergo continued deformation at constant load. This is in contrast to a brittle material, which at failure breaks and loses its load carrying capacity entirely. At the critical state, the behavior of the soil is ductile: the definition of the critical state is that unlimited shear strain can be applied without further changes in stress or specific volume. Between the peak strength and the critical state, however, the behavior of the soil is essentially brittle.

The main purpose of discussing the effects of bonds, peak strength and critical state strength on the stability of residual soil slopes is to highlight recent advances in the understanding of slope stability. It must be understood however, that most of the current procedures and standards commonly used by practising engineers on the subject of slope (e.g. GEO 2000) is based on statistical analysis of data using peak strength. These procedures and standards have generally produced acceptable designs if used correctly and if critical state strength is used based on these procedures and standards, the designs will be too conservative and uneconomical (Gue 2003, Powrie 1997). Therefore, the following sections shall discuss the measurement of shear strength in residual soils for stability analysis using existing procedures and standards on the basis of peak strength. The designer must be aware though on the pitfalls and limitations of these approaches as discussed above in order to exercise better engineering judgment.

**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, effective stress strength parameters, $c'$ and $\phi'$, determined from testing of representative samples of matrix materials are used in analysis. 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 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) and consolidated drained triaxial compression tests (CID) carried out on undisturbed soils (from Mawer sampler without trimming and without side drains). Shear box tests with the direction of shearing in specified orientation are sometimes carried out to explore the effects of anisotropy and shear strength in structural discontinuities. In this chapter, only laboratory tests will be discussed.

Figures 10.4 and 10.5 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 well by one type than by the other. The main features of these two types of tests are summarized in Table 10.3.
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 residual soils are usually of high permeability (usually $10^{-3}$ to $10^{-6}$ m/sec), rainwater can infiltrates with ease into it and it is 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 design of long-term slopes in view of many factors that can cause the loss of suction. It is also important to realize that stiff materials like residual soils usually contain discontinuities which the small scale strength tests may miss in the sampling process and overestimate the 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 material cannot be quantitatively determined and the small scale laboratory 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 is to be taken in the selection of representative soil strength for stability analysis.

Table 10.3. Comparison of direct shear box test and triaxial test (from Brenner et al. 1997).

<table>
<thead>
<tr>
<th>Direct Shear Box Test</th>
<th>Triaxial Test</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Advantages:</strong></td>
<td></td>
</tr>
<tr>
<td>1. Relatively simple and quick to perform.</td>
<td>1. Enables the control of drainage and the measurement of pore pressures.</td>
</tr>
<tr>
<td>2. Enables relatively large strains to be applied and thus the determination of the residual strength.</td>
<td>2. 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.</td>
</tr>
<tr>
<td>3. Less time is required for specimen drainage (especially for clayey soils), because drainage path length is small.</td>
<td>3. Volume changes during shearing can be determined.</td>
</tr>
<tr>
<td>4. Enables shearing along a predetermined direction (e.g. plane of weakness, such as relict bedding)</td>
<td><strong>Disadvantages/Limitations:</strong></td>
</tr>
<tr>
<td><strong>Disadvantages/Limitations:</strong></td>
<td></td>
</tr>
<tr>
<td>1. Drainage conditions during test, especially for less pervious soils, are difficult to control.</td>
<td>1. Influence of value of intermediate principal stress, $\sigma_2$, cannot be evaluated independently. In certain practical problems which approximate the conditions of plane strain, $\sigma_2$, may be higher than $\sigma_3$. This will influence $c'$ and $\phi'$.</td>
</tr>
<tr>
<td>2. Pore pressures cannot be measured.</td>
<td>2. Principal stress directions remain fixed, conditions where the principal stresses change continuously cannot be simulated easily.</td>
</tr>
<tr>
<td>3. 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, $\tau$ versus normal stress, $\sigma$, 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.</td>
<td>3. Influence of end restraint (end caps) causes non-uniform stresses, pore pressures and strains in the test specimens and barrel shape deformation.</td>
</tr>
<tr>
<td>4. Shear stress over failure surface is not uniform and progressive failure may develop.</td>
<td>5. Saturation of fine-grained specimens (e.g. by backpressuring) is not possible. The area of the shearing surfaces changes continuously.</td>
</tr>
<tr>
<td>5. Saturation of fine-grained specimens (e.g. by backpressuring) is not possible. The area of the shearing surfaces changes continuously.</td>
<td><strong>Direct shear box test</strong></td>
</tr>
</tbody>
</table>

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

a) The angle of friction (peak, critical state and residual).

b) The cohesion intercept (peak, critical state and residual) *Note: to use with care.*
c) 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 commencement of a test:

a) Minimum size of shear box
b) Thickness of soil specimen
c) Drainage condition
d) Consolidation and saturation status
e) Shearing rate and displacement allocated
f) Stress level (normal stress).

Shear boxes can either be square (60 mm, 100 mm, 300 mm or more) or circular (50 mm and 75 mm) with the thickness of the sample not more than half of its size. The direct shear box tests can be carried out 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 peak, critical state 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 drained (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 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 is performed with the soil specimen fully immersed in water to eliminate the effects of capillary moisture stresses. Gibson & Henkel (1954) and Head (1982a) recommend a time to failure, \( t_f \), for drained direct shear tests to be 12.7\( t_{100} \), where \( t_{100} \) is the time to 100% primary consolidation and can be extrapolated by linearizing the square root of the cube root of time plot of the consolidation phase of the test. The maximum permissible shearing rate in a drained direct shear test can be estimated to be less than \( \delta_s/t_f \), where \( \delta_s \) is the horizontal displacement of the shear box at peak strength. This value is however, not known prior to testing and has to be estimated or based on experience on similar materials.

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 pass through the origin but for soils with bonded structure, there will usually be an apparent cohesion intercept. For details of carrying out direct shear box test, reference can be made to Head (1982a).

**Triaxial test**

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

a) The angle of friction (peak, critical state and residual).
b) The cohesion intercept (peak, critical state and residual) *Note: to use with care.
c) The pore water pressure response due to shearing (in undrained tests).
d) Initial tangent and secant moduli (unloading and reloading).
e) Consolidation characteristics and permeability.

In normal practice, the following tests are 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, \( \sigma_3 \). 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 one of the most commonly used laboratory tests 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 isotropic consolidation under consolidation stress, \( \sigma_3 \). Full drainage during shearing is permitted so that no excess pore water pressure is generated. Although CID tests are technically superior, they are not often used due to the 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 be about 75 mm. Therefore, the use of Mazer 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 triaxial tests, multi-stage tests should not be used as these tests will usually produce misleadingly 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 (GEO 2000). Sometimes high \( c' \) obtained from testing is often due to the rate of strain or time of shearing to failure being too short. The rate of strain should be estimated from the results during consolidation as discussed earlier. Side drains should not be used as this has shown to produce inconsistency in the sample (Tschebotarioff 1950, GEO 2000). Further details on the laboratory triaxial tests can be obtained from Head (1982b).
Interpretation of Effective Shear Strength from Laboratory Tests

The shear strength of the soil in normal practice is usually represented graphically on a Mohr diagram. The $c'$ and $\phi'$ 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 ($c' - \phi'$ soil strength model) for the concerned stress range as expressed in the equation below:

$$\tau_f = c' + \sigma_{nf} \tan \phi'$$  \hspace{1cm} (10.2)

where

- $\tau_f$ = shear strength of soil (kPa)
- $\sigma_{nf}$ = effective normal stress at failure (kPa)
- $\phi'$ = effective angle of friction (degree)
- $c'$ = apparent cohesion (kPa).

Figure 10.6 shows typical bonding and dilatant characteristics of residual soil at low stress range (low confining and consolidation pressure) which exhibits a peak shear envelope in terms of effective stress which has an apparent cohesion intercept, $c'$ if a linear Mohr-Coulomb $c' - \phi'$ 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 lightly consolidated transported soil. The critical state friction angle is represented as $\phi_{cr}$.

As shown in Figure 10.7, the critical state strength falls 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 concerned using the tangent method. It should be noted that $\phi'$ is different from $\phi_{cr}$ (critical state); and $c'$ is simply the intercept of the peak failure envelope on the shear stress axis, $\tau'$. It is important to realise that $c'$ does not imply that at zero effective 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. Therefore, the in situ stress range and the stress path followed (see details in the next section) must be correctly determined in order to obtain $c' - \phi'$ shear strength envelope that is representative of the field condition.

Another method of determining the shear strength envelope is through the secant method for the stress range concerned as shown in Figure 10.8. In this method, generally the $c'$ is taken as zero unless there are sufficient test results to obtain the representative $c'$. Usually $c'$ should not exceed 10 kPa. This method will yield a more conservative (lower) peak strength value compared to the tangent method at the low stress range and both will yield the same results at high stress range. Therefore, if the stress range 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, 1985) 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 the soil particles have become aligned in directions parallel with the direction of sliding and is relevant only after displacements in the order of several metres (Crabb & Atkinson 1991).

Figure 10.6. Effect of bonding on the apparent cohesion intercept of drained strength (effective stress) failure envelope (from Brenner et al. 1997).

Figure 10.7. Typical shearing characteristics of residual soils during drained shear tests and the tangent method in selection of shear strength envelope.

Figure 10.8. Shear strength envelope through the secant method.
site during design cannot be confirmed, then the secant method shall be used instead of the tangent method.

Influence of Stress Path on Shear Strength

In analysis of geotechnical problems including slope stability, Stress Path Method is the most rational and useful method because it uses laboratory and field data to obtain the stress path of average stresses in 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 and then tests (usually laboratory tests) are performed along the field effective stress path with the specimens at the filed conditions; water content, degree of saturation, stress state, pore pressure, geometry, etc.

Figure 10.9 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 \( F_{AW} \). 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 \( \sigma_n' \) 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 (\( F_L > F_C > F_{AW} \)) and thus yield different factors of safety (FOS). It is very important to note that the vertical stress path (usually used in limit equilibrium computer program, \( F_C \)) does not properly model the other two mechanisms that could occur in the field.

Figure 10.10 shows another example of three different stress paths to failure (for one element of soil):

a) Raising the crest through filling on top of the slope will yield FOS of 2.1.

b) Excavating the toe of the slope will yield FOS of 1.2.

c) Conventional vertical stress path (limit equilibrium analysis assumption) will yield FOS of 1.8.

From the two examples shown (Figures 10.9 and 10.10), three important points have been illustrated:

a) The factor of safety (FOS) for slopes depends on the stress path to failure

b) 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 stability 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 field stress path to failure of slope

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

Groundwater and Pore Water Pressure

The hydrological effects of rainfall on a permeable slope are shown in Figure 10.11. Some of the rainwater runs off the slope and may cause surface erosion if there is inadequate surface protection. In view of the high soil permeability, much of the wa-
ter will infiltrate into the subsoil. This causes the water level in the slope to rise or it may cause a 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 and thus reduces the soil suction (i.e. negative pore pressure).

Failure in residual soils cut slopes might be caused by ‘wetting-up’ process which causes a decrease in soil suction and hence, decreases the soil strength. 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) also reported that over the 60 slope failures they investigated, three-quarters of these failures were due to an increase in pore water pressure.

Slopes should be designed for the representative groundwater level through observation and estimation. However, to predict pore water pressure or groundwater level in cut slopes is one of the most difficult tasks because there are many unknown variables that require long-term monitoring (especially high cut slopes with catchments behind them) and are usually not available during design stage. Therefore, sensitivity analysis on the effect of water levels to the stability of slopes in the high risk-to-life and high economic risk category (GEO 2000) should be carried out and this requires prediction of the worst groundwater conditions.

Transient perched water tables might be formed at the interface of layers of differing permeability. Therefore an examination of the material profiles within a slope and the catchments 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-sites 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 and thus increases 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 in efficiency caused by siltation, deterioration of seals or growth of vegetation blocking the water outlet.

10.3 DESIGN AND STABILIZATION OF RESIDUAL SOIL SLOPES

Analysis of Stability

There are generally two types of failure mechanisms:

a) Planar slide
b) Rotational slip or sliding block

If the residual soil mantle is shallow in comparison with the length of the slope, a planar slide may result (Blight 1997b). The slip surface is commonly between the transition zones of soil (Grade IV to VI) with rock and the thickness of the sliding mass is usually roughly constant. Skempton (1957) presented a simple method of stability analysis for planar slide failure mechanism as illustrated in Figure 10.12. With reference to Figure 10.12, the factor of safety (FOS) with respect to planar slide failure is given as:

\[
FOS = \frac{c' + (\gamma - \gamma_w)z\cos^2\beta \tan\phi'}{\gamma z \sin\beta \cos\phi'}
\] (10.3)

In addition, Skempton (1957) also demonstrated the possibility of slope failure occurring in a relatively gentle gradient due to the effect of rising groundwater level:

If \( c' = 0 \) (critical state), the critical slope is given by the expression:

\[
\tan \beta_c = \frac{\gamma - m\gamma_w}{\gamma} \tan \phi'
\] (10.4)

If, in addition, \( m = 1 \) (i.e. groundwater level coincides with ground surface), then
\[ \tan \beta_c = \frac{\gamma'}{\gamma} \tan \phi' \quad (10.5) \]

where \( \gamma' \) is the submerged density of the soil. Assuming that \( \gamma = 18 \text{ kN/m}^3 \) and \( \phi' = 20^\circ \) and \( c' = 0 \), the critical slope of clay in accordance to the above equation is approximately \( 9^\circ \). When the groundwater level is below the surface, the maximum stable slope will be greater than this value. For example if \( m = \frac{1}{2} \), then \( \beta_c = 14.7^\circ \). This demonstrates the significant influence of groundwater level in stability of slope.

The second typical failure mechanisms is the rotational slip or sliding block which occurs when the residual soil mantle is deep (Blight 1997b). Stability analysis for such failure mechanisms can be carried out using conventional methods such as Modified Bishop, Morgenstern-Price, Janbu, Spencer and Sliding Block method, which are readily available in commercial software. Generally, different analysis methods make different assumptions with regard to the interslice forces and the failure mechanism. For a detailed discussion on the various types of methods of analysis, reference can be made to Fredlund & Krahn (1977).

As a summary, for stability analysis in residual soil slopes, it is important to carry out analyses for different modes of failure if the designer is unsure of the likely failure mechanism in the field, i.e.: a) Planar slide b) Rotational slip (circular) c) Sliding block (wedge)

Methodologies for Stabilization of Slopes
In the following sections, different methodologies for stabilization of slopes will be presented, i.e.: a) Regrading of slope profile b) Rock berms (toe counterweight) c) Reinforced soil wall d) Soil nailing

These methods are by no means exhaustive and other methods such as contiguous bored pile, micropile, sheet pile, etc. can also be used as slope stabilization measures. The four methods above only represent some methods normally used in Malaysia.

The general concept of the stabilization measures shall be presented together with some recommendations on design and good practices. For stabilization work, it is important to bear in mind that the repair work itself may sometimes lead to larger failures, especially during removal of failed material. Therefore, proper caution must be exercised when carrying out stabilization work and work shall be carried out as fast as possible to place material back at the toe of the failure. Removal of failed materials shall not be more than 15 to 30 linear metres (if practical) of the failed material and stabilization work should then be carried out immediately. The minimum distance for machineries operation, site constraints, etc. often governs the practical limits for the removal of the failed material.

10.4 REGRADING OF SLOPE PROFILE
This is the easiest and cheapest method conventionally adopted for stabilizing slopes. Typically, the slope is regraded to a gentler gradient either by trimming or placing of soil/rock onto the existing slope. The regraded slope profile shall be chosen based on the required factor of safety. Important considerations in adopting this method include: a) Failed materials within the failed slope shall be removed prior to placing of soil/rock. If trimming is carried out, the trimming work shall extend beyond the failure plane. This is because the strength of the soil within the failure plane will tend to soften to a value which is significantly lower than the representative soil strength and for repeated failures, this value is close to the residual strength (Skempton 1985). b) The slope shall be benched prior to placement of fills. c) Any structural features on the slope shall be identified. If the failure of the slope is influenced by such features, the required slope profile will be governed by the structural features, e.g. angle of the bedding plane. This may result in a significantly gentler slope profile required for stability as compared to conventional analysis using slope stability methods for homogeneous soils.

A typical schematic sketch illustrating the concept of regrading of slope profile is shown in Figure 10.13.

10.5 ROCK BERMS (TOE COUNTERWEIGHT)
Rock berm (toe counterweight) stabilizes the slope by providing a counterweight in the toe area of the slope. This method is effective in stabilizing slopes which failed due to inadequate lateral resistance or deep rotational failures. This method is also used to repair small slips where the toe area of the slope may be over steepened as a result of erosion or poor construction (FHWA 1988).

The design of rock berms shall take into consideration the location of the berms, i.e.: a) The berms shall be placed in a manner that would contribute to the stabilizing force (NOT destabilizing). b) The berms shall extend beyond the toe area of the failure.
10.6 REINFORCED SOIL WALL

Reinforced soil wall consists of a composite material of frictional soils and reinforcing strips that enable the gravity forces on a wall to be resisted by tensile forces generated in the strip and hence transferred by friction to the soil (Puller 1996). For the design of reinforced soil walls, the following criteria have to be satisfied:

a) Internal stability (Figure 10.15)

- Tension failure of reinforcement strips
- Pull-out failure of reinforcement strips

b) External stability (Figure 10.16)

- Sliding failure
- Overturning failure
- Bearing and tilt failure
- Slip failure

c) Settlement of the wall is within acceptable limits in accordance to its intended function (serviceability limit state).

To satisfy the above criteria, design of reinforced soil walls involves determining:

a) Reinforcement length and lateral/vertical spacing of reinforcement, and
b) Embedment depth of the wall.

Various codes of practice and design manuals such as listed below are available for design of reinforced soil walls:

b) U.S. Department of Transportation, Federal Highway Administration (FHWA 2001); Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines. In this chapter, only the recommended design approach by BS8006 is discussed and some of the important recommendations by BS8006 are summarised in the following sections (Fong 2003).

**Reinforcement length**

BS8006 recommends that:

a) The minimum reinforcement length is 0.7*H for normal retaining structures where H is the maximum height of the wall or higher than the wall if there is a sloping backfill.

b) For abutments (bridges), the minimum length shall be (whichever is longer):
   a. 0.6*H + 2 metres
   b. 7.0 metres

c) If the reinforcement length is to be stepped, the maximum difference between the steps shall be less than 0.15*H.

Figure 10.17 illustrates the definition of the above criteria as recommended by BS8006.

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Figure 10.15. Internal stability (from Clayton et al. 1993).
```

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Figure 10.16. External stability (from Das 1999).
```

**Reinforcement type**

BS8006 differentiates reinforcement into two types:

a) Extensible reinforcement
   Reinforcement that sustains design loads at strains greater than 1% (e.g. geosynthetics).

b) Inextensible reinforcement
   Reinforcement that sustains design loads at strains less or equal to 1% (e.g. metallic reinforcement).

**Embedded depth**

The minimum depth is governed by the mechanical height of the wall and also the factored bearing pressure. The recommendations by BS8006 are summarised in Table 10.4.

Table 10.4. Minimum embedment depth (from Table 20, BS8006 1995).

<table>
<thead>
<tr>
<th>Slope of the ground at toe, $\beta_s$</th>
<th>Minimum embedment, $D_{m}$ (m)</th>
<th>Minimum embedment, $D_{m}/q_r$ (m$^3$/kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\beta_s = 0$</td>
<td>Walls</td>
<td>H/20</td>
</tr>
<tr>
<td>$\beta_s = 0$</td>
<td>Abutments</td>
<td>H/10</td>
</tr>
<tr>
<td>$\beta_s = 18^\circ$</td>
<td>Walls</td>
<td>H/10</td>
</tr>
<tr>
<td>(cot $\beta_s = 3/1$)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\beta_s = 27^\circ$</td>
<td>Walls</td>
<td>H/7</td>
</tr>
<tr>
<td>(cot $\beta_s = 2/1$)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\beta_s = 34^\circ$</td>
<td>Walls</td>
<td>H/5</td>
</tr>
<tr>
<td>(cot $\beta_s = 3/2$)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note:
1. $q_r$ – factored bearing pressure
2. For definition of notation, refer to Figure 10.18.

**External stability**

External stability checks for reinforced soil wall can be carried out using conventional analysis methods used for a gravity retaining wall. BS8006 recommendations on external loads and partial safety factors should be taken into consideration when carrying out the external stability checks.

**Internal stability**

BS8006 provides internal stability checks using two methods:

a) Coherent gravity method (Figure 10.19)

b) Tie back wedge method (Figure 10.20)

The tie back wedge method is based on the principles currently employed for classical or anchored retaining walls. Meanwhile, the coherent gravity method is based on the monitored behavior of structures using inextensible reinforcements and has evolved over a number of years from observations on a large number of structures, supported by theoretical analysis.

specialist contractors use 1.5 m x 1.5 m concrete face panel with two anchor points per panel per level.
Coherent Gravity method should only be used for inextensible reinforcements and for simple wall geometry. For complex wall geometry, curved walls or multi-tiered wall, comparison should also be made using the Tie Back Wedge method and the design which gives longer reinforcement length or closer reinforcement spacing is to be adopted (i.e. whichever is more conservative).

**Settlement**

Settlement of the wall, especially differential settlement should be assessed to ensure that the long-term settlement of the wall is within tolerable limits so as not to cause distress to the wall or impair its functionality. For walls with reinforced concrete panels and metallic reinforcements, the maximum differential settlement is limited to 1% or $\Delta L = 1/100$.

Typical section of a reinforced soil wall is as shown in Figure 10.21.

### 10.7 SOIL NAILING

Stabilization of slopes using soil nailing has the distinct advantage of strengthening the slope without causing further disturbance to the existing slope. Therefore, this method is very popular for strengthening work involving distressed slopes. The basic concept of soil nailing is to reinforce and strengthen the existing ground by installing closely-spaced steel bars, called 'nails', into a slope as construction proceeds from 'top-down'. This process creates a reinforced section that is in itself stable and able to retain the ground behind it. The reinforcements are passive and develop their reinforcing action through nail-ground interactions as the ground deforms during and following construction.
Various codes of practice and design manuals such as listed below are available for design of soil nailing:

While BS8006 provides some guidelines for the design of soil nailing, it is not as comprehensive and user-friendly compared to the recommendations of FHWA’s manual. FHWA’s manual provides a very comprehensive and systematic design approach and major steps involved in the design using Service Load Approach (SLD) are summarized as follows:

**Step 1: Set up critical design cross-section(s) and select a trial design**

This step involves selecting a trial design for the design geometry and loading conditions. The ultimate soil strength properties for the various subsurface layers and design water table location should also be determined. Table 10.5 provides some guidance on the required input such as the design geometry and relevant soil parameters. Subsequently, a proposed trial design nail pattern, including nail lengths, tendon sizes, and trial vertical and horizontal nail spacing, should be determined.

**Step 2: Compute the allowable nail head load**

The allowable nail head load for the trial construction facing and connector design is evaluated based on the nominal nail head strength for each potential failure mode of the facing and connection system, i.e. flexural and punching shear failure. The flexural and punching strength of the facing is evaluated as follows in accordance to the recommendations of FHWA (1998):

**Flexural strength of the facing:**
Critical nominal nail head strength, $T_{FN}$

$$T_{FN} = C_F (m_{V,NEG} + m_{V,POS}) \left( \frac{S_H}{S_V} \right)$$

(10.6)

Where:
- $m_{V,NEG}$ = vertical unit moment resistance at the nail head
- $m_{V,POS}$ = vertical unit moment resistance at mid-span locations
- $S_H$ = horizontal nail spacings
- $S_V$ = vertical nail spacings
- $C_F$ = pressure factor for facing flexure (Table 10.8)

**Vertical nominal unit moment**

$$m_v = \left( \frac{A_s F_y}{b} \right) \left[ d - \left( \frac{A_s f'c}{1.7 f'c b} \right) \right]$$

(10.7)

Where:
- $A_s$ = area of tension reinforcement in facing panel width ‘b’
- $b$ = width of unit facing panel (equal to $S_H$)
- $d$ = distance from extreme compressive fiber to centroid of tension reinforcement
- $f'c$ = concrete compressive strength
- $F_y$ = tensile yield stress of reinforcement

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Figure 10.21. Typical details of reinforced soil wall.

Punching shear strength of the facing:
Nominal internal punching shear strength of the facing, $V_N$

$$V_N = 0.33 (f'_c \text{ (MPa)})^{1/2} \pi (D'_c) (h_c) \quad (10.8)$$

$$D'_c = b_{PL} + h_C$$

Nominal nail head strength, $T_{FN}$

$$T_{FN} = V_N \{1/1-[C_S(A_C-A_GC)/(S_H S_V)]\} \quad (10.9)$$

$C_S =$ pressure factor for punching shear (Table 10.8)

$A_C, A_GC =$ refer Figure 10.23.

The allowable nail head load is then the lowest calculated value for the two different failure modes.

Step 3: Minimum allowable nail head service load check

This empirical check is performed to ensure that the computed allowable nail head load exceeds the estimated nail head service load that may actually be developed as a result of soil-structure interaction. The nail head service load actually developed can be estimated by using the following empirical equation:

$$t_f = F_f K_A \gamma H S_H S_V \quad (10.10)$$

$F_f =$ empirical factor (= 0.5)

$K_A =$ coefficient of active earth pressure

$\gamma =$ bulk density of soil

$H =$ height of soil nail wall

$S_H =$ horizontal spacing of soil nails

$S_V =$ vertical spacing of soil nails

Step 4: Define the allowable nail load support diagrams

This step involves the determination of the allowable nail load support diagrams. The allowable nail load support diagrams are useful for subsequent limit equilibrium analysis. The allowable nail load support diagrams are governed by:

a) Allowable Pullout Resistance, $Q$ (Ground-Grout Bond)

$$Q = \alpha_Q \times \text{Ultimate Pullout Resistance, } Q_u$$

b) Allowable Nail Tendon Tensile Load, $T_N$

$$T_N = \alpha_N \times \text{Tendon Yield Strength, } T_{YN}$$

c) Allowable Nail Head Load, $T_F$

$$T_F = \alpha_F \times \text{Nominal Nail Head Strength, } T_{FN}$$

where $\alpha_Q, \alpha_N, \alpha_F =$ strength factor (Table 10.10).

Next, the allowable nail load support diagrams shall be constructed according to Figure 10.24.

Step 5: Select trial nail spacing and lengths

Performance monitoring results carried out by FHWA have indicated that satisfaction of the strength limit state requirements will not in itself ensure an appropriate design. Additional constraints are required to provide for an appropriate nail layout. The following empirical constraints on the design analysis nail pattern are therefore recommended for use when performing the limiting equilibrium analysis:

a) Nails with heads located in the upper half of the wall height should be of uniform length.

b) Nails with heads located in the lower half of the wall height shall be considered to have a shorter length in design even though the actual soil nails installed are longer due to incompatibility of strain mobilized compared to the nails at the upper half. This precautionary measure is in accordance with the recommendations given by Figure 10.25. However, further refinement in the nail
lengths can also be carried out if more detailed analyses are being carried out, e.g. using finite element method (FEM) to verify the actual distribution of loads within the nails.

The above provision ensures that adequate nail reinforcement (length and strength) is installed in the upper part of the wall. This is due to the fact that the top-down method of construction of soil nail walls generally results in the nails in the upper part of the wall being more significant than the nails in the lower part of the wall in developing resisting loads and controlling displacements. If the strength limit state calculation overstates the contribution from the lower nails, then this can have the effect of indicating shorter nails and/or smaller tendon sizes in the upper part of the wall, which is undesirable since this could result in less satisfactory in-service performance. The above step is essential where movement sensitive structures are situated close to the soil nail wall. However, for stabilization work in which movement is not an important criterion, e.g. slopes where there is no nearby buildings or facilities, the above steps may be ignored.

Figure 10.22. Definition of notation used in Table 10.5.

**Step 6: Define the ultimate soil strengths**
The representative soil strengths shall be obtained using conventional laboratory tests, empirical correlations, etc. The limit equilibrium analysis shall be carried out using the representative soil strengths (NOT factored strengths).

**Step 7: Calculate the factor of safety**
The Factor of Safety (FOS) for the soil nail wall shall be determined using the ‘slip surface’ method (e.g. Bishop’s circular). This can be carried out using commercially available software to perform the analysis. The stability analysis shall be carried out iteratively until convergence, i.e. the nail loads corresponding to the slip surface are obtained. The required factor of safety (FOS) for the soil nail wall shall be based on recommended values for conventional retaining wall or slope stability analyses (e.g. 1.4 for slopes in the high risk-to-life and economic risk as recommended by GEO (2000)).

Figure 10.23. Bearing plate connection details (from FHWA 1998).

**Step 8: External stability check**
The potential failure modes that require consideration with the slip surface method include:

a) Overall slope failure external to the nailed mass (both ‘circular’ and ‘sliding block’ analysis are to be carried out outside the nailed mass). This is especially important for residual soil slopes which often exhibit specific slip surfaces, defined by relict structure, with shear strength characteristics that are significantly lower than those applied to the ground mass in general. Therefore, for residual soil slopes, the analyses must consider both general and non-structurally controlled slip surfaces in association with the strength of the ground mass, together with specific structurally controlled slip surfaces in association with the strength characteristics of the relict joint surfaces themselves. The soil nail reinforcement
must then be configured to support the most critical condition of these two conditions.
b) Foundation bearing capacity failure beneath the laterally loaded soil nail ‘gravity’ wall. As bearing capacity seldom controls the design, therefore, a rough bearing capacity check is adequate to insure global stability.

**Step 9: Check the upper cantilever**
The upper cantilever section of a soil nail wall facing, above the top row of nails, will be subjected to earth pressures that arise from the self-weight of the adjacent soil and any surface loadings acting upon the adjacent soil. Because the upper cantilever is not able to redistribute load by soil arching to adjacent spans, as can the remainder of the wall facing below the top nail row, the strength limit state of the cantilever must be checked for moment and shear at its base, as described in Figure 10.26.

For the cantilever at the bottom of the wall, the method of construction (top-down) tends to result in minimal to zero loads on this cantilever section during construction. There is also the potential for any long-term loading at this location to arch across this portion of the facing to the base of the excavation. It is therefore recommended by FHWA (1998) that no formal design of the facing be required for the bottom cantilever. It is also recommended, however, that the distance between the base of the wall and the bottom row of nails not exceed two-thirds of the average vertical nail spacing.

**Step 10: Check the facing reinforcement details**
Check waler reinforcement requirements, minimum reinforcement ratios, minimum cover requirements, and reinforcement anchorage and lap length as per normal recommended procedures for structural concrete design.

It is recommended that waler reinforcement (usually 2T12) to be placed continuously along each nail row and located behind the face bearing plate at each nail head (i.e. between the face bearing plate and the back of the shotcrete facing). The main purpose of the waler reinforcement is to provide additional ductility in the event of a punching shear failure, through dowel action of the waler bars contained within the punching cone.

**Step 11: Serviceability checks**
Check the wall function as related to excess deformation and cracking (i.e. check the service limit states). The following issues should be considered:
a) Service deflections and crack widths of the facing.
b) Overall displacements associated with wall construction.
c) Facing vertical expansion and contraction joints.

Typical section of a soil nail wall is as shown in Figure 10.27.

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**Table 10.5. Input required for soil nail design.**

<table>
<thead>
<tr>
<th>Soil Properties</th>
<th>Bulk density, ( \gamma )</th>
<th>Ultimate friction angle, ( \phi_{ult} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultimate soil cohesion, ( c_{ult} )</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Wall height, ( H )</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Wall inclination, ( \alpha )</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Height of upper cantilever, ( C )</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Height of lower cantilever, ( B )</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Backslope angle, ( \beta )</td>
<td>( \beta ) ( \geq ) ( \phi_{ult} )</td>
<td></td>
</tr>
<tr>
<td>Soil-to-wall interface friction angle, ( \delta )</td>
<td>Typically 2/3 ( \phi_{ult} )</td>
<td></td>
</tr>
<tr>
<td>Nail inclination, ( \eta )</td>
<td>Typically 15°</td>
<td></td>
</tr>
<tr>
<td>Vertical spacing of nail, ( S_v )</td>
<td>Typically 1.5 m to 2.5 m</td>
<td></td>
</tr>
<tr>
<td>Horizontal spacing of nail, ( S_h )</td>
<td>Typically 1.5 m to 2.5 m</td>
<td></td>
</tr>
<tr>
<td>Characteristic strength of nail, ( F_y )</td>
<td>Typically 460 N/mm²</td>
<td></td>
</tr>
<tr>
<td>Nail size/diameter</td>
<td>Minimum ( \phi_{20} ) mm</td>
<td></td>
</tr>
<tr>
<td>Ultimate bond stress, ( Q_u ) (kN/m)</td>
<td>Values given in Tables 10.6 &amp; 10.7 in kN/m²</td>
<td></td>
</tr>
<tr>
<td>Multiply with perimeter of grout column ( \pi \times D_{GC} ) to obtain value in kN/m</td>
<td>Tables 10.6 &amp; 10.7</td>
<td></td>
</tr>
<tr>
<td>Thickness of shotcrete</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Depth / Width of steel plate</td>
<td>Minimum plate width 200mm</td>
<td></td>
</tr>
<tr>
<td>Thickness of steel plate</td>
<td>Minimum plate thickness 19mm</td>
<td></td>
</tr>
<tr>
<td>Reinforcement for shotcrete</td>
<td>Use BRC reinforcement</td>
<td></td>
</tr>
<tr>
<td>Waler bars</td>
<td>Typically 2T12</td>
<td></td>
</tr>
<tr>
<td>Concrete cover</td>
<td>Typically 50 – 75mm</td>
<td></td>
</tr>
<tr>
<td>Diameter of grout column, ( D_{GC} )</td>
<td>Typically 125mm</td>
<td></td>
</tr>
<tr>
<td>Soil strength</td>
<td>Table 10.10</td>
<td></td>
</tr>
<tr>
<td>Nail tendon tensile strength, ( a_{nt} )</td>
<td>Table 10.10</td>
<td></td>
</tr>
<tr>
<td>Ground-grout pullout resistance, ( a_{gg} )</td>
<td>Table 10.10</td>
<td></td>
</tr>
<tr>
<td>Facing flexure pressure, ( C_F )</td>
<td>Table 10.8</td>
<td></td>
</tr>
<tr>
<td>Facing shear pressure, ( C_s )</td>
<td>Table 10.8</td>
<td></td>
</tr>
<tr>
<td>Nail head strength facing flexure / punching shear, ( a_{fn} )</td>
<td>Table 10.9</td>
<td></td>
</tr>
<tr>
<td>Nail head service load, ( F_F )</td>
<td>Section 2.4.5 (FHWA 1998)</td>
<td></td>
</tr>
<tr>
<td>Bearing capacity</td>
<td>Typically 0.5</td>
<td></td>
</tr>
<tr>
<td>Factors of Safety</td>
<td>Typically 2.5</td>
<td></td>
</tr>
</tbody>
</table>
Table 10.6. Suggested ultimate bond stress – rock (from Table 3.2 and 3.3, FHWA 1998).

<table>
<thead>
<tr>
<th>Construction method</th>
<th>Soil type</th>
<th>Suggested unit ultimate bond stress kN/m² (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-plastic silt</td>
<td>20 – 30 (3.0 – 4.5)</td>
<td></td>
</tr>
<tr>
<td>Medium dense sand</td>
<td>50 – 75 (7.0 – 11.0)</td>
<td></td>
</tr>
<tr>
<td>and silty sand/sandy silt</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dense silty sand and gravel</td>
<td>80 – 100 (11.5 – 14.5)</td>
<td></td>
</tr>
<tr>
<td>Very dense silty sand and gravel</td>
<td>120 – 240 (17.5 – 34.5)</td>
<td></td>
</tr>
<tr>
<td>Loess</td>
<td>25 – 75 (3.5 – 11.0)</td>
<td></td>
</tr>
</tbody>
</table>

Note: In Malaysia, the ultimate bond stress is usually obtained based on correlations with SPT ‘N’ values and typically ranges from 3N to 5N.

Table 10.7. Suggested ultimate bond stress – rock (from Table 3.4, FHWA 1998).

<table>
<thead>
<tr>
<th>Construction method</th>
<th>Soil type</th>
<th>Unit ultimate bond stress kN/m² (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Marl / Limestone</td>
<td>300 – 400 (43.5 – 58.0)</td>
<td></td>
</tr>
<tr>
<td>Phillite</td>
<td>100 – 300 (14.5 – 43.5)</td>
<td></td>
</tr>
<tr>
<td>Chalk</td>
<td>500 – 600 (72.0 – 86.5)</td>
<td></td>
</tr>
<tr>
<td>Soft Dolomite</td>
<td>400 – 600 (58.0 – 86.5)</td>
<td></td>
</tr>
<tr>
<td>Fissured Dolomite</td>
<td>600 – 1000 (86.5 – 144.5)</td>
<td></td>
</tr>
<tr>
<td>Weathered Sandstone</td>
<td>200 – 300 (29.0 – 43.5)</td>
<td></td>
</tr>
<tr>
<td>Weathered Shale</td>
<td>100 – 150 (14.5 – 21.5)</td>
<td></td>
</tr>
<tr>
<td>Weathered Schist</td>
<td>100 – 175 (14.5 – 25.5)</td>
<td></td>
</tr>
<tr>
<td>Basalt</td>
<td>500 – 600 (72.0 – 86.5)</td>
<td></td>
</tr>
</tbody>
</table>

Table 10.8. Recommended value for design – facing pressure factors (from Table 4.2, FHWA 1998).

<table>
<thead>
<tr>
<th>Nominal facing thickness (mm)</th>
<th>Temporary facings</th>
<th>Permanent facings</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Flexure pressure factor $C_F$</td>
<td>Shear pressure factor $C_S$</td>
</tr>
<tr>
<td>100</td>
<td>2.0</td>
<td>2.5</td>
</tr>
<tr>
<td>150</td>
<td>1.5</td>
<td>2.0</td>
</tr>
<tr>
<td>200</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Table 10.9. Nail head strength factors – SLD (from Table 4.4, FHWA 1998).

<table>
<thead>
<tr>
<th>Failure mode</th>
<th>Nail head strength factor (Group I)</th>
<th>Nail head strength factor (Group IV)</th>
<th>Nail head strength factor (Group VII) (Seismic)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Facing Flexure</td>
<td>0.67</td>
<td>1.25(0.67) = 0.83</td>
<td>1.33(0.67) = 0.89</td>
</tr>
<tr>
<td>Facing</td>
<td>0.67</td>
<td>1.25(0.67) = 0.83</td>
<td>1.33(0.67) = 0.89</td>
</tr>
<tr>
<td>Punching Shear</td>
<td>0.67</td>
<td>1.25(0.67) = 0.83</td>
<td>1.33(0.67) = 0.89</td>
</tr>
<tr>
<td>Headed Stud</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tensile Fracture</td>
<td>0.50</td>
<td>1.25(0.50) = 0.63</td>
<td>1.33(0.50) = 0.67</td>
</tr>
<tr>
<td>ASTM A307</td>
<td>0.50</td>
<td>1.25(0.50) = 0.63</td>
<td>1.33(0.50) = 0.67</td>
</tr>
<tr>
<td>Bolt Material</td>
<td>0.50</td>
<td>1.25(0.50) = 0.63</td>
<td>1.33(0.50) = 0.67</td>
</tr>
<tr>
<td>ASTM A325</td>
<td>0.50</td>
<td>1.25(0.50) = 0.63</td>
<td>1.33(0.50) = 0.67</td>
</tr>
<tr>
<td>Bolt Material</td>
<td>0.50</td>
<td>1.25(0.50) = 0.63</td>
<td>1.33(0.50) = 0.67</td>
</tr>
</tbody>
</table>

Table 10.10. Strength factors and factors of safety (from Table 4.5, FHWA 1998).

<table>
<thead>
<tr>
<th>Element</th>
<th>Strength Factor (Group I)</th>
<th>Strength Factor (Group IV)</th>
<th>Strength Factor (Group VII) (Seismic)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nail head strength</td>
<td>$\alpha$ = Table 12.9</td>
<td>see Table 12.9</td>
<td>see Table 12.9</td>
</tr>
<tr>
<td>Nail tendon tensile strength</td>
<td>$\alpha_s = 0.55$</td>
<td>1.25(0.55) = 0.69</td>
<td>1.33(0.55) = 0.73</td>
</tr>
<tr>
<td>Ground-grout pullout resistance</td>
<td>$\alpha_o = 0.50$</td>
<td>1.25(0.55) = 0.63</td>
<td>1.33(0.50) = 0.67</td>
</tr>
<tr>
<td>Soil</td>
<td>F = 1.35</td>
<td>1.08</td>
<td>1.01</td>
</tr>
<tr>
<td>(1.50*)</td>
<td></td>
<td>(1.20*)</td>
<td>(1.13*)</td>
</tr>
<tr>
<td>Soil-temporary construction condition†</td>
<td>F = 1.20</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>(1.35*)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: 
- a) Group I: General loading conditions
- b) Group IV: Rib shortening, shrinkage and temperature effects taken into consideration
- c) Group VII: Earthquake (seismic) effects (Not applicable in Malaysia)
- d) *Soil Factors of Safety for Critical Structures
- e) † Refers to temporary condition existing following cut excavation but before nail installation. Does not refer to ‘temporary’ versus ‘permanent’ wall.
Note: “r” values determined by linear interpolation between a value of 1.0 at wall mid-height and “R” at base of wall.

L = Maximum Nail Length
H = Wall Height
QD = Dimensionless Pullout Resistance
   = αQU / (γ SH SV)

where

αQ = pullout resistance strength factor
QU = ultimate pullout resistance
γ = unit weight
SH = horizontal nail spacing
SV = vertical nail spacing

Figure 10.25. Nail length distribution assumed for design (from FHWA 1998).

Figure 10.26. Upper cantilever design checks (from FHWA 1998).
10.8 CONTROL OF DRAINAGE AND SEEPAGE

As discussed in the earlier sections, water is one of the main contributors to slope failure in residual soils. Therefore, slope stabilization measures are usually implemented together with methods to control drainage and seepage. A variety of surface and subsurface drainage may be used to control drainage and seepage as listed below:

a) Surface water drainage measures
   i. Surface ditches
   ii. Divert surface waters
   iii. Seal joints, cracks, fissures
   iv. Regrade slope to eliminate ponding
   v. Shotcrete surface of slope
   vi. Hydroseeding/Turfing

b) Subsurface drainage measures
   i. Horizontal drains
   ii. Weepholes
   iii. Subsoil drains
   iv. Drainage blankets
   v. Cut-off walls
   vi. Vertical well points
   vii. Seepage tunnels

Drainage is important to intercept groundwater seepage before it enters the slope zone and drains away the water that might infiltrate the slope from rainfall. For subsurface drainage, the drainage blanket should be protected from clogging by enclosing it with a proper geotextile filter, or by designing the granular blanket as a filter.

Some typical drawings showing some of the drainage measures commonly adopted are shown in Figures 10.28, 10.29, 10.30 and 10.31 respectively.

10.9 CONCLUSION

Residual soils exhibit characteristics that are quite different from those of transported soils, i.e.:

a) Very heterogeneous, which makes sampling and testing for relevant engineering parameters difficult.

b) Usually high permeability, therefore susceptible to rapid changes in material properties when subjected to changes of external hydraulic condition.

Residual soils also often exhibit structural features such as relict jointing, bedding or slickensiding which it has inherited from the parent rock.

As such, slope stability and stabilization in residual soils is often not a straightforward task and it poses great challenges to geotechnical engineers all around the world. Therefore, proper understanding of the behavior of residual soils is essential and geotechnical engineers should realize the difference in stress-strain behavior between residual soils and transported soils and its influence on shear strength.

Stability analysis for residual soil slopes are governed by the likely mode of failure and there are generally three types of possible failure mechanisms:

a) Planar slide
b) Rotational slip (circular)
c) Sliding block (wedge)
Figure 10.28. Weepholes.

Figure 10.29. Horizontal drain.
Common methods of stabilization for transported soil slopes can also be adopted for residual soil slopes. However, for residual soil slopes, the influence of structural features shall be properly identified and stabilization measures shall be modified to cater for the most critical conditions. Four different types of stabilization measures are presented, i.e.:

a) Regrading of slope profile
b) Rock berms (toe counterweight)
c) Reinforced soil wall
d) Soil nailing

Finally, drainage measures to control surface runoff and seepage are presented. Drainage measures are usually implemented together with the stabilization measures to effectively stabilize the slope during its intended service.
REFERENCES


