# Program ReSlope (3.0)©: Supplemental Notes

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## 1.0 ABOUT THIS DOCUMENT

This document and the one appearing as Help in ReSlope (3.0)© represent a minor modification of the original report titled "Design Procedure for Geosynthetic Reinforced Steep Slopes," by Dov Leshchinsky, January 1997, US Army Corps of Engineers, Waterways Experiment Station, 3909 Halls Ferry Road, Vicksburg, Mississippi, 39180-6199, Technical Report REMR-GT-23.

## 2.0 INTRODUCTION

Soil is an abundant construction material that, similar to concrete, has high compressive strength but virtually no tensile strength. To overcome this weakness, soils, like concrete, may be reinforced. The materials typically used to reinforce soil are relatively light and flexible, and though extensible, possess a high tensile strength. Examples of such materials include thin steel strips and polymeric materials commonly known as geosynthetics (i.e., geotextiles and geogrids). When soils and reinforcement are combined, a composite material, the so-called 'reinforced soil', possessing high compressive and tensile strength (similar, in principle, to reinforced concrete) is produced.

The increase in strength of the reinforced earth structure allows for the construction of steep slopes. Compared with all other alternatives, geosynthetic reinforced soil slopes are cost-effective. Consequently, various earth structures reinforced with geosynthetics are being constructed worldwide with increased frequency, even in permanent and critical applications (Tatsuoka and Leshchinsky, 1994).

This document describes a design process for geosynthetic reinforced steep slopes. It includes the details of the various stability analyses used to determine the required layout and strength of the reinforcing material. To facilitate the design, these analyses were compiled into a computer program called ReSlope. This program is user-friendly and it contains explanations about input data in windows that appear in response to clicking on 'Help'. ReSlope is interactive, allowing the user to optimize the design with ease. It accounts for elements such as user-specified reduction and safety factors, ultimate strength of geosynthetics, cohesive soils, approximate porewater pressure as determined from a piezometric line or porewater coefficient, external loads, and seismicity.

These note provide suggestions regarding the selection of soil shear strength parameters, definitions of the various safety factors, and practical specifications for reinforcement layout. Design aspects related to erosion control and construction is also discussed. Tips regarding arrest of tension cracks and an economical procedure for repairing a failed slope are given. Most importantly, limitations of the analyses are clearly stated.

## 3.0 ANALYSES USED FOR DESIGN

### 3.1 General

Limit equilibrium (LE) analysis has been used for decades in the design of earth slopes. Attractive features of LE analysis include experience of practitioners with its application, simple input data, useful (though limited) output design information, tangible modeling of reinforcement, and results that can be checked for 'reasonableness' through a different LE analysis method, charts, or hand calculations. Consequently, extension of LE analysis to the design of geosynthetics reinforced steep slopes is desirable. The main drawback of LE analysis is its inability to deal directly with displacements. However, adequate selection of properties of materials and factors of safety should assure acceptable displacements, including safe level of reinforcement deformation.

In principle, inclusion of geosynthetic reinforcement in LE analysis is a straightforward process; the tensile force in the geosynthetic material is included directly in the limit equilibrium equations to assess its effects on stability. However, the inclination of this tensile force must be assumed. Physically, its angle may vary between the as installed (typically horizontal) and tangent to the potential slip surface. Leshchinsky and Boedeker (1989) and Wright and Duncan (1991) have demonstrated that for cohesionless backfill, this inclination has little effects on both the required strength and the layout of reinforcement. They have shown that for cohesionless soil, horizontal tensile force yields slightly conservative results with respect to the required strength of the geosynthetics. Conversely, Leshchinsky (1992) pointed out that for problems such as reinforced embankments over soft soil ($\phi_u$=0; undrained shear strength, $c_u$, is used), the inclination of the reinforcing geosynthetic, located at the foundation and backfill interface, plays a significant role. Since in manmade reinforced slopes the long-term value of cohesion used in design is typically small, inclination has little effects and therefore, it may be assumed horizontal without being overly conservative.

A potential problem in LE analysis of reinforced soil is the need to know the force in each reinforcement layer at the limit-state. Physically, this force may vary between zero and ultimate strength when the slope is at a *global* limit equilibrium state. Assuming the actual force is known in advance implies the reinforcement force is actually an active one, regardless of the problem. The designer then assumes the active force of each reinforcement layer so that *overall* limit equilibrium-state is obtained. The end result may be a slope in which some layers actually provide more force than their allowable strength while other layers are hardly stressed. To overcome this potential problem, a rational methodology to estimate the required (i.e., *reactive*) reinforcement tensile resistance of each layer is introduced via a '*tieback analysis*.' Consequently, the designer can verify whether an individual layer is overstressed or understressed, regardless of the *overall* stability of the slope. Once this problem of 'local stability' is resolved, overall stability of the slope is assessed through rotational and translational mechanisms. In this rotational mechanism (termed here as 'compound failure'), slip surfaces extending between the slope face and the retained soil are examined. The force in the geosynthetic layers in this conventional slope stability analysis is taken directly as the maximum allowable long-term value for each layer. The translational analysis ('direct sliding') is based on the two-part wedge method in which the passive wedge is sliding either over or below the bottom reinforcement layer, or along the interface with the foundation soil. The following is a brief presentation of the various analyses and a summary of their limitations. More information is available in Leshchinsky (1997) and Leshchinsky et al. (1995).

### 3.2 Tieback Analysis

Tieback analysis (more correctly termed 'internal stability analysis') is used to determine the required tensile resistance of each layer needed to assure a reinforced mass that is safe against internal collapse due to its own weight and surcharge loading. This analysis is equivalent to identifying the tensile force needed to resist the 'active lateral earth pressure' at the face of the steep slope. That is, the tensile force needed to restrain the steep slope from sliding along potential slip surfaces that emerge along the face of the slope. The reinforcement tensile force capacity is made possible through a tieback mechanism in which sufficient anchorage of each layer into the stable soil zone is provided. At its front-end, the reinforcement can develop tensile force (i.e., restrain the soil from slipping outwards along the common interface with the reinforcement) only if some type of facing (e.g., wrap around, wire basket) or a trace of cohesion (due to root mat or capillary water) exists.

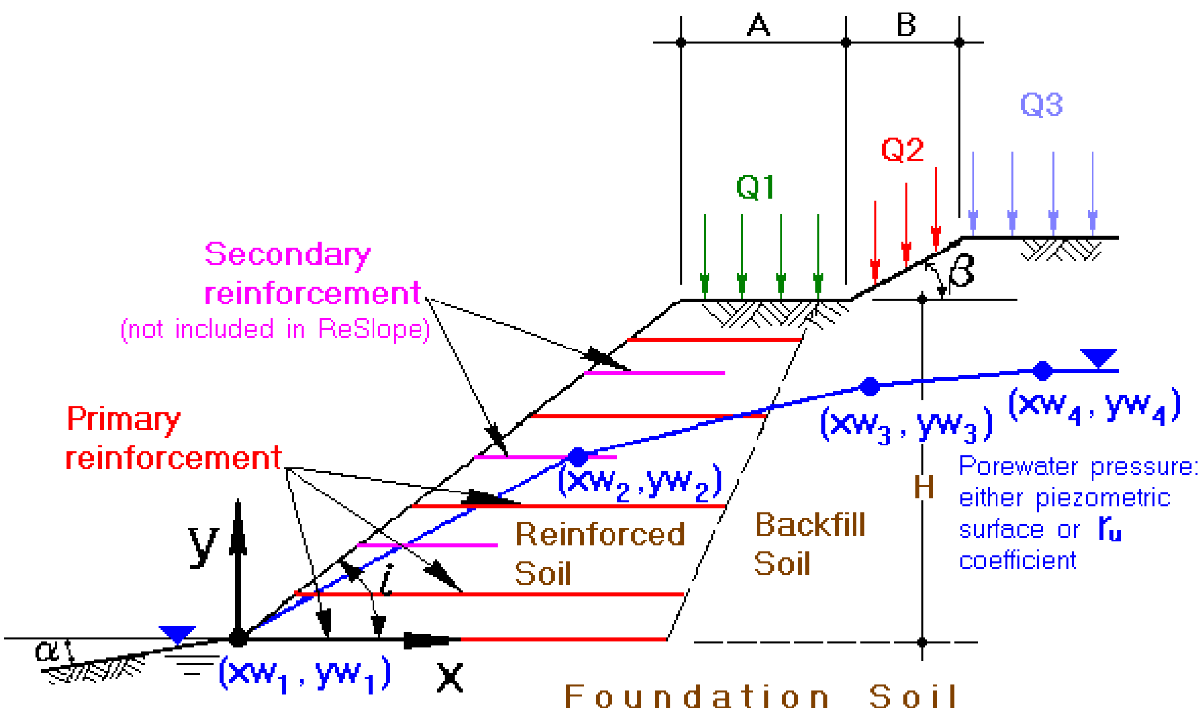
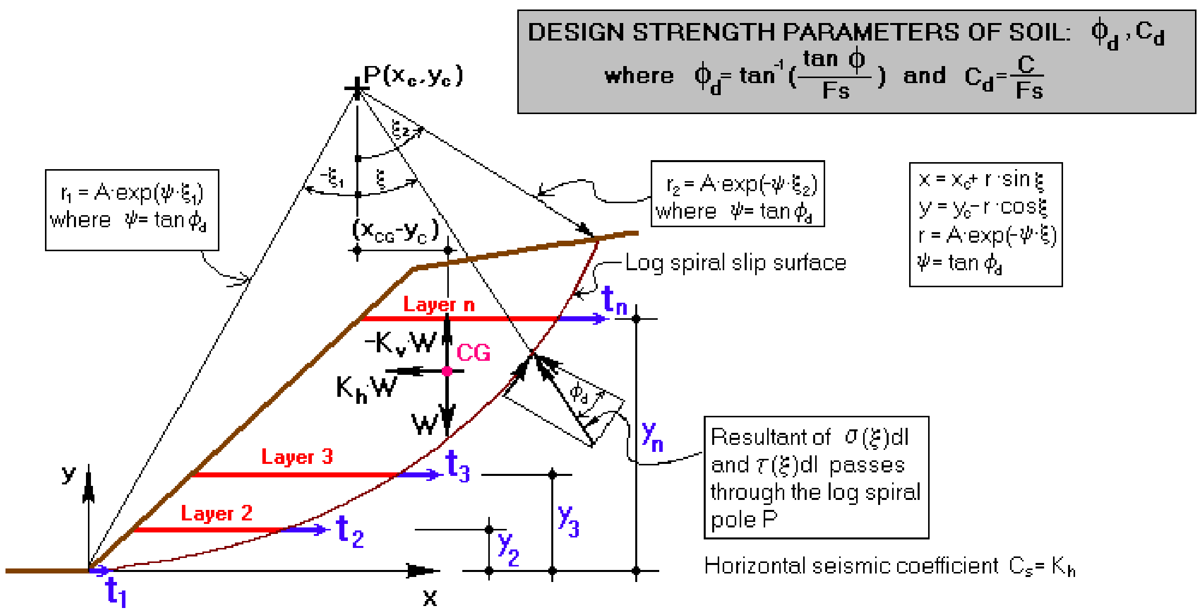



Figure 1. Notation in ReSlope

Figure 1 shows the notation and convention used in ReSlope. Reinforcement is comprised of primary and secondary layers; however, in ReSlope considers only the primary layers. Secondary layers allow for better compaction near the face of the steep slope and thus reduce the potential for sloughing (see Section 3.4). The secondary layers are narrow (typically 1 meter wide) and are installed only if the primary layers are spaced far apart (say, more than about 60 cm apart). At the slope face, the geosynthetic layers may be wrapped around the exposed portion of the soil mass or connected to special prefabricated facing units. If some cohesion exists and the slope is not as steep (e.g., less than 45 degrees), the layers may simply terminate at the slope face as shown in Figure 1. Surcharge loading along the top of the slope may assume three different values as shown in Figure 1. The phreatic surface is defined by a total of four nodes, starting at the origin of the coordinate system (i.e., the toe of the slope) and extending into the slope. Each of the soils (i.e., reinforced soil, backfill or retained soil and foundation soil) may possess different shear strength properties.

In ReSlope analysis, steep slopes are defined as slopes inclined at angles for which they are considered unstable without reinforcement. As an example, for granular backfill a slope would be considered steep if its inclination is steeper than its angle of repose (i.e., $i>\phi_d$ where i and $\phi_d$ are the slope inclination and angle of repose, or design friction angle, respectively). Consequently, in steep slopes the force in each reinforcement layer is activated by an unstable soil mass. That is, the reactive force in each reinforcement layer has to restore a limit equilibrium state. To determine the location of the critical shear surface and subsequently, the necessary reactive force, a log spiral failure surface has been selected. This mechanism is frequently used in geotechnical stability problems.




Moment about P is independent of $\sigma(\xi)$ distribution over the log spiral and therefore, can be written explicitly without resorting to assumptions in statics :

$$\left(+\right. \sum M_P = 0 : W(1+K_v)(x_{CG}-x_C)+K_hW(y_C-y_{CG})-C_d\int(x-x_C)dy-C_d\int(y_C-y)dx+\int u(x-x_C)dx-\int u(y_C-y)dy-\sum_{j=1}^{n} t_j(y_C-y_j)=0$$

Figure 2. Log spiral slip surface: static equilibrium implications

The log spiral mechanism makes the problem statically determinate. For an assumed log spiral failure surface which is fully defined by the parameters $x_c$, $y_c$ and A (e.g., see inset in Figure 2 for definition of terms), the moment equilibrium equation about the pole can be written explicitly without resorting to assumptions in statics. Consequently, by comparing the driving and resisting moments, one can check whether the mass defined by the assumed log spiral is stable for the design values of the shear strength parameters: $\phi_d$ and $c_d$ and the distribution of reinforcement force $t_j$. This check is repeated for other potential log spiral failure surfaces until the least stable system is found, i.e., until the critical slip surface and the associated maximum required restoring reinforcement force are found. The term $C_s$ (see Figure 2) is the horizontal seismic coefficient introducing a pseudo-static force component. It acts at the center of gravity of the critical mass. In ReSlope the notation $K_h$ is used for $C_s$. Also, ReSlope allows for vertical seismic coefficient $K_v$; inclusion of its effects in the moment equilibrium equation is straightforward (see Figure 2). No surcharge is shown in Figure 2 for the sake of clarity of presentation; however, inclusion of its effects in the moment equilibrium equations is straightforward. In this case, $C_s$ is also applied to the surcharge load, rendering a horizontal pseudo-static force at the crest, where the surcharge acts.

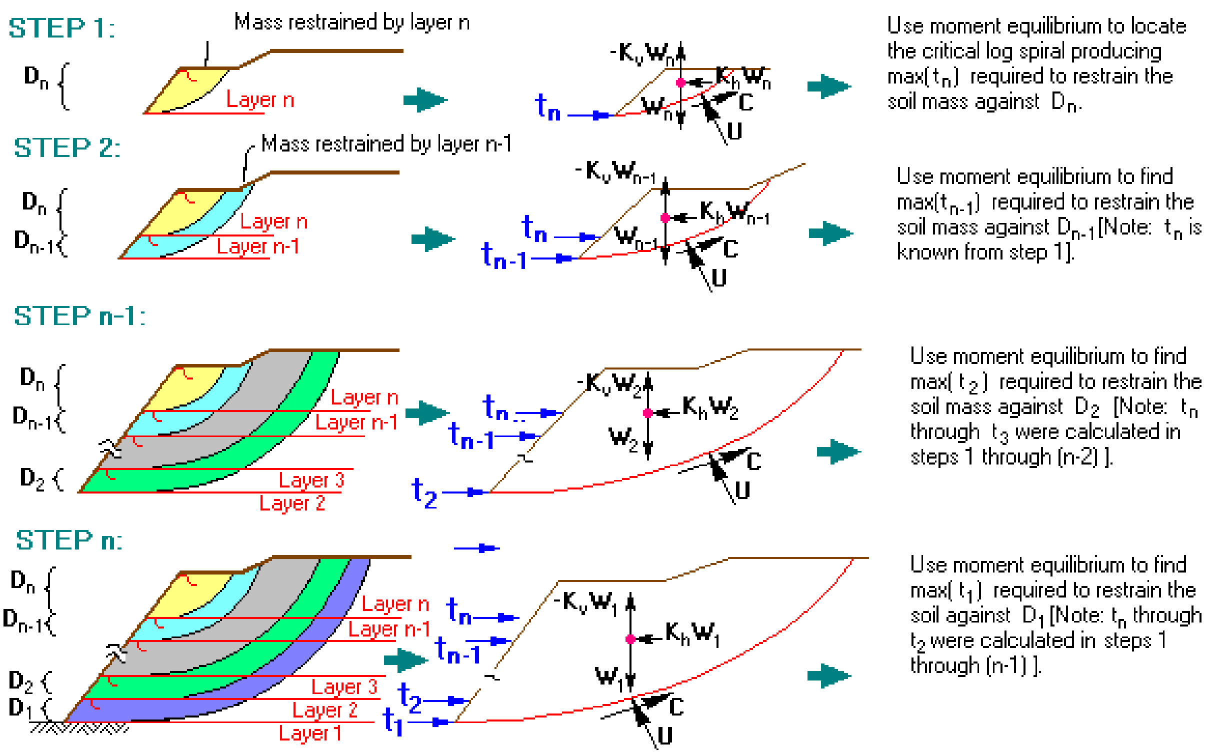



Figure 3. Tensile reaction in reinforcement: computation scheme

Figure 3 illustrates the computational scheme for estimating the tensile reaction in each reinforcement layer. It follows a top-down sequence. In STEP 1, the soil mass acting against $D_n$ is considered. $D_n$ is signified by a reinforcement layer wrapped around the slope face (see Figure 3) thus making it physically feasible for a mass of soil to be laterally supported, resulting in a locally stable mass. That is, $D_n$ is considered as a 'facing unit' (i.e., some facing fixture on the front end of the reinforced soil mass) restraining the unstable soil above from moving outwards. This facing is capable of providing lateral support by being tied to the reinforcement thus its restraining load is transferred into tensile force in the geosynthetic. The moment equilibrium equation is used to find the critical log spiral producing max($t_n$) employing the free-body diagram shown in Figure 3 while examining many potential surfaces. The resulted $t_n$ counterbalances the horizontal force against $D_n$ and thus signifies the reactive force in layer n. That is, the resulted $t_n$ represents the force needed to restore limit equilibrium and hence stability. Note that $D_n$ was chosen to extend down to layer n. This tributary area implies a 'toe' failure activating the largest possible reaction force. It is assumed at this stage that layer n is specified to have strength of exactly $t_n$. Note that the approach presented herein ignores the possibility of front-end pullout in which the soil moves outwards along its interfaces with the reinforcement while the reinforcement is stationary. In other words, the ‘facing’ is assumed to be flexible yet firm enough to prevent outwards slippage of the soil over the reinforcement.

In STEP 2, the force against $D_{n-1}$ is calculated. $D_{n-1}$ extends from layer n to layer (n-1). Using the moment equilibrium equation, max($t_{n-1}$), required to retain the pressure exerted by the unstable soil mass against $D_{n-1}$, is calculated. When calculating $t_{n-1}$, the reaction $t_n$, determined in STEP 1, is known in magnitude and point of action (recall that it is assumed that layer n was ‘installed’ having strength of $t_n$.) Hence, the reactive force in layer (n-1) is the only unknown to be determined from the moment equilibrium equation. It is assumed now that layer (n-1) is specified to have strength $t_{n-1}$.

Figure 3 shows that by repeating the top-down process, the distribution of reactive forces for all reinforcing layers, down to $t_1$, are calculated while supplying the demand for a LE state at each reinforcement level. The end result of tieback is an idealized ‘installation’ of layers with long-term strength varying from $t_n$ to $t_1$, respectively. At this stage, even if the actual strength of the layers is larger, their embedment length beyond the outermost log spiral is just enough to produce the distribution $t_n$ to $t_1$ through pullout mode of failure. Although a situation of higher strength may render the tieback analysis invalid, it is correct along the outermost log spiral. In such a case, the fact that stronger

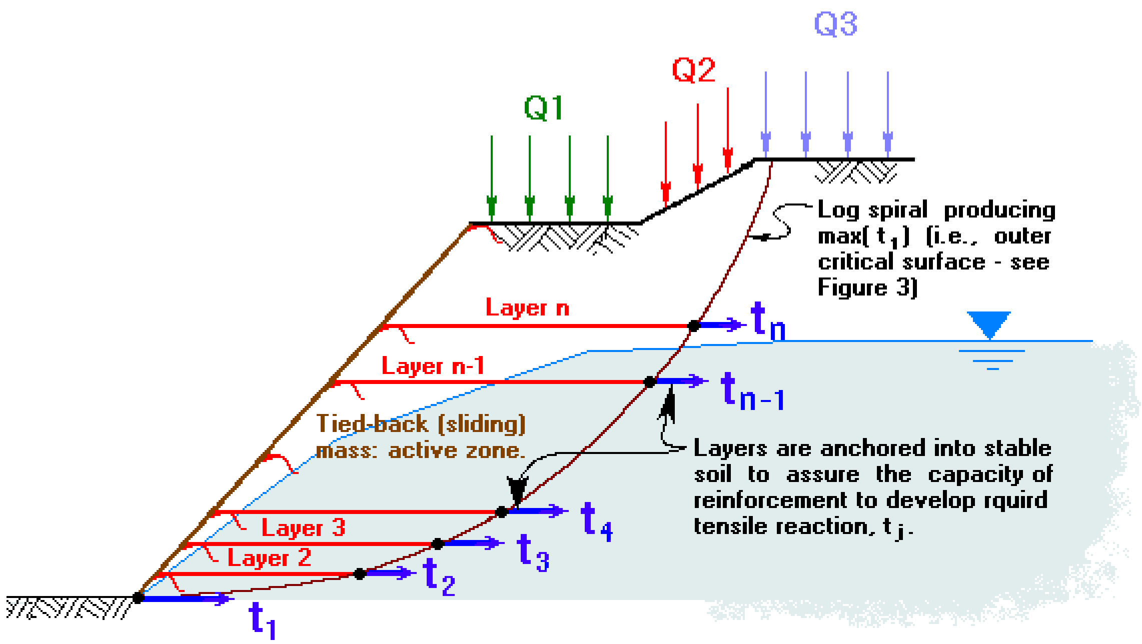



Figure 4. Transfer of tensile reaction into soil adjacent to active zone

layers are installed just produces a safer system inside the active sliding mass (to the left of the outermost log spiral). The main assumption in the analysis procedure is that front-end slippage is not feasible; practically it means that surficial stability is not an issue; ReSlope does not deal with this aspect of stability.

Note that *cohesive* steep slopes are stable up to a certain height. Consequently, the scheme in Figure 3 may produce zero reactive force in top layers. Though these layers may not be needed for local stability (or tieback), they may be needed to resist compound failure as discussed in the next section.

The outermost critical log spiral defines the extreme surface as dictated by Layer 1. In conventional tieback analysis it signifies the extent of the 'active zone'; i.e., it is the boundary between the sliding soil mass and the stable soil. Consequently, reinforcement layers are anchored into the stable soil to assure their capacity to develop the calculated tensile reaction $t_j$ (see Figure 4). In the next section, however, it is shown that the 'stable' soil may not be immediately adjacent to this outermost log spiral and therefore, some layers should be extended further to assure satisfactory stability.

Note in Figures 3 and 4 that the reinforcement layers are wrapped around the overlying layer of soil to form the slope face. However, in slopes that are not as steep (say, i<45°), typically there is no wrap around the face or any other type of facing. In this case, load transfer from each unstable soil mass to the respective reinforcement layer is feasible due to a 'coherent' mass formed at the face. This mass is formed by soil arching or by a trace of cohesion and closely spaced reinforcement layers. The end result is a soil 'plug,' in a sense similar to the one developed at the bottom of a driven open-end pipe pile, that acts *de facto* as a facing unit thus making feasible the load transfer into the primary reinforcement layer. It should be pointed out that 'closely spaced reinforcement' does not necessarily mean closely spaced primary reinforcement layers; simply, this 'plug' can be created by the combination of secondary and primary layers working together to create a coherent mass. Since reinforcement layers, including primary and secondary layers, are typically spaced approximately 30 cm apart, and since the secondary layers extend about 1 meter into the slope, the contribution of secondary layers to the formation of a 'facing' should not be ignored. With time, surface vegetation and its root mat enhances this 'facing.' The end result of forming a coherent face is not just an efficient load transfer from the deep unstable soil mass to the reinforcement, but also improved surficial stability and erosion resistance.

### 3.3 Compound Stability Assessment

For a given geometry, porewater pressure distribution and $\phi_d$ and $c_d$, the tieback analysis provide the minimum required tensile resistance at the level of each reinforcement layer to insure an internally stable structure. It also yields the trace of the outermost log spiral defining the 'active' soil zone, a notion commonly used in conjunction with analysis of retaining walls. In reinforced wall structures, the capacity of the reinforcement to develop the required tensile resistance depends also on its pullout resistance; i.e., the length anchored into the stable soil zone. If the boundary of this stable zone is indeed defined by the 'active' one, then potential slip surfaces that are deeper into the soil mass than the outermost log spiral (outside or within the effective anchorage length) will never be critical. However, since such surfaces will render reduced pullout resistance capacity, they may produce an unstable system. Consequently, a conventional slope stability analysis is used to determine the required reinforcement length so that compound failures will not be likely to occur.

Note that ReSlope assumes that surficial stability is not an issue. The internal stability produces the minimum required long-term strength needed to insure internally stable system at each reinforcement elevation.

The conventional factor of safety in LE is Fs = $\tan(\phi_{available})/\tan(\phi_{design}) = c_{available}/c_{design}$ ; it is also utilized in ReSlope (see general note in Figure 5). The terms $\phi_{design}$ and $c_{design}$ are equivalent to, or interchangeable with, the mobilized shear strength parameters. The specified minimum value of Fs(design) for *soil shear strength* in ReSlope must be satisfied for all rotational slip surfaces, whether tieback or compound.

**GENERAL :** In LE analysis one must assure that for all possible slip surfaces, the following Fs is exceeded:

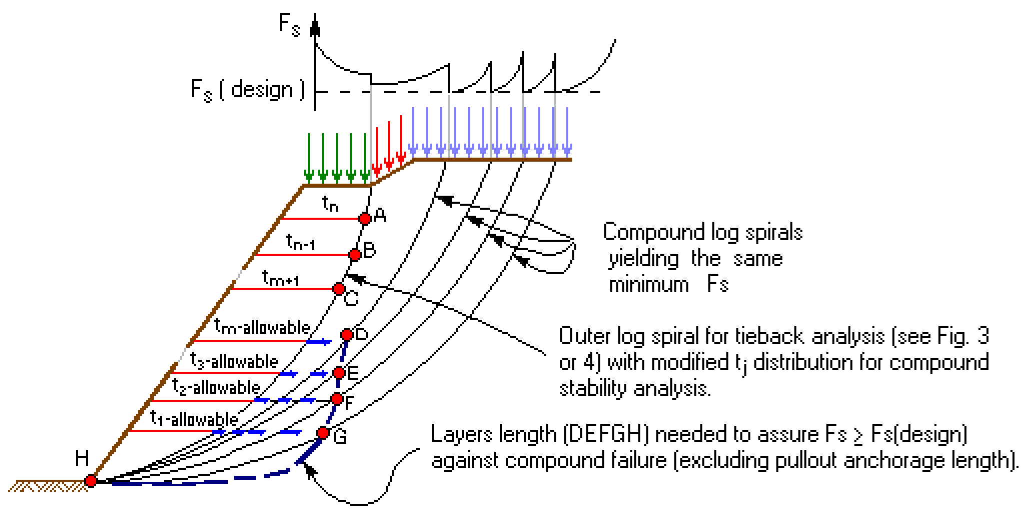
$$\min(F_s) = F_s(\text{design}) = \frac{\tan(\phi_{available})}{\tan(\phi_{design})} = \frac{C_{available}}{C_{design}}$$

NOTE: $\phi_{design} = \phi_{mobilized}$ and $C_{design} = C_{mobilized}$

**STEP 1:** Find $m$ so that $\sum_{j=1}^{m} (t_{allowable})_j \geq \sum_{j=1}^{n} t_j$

where $m$ = minimum number of layers, counting from bottom layer #1, capable of developing total tensile resistance equal to the total force, for all reinforcement layers, as obtained from tieback analysis.

**STEP 2:** Conduct stability analysis according to the following scheme:




**STEP 3:** Repeat **STEP 2** for slip surface emerging at Layer 2, then Layer 3 and so on. Up to Layer n.

**STEP 4:** The longest length (including anchorage) from Steps **2** and **3** is selected for design. assuring adaquate resistance to both tieback and compound failures.

Figure 5. Reinforcement length required assuring compound stability

The tieback analysis results in the *minimum* required allowable strength of reinforcement at each level. The specified reinforcement, therefore, must possess strength equal or exceeding this calculated strength. In reality, the allowable strength of most layers will exceed the required value as determined from the tieback analysis. Consequently, if viewed from global stability, only *m* layers are needed (see Step 1 in Figure 5); i.e., reinforcement selected based on tieback analysis may produce more reinforcement than needed for *global* stability. These bottom *m* layers may contribute their full allowable strength in the compound analysis, which deals only the aspect of global stability. The upper layers *(m+1)* through *n* may contribute only their calculated tieback values. Such a procedure is conservative.

Embedding the layers immediately to the right of the outermost log spiral obtained in the tieback analysis, so that $t_{allowable}$ for layers 1 through *m* and $t_j$ for layers *(m+1)* through *n* could develop through pullout resistance, will produce a system having a factor of safety in excess of Fs(design). Terminating the upper layers *(m+1)* through *n* at points ABC in Figure 5 will decrease the factor of safety. However, since the summation of $t(allowable)_j$ for the outermost log spiral equals or exceeds the required overall value (Step 1 in Figure 5), the resulting safety factor is equal to or slightly larger than Fs(design). Consequently, these upper layers are sufficiently long.

Following a procedure similar to the one detailed by Leshchinsky (1992), lengthen layers 1 through *m* to a test body defined by a log spiral extending between the toe and the crest, deeper than the outermost log spiral (Step 2 in Figure 5). Embed each layer beyond the slip surface so that $t(allowable)_j$ can develop. Fs will increase as a slip surface deeper than the critical one is specified. Truncate layer *m* and check (using the moment equilibrium equation) whether Fs have dropped to the minimum design value. If it has, this layer is sufficiently long (see point D in Figure 5); if it is less than the minimum, lengthen this layer and repeat calculations until a satisfactory length is found. ReSlope repeats this process to determine the required length of layer *(m-1)*, while considering zero contribution from all layers above since they were already truncated. That is, layers above are not effective for deeper slip surfaces. Subsequently, point E is found. Repeating the process for all layers down to layer 1 yields the length (e.g., curve DEFGH in Figure 5) required, assuring that the minimal value of Fs is met or exceeded for all possible log spiral failure surfaces emerging through the toe.

Compound critical surfaces emerging above the toe are also possible. ReSlope verifies that the length of reinforcement will produce safety factors exceeding Fs(design) for potential slip surfaces emerging above the toe. As indicated in Step 3 in Figure 5, the scheme shown in Step 2 is repeated for slip surfaces emerging through the slope face at the location of reinforcement layers. The values of $t_j$ then are taken as determined in Step 2. Subsequently, layers previously truncated will be lengthened, if necessary, to produce a satisfactory safety factor. Note that ReSlope does not consider surfaces emerging to the left of the toe. This implies that competent foundation is assumed. If this is not the case, longer reinforcement might b needed (see discussion in Limitation).

### 3.3.1 Comments: Modes of Failure, Anchorage, and Pullout Interaction Coefficient

At this point it is appropriate to elaborate on terminology used in ReSlope. In its output, the following controlling mode of failure appears next to each layer: Compound Mode of Failure or Tieback Mode of Failure. The first mode implies the full allowable strength of layer j was utilized in analysis to assure resistance to compound failure. In this case the reinforcement force required for tieback stability is smaller than that required for compound failure and therefore, the compound failure is considered critical (i.e., prevails). The second mode indicates that only the tensile force required to assure local stability, as obtained from tieback analysis (assuming the force distribution indeed will correspond to tieback), was needed. Required anchorage length of each layer is then calculated according to the prevailing mode of failure of the respective layer and its associated tensile force. For an adequately designed slope, bottom layer(s) should always correspond to a compound failure. Tieback mode of failure at bottom layer(s) indicates the overall factor of safety for geosynthetic specified by the designer is unattainable for the selected reinforcement and its spacing ('local' rupture may occur). The designer then must either specify a stronger or more closely spaced reinforcement. Alternating modes of failure in ReSlope also indicates inadequate specified strength or spacing of reinforcement.

Anchorage lengths are specified beyond points A, B, C, D, E, F, G, etc. This is slightly conservative since, contrary to the exact procedure of compound analysis, it assures that $t_{allowable}$ can also develop along the envelope D, E, F, G although zero strength is required there. However, since the required anchorage lengths of lower layers are relatively short in realistic problems, this simplification is reasonably conservative. In fact, since pullout resistance depends on overburden pressure that is calculated in an approximated fashion (i.e., the weight of soil column and surcharge above the point of interest, per unit area, is calculated as this pressure), such conservatism is warranted.

Specifying a layout similar to ABCDEFG will contain *m* potential slip surfaces, all having the same minimal safety factor against rotational failure (see Figure 5). However, because of practical considerations, a uniform or linearly varying length of layers is specified by ReSlope (based on Step 4 in Figure 5). As a result, the number of such potential slip surfaces is reduced in the actual structure since most layers are longer, and typically, some are stronger than optimally needed. ReSlope shows all compound slip surfaces.

Finally, anchorage lengths are calculated to resist pullout forces equal to the required allowable strength of each layer multiplied by a factor of safety Fs-po. In these calculations the overburden pressure along the anchored length and the parameter defining the shear strength between soil and reinforcement interface are used. This parameter, $C_i$, termed the interaction coefficient, relates the interface strength to the reinforced soil design strength parameters: $\tan((\phi_d)$ and $c_d$. The interaction coefficient is typically determined from a pullout test. Koerner (1998) gives clear details about the procedure, data reduction and significance of the pullout test. Elias and Christopher (1997) give more details. The required anchorage length of layer j equals to $Le_j = t_j/\{2\sigma_j C_i[\tan((\phi_d)+c_d]\}$ where $\sigma_j$ signifies the average overburden pressure above the anchored length.

### 3.4 Direct Sliding Analysis

Specifying reinforcement layout that satisfies a prescribed Fs against rotational failure does not assure sufficient resistance against direct sliding of the reinforced mass along its interface with the foundation soil or along any reinforcement layer. The length required to yield stable mass, $L_{ds}$, is determined from a LE analysis that satisfies force equilibrium; i.e., the two-part wedge method.

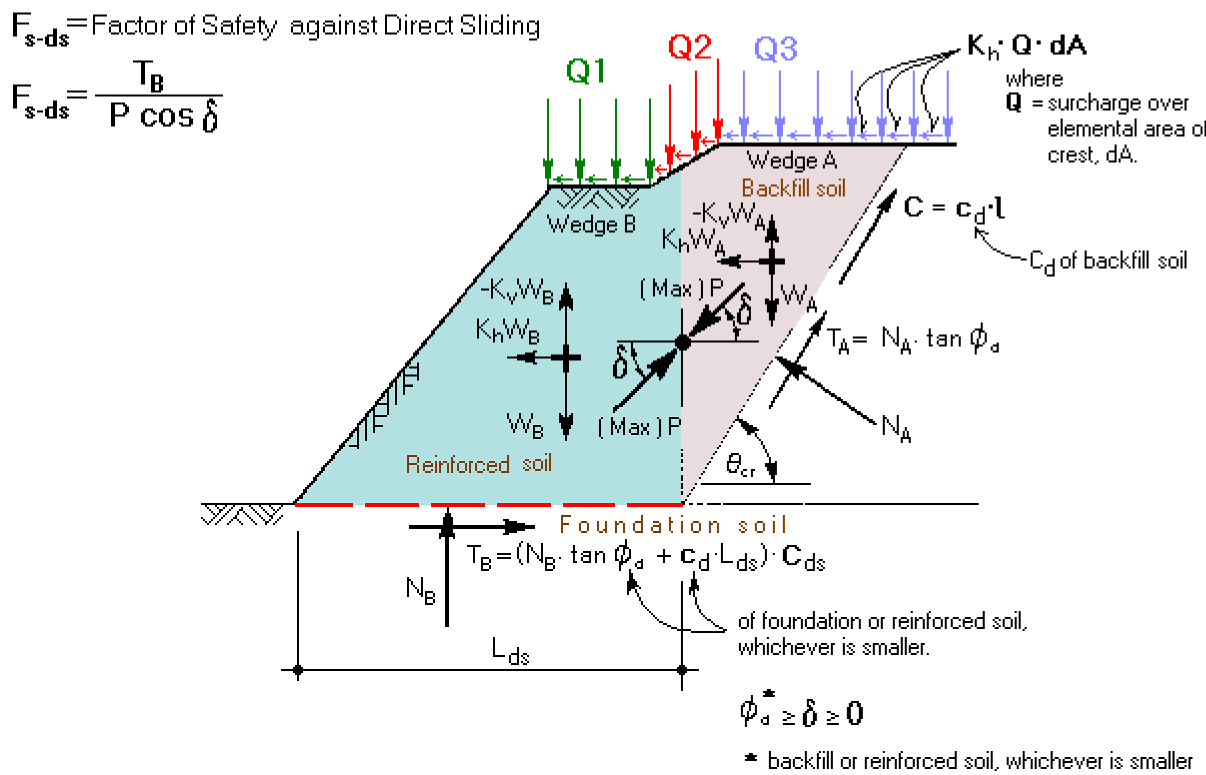



Figure 6. Two-part wedge mechanism for direct sliding analysis

An initial value of $L_{ds}$ is first assumed (see Figure 6). Then, a value for $\delta$, the interwedge force inclination, is chosen: $\delta$ may be specified between zero and $\phi_d$ of the backfill or reinforced soil, whichever is smaller. Subsequently, the maximum value of the interwedge force, P, is found by varying $\theta$ while solving the two force equilibrium equations for the active Wedge A. This interwedge force signifies the resultant of the lateral earth pressure exerted by the backfill soil on the reinforced soil. Next, the vertical force equilibrium equation for Wedge B, which includes the vertical component of the lateral thrust of the active wedge (i.e., $P \cdot \sin\delta$), is solved. After obtaining $N_B$, the sliding resisting force, $T_B$, along the base $L_{ds}$ is calculated.

At this stage, the actual factor of safety against direct sliding of the reinforced mass, Fs-ds, is calculated by comparing the resisting force with the driving force. ReSlope changes $L_{ds}$, repeating the process for Wedge A and Wedge B, until the computed factor of safety against direct sliding equals to the prescribed value. $L_{ds}$ reported by ReSlope correspond to the maximum length obtained from analysis of sliding along the foundation soil (if bottom layer is placed above the foundation) and from analysis of sliding above and below bottom layer.

### 3.4.1 Comments: Coefficient of Direct Sliding, Interwedge Force Inclination, Seismicity, and Factor of Safety

When calculating $T_B$, the coefficient $C_{ds}$ is utilized; it signifies the interaction coefficient between the reinforcement and the soil as determined from a direct shear test. If the bottom layer (i.e., layer 1 in Figure 3) is placed directly over the foundation soil, two values of $C_{ds}$ are needed: one for the interface with the reinforced soil and the other for the interface with the foundation soil. If the bottom layer is above the foundation, ReSlope will ignore the value specified for the interface with the foundation soil; however, it will check stability for direct sliding along the interface between the reinforced and foundation soils, as well as along the reinforcement embedded within the reinforced soil.

The assumed value of $\delta$ may have significant influence on the outcome of the analysis. Selecting $\delta>0$ implies the backfill soil will either settle relative to the reinforced soil or the reinforced soil will slide slightly as a monolithic block thus allowing interwedge friction to develop. Since the effects of reinforcement layers, some of which will typically intersect the interwedge interface, are ignored, selecting a value of $\delta$ between $(2/3)\phi_d$ and $\phi_d$ should be viewed as a conservative choice.

The technique for incorporating seismicity into the force equilibrium analysis is shown in Figure 6. In a pseudo-static approach, however, large seismic coefficient, $C_s$ (or $K_h$ and $K_v$), may produce unrealistically large reinforced soil block, Wedge B. In this case, a permanent displacement type of analysis (i.e., Newmark's slip-stick model) is appropriate. Alternatively, ReSlope allows the user to eliminate inertia from Wedge B, analogous, in a sense, to Mononobe-Okabe model used in analysis of gravity walls. Only the 'dynamic' effects on P are superimposed then on the statics of the problem. Unlike Mononobe-Okabe who used the static $\theta_{cr}$ also for the dynamic case, ReSlope seeks and uses $\theta_{cr}$ producing maximum interwedge force, max(P).

Finally, note that Fs-ds is imposed after reducing the shear strength parameters of the soils by a factor of safety; i.e., using $\phi_d$ and $c_d$. In the context of LE slope stability analysis, this constitutes a 'double taxation.' However, in the analysis of *reinforced* slopes, notions associated with *reinforced* walls are commonly used, including the value of Fs-ds. To be consistent with this practice, ReSlope allows the user to specify Fs-ds. For most slopes, its specified value could range between 1.0 and 1.3.

### 3.5 Deepseated Analysis Using Bishop Method

ReSlope performs conventional unreinforced slope stability analysis, utilizing Bishop method, to assess the minimum factor of safety against deepseated failure. In a sense, this analysis indicates the bearing capacity of the foundation soil.

Circular slip surfaces are examined and the one rendering the lowest factor is selected. The circles examined, however, are restricted to those passing away from the bottom of the reinforced soil zone. The stabilizing effects of intersecting reinforcement layers above the bottom layer with the critical circle are ignored. The user sets the maximum feasible circle penetration. Seismicity is included in the analysis through the coefficient $C_s$ (i.e., through $K_h$ and $K_v$). That is, Bishop's formulation was modified to include pseudo-static forces due to self-weight and surcharge loads.

Deepseated circles tend sometimes to emerge rather steeply. It is well known that in this case, large numerical errors may occur in slope stability methods utilizing slices. ReSlope tests for such potential error through a parameter known as $m_\alpha$. If $m_\alpha<0.1$ for a slice, the slide resistance of this *slice* is set to *zero* thus avoiding potentially large numerical errors.

### 3.5.1 Comments: Deepseated Analysis

Circles describing overhanging 'cliffs' are excluded from consideration. In case of a backslope, a tension crack is introduced between the crest and the elevation of the top of the slope.

ReSlope does not adjust automatically the length of bottom reinforcement layers to meet a certain factor of safety against deepseated failure. In case this factor is less than an acceptable minimum, the user can use the following procedure. Set larger than needed safety factors for direct sliding. This will result in longer reinforcement length and subsequently, larger factor of safety against deepseated failure; i.e., it will 'push' the critical circle away thereby increasing the associated safety factor. Repeat until a satisfactory factor is attained. Before significant lengthening of the reinforcement, however, it is worthwhile to check whether Bishop analysis for the particular problem does not produce overly conservative results. This check can be done using one of the available rigorous slope stability methods (e.g., Spencer's, Janbu's or Morgenstern-Price's). To avoid over-conservatism, stabilizing effects of reinforcement layers intersecting the slip surface should then be included in the analysis.

## 3.6 Limitations of Analyses

Though the analyses in ReSlope follow a rational scheme in the context of design, the following limitations should be highlighted:

**a.** In the compound failure analysis, only log spirals emerging at or above the toe were considered. That is, log spirals emerging away from the toe, signifying deepseated failures that activate the reinforcement were excluded. Toe and above toe potential slip surfaces are typically most critical in steep slopes, especially when the foundation soil is competent. An indication regarding the competency of the foundation is provided in ReSlope by the Bishop deepseated analysis. Furthermore, since the trace of the outermost compound slip surface may be displayed by ReSlope, one can render a judgment whether deepseated failure through the reinforcement is likely to occur. That is, if this surface is deep, penetrating the foundation and yet emerging through the toe, then the critical compound slip surface is likely to be deeper than the one predicted by ReSlope. In this case, a more generalized analysis such as ReSSA (ADAMA Engineering, Inc., 33 The Horseshoe, Newark, DE 19711, 302/368-3197, adama@msew.com). However, such deepseated failures may require extremely strong and long reinforcement rendering, perhaps, an uneconomical reinforced slope.

**b.** The phreatic surface (see Figure 1) can be estimated from a flow net. However, ReSlope utilizes it as a piezometric line with zero head to assess the porewater pressure distribution. That is, the depth of a point relative to this line is used to calculate the pressure. In the strict sense of flow nets, equipotential lines are used to calculate the pressure distribution. Using the phreatic surface as a piezometric surface yields more conservative results (i.e., the calculated pressures are somewhat larger than those predicted by a flow net especially if the surface has steep downward slope, typically near the toe). It should be added that if piezometric data is available, one could establish the location of the surface termed 'phreatic' in ReSlope in a straightforward manner. Finally, as is the case in most stability analysis computer programs, seepage forces are assumed to be negligible.

**c.** The possibility of surficial failure is not assessed by ReSlope. If the reinforcement is wrapped-around at the face of the slope, this type of failure is not likely to occur (provided the backfolded geosynthetic sheet is re-embedded sufficiently deep, usually at least 1 meter away from the slope face). If it is not backfolded (as is typical for slopes inclined at less than 45 degrees), secondary reinforcement and proper erosion control measures (including vegetation) should be used to minimize the risk of surficial failure.

**d.** In the strict sense of analysis, the log spiral slip surface is valid for homogenous soil only. However, in the compound failure analysis (Figure 5), this surface passes through both reinforced and backfill soils and possibly, even through the foundation soil. ReSlope is using a weighed average technique, considering the compound failure lengths in the reinforced soil and in the backfill soil, to find equivalent values for $\phi_d$ and $c_d$ to be used in analysis. The value of the equivalent $\phi_d$ is used to define the trace of the log spiral passing through the reinforced and backfill soils. The weighed average is such that typically the results will be somewhat on the conservative side.

**e.** For low strength of backfills, a segment of the outermost compound failure may pass through the foundation soil. The strength of soil used in analysis then is approximated as described in item d. That is, the strength of the foundation is not considered in the averaging. However, if the foundation soil is relatively strong, such penetration is unlikely. ReSlope allows the user to limit the extent of critical slip surface to just being tangent, at most, to the foundation. The end result then is much shorter length of reinforcement as dictated by compound analysis. The user should use judgment when invoking this option. If the foundation soil is quite soft, deep compound failures are feasible (see discussion in item a).

**f.** Figure 1 shows three different intensities of surcharge loads: Q1, Q2, and Q3. However, the predicted reinforcement force obtained from the tieback analysis will theoretically be more accurate as the loads above the trace of the outermost tieback surface (Figure 4) approach uniformity. The reason for a potential inaccuracy when the loads are grossly nonuniform can be realized using the scheme in Figure 3. Each layer counterbalances a distinctive 'slice' of soil. The slice may be subjected to surcharge load. Consequently, a single reinforcement layer solely counterbalances each portion of surcharge over a particular slice. If this surcharge is quite concentrated (i.e., distributed over a few slices), only a few reinforcement layers will react to this surcharge. However, since soil medium tends to distribute and diminish such surcharge loads with depth, these few layers will actually be subjected to lower forces than predicted by ReSlope while layers above and below (i.e., outside the tributary area defined by the surcharged slices) will carry higher loads than those predicted by the tieback analysis. That is, concentrated loads may lead simultaneously to both conservative and unconservative predictions regarding reactive forces in reinforcement layers. In the rare occasion when a problem involving high intensity Q2 or Q3 over the outermost tieback surface is analyzed, use the following approximating procedure. Run ReSlope twice. First run it without surcharge to obtain baseline results for the reactive force in the reinforcement layers, and then run it with the actual surcharge to obtain the required length of layers. Use an available approximate solution to estimate the lateral earth pressure against each tributary area (Figure 3) due to the concentrated surcharge. Calculate the resultant force over each tributary area resulting from this lateral pressure. Add each resultant force (i.e., superimpose) to the existing force in each respective layer as calculated in the first run (i.e., the surcharge-free run). The overall factor of safety for geosynthetic, for the tieback mode of failure, can be calculated now for each layer. A safe layout, including adequate resistance to compound failure and direct sliding, has been obtained from the second run. It is likely that in slopes less than about 60 degrees the alternative more 'accurate' procedure has negligible effects on the results.

## 4.0 DESIGN CONSIDERATIONS

### 4.1 General

The analyses in ReSlope are all based on a limiting equilibrium state. Such a state deals, by definition, with a structure that is at the verge of failure. Adequate safety factors included in the analyses ensure acceptable margins of safety against the various failure mechanisms analyzed. In the LE analysis it is implicitly assumed that the different materials involved (i.e., the geosynthetics and soils) will all contribute their full design strengths simultaneously. For materials having a constant plastic shear strength after some deformation, such an assumption is realistic. However, the materials in the reinforced soil system do not possess this idealized plasticity. Consequently, the following guide is recommended when specifying material properties for ReSlope analysis.

### 4.2 Soil: Shear Strength and Factor of Safety

Slip surface development in soil is a progressive phenomenon, especially in reinforced soil where reinforcement layers delay the formation of a surface in their vicinity. That is, a slip surface does not develop at the same instant along its full length and thus the peak shear strength of the compacted soil is not being mobilized simultaneously as assumed in the LE analysis. Consequently, it is recommended that the design values of $\phi$ and c will not exceed the residual strength of the soil. This will assure that at the LE state, the shear strength utilized in each analysis is indeed attainable all along the slip surface.

The value of the shear strength parameters reported by laboratories typically corresponds to peak shear strength. In this case, a minimum factor of safety of Fs=1.3 practically assures that the design strength parameters will be at or below their residual values [i.e., $\phi_d = \tan^{-1}(\tan\phi_{peak})/Fs$ and $c_d = c_{peak}/Fs$]. It is recognized that by using the residual values, the gain in soil strength due to compaction is basically ignored in the analysis and thus has an overall conservative impact on the reinforced slope system. However, the complex issue of progressive failure is then avoided while assuring results 'on the safe side.' Use of residual strength in analysis should not undermine the importance of compaction for structural performance.

There are cases in which the soil will not exhibit a peak strength behavior. If the soil is lacking peak strength characteristics or the reported shear strength corresponds to a residual value, a factor of safety of Fs=1.0 can then be used. Note that for residual shear strength parameters, a value of Fs=1.0 is typically specified in design of critical structures such as geosynthetic reinforced walls. Though such a value seems to be low, recall that the stability of a steep slope is hinging on the tensile strength of the reinforcement; that is, without reinforcement a slide will occur. The soil just contributes its shear resistance to slide.

### 4.2.1 Cohesion and Factor of Safety

If cohesive fill is used, extreme care should be used when specifying the cohesion value. Cohesion has significant effects on stability and thus the required reinforcement strength. In fact, a small value of cohesion will indicate that no reinforcement at all is needed at the upper portion of the slope. However, over the long-run cohesion of manmade embankments tends to drop and nearly diminish. Since long- term stability of reinforced steep slopes is of major concern, it is perhaps wise to ignore the cohesion altogether. It is therefore recommended to limit the design value of cohesion to 100 psf (5 kPa). It should be pointed out, however, that end-of-construction analysis must be also conducted if a soft foundation is present. In this case stability against deepseated failure must be assured.

## 4.3 Reduction and Safety Factors Related to Geosynthetics

Limit equilibrium analysis assumes that reinforcement and soil reach their design strengths at the same instant, regardless of deformation characteristics. Though use of residual strength will insure availability of the soil shear resistance at all deformation levels, this may not be the case with the reinforcement. For example, if the reinforcement is very stiff relative to the soil, its strength will be mobilized rapidly, potentially reaching its design strength value before the soil reaches its strength. This may lead to overstressing and subsequently, premature rupture of the reinforcement, violating the premise that its tensile resistance will be available with the soil strength. The result might be local, or even global, collapse. However, since geosynthetics are ductile (typically, rupture strain greater than 15%), large strains may develop locally in response to overstressing thus allowing the soil to deform and mobilize its strength as assumed in the analysis and as needed for stability.

To assure that indeed some overstressing of the reinforcement without breakage is possible, an overall factor of safety for uncertainties is specified in ReSlope. This factor multiplies the calculated minimal required reinforcement strength at each level. Typical values for this factor range from Fs-u=1.3 to 1.5. The strength of the factored reinforcement should be available throughout the design life of the structure. To achieve this, reduction factors for installation damage ($RF_{id}$), durability ($RF_d$), and creep ($RF_c$) should be applied so that geosynthetics possessing adequate ultimate strength, $t_{ult}$, could be selected. That is, the specified geosynthetic should have the following ultimate strength:

$$t_{ult}=t_{required}\cdot(Fs\text{-}u)\cdot(RF_{id})\cdot(RF_c)\cdot(RF_d)$$

Elias and Christopher (1997) give preliminary values for reduction factors for geosynthetics:

| **Polymer Type** | $\mathbf{RF_{id}}$ | $\mathbf{RF_d}$ | $\mathbf{RF_c}$ |
|---|---|---|---|
| Polyester | 1.05 to 3.0 | 1.1 to 2.0 | 2.0 to 2.5 |
| Polypropylene | 1.05 to 3.0 | 1.1 to 2.0 | 4.0 to 5.0 |
| Polyethylene | 1.05 to 3.0 | 1.1 to 2.0 | 2.5 to 5.0 |
| Typical* | 1.05 to 1.5 | 1.05 to 1.5 | 1.5 to 3.0 |

* Elias and Christopher (1997) do not give values appearing in the 'typical' row. Leshchinsky provides this preliminary information to acquaint the novice user with typical range of values specified for inert environment and well-controlled construction.

Note that for normal soil conditions (i.e., mild pH and no biological activity) in steep slopes, degradation should not be a problem when using a typical reinforcing polymeric material. The values of $RF_{id}$ and $RF_d$ are site specific. The creep reduction factor, $RF_c$, depends, to a large extent, on the polymer type and the manufacturing process.

Documented testing on geosynthetics, to be provided by the manufacturer or supplier, will likely result in recommended reduction factors falling within the range suggested as 'typical' shown in the table above. When actual test documentation is not available, however, the following conservative default values are recommended (Berg, 1992):

$RF_{id} = 3.0$ $RF_c = 5.0$ $RF_d = 2.6$

The following provisions apply to these default values:

1. A creep default value may be used only for preliminary design; actual test data is required for final design.
2. Durability default value should not be used for these soils: acid sulfate soil, organic soil, salt affected soil, ferruginous soil, calcareous soil, and modified soils (e.g., soils subjected to deicing salts, and cement stabilized or lime stabilized soils); actual test data should be used for final design.

Documented test data on creep test results should comply with ASTM D5262-92 test procedure. The term ultimate strength, $t_{ult}$, should correspond to the result obtained from the wide-width tensile test, following ASTM D4595-86 procedure. Note that the selected geosynthetic should be installed so that its ultimate strength is available in the potential slide direction (i.e., geosynthetics usually posses different strengths along their principal axes). Typically, the strength at 5% elongation strain in the wide-width test is reported as well. Some designers concerned with performance prefer to use this value as '$t_{ult}$.' In this case, the factor of safety for uncertainties can be reduced to Fs-u=1.1 to 1.3 since the actual strength is significantly larger. It should be noted, though, that performance (i.e., deformations) of steep slopes is less critical than that of walls and therefore, the 5% 'limit' is unnecessary for most practical purposes.

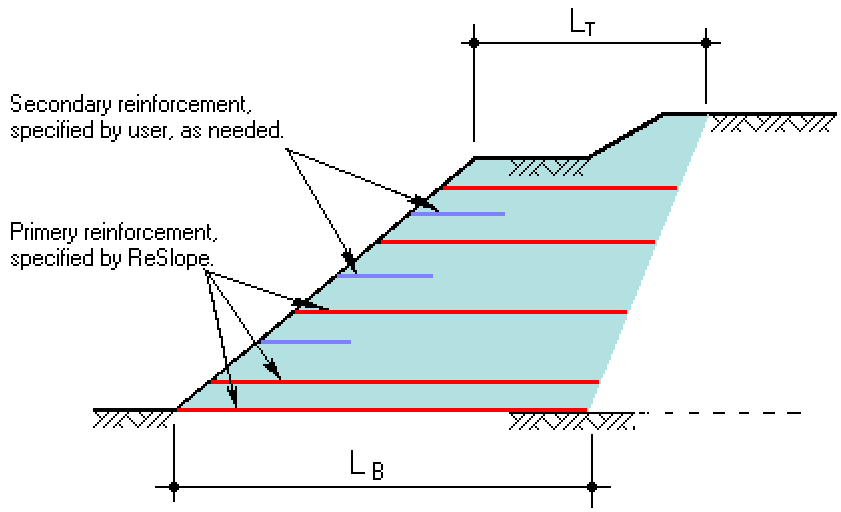
To make the design process more efficient, ReSlope allows the user to specify the ultimate strength of each reinforcement layer. For the selected spacing and strengths, ReSlope reports whether the resulted minimum factor of safety for uncertainties is satisfactory. To be practical, the user should input a realistic value of ultimate strength. A convenient source for such values is available in the Specifier's Guide, published annually in the Geotechnical Fabrics Report by the Industrial Fabrics Association International, 1801 County Road BW, Roseville, Minnesota 55113, Tel. (612) 222-2508. This publication also includes the addresses of manufacturers. The user can then verify further data related to recommended (and documented) reduction factors corresponding to a product.

Finally, if seismicity is considered in the design, the reduction factor for creep at the seismic event can be set to one. Simply, since the duration of the superimposed pseudo-static seismic load is short, significant creep is not an issue. However, the user should run ReSlope again, this time with no seismicity, to verify that the required seismic strength is no less than the required value for static stability where the creep reduction factor is fully specified; the larger strength value from static and seismic runs should be specified. Under seismic conditions, smaller safety factors than those specified for static conditions may be tolerable (see Elias and Christopher, 1997).

### 4.4 Other Specified Safety Factors

ReSlope requires as input data the factor of safety against direct sliding, Fs-ds. This safety factor assures that the force tending to cause direct sliding of the reinforced soil block is adequately smaller than the force available to resist it. It is a straightforward adaptation of analysis from reinforced retaining walls or gravity walls. However, in slope stability analysis, unlike walls, the shear strength parameters of the soil are reduced by Fs. It is recommended to use Fs-ds=1.2 if the soil safety factor, Fs, is 1.3 or less. For large specified values of Fs (i.e., values rendering shear strengths less than the residual strengths), the values for Fs-ds may range from 1.0 to 1.3.

With reference to direct sliding, note the coefficient $C_{ds}$. There are two direct sliding coefficients. The first signifies the ratio of shear strength of the interface between the reinforcement and reinforced soil and the shear strength of the reinforced soil alone. The second coefficient signifies a similar ratio but with respect to the strength of the foundation soil. This coefficient reflects a mechanism in which soil slides over the reinforcement sheet. Its value can be determined by using direct shear tests in which the shear strength of the interface between the relevant type of soil and the reinforcement is assessed under various normal loads. The test procedure is described in ASTM D5321. To avoid the dilemma of the development of progressive failure, it is once again recommended that one use the residual strength values for both interface strength and soil strength when calculating their ratio $C_{ds}$. Typically, $C_{ds}$ will vary between 0.5 and 1.0, depending on the type of soil and reinforcement. For granular soils and common geosynthetics used in reinforcement, $C_{ds}$ is about 0.8. Note that in many cases, the required length of bottom layer (i.e., see $L_B$ in Figure 7) may increase significantly as $C_{ds}$ decreases below 0.8.




**ReSlope Options :**

(1) $L = L_T = L_B =$ longest length required for tieback analysis, compound failure analysis, and direct sliding analysis.

(2) $L_B =$ same as L in (1).

$L_T =$ longest length required for tieback analysis and compound failure analysis.

Figure 7. Reinforcement length specified by ReSlope

The user specified factor of safety against pullout, Fs-po, should multiply the calculated required allowable tensile force of each reinforcement layer. Anchorage length then is calculated to provide pullout resistance up to this increased tensile force. Typically, Fs-po value is specified as 1.5. Under seismic conditions this value should be increased by 20%.

Similar to $C_{ds}$, ReSlope requires the value of $C_i$, the interaction coefficient. It relates the strength of the interface between the reinforcement and soil to the shear strength of the reinforced soil or foundation soil. This coefficient reflects a mechanism in which the reinforcement is being pulled out from a confining stable soil. The required anchorage length is calculated based on $C_i$. The value $C_i$ is normally determined from a pullout test; for test details refer to Koerner (1998) or Elias and Christopher (1997). Typically, the value of $C_i$ varies between 0.5 and 1.0, depending on the type of soil and reinforcement. For granular soils, the typical value of $C_i$ is about 0.7. It should be pointed out that anchorage length for reasonably spaced (e.g., 30 to 60 cm vertical spacing) continuous reinforcing sheets, the typical anchorage length is quite small relative to the total required length in the final layout. The user can easily conduct a parametric study for a particular problem using ReSlope to verify whether a sophisticated procedure to determine accurately $C_i$ is indeed worthwhile.

### 4.5 Specified Layout of Reinforcement

Two practical options for specifying reinforcement length are available in ReSlope (see Figure 7). The first option simplifies construction by specifying all layers to have a uniform length. This length is selected as the longest value obtained from the tieback analysis, the compound failure analysis, or the direct sliding analysis.

The second safe option is to specify $L_B$ and $L_T$ at the bottom and top, respectively, where $L_B$ is the longest length from all analyses and $L_T$ is the longest length obtained from compound and tieback analyses. Length of layers in between is linearly interpolated. This specification is more economical; however, it may result in misplaced layers at the construction site. ReSlope allows the designer to select uniform (option 1) or nonuniform (option 2) lengths.

For instructive purposes, ReSlope allows the user to specify the required minimum length of each layer satisfying all factors of safety. Though such layout should not be used for construction, it provides the designer with a sense of the amount of 'wasted' reinforcement when specifying uniform or linearly varying length.

Figure 7 shows primary and secondary reinforcing layers. In the stability analyses, only primary layers are considered. However, layers spaced too far apart may promote localized instability along the slope face. Therefore, secondary reinforcement layers should be used. Their width should extend at least 1 meter back into the fill and their strength, for practical purposes, may be the same as the adjacent primary reinforcement. The vertical spacing of a secondary reinforcement layer from either another secondary layer or from a primary one should be limited to 30 cm. Secondary reinforcement creates a 'coherent' mass at the slope face, a factor important for local stability. Furthermore, it allows for better compaction of the soil at the face of the steep slope. This, in turn, increases the sloughing resistance and prevents surficial failures. If wrap-around is specified, secondary reinforcement can be used to wrap the slope face as well. It should be backfolded then at least 1 meter back into soil, same as the wrapping primary reinforcement.

### 4.6 Erosion Control

Erosive forces can cause surface sloughing, especially when steep slopes are considered. Consequently, measures to minimize erosion damage must be part of the design process of a reinforced slope system.

The most common method to reduce erosion due to surface water runoff is through use of vegetation. However, establishment and maintenance of vegetative cover over steep slopes can be difficult (Berg, 1992). For example, the steep grades limits the amount of water absorbed by the soil before runoff occurs and thus make it more difficult for germination and establishment of roots. Furthermore, established vegetation must be maintained over the entire slope throughout time.

An effective way to control erosion is to use synthetic mats or blankets. To be considered 'permanent', the mat should be stabilized against ultra-violet radiation and be inert to naturally occurring soil-born chemicals and bacteria. As pointed out by Berg (1992), the erosion control mat serves three functions:

1. Protects the bare soil face against erosion until vegetation is established.
2. Reduces runoff velocity for increased water absorption by the soil thus promoting long-term survival of the vegetative cover.
3. Reinforces the root system of the vegetative cover.

Note that maintenance of vegetation (e.g., re-seeding, mowing, etc.) may be required and therefore, should be considered in design when specifying the slope angle.

For slopes that are less than 45 degrees, low height slopes, and/or moderate runoff, a permanent synthetic mat may not be required (Berg, 1992). A degradable erosion blanket may be specified to promote growth until vegetative cover is firmly established. Such a blanket will typically lose its integrity after about one year.

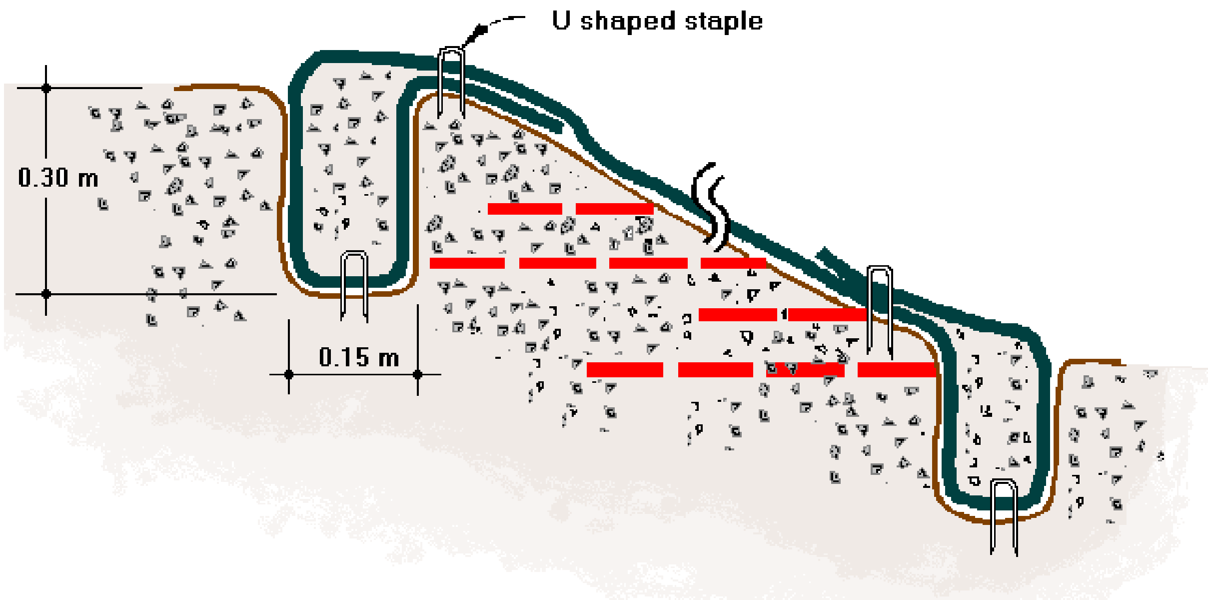



Figure 8. Erosion control mat embedded at upstream and downstream ends

Most manufacturers' literature provides detailed installation guidelines for erosion blankets and mats. As a rule, mats/blankets should be placed over a smooth and compacted grade that is covered by a few inches of topsoil. Anchor trenches should secure the mat/blanket at the upstream and downstream ends; these trenches should be at least 30 cm deep and 15 cm wide (Figure 8). Note that U-shaped ground staples are used in Figure 8 to fasten the blanket to the surface. If the slope is longer than approximately 10 meters, the blanket/mat should be secured by embedding it in slots, maximum 10 meters apart, 15 cm deep and 15 cm wide (Figure 9). Manufacturers according to their product properties and experience give details regarding overlapping, edge anchor, staple patterns, and seeding. Note that prices vary widely. The designer should verify product suitability for a specific project. Upon selection of a product, the designer should specify the layout and installation details.

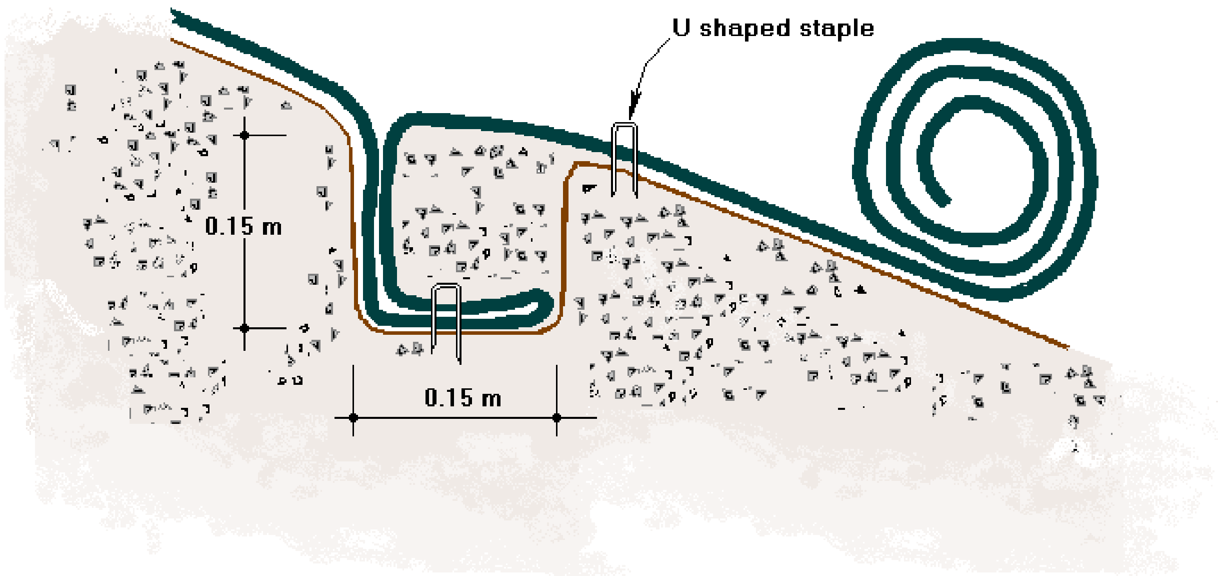



Figure 9.  Erosion control mat secured at intermittent intervals

## 4.7 Tension Cracks

When cohesive soil is used for steep slopes (e.g., levees), tension cracks are likely to develop at the crest.  This likelihood increases when the soil is compacted above its optimal moisture content, as is the typical case in levee construction.

Using Mohr-Coulomb's failure criterion, it can be shown for $\phi$ =0 that the depth to which tensile normal stresses extend, $Z_c$, approximately equals $2c/\gamma$ where c = cohesion and $\gamma$ = moist unit weight of soil.

To reduce the possibility of a tensile crack development, several techniques can be used.  Placing a granular soil cover, $Z_c$ thick, over the crest will provide sufficient overburden pressure to eliminate tensile stresses within the clayey soil.  The granular cover should be considered as a surcharge load, $Q=\gamma Z_c$, in the stability analysis and design.  A more practical solution would be to install geogrid layers, spaced at 15 cm intervals, within the tensile stress zone $Z_c$.  These grid layers should be placed along the entire crest width.  The minimum allowable strength of these grids should exceed $t_{allowable}>c \cdot Z_c/n$ where *n* = number of grid layers within $Z_c$.  If this strength is less than that required for the primary reinforcement layers, it will be less confusing at the construction site to use the same strength as the primary layers.  Such use of geosynthetics will arrest the development of cracks.  The end result will be tension cracks with negligible depth.

## 4.8 Slope Repair

Reinforced soil can be used effectively to repair failed slopes. To lower the cost of repair, minimum excavation into the remaining stable portion of the slope is desired; i.e., the collapsed material is removed and a minimal cut into the undamaged slope is conducted so that the exposed slope is sufficiently stable during the repair (Figure 10). Such a process implies that the length of bottom reinforcement layers is restricted in length. However, ReSlope provides unrestricted length of grids as obtained from analysis. To make use of ReSlope for restricted reinforcement length, follow this procedure:

a. Specify reinforcement layers at lower elevations (see Figure 11). Run ReSlope and verify that the calculated length is adequate. If length is too long, lower the elevation of specified reinforcement. Conversely, if unacceptably short, run with higher specified elevations.
b. Run ReSlope again, this time for a slope H1 high (see Figure 11). In this run, the reinforcement required to assure stability above point A (Figure 11) will be determined.
c. Specify final layout based on maximum required lengths as obtained from a and b. Use the layout option as shown in Figure 7.

Note that this procedure utilizes only the lower layers to stabilize the full height of the slope. The layers above A provide just local stability to the upper portion of the slope. The end result is shorter reinforcement length. The trade-off is higher required strength of bottom layers.

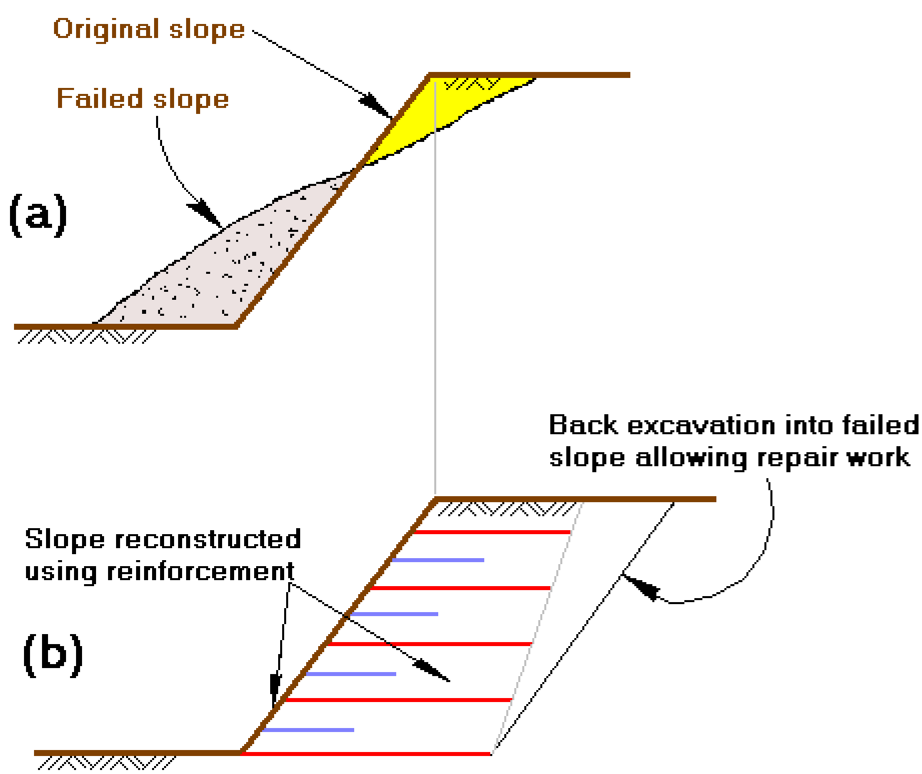



Figure 10. Slope repair: a) Failed section, and b) Reconstructed slope (may be deeper than, or same as, original grade)

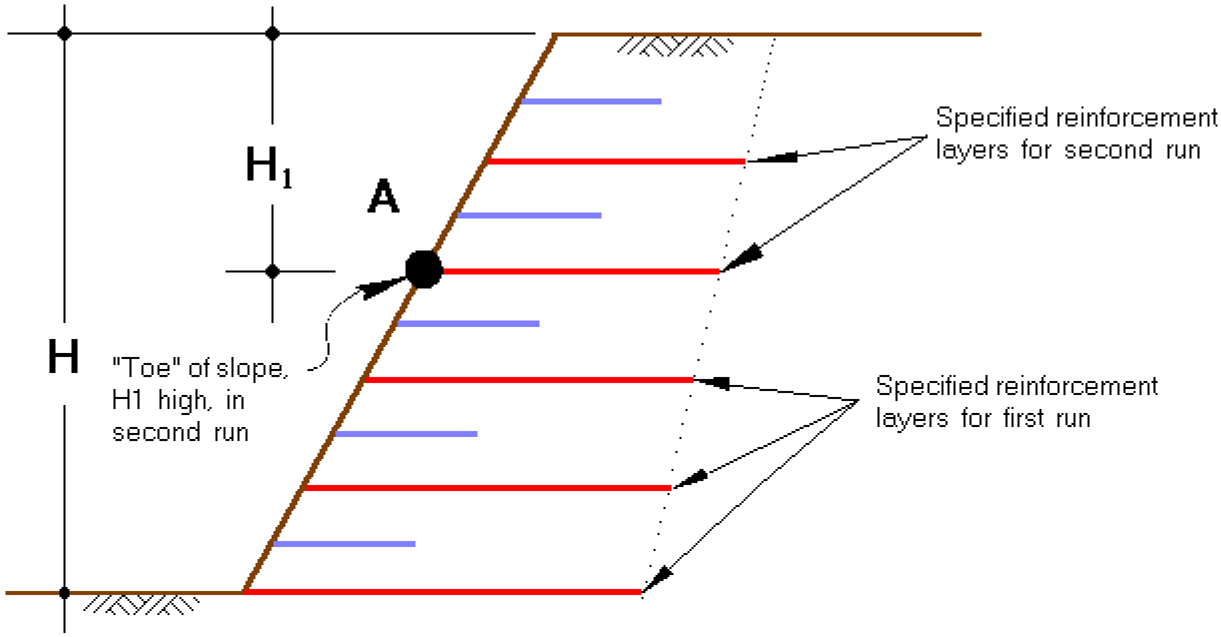



Figure 11. Procedure for running ReSlope when length of reinforcement is restricted

### 4.9 Construction of Slope Face

Since the reinforced structure cost will depend also on the construction procedure near the slope face, it is important to consider this factor in the design phase; i.e., when selecting the slope angle. Depending on soil properties (mainly cohesion), the reinforcement spacing and the slope inclination, temporary support near the slope face may be needed to make the construction of steep slopes feasible. That is, adequate compaction near the face, without using some type of facing to support the constructed layer, may be impossible for steep slopes. A typical removable support is shown in Figure 12. An L-shaped bracing supports a wooden board, 5 by 30-cm. The base of this bracing is a metal flange, 10 cm wide, 6 mm thick, and 60 to 100 cm long. A metal pipe, 30 cm high, is welded to this flange about 3-5 cm from its end. L-shaped bracing is placed on top of the last completed layer approximately every 1 meter. A small mound of soil can be placed on each bracing to secure its position. After placing the wooden board adjacent to the metal pipe (Figure 12), the geosynthetic sheet is placed over it. Then, the reinforced soil can be placed, evenly spread and compacted to the desired density. If a wrapped-faced slope is constructed (Figure 12), the overhanging (unburied) sheet should be folded back and anchored into the reinforced soil. Now, the supporting board can be taken out and the bracing pulled out for reuse in the construction of the next layer. It is quite possible that manually operated compaction equipment should be used up to a distance of 1 to 2 meters from the facing. In any event, no construction equipment should be allowed directly on the geosynthetic.

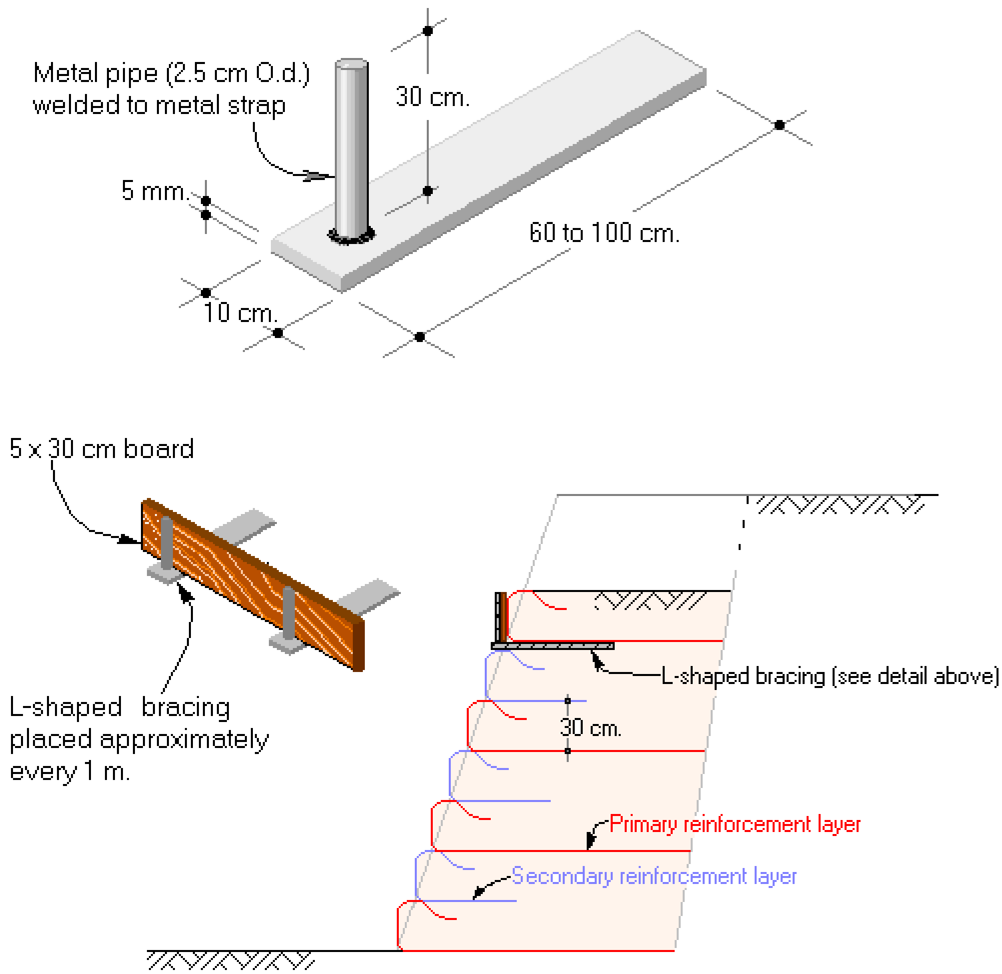



Figure 12. Removable facing support (needed for very steep slopes)

The same procedure can be also used when no wrap-around face is used; i.e., when the reinforcement terminates at the face. Also, for very steep slopes, left in-place welded wire mesh forms (i.e., facings) may be more economical. Information about other types of permanent or temporary facings can be obtained from geosynthetic manufacturers.

## 5.0 CONCLUSION

A method for the design of steep slopes reinforced with geosynthetic materials has been presented. The analyses involved in the design process are based on limit equilibrium. These analyses leads to reinforced mass that is internally and externally stable. To make the application of these analyses practically possible, a computer program ReSlope was developed. Program ReSlope allows the user to optimize the layout of the reinforcement layers by accounting for elements such as user-specified reduction and safety factors, selected ultimate strength of geosynthetic, cohesive soil, porewater pressure, external loads and seismicity.

In addition to description of the analyses conducted by ReSlope, this document also provides recommendations regarding the selection of soil shear strength parameters and safety factors. Recognizing the limitations of limit equilibrium analysis, especially when applied to slopes comprised of materials posing different properties (i.e., soil and polymeric materials), it is recommended that the soil shear strength parameters should correspond to residual strength. It is also recommended to limit the value of cohesion used in the design of reinforced slopes.

This document presents briefly the design aspects related to erosion control of steep slopes. Also, a schematic procedure for the construction of reinforced steep slopes is illustrated. Finally, tips regarding arrest of tension cracks and an economical procedure for repairing a failed slop are given.

Program ReSlope combined with this document produces an efficient design tool for steep slopes reinforced with geosynthetic layers. Only qualified engineers, who are familiar with slope stability analysis and soil reinforcing, should use this tool.

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## Appendix A: Superimposed Reinforced Slopes

The suggested APPROXIMATED procedure is for preliminary purposes only. As an example, assume a problem with two superimposed steep slopes with a setback D as shown in Figure A1. Use the following steps:

1. Run ReSlope for the 'equivalent' problem shown in Figure A2. If all outermost compound and tieback slip surfaces intersect the setback segment D, the strength and layout of reinforcement can be designed as if there are two independent slopes. Note that the upper slope may affect the length of reinforcement needed to resist direct sliding at the bottom slope. This, however, has no effect on the strength or spacing required for the lower slope. Running ReSlope for the equivalent problem should capture the length for direct sliding due to the upper slope. Run ReSlope again, this time only for the geometry of the upper slope and crest (Figure A3), to determine the required strength and layout of reinforcement. The lower and upper slopes may each have different length of reinforcement.

2. If setback D renders the lower reinforced slope dependent on the upper one (see Step 1), use ReSlope to find the required layout and strength of reinforcement for the upper slope. That is, run ReSlope with the geometry of Steep Slope 2 including its crest and surcharge (Figure A3). Next, run ReSlope for the lower slope specifying the 'equivalent' geometry as shown in Figure A4. This run should use the 'manual' option in ReSlope; introduce an equivalent geosynthetic layer near the crest having the strength equal to the summation of all layers in the upper slope. This equivalent geosynthetic layer will approximately represent the upper slope reinforcement in compound stability computations. The pullout length of this layer can be ignored since it has been accounted for already in the upper slope.

Note that the basic slope geometry used in ReSlope allows for other methods of approximation. However, such approximations are based on judgment rather than direct calculations. Hence, such an approach should be used for preliminary purposes only.

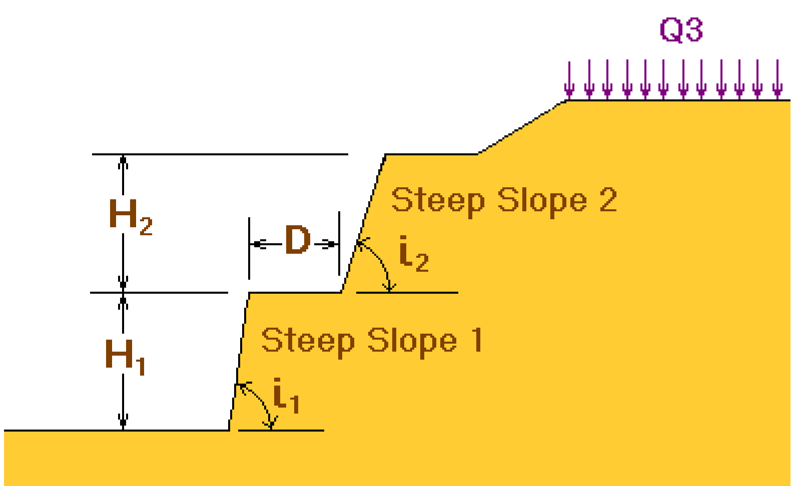



Figure A1. Example of two superimposed steep slopes.

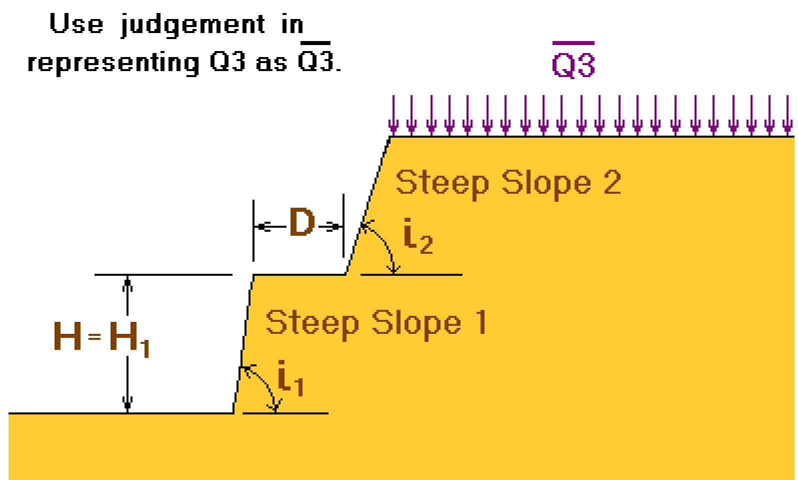



Figure A2. First ‘equivalent’ problem.

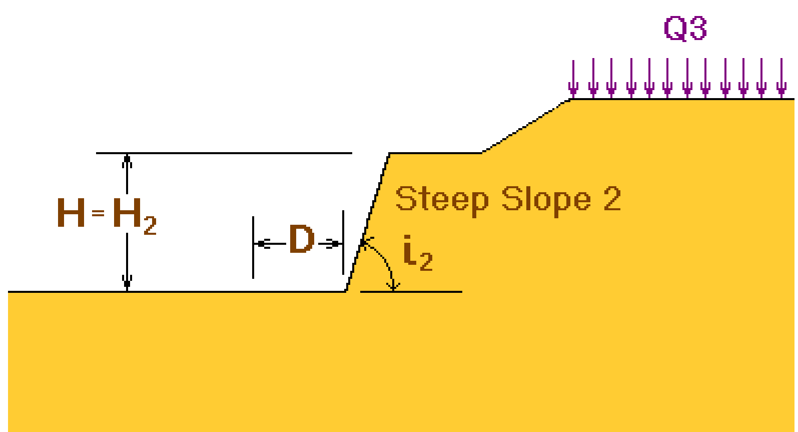
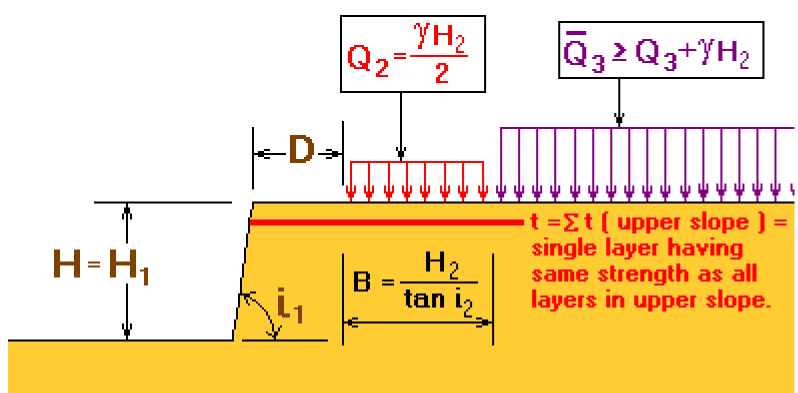



Figure A3. Geometry of upper slope.

Use judgement in specifying $\overline{Q}_3$ and $Q_2$. Note that $Q_2$ represents average uniform pressure under the slope face.




( Note: $\beta$ can be as negligibly small value as, say, 0.1° ).

Figure A4. Equivalent geometry of lower slope.