MIRAFI®

Geosynthetics for soil reinforcement

Ten Cate Nicolon

reinforced soil slopes

Steepened slopes have become increasingly advantageous due to the desire to increase land usage and decrease site development costs. The proven concept of tensile reinforcement allows construction of slopes with far steeper face angles than are permitted by the soils natural angle of repose. Steepened slopes reinforced with **Mirafi®** geosynthetics can increase land usage substantially while providing a natural appearance.

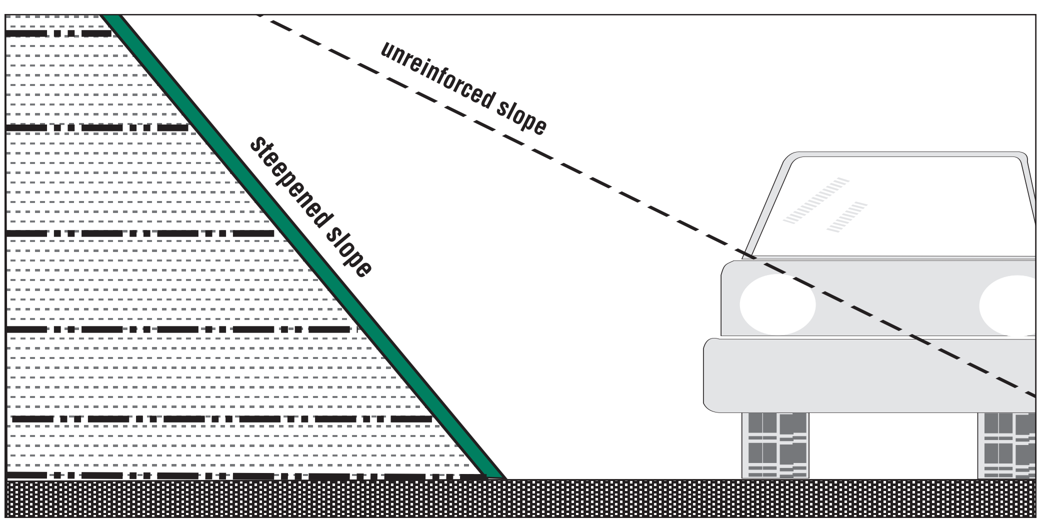
The stability of a reinforced soil slope can be threatened by erosion due to surface water runoff, or more severe forces associated with water currents and wave attack. Slope face erosion may create rills and gullies, and result in surface sloughing and possibly deep-seated failure (Berg.1993). Erosion control and re-vegetation measures must, therefore, be an integral part of all reinforced soil slope system designs. The type of erosion control facing option selected depends on the finished slope face angle.

ADVANTAGES

- **Economics**: significantly lowers site development costs by providing soil retaining solutions without the costs of retaining wall facade materials
- **Usability**: drastically increases the amount of usable land within a given parcel without the cost of a traditional retaining wall
- **Aesthetics**: allows incorporation of 'green' surface
- **Efficiency**: speeds development and construction of site
- **Reliability**: proven design methodologies lead to successful implementation of steepened slopes.

APPLICATIONS

- Highway embankments
- Dikes and Levees
- Landslide repair
- Residential developments
- Commercial/ Office parks
- Landfills



erosion protection options

Soft Armor (Slope Face Angle less than 45°)

Reinforced soil slopes with face angles less than 45 degrees are typically protected with soft armor systems. The function of a soft armor system is to facilitate vegetation growth that provides long term erosion protection to the slope face. A soft armor system consists of a temporary or permanent erosion blanket, a cellular confinement system, or other type of erosion control device along with natural vegetation. Two common soft armor facing options are shown below.

Rolled-Erosion Control Product (RECP)

Cellular Confinement Stabilization

Hard Armor (Slope face angle greater than 45°)

When slope face angles increase to greater than 45 degrees, a more durable facing system is required. Reinforced soil slopes with face angles greater than 45 degrees are typically protected with hard armor systems. The function of a hard armor system is to provide long term erosion protection to the slope face. A hard armor system may also use vegetation as a means of erosion control. A hard armor system consists of a welded wire mesh or basket, stacked cellular confinement panels, an open face SRW unit, or other type of protection device. These systems may also use natural vegetation as a means of protection. Four common hard armor facing options are shown below.

Welded Wire Basket Facing

Open Face SRW Facing

Stacked Cellular Confinement Facing

Gabion Basket Facing

REINFORCED STEEPENED SLOPE SYSTEM ON A FIRM FOUNDATION

Applications of a Reinforced Steepened Slope System

Slopes are common geographic features located adjacent to highways and along the periphery of building sites in many areas of the country. For construction on highway and building projects, relatively flat areas are preferred. These areas must be excavated out of the existing terrain, often leaving significant grade changes at the edges of the excavation. The economic feasibility of constructing a particular highway alignment or the development of a parcel of land may be determined by the ability to create sufficient flat, or level, land to satisfy space safety, or access requirements. Reinforced steepened slopes provide a cost-effective means to achieve more efficient grade changes than is possible with unreinforced slopes. Figure 1 illustrates some of the applications of reinforced steepened slopes.

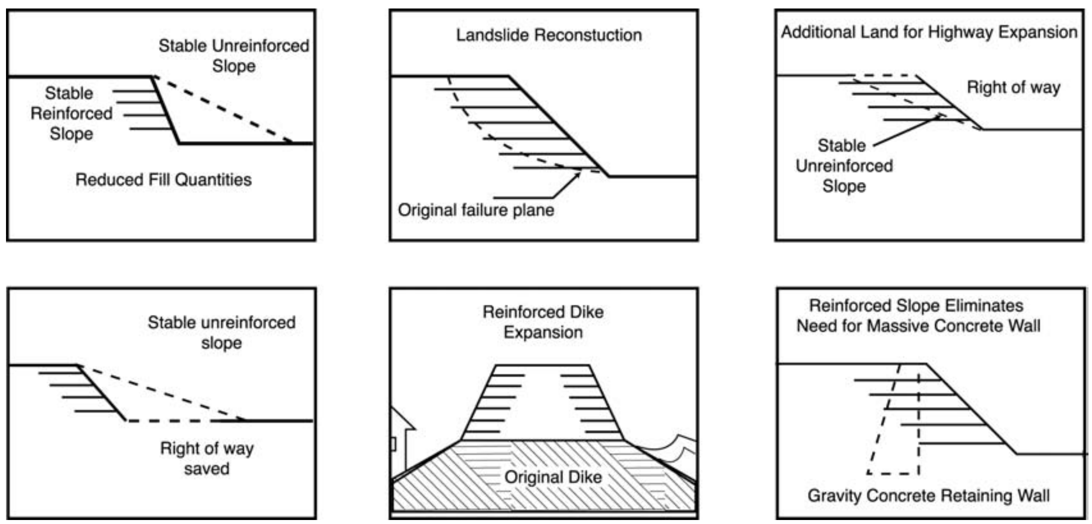


Figure 1: Applications Using Reinforced Steepened Slopes

Details of a Reinforced Steepened Slope System

Overview. Geosynthetic reinforced steepened slopes are soil structures constructed with slope face angles up to as high as 70 degrees from horizontal. Typical unreinforced soil slopes are limited to slope face angles of approximately 25 to 30 degrees or less, depending on the slope soil. The additional steepness provided by reinforced slopes minimizes the extent to which grade change structures, i.e. slopes or walls, must encroach into highway right-of-way or onto building sites as shown in Fig 2.

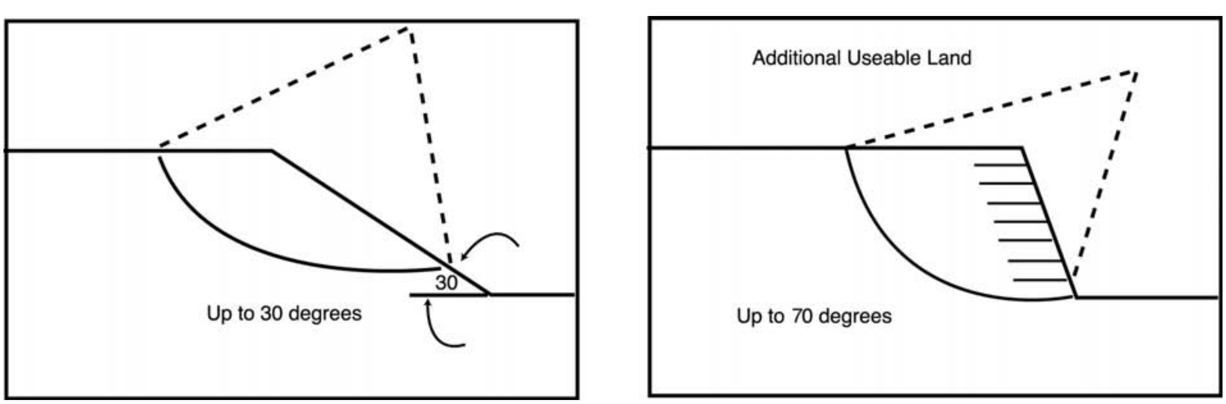


Figure 2: Conventional vs. Steepened Slopes

System Components. Like conventional soil slopes, reinforced slopes are constructed by compacting soil in layers while shifting the face of the slope back to create the desired angle. Subsequently, the face is protected from erosion by vegetation or other means. Additional geosynthetic elements maybe incorporated into reinforced steepened slopes to minimize ground water seepage and to enhance the stability of the steepened slope and the erosion resistance of the facing. The following are the typical components of a geosynthetic reinforced steepened slope system:

- Foundation - Stable soil or bedrock upon which the slope is constructed. Stability in the foundation is assumed.
- Retained Soil - The soil which remains in place beyond the limits of the excavation.
- Subsurface Drainage - Geosynthetic drainage medium installed at the limits of the reinforced soil zone to control, collect, and route ground water seepage.
- Reinforced Soil - The soil which is placed in lifts adjacent to the retained soil and incorporates horizontal layers of reinforcement to create the sloped structure.
- Primary Reinforcement- Geosynthetic, either geogrid or geotextile with sufficient strength and soil compatible modulus, placed horizontally within the slope to provide tensile forces to resist instability.
- Secondary Reinforcement - Geosynthetic, either geogrid or geotextile that is used to locally stabilize the slope face during and after slope construction.
- Surface Protection - The erosion resistant covering of the finished slope surface.

Figure 3 shows typical components of a reinforced steepened slope system and their relative locations. The primary reinforcement provides a tensile strength component within the reinforced soil zone that allows a slope to stand at steeper angles than would normally be achieved without reinforcement.

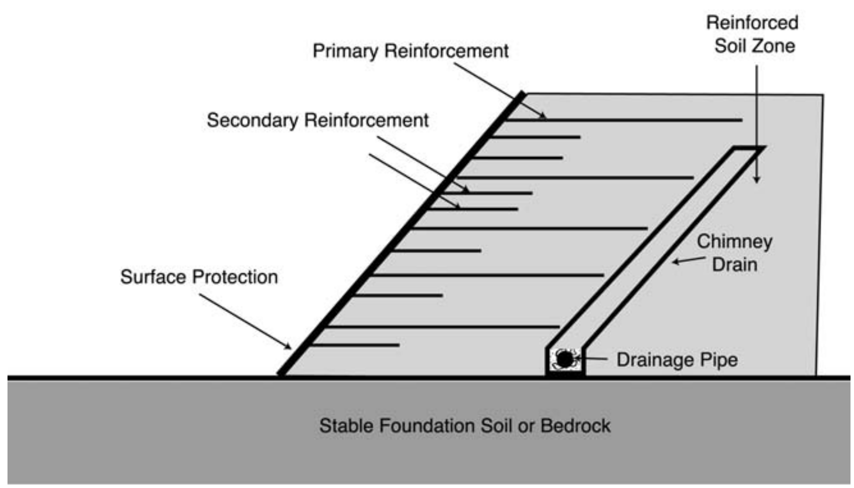


Figure 3: Typical Components of a Reinforced Steepened Slope System

Site Specific Design Considerations

Slope Geometry. The actual steepness requirements for a slope will result from the site layout and will be determined by assessing the topographic relationship between the toe line and the crest line. The grades, or steepness, of the slope as well as slope height, will generally vary along the slope alignment requiring the designer to select reasonably spaced, representative cross-sections for reinforcement design. When selecting slope angle, β, and slope height, H, slope angles should be no steeper than 70° and slope heights may be limited by surface water runoff considerations

Foundation Conditions. Slope stability analysis generally assumes that the foundation is firm, i.e. strong and stable, relative to the slope fill soils and thus deep-seated failure modes are not typically a concern. Still, the designer must assess the foundation conditions in the proximity of a proposed reinforced slope to assure that a failure plane passing through the foundation is unlikely. Soil test borings can be made to estimate subsoil strength and to locate geologic faults and ground water elevations.

Ground Water. Ground water is a potential source of problems in soil structures. Unexpected ground water seepage can alter fill and foundation properties, cause internal erosion, "slicken" potential failure surfaces or increase horizontal and vertical loadings. All of these conditions can be minimized by identifying ground water sources, controlling seepage from them, and designing for the resulting expected soil moisture conditions. Whenever possible, ground water elevations should be maintained well below the foundation level.

Fill. Reinforced slopes can be constructed from a wide variety of soils. This often allows on-site soils to be used, minimizing the need to transport material on or off site. Preferred fill materials are predominately granular or low plasticity fine-grained soils. Fill soil properties should be obtained from laboratory testing of the candidate soils. Selecting soil properties for use in design is discussed later.

Surcharge Loading. Additional vertical and horizontal loads are applied to the reinforced slope system by any surcharge or externally applied loading, that is imposed upon the system. These loadings can result from structures, vehicles, or even additional soil masses. Applicable surcharge loadings must be resolved into corresponding horizontal and vertical forces on the reinforced slope system. One way of doing this is to transform the surcharge load, q, into an equivalent additional soil layer equal to q/γ.

Other External Loading. Other externally applied loading such as point loads, seismic loads, or hydrostatic loads are beyond the scope of this document, but must still be addressed by the designer if they are present.

Fill Soil and Geosynthetic Reinforcement Properties

Material Selection. Each prospective fill type will develop unique strength and reinforcement interaction properties under the expected compaction and soil moisture conditions. Therefore, the cost-effectiveness of a reinforced slope can be affected by the fill and corresponding reinforcement type selected. A thorough evaluation of potential fill and reinforcement materials is necessary to identify the best possible combination.

Soil Properties. The critical equilibrium for steep reinforced slopes is usually governed by long term stability conditions. The soil strength is thus described in terms of its maximum unit weight, γ_{max}, effective friction angle ϕ'_f, and effective cohesion, c'[2]. These properties are used to determine the stability of soil layers under design loadings. Table 1 outlines some typical soil types and ranges of associated soil properties. This information is for general groups of soils and should be used only as a guide. Specific soil properties for the foundation, fill and embankment soils on a given project should be determined from field and laboratory testing.

Table 1: Typical Soil Properties [1]

Soil Description	USCS Class*	(Deg) 0’	MDD** Std Compact (lb/ft3)	Optimum Moisture Content (%)
Well-graded sand-gravel	GW	>38	125-135	11 - 8
Poorly-graded sand-gravel	GP	>37	115-125	14 - 11
Silty gravels, poorly graded sand-gravel-silt	GM	>34	120-135	12 - 8
Clayey gravels, poorly graded sand-gravel-clay	GC	>31	115-130	14 - 9
Well graded clean sand, gravelly sands	SW	38	110-130	16 - 9
Poorly-graded clean sands, gravelly sands	SP	37	100-120	21 - 12
Silty clays, sand-silts - clays	SM	34	110-125	16 - 11
Clayey sands, sand-clays	SC	31	105-125	19 - 11
Silts and clayey silts	ML	32	95-120	24 - 12
Clays of low plasticity	CL	28	95-120	24 - 12
Clayey silts, elastic silts	MH	25	70-95	40 - 24
Clays of high plasticity	CH	19	75-105	36 - 19

**Unified Soil Classification System*
***MDD=max dry density*

Soil properties used in the design of reinforced slopes must reflect the expected *in-situ* conditions. Cohesion in the soil is often neglected which provides additional conservatism to the design. The controlled placement of the fill and the flexibility of the finished structure generally assures a drained, large strain condition. The soil strength is properly described by either a large strain or a factored peak effective soil friction angle, ϕ'_f. The factored soil friction angle is calculated using Equation 1.

$$\phi'_f = \tan^{-1} [(\tan\phi')/FS] \qquad \text{(Eqn 1)}$$

Geosynthetic Reinforcement. The geosynthetic reinforcement, i.e. geogrids or geotextiles, used in slopes must satisfy both strength and soil interaction requirements. The strength requirements focus on the long term design strength (LTDS) of the reinforcement. Soil interaction properties include coefficients of direct sliding, C_{ds}, and pullout, C_i.

Strength Properties. For reinforced soil structures it is important that the reinforcement be "compatible" with the soil. This means that the long term design strength of the reinforcement should be acheived at a total strain level (elastic + creep) corresponding to a strain in the soil matching peak soil strength. For most soils the strain level at peak soil strength is between 3% and 10% and is easily determined by laboratory testing. As a result, a total strain level not to exceed 10% is commonly used for steepened slopes, though a limiting strain of 5% may be appropriate if sensitive structures are adjacent to the slope.

The long term design strength (LTDS) of a reinforcement is determined by applying partial factors of safety to the ultimate tensile strength. These partial factors of safety account for creep, chemical and biological durability and installation damage. Equation 2 is used to calculate LTDS. Table 2 provides LTDS values in the primary machine strength direction (MD) for selected Mirafi® reinforcement products.

$$LTDS = T_{ult} \;/\; [RF_{cr} \times RF_{id} \times RF_{d}]$$
(Eqn 2)

where: T_{ult} = ultimate wide width tensile strength; RF_{cr} = reduction factor for creep deformation; RF_d = reduction factor for durability; RF_{id} = reduction factor for installation damage

Table 2- LTDS for Selected Geosynthetics

Geosynthetic	LTDS (In sand) (lb/ft)
Miragrid® 2XT	949
Miragrid® 3XT	1558
Miragrid® 5XT	2234
Miragrid® 7XT	2961
Miragrid® 8XT	3636
Miragrid® 10XT	4312

Soil Interaction Properties. The coefficient of direct sliding, C_{ds}, and the pullout interaction coefficient, C_i , are both measures of the interaction between the geosynthetic and the soil and are determined by laboratory testing. The value C_{ds} is used in the calculation of factors of safety involving a block of soil sliding over a geosynthetic layer. C_i is used to determine the length of geosynthetic which must extend beyond the critical failure surface to fully develop, or anchor, the reinforcement. Equation 3 is used to calculate this 'embedment' length, L. Table 3 provides C_{ds}, and C_i values for selected Mirafi® geosynthetics in typical soils (4,5,6).

$$T_{pull} = 2 \times C_i \times L \times o'_v \times \tan\phi'_f \qquad \text{(Eqn 3)}$$

Table 3- Coefficient of Shear Stress Interaction for Mirafi® Construction Products

Soil Type	Coefficient of Shear Stress Interaction C_i: Miragrid® Geogrids	Coefficient of Direct Sliding C_{ds}
Sands	0.9-1.0	0.9
Silts	0.8-0.9	0.8
Clays	0.7-0.8	0.7

Note: These values are for preliminary design purposes only. Specific test results are available from the Mirafi® Technical Services Department, upon request.

STABILITY ANALYSIS FOR STEEPENED SLOPES AND EMBANKMENTS OVER STABLE FOUNDATIONS

Two-part Wedge Analysis

The two-part wedge analysis method for a soil slope or embankment over a stable foundation can be referenced to Figure 4. A trial failure mechanism is defined by potential linear failure surfaces that are assumed to propagate from a point on the slope (point A) to a breakpoint (B) and then exit at the slope surface at point (C) located at or beyond the slope crest. The potential failure zone therefore comprises two soil masses (wedges) identified as regions 1 and 2 in the figure. If a reinforcement layer intersects a potential failure surface then it provides a horizontal restraining force that is included in the overall calculation of horizontal force equilibrium.

In a typical analysis a large number of two-part wedge geometries must be inspected in order that the critical geometry is found (i.e. the two-part wedge giving the lowest factor-of-safety against slope failure). It is clear that the only practical method of identifying the critical failure mechanism is to use a computer program. Computer programs RSS, available from Federal Highway Administration by ADAMA (FHWA NH1-00-043) engineering can be used to carry out two and three part wedge analysis for slopes with varying geometrics, soil properties, and groundwater elevations. Other commercial software programs are also available.

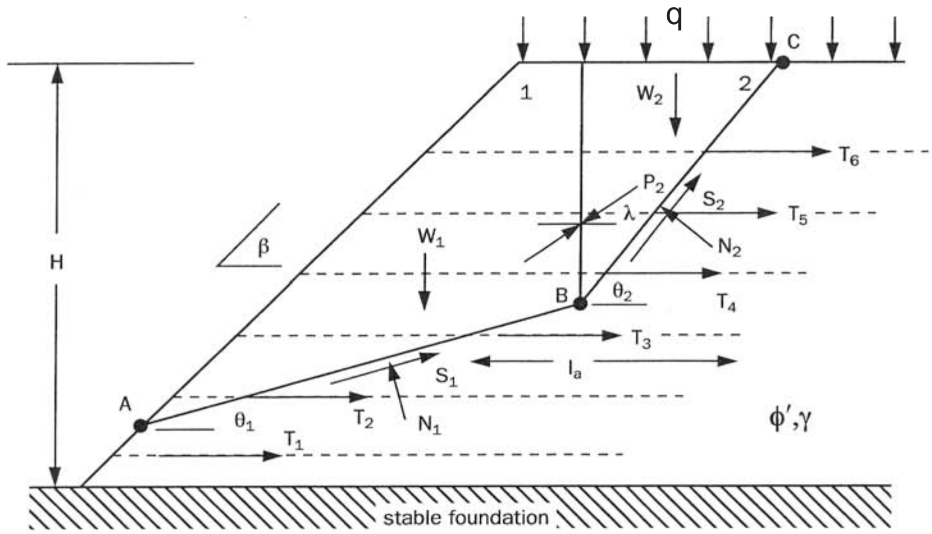


Figure 4: Two-part wedge analysis

Stability Calculations

The stability calculations for an assumed two-part wedge failure mechanism can be referenced to Figure 4.

For illustration purposes, the procedures described in the section are restricted to reinforced slopes with uniform, cohesionless soils (i.e., c'= 0, ϕ'>0) and the groundwater table well below the toe elevation.

The destabilizing forces acting on the slope include the bulk weight of the trial wedges W_1 and W_2 and any uniformly distributed surcharge q. The resisting forces include the shear resistance developed along the bottom and top failure planes, S_1 and S_2 and the horizontal tensile forces developed by the intersected reinforcement layers. The shearing resistance along the failure planes AB and BC are assumed to be Coulomb type with $S_1 = N_1 \times \tan\phi'_f$ and $S_2 = N_2 \times \tan\phi'_f$. The soil friction angle used in the computation is the factored soil friction angle (ϕ'_f) calculated according to Equation 1.

The quantity P_2 in Figure 4 is the unbalanced force that is required to keep the upper wedge at limit equilibrium. In general, the orientation of the interslice friction angle will be $0 < \lambda < \phi'_f$. A conservative assumption is $\lambda = 0$ (i.e., results in a safer design).

The factor-of-safety (FS) against failure of a trial two-part wedge is the minimum value that can be applied to the peak soil friction coefficient so that the horizontal destabilizing force P Is just equal to the sum of the factored horizontal tensile capacities of the reinforcement layers ST/FS. The sum ΣT is calculated from the tensile capabilities of the reinforcement layers that are intersected by the trial failure surfaces (i.e., T_2 through T_6 in Figure 4).

The out-of-balance horizontal force P is calculated using Equation 4a, 4b and 4c. The wedge weights W_1 and W_2 include the net vertical force due to any uniformly distributed surcharge load acting over the slope surface.

The maximum tensile force T_i available from any individual reinforcement layer is the lesser of the long term design strength (LTDS) or the design pullout capacity of the geosynthetic, T_{pull}.

$$P = P_2\cos\lambda + (P_2\sin\lambda + W_2)\left\{\frac{\sin\theta_2 - \cos\theta_2\tan\phi'_f}{\cos\theta_2 + \sin\theta_2\tan\phi'_f}\right\} \quad \text{(Eqn 4a)}$$

$$\text{where, } P_2\cos\lambda = W_1\left\{\frac{\tan\theta_1 - \tan\phi'_f}{1 + \tan\theta_1\tan\phi'_f}\right\} \quad \text{(Eqn 4b)}$$

$$\text{and } \phi'_f = \tan^{-1}\left\{\frac{\tan\phi'_f}{FS}\right\} \quad \text{(Eqn 4c \& Eqn 1)}$$

A summary of possible failure mechanisms that must be examined to find the critical mechanism is illustrated in Figure 5. Other permutations include external base sliding in which no reinforcement layers are intersected by the upper wedge.

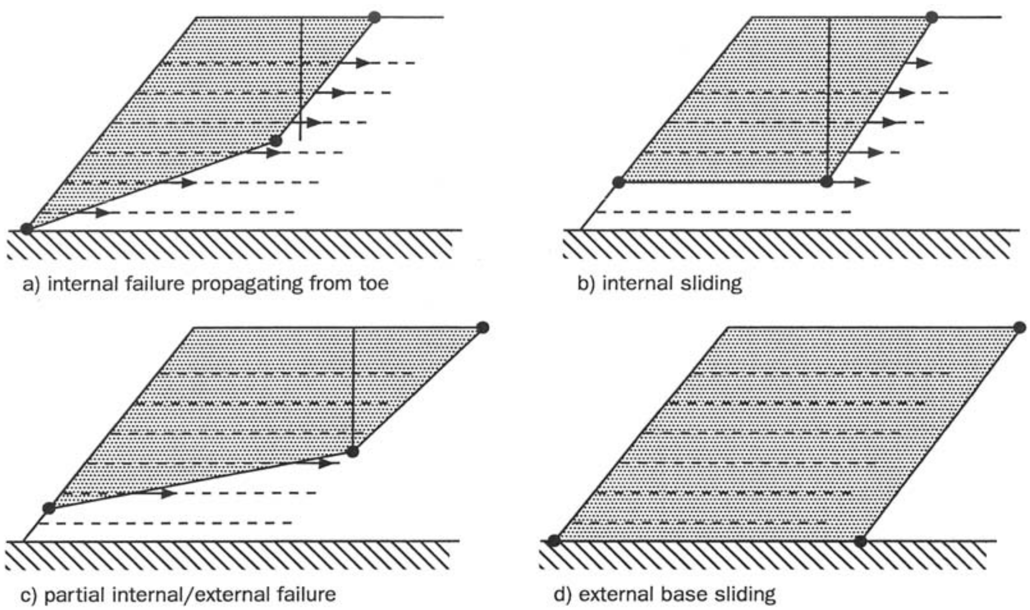


Figure 5: Some Two-Part Wedge Failure Mechanisms

Internal Sliding

Figure 5b illustrates an internal direct sliding mechanism in which the bottom wedge boundary coincides with a layer of reinforcement. Conventional practice is to assume that the potential shear resistance along this bottom surface is modified by the presence of the reinforcement layer. For this condition the friction coefficient term ($\tan\phi'_f$) in the denominator and numerator of Equation 4a becomes ($\alpha \times \tan\phi'_f$) where α is the direct sliding coefficient. The magnitude of the direct sliding coefficient is restricted to $\alpha < \phi'_f$. For Miragrid® geogrid products in combination with well-compacted granular soils, Mirafi® Construction Products recommends a value of α =0.9 for preliminary design purposes. For final design and analysis purposes a representative value of the direct sliding coefficient can be determined from the results of direct shear box testing. These tests should use the proposed geosynthetic reinforcement material and slope soils prepared to the same conditions as in the field.

Factor-of-Safety

A minimum factor-of-safety for reinforced slopes with frictional soils is FS=1.5 applied to Eqn 1. The actual choice of factor-of-safety should be based on the recommendation of a geotechnical engineer who is familiar with the soils at the site, slope function, additional loads, proposed reinforcement material and method of construction.

Circular Slip Analysis

This section reviews circular slip methods of analysis for the design and analysis of steepened slopes and embankments over stable foundations. For design purposes, the slopes are assumed to be seated on competent foundation soils or rock that are incompressible. Potential failure surfaces are assumed to be restricted to the slope soils or embankment fill above the stable foundation.

The method of analysis described in the manual is based on a modified "Bishop's Simplified Solution" in which the factor-of-safety against slope failure is described by the ratio of the sum of resisting moments to the sum of driving moments calculated using the method of slices. The driving moments are due to soil self weight and any surface loadings. The resisting moments are proportional to the mobilized soil shearing resistance developed along the failure surface. This conventional and widely used method of analysis can be easily modified to include the resisting moment due to any reinforcement layer that intersects a trial failure surface. The methodology described in this section follows the recommendations contained in the FHWA guidelines[3] for reinforced slopes.

In the examples to follow the soils are assumed to be granular materials and stability calculations are based on an effective stress analysis. The analysis are therefore appropriate for drained soils.

Unreinforced Slope

The factor-of-safety FS_u for an unreinforced slope is expressed as:

$$FS_u = \frac{\text{Resisting Moment}}{\text{Driving Moment}} = \frac{M_r}{M_d} \qquad \text{(Eqn 5)}$$

The slope can be divided into a convenient number of slices as illustrated in Figure 6 for a prescribed center-of-rotation 0 and Radius R. The factor-of-safety Equation 5 can be expanded as shown in Equation 6. Here the summation signs are with respect to the vertical slices.

The parameters shown in Figure 6 and in Equation 6 are:

- W = total weight of slice based on bulk unit weight of soil plus surcharge loading (q x b) if present
- q = uniformly distributed surcharge acting at crest of slope
- b = the horizontal width of the slice
- ψ = the angle formed by the tangent to the midpoint of the slice and the horizontal
- c' = soil cohesion at base of slice
- ϕ' = peak soil friction angle at base of slice
- r_u = dimensionless pore water pressure coefficient

$$FS_u = \frac{1}{\Sigma W \sin\psi} \Sigma \left[\{c'b + W(1 - r_u)\tan\phi'\} \frac{\sec\psi}{1 + \frac{\tan\psi\tan\phi'}{FS_u}} \right] \qquad \text{(Eqn 6)}$$

The porewater pressure coefficient r_u can be approximated using the approach illustrated in Figure 6. The result may be a small error that is conservative. For any slice that does not intersect the groundwater table $r_u = 0$.

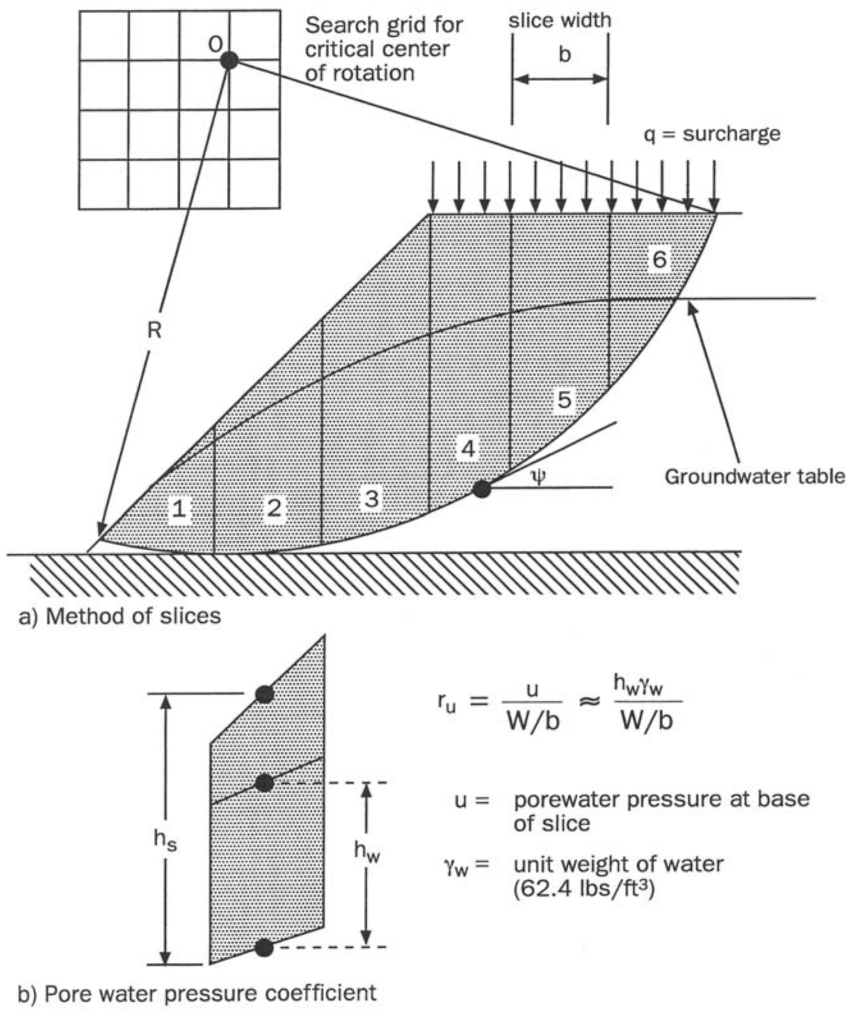


Figure 6: Circular Slip Analysis and Method of Slices for Unreinforced Slope

The presence of the factor-of-safety term on both sides of Equation 6 means that for a prescribed trial failure circle, a process of successive iterations is required until the solution converges to a unique value of FS_u. Clearly, the computations required to perform this calculation and to inspect a potentially large number of critical slip circles means that the analysis is best performed using a computer program.

Commercially available computer programs such as G Slope from Mitre Software Corporation, RSS from FHWA, ReSSA and ReSlope from ADAMA Engineering, STABL from Purdue Universiy, and UTEXAS4 from the University of Texas, as well as others, can be used fro this purpose.

Reinforced Slope

The factor-of-safety FS_r for a reinforced slope is expressed as:

$$FS_r = FS_u + \frac{\text{resisting movement due to reinforcement}}{\text{driving moment}} \qquad \text{(Eqn 7)}$$

The right hand term represents the additional factor-of-safety against slope failure due to the stabilizing effect of the tensile geosynthetic reinforcement. Referring to Figure 7, the factor-of-safety expression for the reinforced slope case can be expressed as:

$$FS_r = FS_u + \left(\frac{\Sigma T_i R_{T_i} / \cos\psi_1}{M_D}\right) \qquad \text{(Eqn 8)}$$

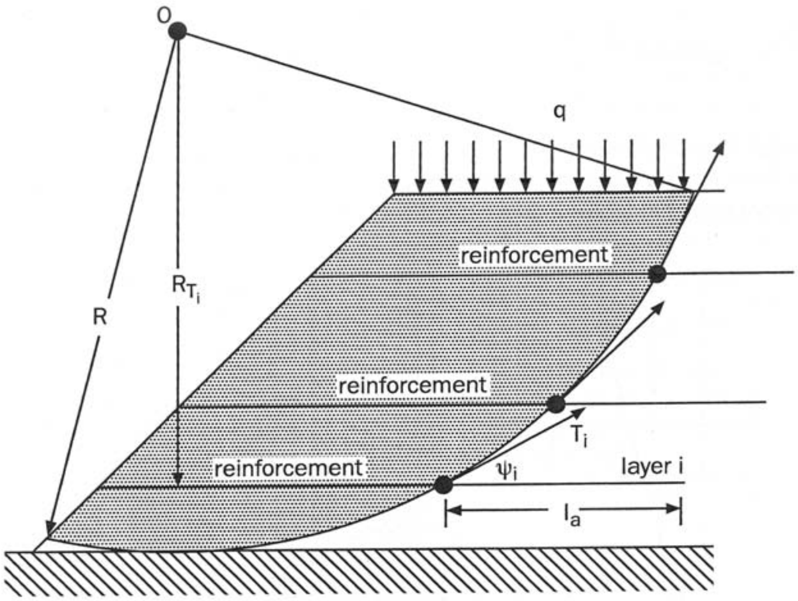


Figure 7: Circular Slip Analysis and Method of Slices for Reinforced Slope

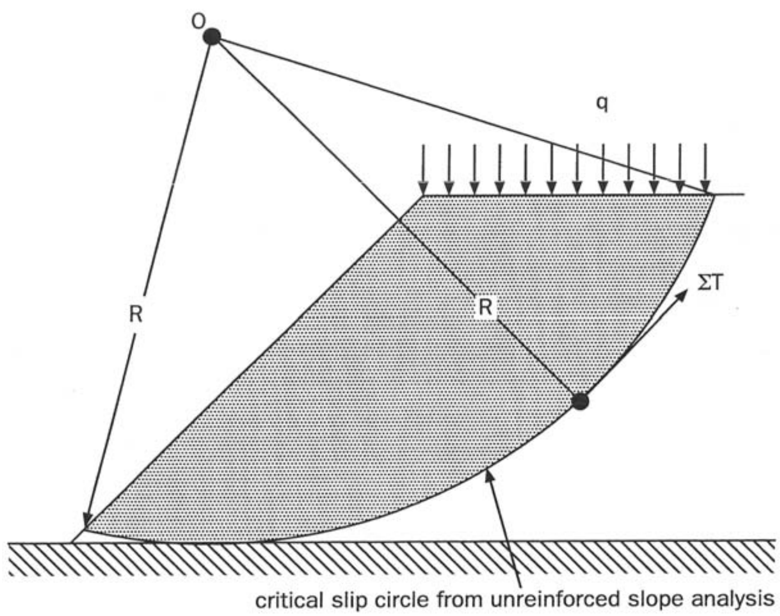


Figure 8: Approximate Method to Calculate the Factor-of-Safety for a Reinforced Slope[3]

Here the summation term is with respect to the reinforcement layers and the tangent slopes ψ_i of the circular slip surface at the point of intersection with each reinforcement layer i. Some engineers argue that the restoring force T_i will act parallel to the slip surface if the reinforcement products are extensible materials (e.g. Miragrid®). Extensible reinforcement products are able to conform to the geometry of the failure surface at incipient collapse of the slope.

The magnitude of the tensile force T_i used for each layer in the summation term in Equation 8 is the lesser of:

1. The long term design strength of the reinforcement (LTDS). This is the working tensile load level below which the reinforcement remains intact and does not undergo excessive straining.

2. The pullout capacity of the embedded length of the reinforcement beyond the slip circle (i.e. length l_a in Figure 7). The quantity T_i must not exceed the pullout capacity (T_{pull}) of the reinforcement. The calculation of pullout capacity is performed by using equation 3.

The above method can be used with commercially available software for circular slip analysis of unreinforced slopes provided that the magnitude of the driving moment M_D is available in the output. The magnitude of the right hand term in Equation 8 can then be computed by hand or by using a simple computer spreadsheet. Alternatively, the approximate method described in the following section can be used to give a reasonably conservative estimate of the reinforced slope factor-of-safety.

Approximate Method to Calculate Factor-of-Safety for Trial Reinforced Slope

For preliminary design purposes the factor-of-safety from the results of an unreinforced slope stability analysis can be modified to estimate the factor-of-safety for the corresponding reinforced slope using extensible reinforcement (refer to Figure 8):

Step 1. Calculate FS_u for the unreinforced slope and determine the geometry of the corresponding critical slip circle.

Step 2. Calculate the total available restoring force ΣT_i based on the sum of the LTDS of all reinforcement layers that intersect the critical slip circle from Step 1.

Step 3. Assume that ΣT_i acts parallel to the critical unreinforced slip circle and calculate the factor-of-safety for the reinforced slope as follows:

$$FS_r = FS_u + R \times \frac{\Sigma T_i}{M_D} \qquad \text{(Eqn 9)}$$

External Stability of a Reinforced Soil Mass over a Stable Foundation[3]

The FHWA guidelines contain recommendations for the analysis of external sliding stability of a reinforced soil mass over a stable foundation. The sliding mechanism assumed in these calculations is conceptually identical to the sliding mechanism illustrated in Figure 5d. The reinforced soil mass is treated as an equivalent gravity structure with a mass equal to the reinforced zone above the base. The factor-of-safety against base sliding is calculated as the ratio of base sliding resistance (force/unit width of slope) to driving force resulting from the retained slope materials.

The limits of the reinforced soil mass can be estimated using the Design Chart Method described in the next section or the results of circular sup analysis described in the previous section.

CHARTS FOR PRELIMINARY DESIGN OF STEEPENED SLOPES AND EMBANKMENTS OVER STABLE FOUNDATIONS

This section describes how the designer can use a series of charts to carry out a preliminary design of a reinforced soil slope or embankment. The preliminary designs that result from this approach are restricted to slopes or embankments composed of free-draining granular soils and constructed over stable foundations. The charts have been generated using a conventional two-part wedge limit equilibrium method of analysis and cover the case of simple geometry with a range of slopes from 90 degrees (vertical) to 30 degrees and a range of soils with friction angles from 15 degrees to 50 degrees. A factored soil friction angle ϕ'_f (Equation 1) should be used with the design charts to account for variability in soil properties and uncertainty in slope geometry and loading.

Principal Assumptions

The basic assumptions used to generate the charts are as follows:

1. The foundation soils below the toe of the slope are stable and any potential instability is restricted to the free-draining cohesionless granular soil mass above the elevation of the toe.
2. The groundwater table is well below the toe of the slope.
3. The properties of the soil are uniquely described by a uniform bulk unit weight λ and a peak friction angle ϕ' (degrees).
4. The intersection of failure surfaces with the slope boundaries occurs at the toe of the slope and at points beyond the crest.
5. Interslice forces have been assumed to act at an angle of $\lambda = \phi'$ to the horizontal.
6. No additional slope loadings due to seismic forces are present.
7. The primary reinforcement utilized is a Mirafi® reinforcement product.

Calculation of Factored Soil Friction Angle for Design

A factor-of-safety FS should be applied to the soil peak friction angle to account for variability in soil properties and uncertainty in slope geometry and loading. For routine slopes a value of FS = 1.5 is typical. However, it is the responsibility of the geotechnical engineer to recommend an appropriate factor-of-safety based on site conditions, external loading and slope function. The factored soil friction angle ϕ'_f is used in the calculations described in the following text. The factored soil friction angle is calculated as follows:

$$\phi'_f = \tan^{-1}\{(\tan\phi')/FS\} \qquad \text{(Eqn 10)}$$

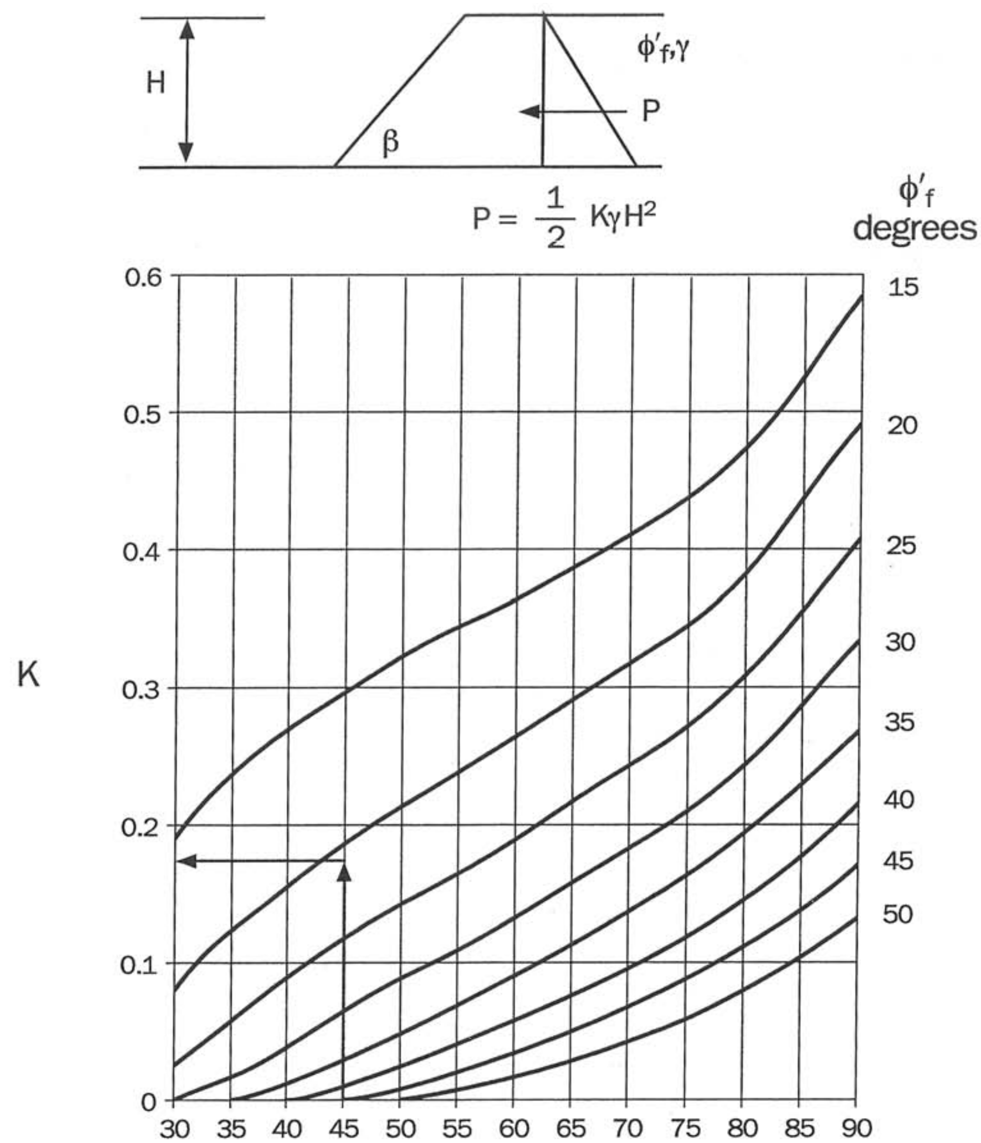


Chart 1: Equivalent Coefficient of Earth Pressure, K= f(ϕ'_f, β)

Design Charts

Coefficient of Earth Pressure (Chart 1)

In order to estimate the minimum number of primary reinforcement layers in a slope it is necessary to calculate the net horizontal force P required to just maintain the slope at limit equilibrium. The approach adopted to generate the design charts is to determine the critical two-part wedge that yields the maximum required horizontal force P. The geometry used in the two-part wedge analysis is illustrated in Figure 9.

Chart 1 gives the maximum equivalent coefficient of active earth pressure K based on a search of all potential two-part wedges for a given β and factored friction angle (i.e $\phi' = \phi'_f$). The magnitude of force P is determined by examining a very large number of wedge geometries defined by a grid of break-points superimposed on the slope cross-section. For each break-point the angle θ_2 was varied to determine the maximum out-of balance force P. The analysis assumed that interslice forces act at 1/3 the height of the interslice boundary (the point of application is a concern in eccentricity calculations described in the next section).

The constraint that the width L of the reinforced zone must be sufficient to capture the critical two-part wedge is illustrated in Figure 10. The calculation for adequate reinforcement zone width to prevent base sliding can be referred to Figure 11. The calculations to determine the minimum base width to prevent sliding were based on the following factor-of-safety relationship:

$$FS = \frac{S}{P_2\cos\lambda} \qquad \text{(Eqn 11)}$$

$$= \frac{\alpha\,(W_1 = P_2\sin\lambda)\,\tan\phi'}{P_2\cos\lambda} = 1$$

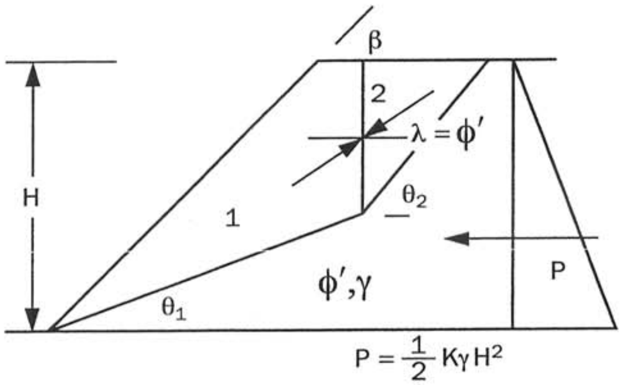


Figure 9:
Two-Part Wedge Anaylysis to Calculate Unbalanced Horizontal Force P.

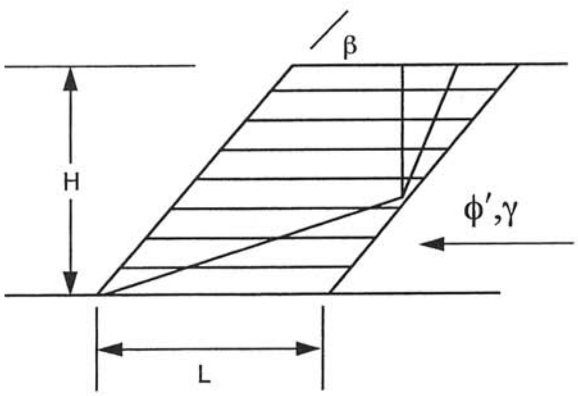


Figure 10:
Reinforced Zone Containing Critical Two-Part Wedge

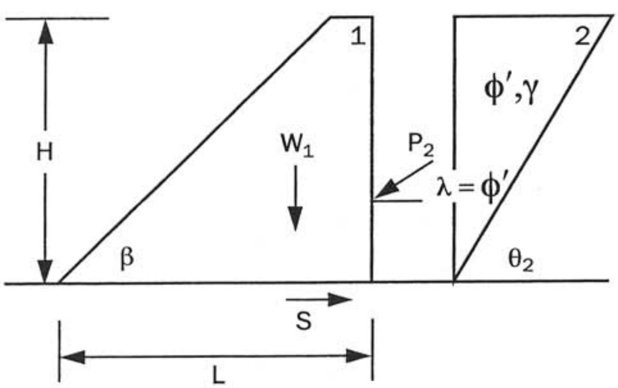


Figure 11:
Free Body Diagram for Calculation of Base Sliding

Minimum Reinforcement Length (Chart 2)

The calculation of the minimum length of reinforcement was based on the following criteria:

1. All reinforcement lengths are equal (i.e. truncation parallel to the slope face).
2. The reinforced zone must have sufficient length L to contain the critical unreinforced two-part wedge.
3. The reinforced zone must have sufficient length L that the slope does not slide outward.
4. The reinforced zone must have sufficient length L that tensile vertical stresses are not developed along the surface of the foundation soils (i.e., base eccentricity must fall within the middle third of the base width L).

Here the parameter S is the shearing resistance acting at the base of the slope and is controlled by the friction angle of the slope soils ϕ', the weight of wedge W_1 (hence width of the reinforced zone L) and the coefficient of direct sliding (which has been taken as $\alpha = 0.9$). The quantity P_2 is the unbalanced interslice force acting on wedge 1 by the right hand side wedge 2.

The calculation for base eccentricity can be referenced to Figure 12. The analysis involves progressively increasing the base dimension L until the linear distribution of vertical base pressure σ'_v, is compressive everywhere for maximum values of P_2.

The results of analysis are presented in normalized form L/H on Chart 2 for the condition $\lambda = \phi'$ (where $\phi' = \phi'_f$). Here, L is the length of reinforcement and H is the height of the slope.

Calculation of Minimum Number of Reinforcement Layers

The equivalent coefficient of earth pressure K for design is determined from Chart 1 using ϕ'_f. The minimum number of reinforcement layers N_{min} can be calculated as follows:

$$N_{min} >= \frac{P}{LTDS} = \frac{(1/2)\,K\gamma H^2}{LTDS} \qquad \text{(Eqn 12)}$$

Here term LTDS denotes the long term design strength (allowable working stress) of the Mirafi® reinforcement products.

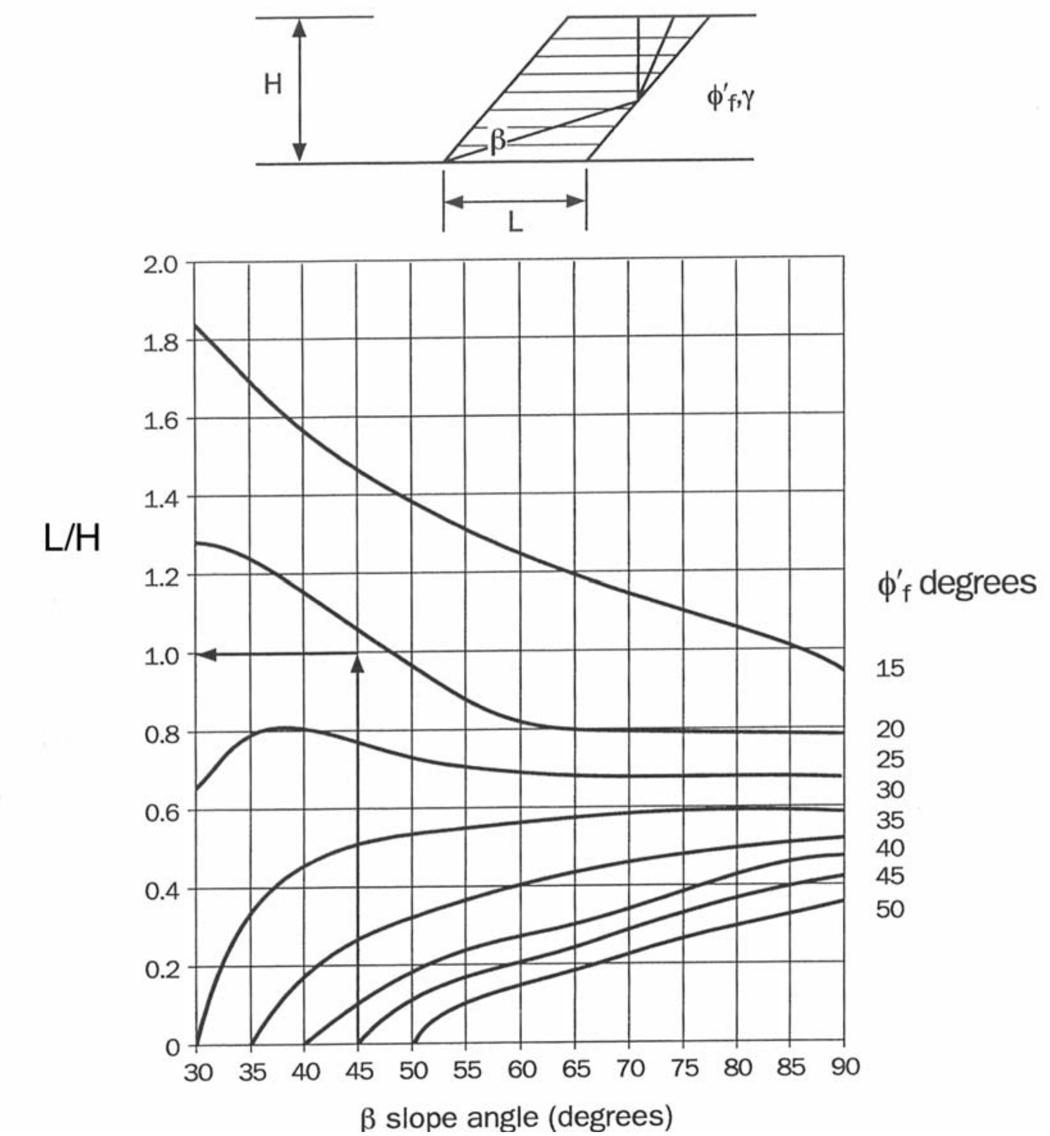


Chart 2: Minimum Ration of Reinforcement Length to Slope Height L/H to contain Critical Two-Part Wedge and Satisfy Sliding and Eccentricity Criteria.

Calculation of Minimum Length of Reinforcement

The minimum reinforcement length L is calculated from Chart 2 based on ϕ'_f, β, and the height of the slope H.

Calculation of Maximum (Primary) Reinforcement Spacing

The calculation of maximum reinforcement spacing S_{vmax} at any depth z below the crest of the slope can be carried out using the following relationship:

$$S_{vmax} = > \frac{LTDS}{K\gamma z} \qquad \text{(Eqn 13)}$$

Here the quantity LTDS refers to the long term design strength of the reinforcement and parameter K to the coefficient of earth pressure established from Chart 1. The value of S_{vmax} in Equation 23 is dependent on the magnitude of bulk moist unit weight of the soil γ, the value K and the LTDS of the reinforcement. Hence, it is not practical to provide a general chart to estimate S_{vmax}. A spacing design chart based on the example problem at the end of this chapter illustrates the procedure.

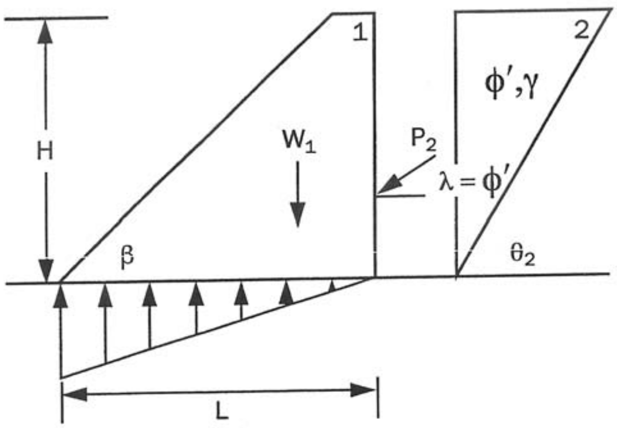


Figure 12
Free body diagram associated with calculation of minimum reinforcement length L to ensure compressive bearing pressures at base of slope (i.e., base eccentricity <L/6)

Uniform Surcharge

The influence of a uniformly distributed surcharge q acting at the crest of the slope (Figure 13) can be considered by analyzing a slope with an equivalent unsurcharged height H' where:

$$H' = H + q/\gamma \qquad \text{(Eqn 14)}$$

The replacement of the surcharged slope height by an equivalent unsurcharged height H' is valid for $q/\gamma < 0.2H$. For greater surcharge pressures a more detailed slope stability analysis should be carried out.

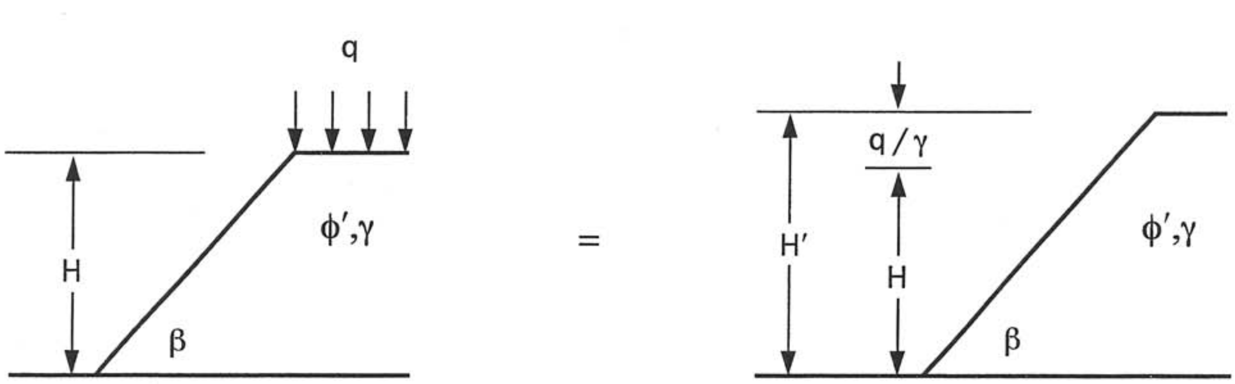
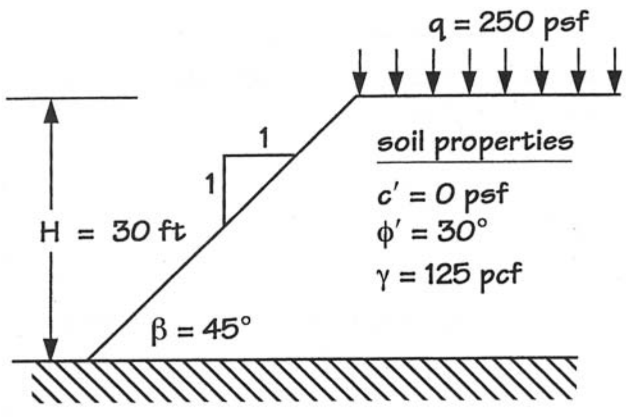


Figure 13: Modified Slope Height to Incline Influence of Uniformly Distributed Surcharge

Example Design Problem

The following design example is related to the proposed slope geometry and site parameters shown in Example Figure 1. The engineer is required to recommend a reinforcement layout using Miragrid® geogrid reinforcement products.



Example Figure 1:
Proposed slope geometry and soil parameters for design example.

Step 1

Select design parameters for soil and Miragrid properties

Embankment soil properties:

Peak friction angle, $\phi' = 30^\circ$
Cohesion. $c' = 0$ psf
bulk unit weight, $\gamma = 125$ pcf
Slope factor-of-safety, FS = 1.5

Slope height, H = 30 ft
Uniform surcharge pressure, q = 250 psf
Slope angle, $\beta = 45^\circ$

Geogrid properties:

Long term design strength (LTDS), in Type 3 backfill: sand, silt, clay:
Miragrid 2XT = 949 lb/ft
Miragrid 3XT = 1558 lb/ft
Miragrid 5XT = 2234 lb/ft
Miragrid 7XT = 2961 lb/ft
Miragrid 8XT = 3636 lb/ft
Miragrid 10XT = 4312 lb/ft

Step 2

Calculate factored friction angle ϕ'_f:

$\phi'_f = \tan^{-1}\{(\tan\phi')/\mathrm{FS}\}$

$\phi'_f = \tan^{-1}\{(\tan 30^\circ)/1.5\} = 21.0^\circ$

Step 3

Calculate equivalent slope height H:

$H' = H + (q/\gamma)$
$H' = 30 + (250/125) = 32$ ft

Step 4

Determine the force coefficient K, from Chart 1 using the slope angle β, and the factored friction angle ϕ'_f:

$K = 0.18$

Step 5

Determine the total horizontal force P that must be resisted by the Miragrid® reinforcement layers:

$P = (1/2)\, K\gamma\, (H')^2$
$P = (1/2)\, 0.18\, (125)(32)^2 = 11520$ lb/ft

Step 6

Calculate minimum number of Miragrid® layers N_{min} required to counter unbalanced force P:

$N_{min} = P/(LTDS)$
N_{min} = for Miragrid 2XT = 11520/949 = 12.1 layers - 13 layers
N_{min} = for Miragrid 3XT = 11520/1558 = 7.4 layers - use 8 layers
N_{min} = for Miragrid 5XT = 11520/2234 = 5.2 layers - use 6 layers
N_{min} = for Miragrid 7XT = 11520/2961 = 3.89 layers - use 4 layers
N_{min} = for Miragrid 8XT = 11520/3636 = 3.2 layers - use 4 layers

Step 7

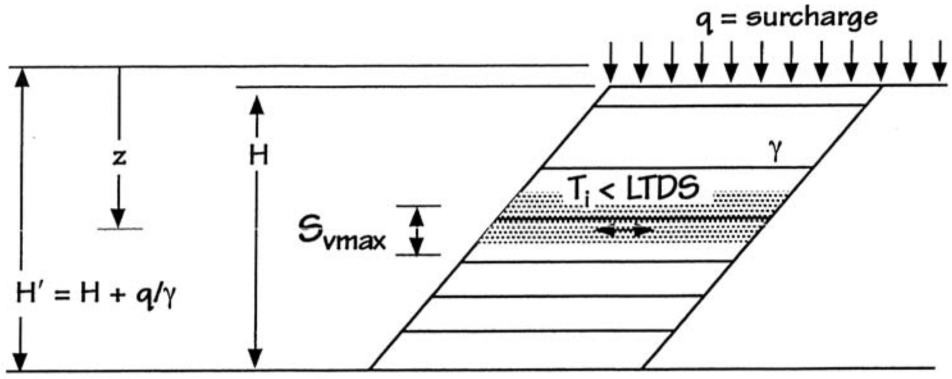
Determine the required embedment length of primary geogrid from Chart 2 using factored friction angle ϕ'_f,slope angle β and modified slope height H':

L/H' ratio from Chart 2 = 1.0
$L = (L/H')\,(H')$
$L = (1.0)\,(32) = 32.0$ ft

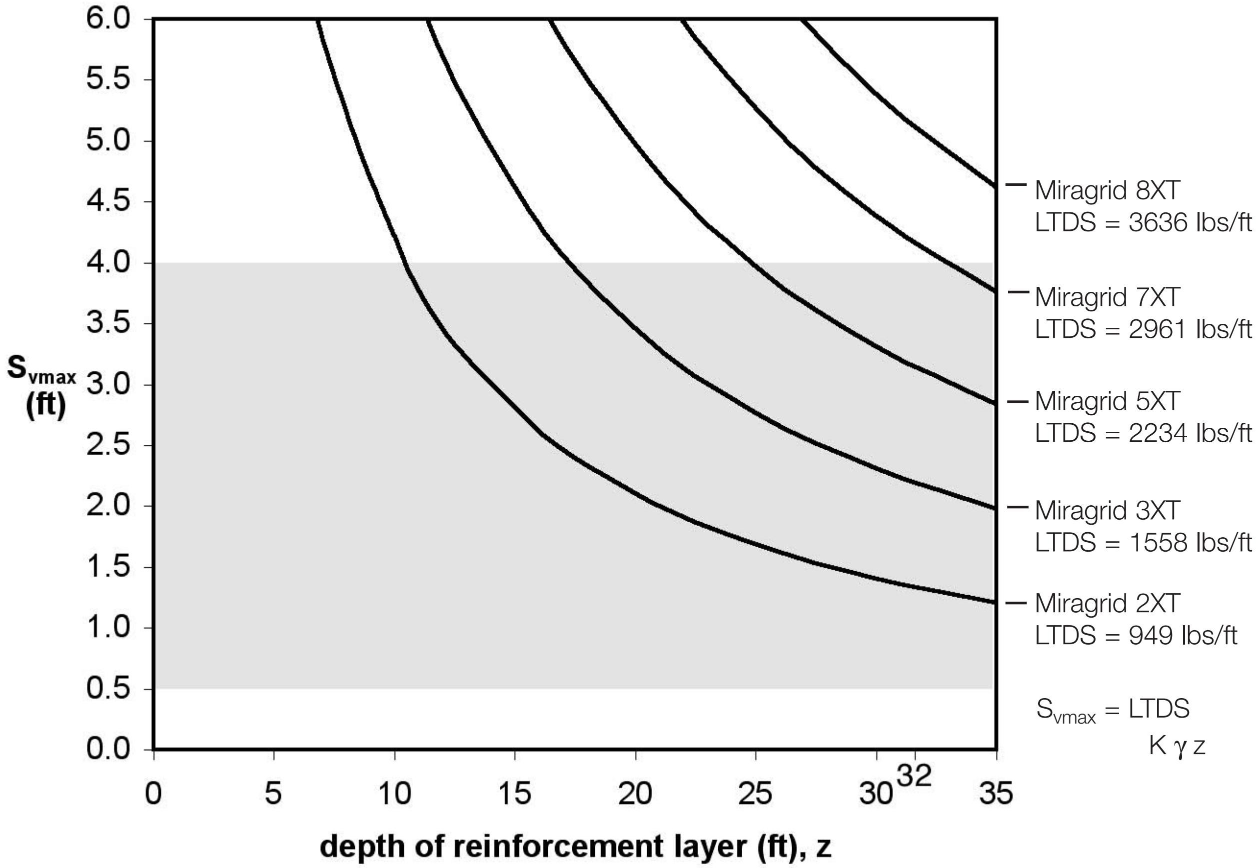
Step 8

Calculate the maximum allowable vertical spacing for each Miragrid product using:

$S_{vmax} = (LTDS) / (K\gamma z)$



Example design problem - Calculation of maximum spacing for primary reinforcement (K = 0.18, γ = 125 lbs/ft^3 and H' = 32 ft)



Example Figure 2: Example Design Problem
Calculation of maximum spacing for primary reinforcement
(K - 0.18, γ = 125 lbs/ft^3 and H' = 32 ft)

Where z is the distance from the top of the slope with height H'. It may be convenient to develop a chart such as that shown in Example Figure 2. Select spacing of geogrid layers starting from the bottorn of the slope and working up. For example: the spacing chart illustrates that Miragrid® 7XT, 8XT, and 10XT would not be recommended since at any elevation in the slope they are too strong and the spacing would be controlled by the 4 foot maximum spacing criterion recommended in the FHWA guidelines. A more reasonable selection would be Miragrid® 2XT, 3XT, 5XT or a combination of these products.

Step 9

Option 1: Select Miragrid® 2XT and calculate maximum geogrid spacing at bottom of embankment zone:

S_{vmax} = LTADL / $K\gamma H'$

S_{vmax} = 949 / (0.18 x 125 x 32) = 1.3; use S_{vmax} = 1.3 ft

Continue with Miragrid® 2XT. For layers within 12 feet of the modified slope crest, the layer spacing will be controlled by the 4 foot maximum spacing criterion.

Option 2: Select Miragrid® 3XT and calculate maximum geogrid spacing at bottom of embankment zone:

S_{vmax} = LTDS/ $K\gamma H'$
S_{vmax} = 1558 / (0.18 x 125 x 32) = 2.2; use S_{vmax} = 2.0 ft

Continue with Miragrid® 3XT. For layers within 18 feet of the modified slope crest, the layer spacing will be controlled by the 4 foot maximum spacing criterion.

Option 3: Select Miragrid® 5XT and calculate maximum geogrid spacing at bottom of embankment zone:

S_{vmax} = LTDS/ $K\gamma H'$
S_{vmax} = 2234 / (0.18 x 125 x 32) = 3.1; use S_{vmax} = 3.0 ft

Continue with Miragrid® 5XT. For layers within 26 feet of the modified slope crest, the layer spacing will be controlled by the 4 foot maximum spacing criterion.

Option 4: Break the embankment into top, middle and bottom zones and reduce the strength of the reinforcement in each layer starting with the strongest reinforcement at the bottom (i.e. 5XT). The bottom zone can be assumed to have a thickness of 12 feet, the middle zone a thickness of 10 feet and the top zone a thickness of 10 feet. Refine reinforcement spacing to minimize the number of primary reinforcement layers and to simplify construction:

Bottom zone (z = 32 to 20 feet):
use 5XT
S_{vmax} = 3.0 ft (as before)

Middle zone (z =20 to 10 feet):
use 3XT
S_{vmax} = 1558 / (0.18 x 125 x 20) = 3.5; use S_{vmax} = 3.0 ft

Top zone (z = 0 to 10 feet):
use 2XT
S_{vmax} = 949 / (0.18 x 125 x 10) = 4.2; use S_{vmax} = 4.0 ft

Step 10

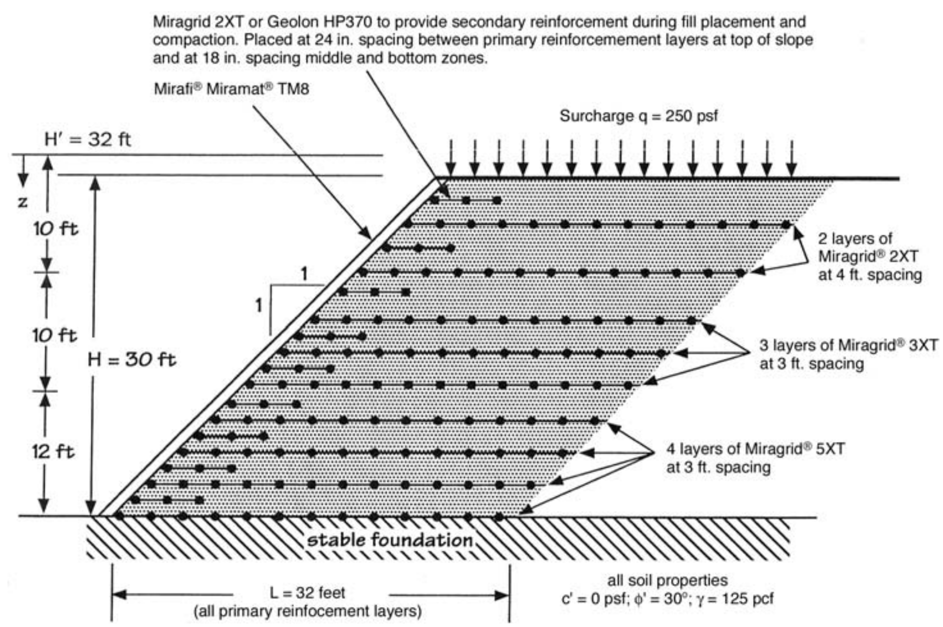
Add details to depict completed slope. If primary reinforcement spacing exceed 18 inches, use Miragrid® 2XT at 18 to 24 inch intervals as secondary slope reinforcement. If $\beta < 45^\circ$, treat slope surface with an appropriate surficial erosion control/revegetation system such as Miramat® TM8. If $\beta > 45^\circ$, wrap slope face with geogrid and provide appropriate erosion control/revegetation to provide additional slope protection.

Step 11

Sketch slope showing primary and secondary reinforcement as noted on Example Figure 3. The final recommended design will be based on a layout that is a compromise between the requirement layers and the desire to keep the layout as simple as possible to ease construction.

Step 12

Verify the internal stability and calculate the external stability of the cross section using slope stability methods. Since the computations required to perform the required number of calculations would be prohibitive to perform by hand, the analyses are best performed by computer program. Failure modes including, by not limited to, multiple wedge type internal faiure modes, sliding block or translational and circular arc rotational external failure modes should be considered. Site-specific conditions will often control the reinforcement scheme chosen for the final reinforced slope cross sections.



Example Figure 3: Example Miragrid® Reinforcement Layout

References

1. Carter, M. and S.P. Bentley (1991). *Correlations of Soil Properties.* Pentech Press, London. pp. 46, 90-91.
2. Jewell, R.A. (1991). *Revised Design Charts for Steep Reinforced Slopes.* Reinforced Embankments: Theory and Practice. D.A. Shercliff, Editor, Thomas Telford LTD., London. pp. 1-30.
3. *Geotextile Design and Construction Guidelines (1989a)*. NHI Course No. 13213, Pub. No. FHWA-HI-95-039, Revised April 1998.
4. *Mirafi Miragrid Reinforced Soil Submittal Binder*, TC Mirafi. 1998.
5. Geosyntec Consultants. *Final Report Geosynthetic Pullout Testing - Select Miragrid XT Geogrids with Concrete Sand,* April, 2001.
6. Koutsourais, M., Sandri, D., and Swan, R. *Soil Interation Characteristics of Geotextiles and Geogrids*. Conference Proceedings from the Sixth International conference of Geosynthetics, vol 2, pp. 739-744.

Specification for Geosynthetic Used as Soil Reinforcement in Mechanically Stabilized Earth Retaining Structures

1 GENERAL

1.1 SECTION INCLUDES

A. Geosynthetic to provide reinforcement for mechanically stabilized earth retaining structures. The primary function of the geosynthetic is reinforcement.

1.2 RELATED SECTIONS

A. Section 02050 - Basic Site Materials and Methods
B. Section 02100 - Site Remediation
C. Section 02200 - Site Preparation
D. Section 02300 - Earthwork
E. Section 02830 - Retaining Walls

1.3 UNIT PRICES

A. Method of Measurement: By the square meter (or square yard - as indicated in contract documents) including seams, overlaps, and wastage.
B. Basis of Payment: By the square meter (or square yard - as indicated in contract documents) installed.

1.4 REFERENCES

A. AASHTO Standards
1. T88 - Particle Size Analysis of Soils
2. T90 - Determining the Plastic Limit and Plasticity Index of Soils
3. T99 - The Moisture-Density Relations of Soils Using a 5.5lb (2.5 kg) Rammer and a 12 in (305 mm) Drop
4. Standard Specifications for Highway Bridges

B. American Society for Testing and Materials (ASTM):
1. D 123 - Standard Terminology Relating to Textiles
2. D 276 - Test Method for Identification of Fibers in Textiles
3. D 4354 - Practice for Sampling of Geosynthetics for Testing
4. D 4355 - Test Method for Deterioration of Geotextiles from Exposure to Ultraviolet Light and Water (Xenon-Arc Type Apparatus)
5. D 4439 - Terminology for Geotextiles
6. D 4595 - Test Method for Tensile Properties of Geotextiles by the Wide-Width Strip Method
7. D 4759 - Practice for Determining the Specification Conformance of Geosynthetics
8. D 4873 - Guide for Identification, Storage, and Handling of Geotextiles
9. D 5262 - Test Method for Evaluating the Unconfined Tension Creep Behavior of Geosynthetics
10. D 5321 - Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosyn thetic Friction by the Direct Shear Method

C. National Concrete Masonry Association (NCMA) - Design Manual for Segmental Retaining Walls, Second Edition, 1997.

D. Geosynthetic Research Institute:
1. GRI-GT6 - Geotextile Pullout
2. GRI-GT7 - Determination of the Long-Term Design Strength of Geotextiles
3. GRI-GG4 (b) - Determination of the Long-Term Design Strength of Flexible Geogrids
4. GRI-GG5 - Test Method for Geogrid Pullout

E. Federal Highway Administration (FHWA)
1. FHWA NHI-00-043 March 2000 - Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines
2. FHWA NHI-00-044-Sept. 2000 - Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes

F. American Association for Laboratory Accreditation (A2LA)
G. Geosynthetic Accreditation Institute (GAI) - Laboratory Accreditation Program (LAP).

1.5 DEFINITIONS

A. Minimum Average Roll Value (MARV): Property value calculated as mean minus two standard deviations. Statistically, it yields a 97.5 percent degree of confidence that any sample taken during quality assurance testing will exceed value reported.

1.6 SUBMITTALS

A. Submit the following:
   1. Certification: The contractor shall provide to the Engineer a certificate stating the name of the manufacturer, product name, style number, chemical composition of the filaments or yarns and other pertinent information to fully describe the geosynthetic. The Certification shall state that the furnished geosynthetic meets MARV requirements of the specification as evaluated under the Manufacturer's quality control program. The Certification shall be attested to by a person having legal authority to bind the Manufacturer.

1.7 QUALITY ASSURANCE

A. Manufacturer Qualifications:
   1. Geosynthetic Accreditation Institute (GAI)- Laboratory Accreditation Program (LAP)
   2. American Association for Laboratory Accreditation (A2LA)

1.8 DELIVERY, STORAGE, AND HANDLING

A. Geosynthetic labeling, shipment, and storage shall follow ASTM D 4873. Product labels shall clearly show the manufacturer or supplier name, style name, and roll number.
B. Each geosynthetic roll shall be wrapped with a material that will protect the geosynthetic from damage due to shipment, water, sunlight, and contaminants.
C. During storage, geosynthetic rolls shall be elevated off the ground and adequately covered to protect them from the following: site construction damage, precipitation, extended ultraviolet radiation including sunlight, chemicals that are strong acids or strong bases, flames including welding sparks, excess temperatures, and any other environmental conditions that may damage the physical property values of the geosynthetic.

2 PRODUCTS

2.1 MANUFACTURERS

A. MIRAFI® Construction Products
365 South Holland Drive
Pendergrass, GA, 30567
United States of America
1-888-795-0808
1-706-693-2226
1-706-693-2083, fax
www.mirafi.com
www.miragrid.com

2.2 MATERIALS

A. Primary Reinforcement Geosynthetic:
   1. The geosynthetic shall be manufactured with fibers consisting of long-chain synthetic polymers composed of at least 95 percent by weight of polyolefins or polyesters. They shall form a stable network such that the filaments or yarns retain their dimensional stability relative to each other, including selvages.
   2. The geosynthetic shall meet the requirements of Table 1. All numeric values in Table 1 represent MARV in the principal reinforcement direction.

TABLE 1 - PRIMARY REINFORCEMENT GEOSYNTHETIC

Type	Long Term Design Strength (LTDS) kN/m (lbs/ft)	UV Resistance % strength	Ci	Cds
P1	186.32 (12,776)	70	0.8	0.8
P2	130.38 (8,940)	70	0.8	0.8
P3	91.18 (6,252)	70	0.8	0.8
P4	70.91 (4,862)	70	0.8	0.8
P8	32.58 (2,234)	70	0.8	0.8
P9	22.72 (1,558)	70	0.8	0.8
P10	13.84 (949)	70	0.8	0.8

3. Approved primary reinforcement geosynthetics are as follows:

	Geogrid	Geotextile
Type P1	Miragrid® 24XT	Geolon® HS2400
Type P2	Miragrid® 22XT	Geolon® HS1715
Type P3	Miragrid® 20XT	Geolon® HS1150
Type P4	Miragrid® 18XT	Geolon® HS800
Type P5	Miragrid® 10XT	Geolon® HS800
Type P6	Miragrid® 8XT	Geolon® HS600
Type P7	Miragrid® 7XT	Geolon® HS600
Type P8	Miragrid® 5XT	Geolon® HS400
Type P9	Miragrid® 3XT	Geolon® HS400
Type P10	Miragrid® 2XT	Geolon® HS400

4. Long-Term Design Strength (LTDS) and Allowable Tensile Strength (Ta) are determined per AASHTO, FHWA, GRI, and NCMA guidelines where;

$$LTDS = \frac{T_{ULT}}{(RF_{CR})(RF_{ID})(RF_D)}$$

a. T_{ULT}, Ultimate Tensile Strength, shall be the minimum average roll value (MARV) ultimate tensile strength as tested per ASTM D6637 or D4595.

b. RF_{CR}, Reduction Factor for Creep Deformation, is the ratio of TULT to creep limited strength determined in accordance with ASTM D 5262. The results shall be extrapolated for a 75 year design life using elevated temperature and/or stress rupture testing for 10,000 hours or room temperature testing for 65,700 hours per GRI-GG4(b) or GRI-GT7. Total reinforcement strain shall be less than 10% over the 75-year design life.

c. RF_{ID}, Reduction Factor for Installation Damage, shall be determined from construction damage tests for each product or product family proposed for use with project specific, representative or more severe backfill materials and construction techniques. Testing shall be consistent with ASTM D5818, GRI-GG4 (b) or GRI-GT7. A default RFID value of 2.0 shall be used if such testing has not been conducted. The minimum RF_{ID} shall not be less than 1.05.

d. RF_D, Reduction Factor for Durability, shall be determined by testing before and after immersion in the specific liquid environment under consideration. The immersion procedure to be used follows the EPA 9090 Test Method. This testing method shall only be performed by an independent testing laboratory. RFD shall be determined for polymer specific (PET as identified by molecular weight, CEG, and intrinsic viscosity and HDPE and PP as identified by specific gravity and melt flow index) durability testing covering the range of expected soil environments per EPA 9090 testing at temperatures of 23°C and 50°C. In absence of adequate chemical degradation testing and long-term extrapolation a default RFD value of 2.0 shall be used. The minimum RF_D shall not be less than 1.1.

5. Soil Interaction Coefficient, Ci value shall be determined from short-term effective stress pullout tests per ASTM D6706, GRI-GG5 or GRI-GT6 over the range of normal stresses encountered. The maximum pullout force used

to determine Ci shall be limited to the lesser of Ta or the force that yields 1.5 inches displacement. The minimum Ci value shall not be less than 0.8, determined as follows:

$$C_i = \frac{F}{2L\sigma_N \tan\varphi}$$

where F = Pullout Force (lb/ft), per GRI-GG5 or GRI-GT6
L = Geosynthetic Embedment Length in Test (ft)
σ_N = Effective Normal Stress (psf)
φ = Effective Soil Friction Angle, Degrees

6. Direct Sliding Coefficient, Cds value shall be determined in accordance with ASTM D 5321 over the range of normal stresses encountered. The minimum Cds value shall not be less than 0.8, determined as follows:

$$Cds = \frac{R_{ds}}{L\sigma_N \tan\varphi}$$

where R_{ds} = Maximum Shear Resistance (lb/ft), per ASTM D 5321
L = Stationary Length of Geosynthetic (ft)
σ_N = Effective Normal Stress (psf)
φ = Effective Soil Friction Angle, Degrees

7. UV Resistance shall be determined in accordance with ASTM D 4355. Geosynthetics shall retain a minimum of 70% of the Ultimate Tensile Strength per ASTM D 4595 after UV exposure.

B. Secondary Reinforcement Geosynthetic:
1. The geosynthetic shall be manufactured with fibers consisting of long-chain synthetic polymers composed of at least 95 percent by weight of polyolefins or polyesters. They shall form a stable network such that the filaments or yarns retain their dimensional stability relative to each other, including selvages.
2. The geosynthetic shall meet the requirements of Table 2. All numeric values in Table 2 represent MARV in the principal reinforcement direction.

TABLE 2 - SECONDARY REINFORCEMENT GEOSYNTHETIC

Type	Ultimate Tensile Strength ASTM D 4595 kN/m (lbs/ft)	UV Resistance ASTM D 4355 % strength retained
S1	58.33 (4000)	70
S2	39.38 (2700)	70
S3	29.17 (2000)	70
S4	21.88 (1500)	70

3. Approved geosynthetics are as follows:

	Geogrid	Geotextile
Type S1	BasXgrid® 12	Geolon® HP570
Type S2	BasXgrid® 11	Geolon® HP370
Type S3	BasXgrid® 11	Geolon® HP370
Type S4	BasXgrid® 11	Geolon® HP370

2.3 QUALITY CONTROL

A. Manufacturing Quality Control: Testing shall be performed at a laboratory accredited by GAI-LAP and A2LA for tests required for the geosynthetic, at frequency meeting or exceeding ASTM D 4354.

B. Ultraviolet Stability shall be verified by an independent laboratory on the geosynthetic or a geosynthetic of similar construction and yarn type.

3 EXECUTION

3.1 PREPARATION

A. Foundation soil shall be excavated to the lines and grades as shown on the construction drawings or as directed by the Engineer. Over-excavated areas shall be filled with compacted backfill material as per project specifications or as directed by the Engineer. As a minimum, foundation soil shall be proof rolled prior to backfill and geosynthetic placement.

3.2 INSTALLATION

A. Geosynthetic shall be laid at the proper elevation and orientation as shown on the construction drawings or as directed by the Engineer. Contractor shall verify correct orientation of the geosynthetic.

B. Geosynthetic may be temporarily secured in-place with staples, pins, sand bags or backfill as required by fill properties, fill placement procedure or weather condition, or as directed by the Engineer.

C. Primary geosynthetic may not be overlapped or connected mechanically to form splices in the primary strength direction. Single panel lengths are required in the primary strength direction. No overlapping is required between adjacent rolls unless specified by the Engineer.

D. Backfill material shall be placed in lifts and compacted as directed under project specifications. Backfill shall be placed, spread and compacted in such a manner as to minimize the development of wrinkles in and/or movement of the geosynthetic. A minimum fill thickness of 150 mm (6 in) is required prior to the operation of tracked vehicles over the geosynthetic.

E. Turning of tracked vehicles should be kept to minimum to prevent tracks from displacing the fill and damaging the geosynthetic. Rubber tired equipment may pass over the geosynthetic reinforcement at low speeds, less than 16 km/hr (10 mph). Sudden braking and sharp turns shall be avoided. Any geosynthetic damaged during installation shall be replaced by the Contractor at no additional cost to the Owner.

END OF SECTION

INSTALLATION GUIDELINES FOR GEOSYNTHETIC REINFORCED STEEPENED SLOPES

This document is prepared to help ensure that the geosynthetic reinforced soil slope, once installed, will perform its intended design function. To do so, the geosynthetic must be identified, handled, stored, and installed in such a way that its physical property values are not affected and that the design conditions are ultimately met as intended. This document contains information consistent with generally accepted methods of identifying, handling, storing and installing geosynthetic materials. Failure to follow these guidelines may result in the unnecessary failure of the geosynthetic in a properly designed application.

Material Identification, Storage and Handling

The geotextile shall be rolled on cores having strength sufficient to avoid collapse or other damage from normal use. Each roll shall be wrapped with a plastic covering to protect the geosynthetic from damage during shipping and handling, and shall be identified with a durable gummed label or the equivalent, clearly readable on the outside of the wrapping for the roll. The label shall show the manufacturer's name, the style number, and the roll number. Roll identification corresponding to the proposed location of the roll as shown on the construction drawings and as approved by the Engineer, Owner and Contractor can be provided.

While unloading or transferring the geosynthetic from one location to another, prevent damage to the wrapping, core, label, or to the geosynthetic itself. If the geosynthetic is to be stored for an extended period of time, the geosynthetic shall be located and placed in a manner that ensures the integrity of the wrapping, core, and label as well as the physical properties of geosynthetic. This can be accomplished by elevating the geosynthetic off the ground on dunnage and ensuring that it is adequately covered and protected from ultraviolet radiation including sunlight, chemicals that are strong acids or strong bases, fire or flames including welding sparks, temperatures in excess of 60C (140F), and human or animal destruction.

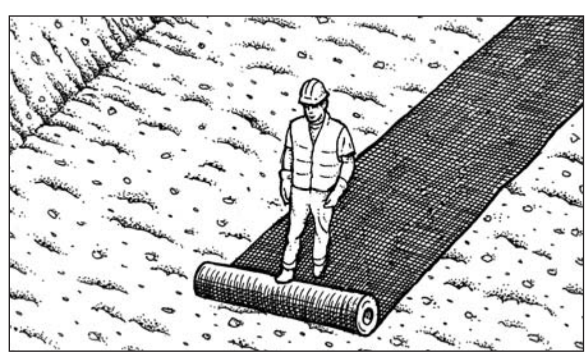
Foundation Soil/Subgrade Preparation

Prepare the surface on which the geosynthetic is to be placed so that no damage to the geosynthetic will occur. Foundation/subgrade soil should be excavated to the lines and grades as shown on the construction drawings or as directed by the Engineer. Over excavated areas should be filled with compacted backfill material as directed by the Engineer. The foundation/subgrade soil should be cleared of all deleterious materials and the surface should be smooth and level such that any shallow depressions and humps do not exceed 6 in (15 cm) in depth and height. The foundation/subgrade soils should be proofrolled prior to geosynthetic and backfill placement. This exercise should be performed prior to each successive geosynthetic layer that is installed.

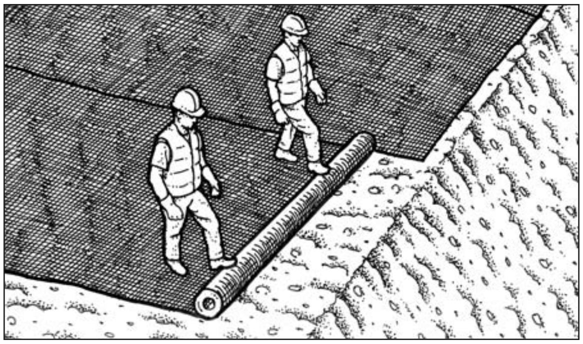
The foundation soils shall be compacted to 95 percent of optimum dry density and plus or minus three (3) percentage points of the optimum moisture content, according to test method ASTM D698 or as specified by the Engineer. It is recommended that cohesive soils be compacted in maximum lifts of 6 in (15 cm) to 8 in (20 cm) and granular soils in lifts of 9 in (23 cm) to 12 in (30 cm) compacted thickness.

Geosynthetic Installation

Before unrolling the geosynthetic, verify the roll identification, length, installation orientation (strength direction) and the installation location using the construction drawings. While unrolling the geosynthetic, inspect it for damage or defects. Damage that occurred during storage, handling or installation shall be repaired as directed by the Engineer.



The geosynthetic should be placed at the correct elevation and orientation as shown on the construction drawings or as directed by Engineer. Correct orientation of the geosynthetic is of utmost importance and shall be verified by the Contractor. The geosynthetic shall be cut to length as shown on the construction drawings using a razor knife, scissors, sharp knife, or other Engineer approved cutting tool.

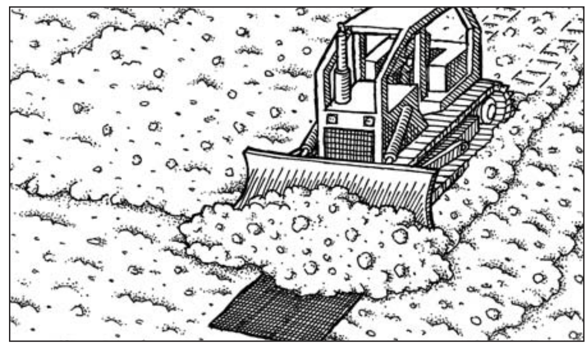


After the geosynthetic has been situated in place it should be tensioned by hand until taut, (i.e. free of wrinkles and lying flat). Adjacent geosynthetic panels, in the case of 100 percent coverage in plan view, should be overlapped as necessary to ensure 100 percent coverage, unless otherwise specified in the construction documents. Geosynthetic panels may be secured in-place with staples, pins, sand bags, or backfill as required by fill properties, fill placement procedures, or weather conditions, or as directed by the Engineer.

The geosynthetic may not be spliced in the primary strength direction through overlap, sewing, or other mechanical connection unless otherwise directed by the Engineer. Therefore, the geosynthetic should be installed in one continuous piece with the primary strength direction extending the full length of the reinforced area.

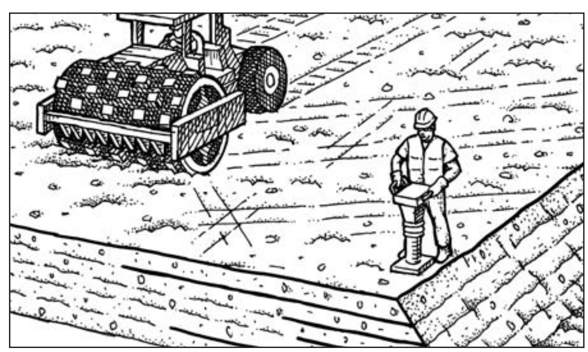
Place only the amount of geosynthetic needed to complete immediately pending work in order to minimize unnecessary exposure to the reinforcement. After a layer of geosynthetic has been placed, the succeeding layer of soil shall be prepared, placed and compacted as indicated in the construction documents. After installation of the soil layer has been completed, the next geosynthetic layer can be installed. The process is repeated for each subsequent layer of geosynthetic and compacted soil.

Backfill Placement



The geosynthetic is laid directly on the horizontal surface of a layer of compacted fill and covered with the next layer of fill. Deployment of fill should be performed as directed by the Engineer in charge of construction quality assurance. Soil fill shall be 95 percent of optimum dry density and plus or minus three (3) percentage points of the optimum moisture content, according to test method ASTM D698 or as specified by the Engineer. It is recommended that cohesive soils be compacted in maximum lifts of 6 in (15 cm) to 8 in (20 cm) and granular soils in lifts of 9 in (23 cm) to 12 in (30 cm) compacted thickness. The minimum compacted fill thickness between adjacent layers of geosynthetic should not be less than 6 in (15 cm) or twice the size of the larger fill particles, whichever is larger. Fill should be compacted as defined by the project specifications or as directed by the Engineer.

Backfill should be placed, spread, and compacted in such a manner that minimizes the development of wrinkles in and/or movement of the geosynthetic. Care should be taken to control the timing and rate of placement of fill material to ensure that construction activities or site vehicles traveling on any exposed geosynthetic do not damage the material.



Backfill within 3 feet (1 m) of the slope face should be compacted with hand compaction equipment. Soil compaction tests shall be performed on every soil lift or as other wise directed by the Engineer. Backfill shall be graded away from the slope crest and rolled at the end of each workday to prevent ponding of water on the surface of the reinforced soil mass. The site shall be maintained to prevent the flow of water from overtopping the slope crest during construction and after the completion of the slope.

Most rubber-tired vehicles can be driven at slow speeds, less than 10 mph and in straight paths over the exposed geosynthetic without causing damage to the geosynthetic. Sudden braking and sharp turning should be avoided. Tracked construction equipment may not be operated directly upon the geosynthetic. A minimum fill soil thickness of six 6 in (15 cm) is required prior to operation of tracked vehicles over the geosynthetic. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and damaging the geosynthetic.

Drainage

Groundwater infiltration and/or surface water runoff can cause saturation of the reinforced fill soil that will significantly reduce soil strength and reduce the stability of the reinforced mass. If the slope was not designed with extra reinforcement to handle these reduced soil strengths, then an engineered drainage system should be provided to prevent the fill from becoming saturated.

Protection of the Slope Face

For reinforced slopes, 1:1 V or flatter, the slope face is hydroseeded and covered with a material that will retain soil particles and promote vegetative growth. For slopes steeper than 1:1 V or in areas where vegetation is difficult to establish, the slope can be treated with durable facing (i.e. wire L-baskets, shotcrete, landscaping timbers, gabions, etc.).