CHAPTER 8

Reinforced Slopes and Embankments

Reinforcement can be used to improve the stability of slopes and embankments, making it possible to construct slopes and embankments steeper and higher than would otherwise be possible. Reinforcement has been used in four distinct types of applications:

1. *Reinforced slopes.* Multiple layers of reinforcement at various elevations within fill slopes have been used to increase the factor of safety for slip surfaces that cut through the reinforcement, making it possible to construct slopes steeper than would be possible without reinforcement.
2. *Reinforced embankments on weak foundations.* Reinforcement at the bottom of an embankment on a weak foundation can increase the factor of safety for slip surfaces passing through the embankment, making it possible to construct the embankment higher than would be possible without reinforcement.
3. *Reinforced soil walls or mechanically stabilized earth walls.* Several different proprietary systems have been developed for reinforced soil walls, which are used as alternatives to conventional retaining walls. Most of the companies that market MSE walls have developed proprietary design procedures. The stability of MSE walls can also be evaluated using the methods described in this chapter.
4. *Anchored walls.* Vertical soldier pile walls or slurry trench concrete walls can be ''tied back'' or anchored at one or more levels to provide vertical support for excavations or fills. Anchored walls have been used in both temporary and permanent applications. The methods described in this chapter can be used to evaluate the stability of anchored walls.

LIMIT EQUILIBRIUM ANALYSES WITH REINFORCING FORCES

Reinforced slopes can be analyzed using the procedures described in Chapter 6 by including the reinforcement forces in the analyses as known forces. Zornberg et al. (1998*a*,*b*) have shown through centrifuge tests that limit equilibrium analyses provide valid indications of factor of safety and failure mechanisms for reinforced slopes. Their analyses, which agreed well with the results of their tests, were performed using peak values of ϕ' rather than the lower critical-state friction angle of the backfill soil.

The amount of force required to achieve a target value of factor of safety can be determined using repeated trials, varying the magnitude of the force until the factor of safety computed is the one desired. Some computer programs can perform this operation automatically—the input is the desired factor of safety, and the output is the required reinforcement force. This type of program is better adapted to design of reinforced slopes, since there is no need for repeated analyses.

FACTORS OF SAFETY FOR REINFORCING FORCES AND SOIL STRENGTHS

Two methods have been used for limit equilibrium analyses of reinforced slopes.

- *Method A.* The reinforcement forces used in the analysis are *allowable* forces and *are not divided* by the factor of safety calculated during the slope stability analysis. Only the soil strength is divided by the factor of safety calculated in the slope stability analysis.

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- *Method B.* The reinforcement forces used in the analysis are *ultimate* forces, and *are divided* by the factor of safety calculated in the slope stability analysis. Both the reinforcing force and the soil strength are divided by the factor of safety calculated in the slope stability analysis.

Method A is preferable, because the soil strength and the reinforcement forces have different sources of uncertainty, and they therefore involve different amounts of uncertainty. Factoring them separately makes it possible to reflect these differences.

When a computer program is used to analyze reinforced slopes, it is essential to understand which of these methods is being used within the program, so that the appropriate measure of reinforcing force (allowable force or ultimate force) can be specified in the input for the analysis.

If the documentation of a computer program does not specify whether the reinforcement force should be allowable or ultimate, this can be deduced from the equations employed to compute the factor of safety.

Method A Equations

If the factor of safety for circular slip surfaces is defined by an equation of the form

$$F = \frac{\text{soil resisting moment}}{\text{overturning moment} - \text{reinforcement moment}} \tag{8.1}$$

or, more generally, if the factor of safety is defined by an equation of the form

$$F = \frac{\text{shear strength}}{\text{shear stress required for equilibrium} - \text{reinforcement resistance}} \tag{8.2}$$

the program uses method A, and the reinforcement forces specified in the input should be allowable forces, denoted here as P_{all}.

Method B Equations

If the factor of safety for circular slip surfaces is defined by an equation of the form

$$F = \frac{\text{soil resisting moment} + \text{reinforcement moment}}{\text{overturning moment}} \tag{8.3}$$

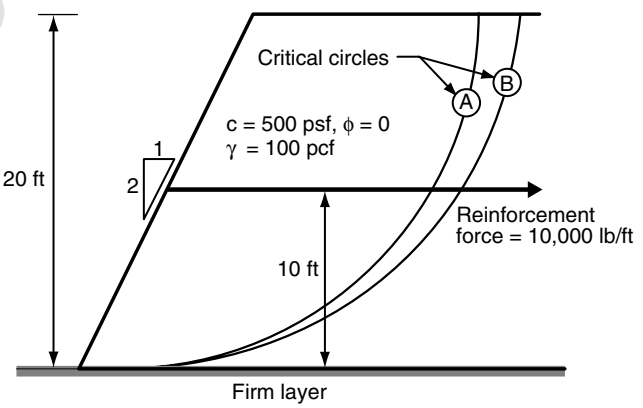
or, more generally, by an equation of the form

$$F = \frac{\text{shear strength} + \text{reinforcement resistance}}{\text{shear stress required for equilibrium}} \tag{8.4}$$

the program uses method B, and the reinforcement forces specified in the input should be the unfactored long-term load capacity of the reinforcement, denoted here as P_{lim}.

If it is not clear which method is used by a computer program, this can be determined by analyzing the reinforced slope problem shown in Figure 8.1. This slope is 20 ft high and is inclined at 2.0 vertical on 1.0 horizontal. The soil within the slope is uniform, with $\gamma =$ 100 pcf, $\phi = 0$, and $c = 500$ psf. There is a firm layer beneath the toe. A reinforcing force of 10,000 lb/ft acts horizontally at midheight, 10 ft above the toe of the slope. Results for two analyses are shown in Figure 8.1. The analyses considered only circular slip surfaces tangent to the top of the firm layer. The critical circles, located as shown in Figure 8.1, exit slightly above the toe of the slope.

The method used by any computer program can be determined based on the computed factor of safety:



Method A: The reinforcement force used in the analysis is an allowable force, and is not divided by the factor of safety calculated during the slope stability analysis.
The factor of safety computed using method A is F = 2.19.

Method B: The reinforcement force used in the analysis is an ultimate force, and is divided by the factor of safety calculated during the slope stability analysis.
The factor of safety computed using method B is F = 1.72.

Figure 8.1 Check problem for determining whether a computer program is using method A or method B for analysis of reinforced slopes.

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- If the program computes $F = 2.19$, the program uses method A. Allowable reinforcement forces should be used with the program.

- If the program computes $F = 1.72$, the program uses method B. Ultimate reinforcement forces should be used with the program.

Slight deviations from $F = 2.19$ or $F = 1.72$ can be expected, depending on the number of slices used by the program, and the method used to locate the critical circle. The differences should be no more than 1 or 2%, however.

TYPES OF REINFORCEMENT

The principal types of reinforcing materials that have been used for slopes and embankments are geotextile fabrics, geogrids, steel strips, steel grids, and high-strength steel tendons. *Geotextiles* are manufactured by weaving polymeric fibers into a fabric or by matting the fibers together to form a continuous nonwoven fabric. Woven geotextiles are stiffer and stronger than nonwoven geotextiles and more useful for reinforced slope applications. *Geogrids* are manufactured by stretching sheets of polymer plastic in one or both directions to form a high-strength grid. Stretching the polymeric materials makes them stiffer and stronger. *Galvanized* or *epoxy-coated steel strips* have been used for slope reinforcement. The strips usually have raised ribs to increase their pullout resistance. Welded mild steel mats or grids have also been used for reinforcing slopes and embankments.

Key sources of information about geosynthetic reinforcing for slopes are the book by Koerner (1998), which contains a great deal of information on the fundamental characteristics and properties of polymers, geotextiles, and geogrids, and the FHWA (2000) publication entitled *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes: Design and Construction Guidelines,* which covers a wide range of subjects related to geotextiles, geogrids, steel strips, and steel grids, and their use in reinforced walls and slopes.

REINFORCEMENT FORCES

The long-term capacity of reinforcement, denoted here as T_{lim}, depends on the following factors:

- *Tensile strength.* For steel, the tensile strength is the yield strength. For geosynthetics, the tensile strength is measured using short-term wide-width tensile tests.
- *Creep characteristics.* Steel does not creep appreciably, but geosynthetic materials do. The tensile loads used for design of geotextile- and geogrid-reinforced walls must be reduced to values lower than those measured in short-term tensile tests, to stresses that are low enough so that little or no creep deformation will occur over the design life of the structure.
- *Installation damage.* Geotextiles and geogrids are subject to damage during installation that results in holes and tears in the material. Epoxy-coated and PVC coatings on steel are subject to damage during installation, and galvanization is therefore preferred for corrosion protection.
- *Durability.* The mechanical properties of geosynthetics are subject to deterioration during service as a result of attack by chemical and biological agents. Steel is subject to corrosion.
- *Pullout resistance.* Near the ends of the reinforcement, capacity is limited by the resistance to pullout, or slip between the reinforcement and the soil within which it is embedded.
- *Reinforcement stiffness and tolerable strain within the slope.* To be useful for slope reinforcement, the reinforcing material must have stiffness as well as strength. A very strong but easily extensible rubber band would not provide effective reinforcement, because it would have to stretch so much to mobilize its tensile capacity that it would not be able to limit the deformation of the slope.

Values of T_{lim}, the long-term capacity of reinforcing materials, must satisfy the following three criteria:

1. $T_{lim} \leq$ capacity determined by short-term tensile strength, creep, installation damage, and deterioration of properties over time.
2. $T_{lim} \leq$ capacity determined by pullout resistance.
3. $T_{lim} \leq$ capacity determined by stiffness and tolerable strain.

Methods of applying these requirements to geosynthetics and steel reinforcing are described in the following sections.

Criterion 1: Creep, Installation Damage, and Deterioration in Properties over Time

Geotextiles and geogrids. The effects of creep, installation damage, and long-term deterioration on geosynthetic materials can be evaluated using the expression

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$$T_{\text{lim}} = \frac{T_{\text{ult}}}{(\text{RF}_{\text{CR}})(\text{RF}_{\text{ID}})(\text{RF}_{\text{D}})} \tag{8.5}$$

where T_{lim} is the long-term limit load (F/L); T_{ult} the short-term ultimate strength, measured in a wide-strip tension test (F/L); RF_{CR} the strength reduction factor to allow for creep under long-term load; RF_{ID} the strength reduction factor to allow for installation damage; RF_{D} the strength reduction factor to allow for deterioration in service. Values of RF_{CR}, RF_{ID}, and RF_{D} recommended by the FHWA are given in Table 8.1. The units of T_{lim} and T_{ult} are force per unit length of reinforced slope.

Steel reinforcement. Steel reinforcing does not creep appreciably and is not subject to installation damage. Epoxy and PVC coatings are subject to installation damage, but galvanized coating is not. The effects of long-term deterioration of steel due to corrosion can be evaluated using the expression

$$T_{\text{lim}} \leq A_C f_y \tag{8.6}$$

where T_{lim} is the allowable long-term reinforcement tension load (F/L); A_c the cross-sectional area of reinforcement after corrosion, calculated by reducing the metal thickness by the loss expected during the life of the installation [A_c is the area per unit length of slope (L^2/L)]; and f_y the yield strength of steel (F/L^2). Corrosion rates for steel in mildly corrosive backfill materials are given in Table 8.2.

Table 8.1 Reduction Factors for Tensile Strengths of Geotextiles and Geogrids for Use in Eq. (8.5)

Reduction for:	Factor	Polymer	Range of values[a]
Creep	RF_{CR}	Polyester	1.6–2.5
		Polypropylene	4.0–5.0
		Polyethylene	2.6–5.0
Installation damage	RF_{ID}	Any polymer	1.1–3.0
Deterioration in service	RF_{D}	Any polymer	1.1–2.0

[a]These values (from FHWA, 2000) are applicable to reinforcement in granular soils with maximum particle sizes up to 19 mm, values of pH from 4.5 to 9.0, and in-service temperatures below 30°C. Geotextiles weighing less than 270 g/m^2 are subject to greater damage during installation and should not be used for reinforcement.

Table 8.2 Corrosion Rates for Steel Reinforcement in Mildly Corrosive Backfill

Material corroding	Period of time	Corrosion rate (μm/yr)	Corrosion rate[a] (in./yr)
Zinc	First two years	15	5.9×10^{-4}
Zinc	Thereafter	4	1.6×10^{-4}
Carbon steel	Thereafter	12	4.7×10^{-4}

[a]These corrosion rates are applicable for steel reinforcement in backfill with these electrochemical properties: Resistivity greater than 3000 $\Omega \cdot$ cm, pH between 5 and 10, chlorides less than 100 ppm, sulfates less than 200 ppm, organic content less than 1%.

Source: After FHWA (2000).

Criterion 2: Pullout Resistance

To develop tensile capacity, reinforcement must be restrained sufficiently by friction in the soil. The maximum possible resistance (T_{po}) is proportional to the effective overburden pressure. T_{po} begins from zero at the end of the reinforcement, where the embedded length is zero and increases with distance from the end, as shown in Figure 8.2. The slope of the curve representing the variation of T_{po} with distance can be expressed as

$$\frac{dT_{\text{po}}}{dL} = 2\gamma z \alpha F^* \tag{8.7}$$

where T_{po} is the pullout resistance (F/L); L the length of embedment, or distance from the end of the reinforcement (L); γ the unit weight of fill above the reinforcement (F/L^3); z the depth of fill above the reinforcement (L); α the adjustment factor for extensible reinforcement (dimensionless); and F^* the pullout resistance factor (dimensionless).

Values of α and F^* recommended by the FHWA (2000) are listed in Table 8.3. These values are conservative estimates. Larger values may be applicable and can be used if they are supported by tests performed on the specific soil and reinforcing material.

Equation (8.7) gives the slope of the pullout resistance curve at any location. If the thickness of fill above the reinforcement is constant, the slope of the pullout curve is constant, and the pullout resistance can be expressed as

$$T_{\text{po}} = 2\gamma z \alpha F^* L_e \tag{8.8}$$

where L_e is the distance from the end of the reinforcement or length of embedment (L).

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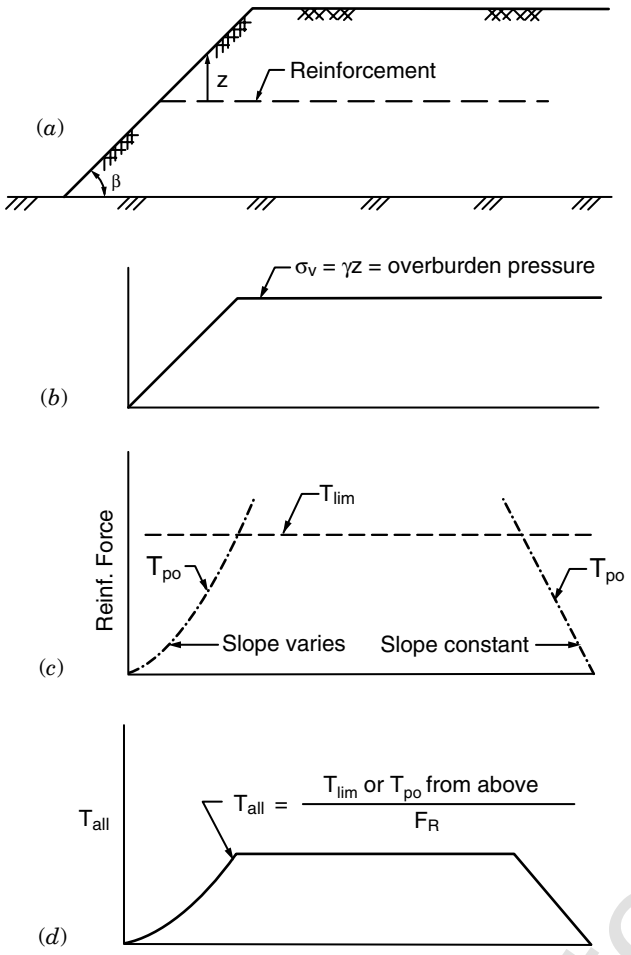


Figure 8.2 Variation of T_{lim} and T_{all} with distance along reinforcement.

Table 8.3 Pullout Resistance Factors α and F^* for Use in Eq. (8.7)

Pullout resistance factor[a]	Type of reinforcement	Resistance factor value
α	Geotextiles	0.6
	Geogrids	0.8
	Steel strips and steel grids	1.0
F*	Geotextiles	0.67 tan ϕ
	Geogrids	0.8 tan ϕ
	Steel strips and steel grids	1.0 tan ϕ

[a]Higher values of F^* generally apply at depths shallower than 6 m. Larger values of both α and F^* may be applicable and can be used if they are supported by tests performed on the specific soil and reinforcing material.

Source: FHWA (2000).

If the overburden pressure increases with distance from the end of the reinforcement, as it does beneath the slope on the left in Figure 8.2, the slope of the T_{po} curve also increases with distance, and the pullout resistance diagram is a curve. In this case the pullout resistance can be expressed as

$$T_{po} = \gamma \tan \beta \; \alpha F^* (L_e)^2 \tag{8.9}$$

where β is the slope angle in degrees, as shown in Figure 8.2.

Criterion 3: Reinforcement Stiffness

Reinforcing materials must be stiff enough so that reinforcement forces can be mobilized without excessive strain. The value of T_{lim} should not exceed the product of the long-term secant modulus of the reinforcement multiplied by the tolerable strain for the slope:

$$T_{lim} \leq E_{secant} \varepsilon_{tolerable} \tag{8.10}$$

where E_{secant} is the secant modulus of reinforcing at axial strain = $\varepsilon_{tolerable}$ (F/L) and $\varepsilon_{tolerable}$ is the strain within the slope at the location of the reinforcing that can be tolerated without excessive slope deformation or failure (dimensionless).

Steel reinforcing is often described as *inextensible* because it stiffness is very high compared to its yield strength. With a modulus equal to 500 to 1000 times its yield strength, the yield strength of steel is mobilized at a strain of only 0.1 to 0.2%, far less than the tolerable strains for soil reinforcing applications. As a result, criterion 3 never governs the value of T_{lim} for steel reinforcing. The stiffness of geosynthetic materials, on the other hand, may be low enough so that criterion 3 governs the value of T_{lim} for applications where the tolerable strain is small.

As shown in Figure 8.3, E_{secant} is the slope of a line extending from the origin to the point on the T–ε curve where the strain is equal to $\varepsilon_{tolerable}$. Note that the units of E_{secant}, like the units of T_{lim}, are force per unit length.

Values of tolerable strain are based on the results of finite element analyses (Rowe and Soderman, 1985); and on experience (Fowler, 1982; Christopher and Holtz, 1985; Haliburton et al., 1982; Bonaparte et al., 1987). A summary of published recommendations is given in Table 8.4.

ALLOWABLE REINFORCEMENT FORCES AND FACTORS OF SAFETY

The preceding section is concerned with the long-term capacity of reinforcement (T_{lim}). These values of T_{lim}

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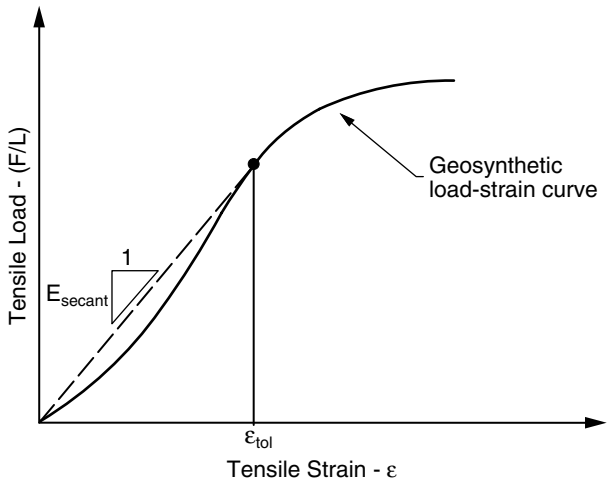


Figure 8.3 Definition of E_{secant} for geosynthetic reinforcement.

Table 8.4 Tolerable Strains for Reinforced Slopes and Embankments

Application	ε_{tol} (%)
Reinforced soil walls	10
Reinforced slopes of embankments on firm foundations	10
Reinforced embankments on nonsensitive clay, moderate crest deformations tolerable	10
Reinforced embankments on nonsensitive clay, moderate crest deformations not tolerable	5–6
Reinforced embankments on highly sensitive clay	2–3

Source: Compiled from Fowler (1982), Christopher and Holtz (1985), Haliburton et al. (1982), Rowe and Soderman (1985), and Bonaparte et al. (1987).

reflect consideration of long-term loading, installation damage, deterioration in properties over time, pullout resistance, and tolerable strains, but they do not include a factor of safety.

The allowable load assigned to reinforcing materials should include a factor of safety, as indicated by

$$T_{all} = \frac{T_{lim}}{F_R} \tag{8.11}$$

where T_{all} is the allowable long-term reinforcement force (F/L) and F_R is the factor of safety for reinforcement force. The value of F_R should reflect (1) the degree of uncertainty involved in estimating the value of T_{lim}, (2) the degree of uncertainty involved in estimating the load that the reinforcement must carry, and (3) the consequences of failure. Recommended values of F_R are given in Table 8.5.

ORIENTATION OF REINFORCEMENT FORCES

Various orientations of reinforcement forces have been suggested (Schmertmann et al., 1987; Leshchinsky and Boedeker, 1989; Koerner, 1998; FHWA, 2000). The extremes are (1) reinforcement forces that are aligned with the original orientation of the reinforcement, and (2) reinforcement forces that are parallel to the slip surface. The latter assumption, which results in larger factors of safety, has been justified by the concept that the reinforcement will be realigned where the slip surface crosses the reinforcement. This is more likely if the reinforcement is very flexible. The assumption that the orientation of the reinforcement force is the same as the orientation of the reinforcement is more conservative, is supported by the findings of Zornberg et al. (1998a), and is the more logical, reliable choice. This is the approach recommended here.

REINFORCED SLOPES ON FIRM FOUNDATIONS

Reinforcement in embankments can be used to construct slopes steeper than would be possible without reinforcing. Usually, several layers of reinforcing are used, spaced more closely near the base of the slope and farther apart near the top, as shown in Figure 8.4. Secondary shorter lengths of lower-strength reinforcement, between the primary reinforcement layers, can be used to improve surficial stability (FHWA, 2000). Zornberg et al. (1998a) showed that such layers near the bottom of the slope significantly enhance stability if they are wrapped around at the face and overlap the adjacent layers. In this configuration they are anchored firmly and not subject to pullout failure.

Table 8.5 Recommended Values of F_R

Consequences of failure	Uncertainties in T_{lim} and load in reinforcement	Appropriate value of F_R
Minimal	Small	1.5
Minimal	Large	2.0
Great	Small	2.0

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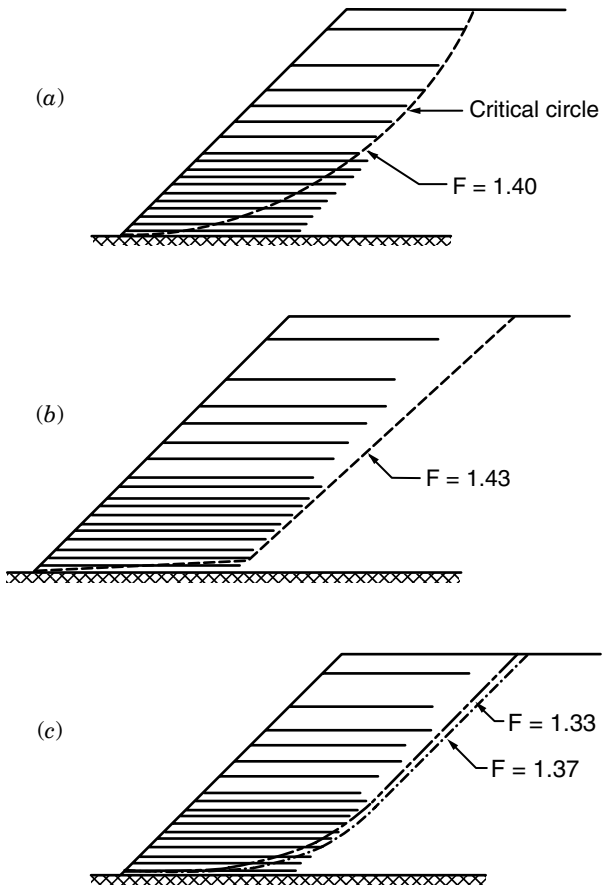


Figure 8.4 Limit equilibrium analyses of a reinforced slope using circular, wedge, and smooth noncircular slip surfaces: (After Wright and Duncan, 1991). (*a*) critical circular slip surface; (*b*) critical two-part wedge; (*c*) critical noncircular slip surfaces.

The stability of reinforced slopes can be evaluated using the procedures outlined in Chapter 6. An example is shown in Figure 8.4. It can be seen that the factor of safety varies slightly with the shape of the slip surface, from $F = 1.43$ for the most critical two-part wedge slip surface, to $F = 1.33$ for the most critical noncircular slip surface, a difference of about 7%. The most critical circle gives a factor of safety $F = 1.40$, which is sufficiently accurate for practical purposes.

Schmertmann et al. (1987) Charts

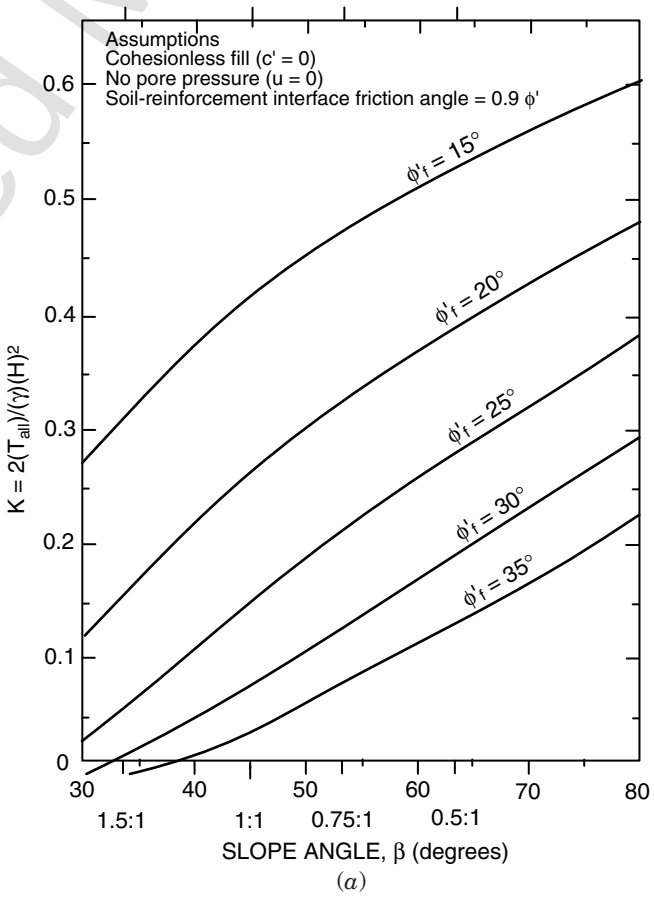
Before an analysis of the type illustrated in Figure 8.4 can be performed, the strength, length, and spacing of the reinforcement must be estimated. Determining these by trial and error can be time consuming because many trials can be required to determine strength, length, and spacing.

Designing reinforced slopes is facilitated greatly by slope stability charts of the type developed by Schmertmann et al. (1987), which are shown in Figure 8.5*a* and *b*. Figure 8.5*a* can be used to determine the total reinforcing force, and Figure 8.5*b* can be used to determine the length of reinforcing required for a given factor of safety. The terminology used in these charts is:

$$K = \frac{2T_{\text{all}}}{\gamma(H')^2} = \text{dimensionless force coefficient} \tag{8.12}$$

$$H' = H + \frac{q}{\gamma} = \text{effective height, including effect of surcharge (L)} \tag{8.13}$$

$$\phi'_f = \arctan\frac{\tan\phi'}{F} = \text{factored friction angle (degrees)} \tag{8.14}$$

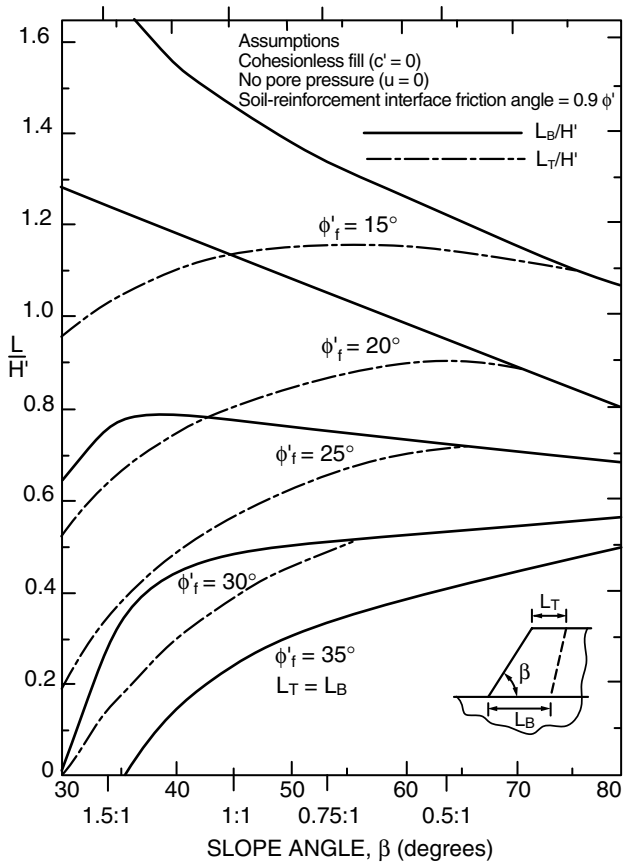


(*a*)

Figure 8.5 (*a*) Reinforcement force coefficients; (*b*) reinforcement length coefficients. (After Schmertmann et al., 1987.)

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(*b*)

Figure 8.5 (*Continued*)

L_B = required length of reinforcement at the bottom of the slope (L)

L_T = required length of reinforcement at the top of the slope (L)

β = slope angle (degrees)

γ = unit weight of soil (F/L^3)

q = surcharge pressure (F/L^2)

u = pore pressure, assumed to be zero throughout the slope

Example. As an example of the use of these charts, consider the slope shown in Figure 7.27. Pertinent parameter values from Figure 7.27 are H = 24 ft, γ = 130 pcf, ϕ = 37°, β = arctan(1.25) = 39°, q = 0, c = 0, and u = 0. The factor of safety computed in the STABGM user's manual and Chapter 7 is F = 1.71.

The charts in Figure 8.5*a* and *b* can be used to determine the total reinforcement force and the length of reinforcement required for a factor of safety F = 1.71 using the following steps:

1. Compute ϕ_f' = arctan tan ϕ'/F = arctan 0.75/1.71 = 24°.
2. Compute H' = H + q/γ = 24 ft + 0/130 pcf = 24 ft.
3. From Figure 8.5*a*, determine K = 0.11.
4. Compute T_{all} = (0.11)($\frac{1}{2}$)(130 pcf)(24 ft)^2 = 4100 lb/ft.

In Figure 7.27 there are five active layers of reinforcement. The sixth layer shown in the STABGM manual and Figure 7.27, at the elevation of the toe, does not cut across any of the slip surfaces shown in Figure 7.28 and therefore does not influence the factor of safety. The factors of safety computed using STABGM and UTEXAS4 are the same whether this bottom layer of reinforcement is included or not. The total reinforcement force from step 4, 4100 lb/ft, agrees well with the total of 4000 lb/ft provided by the five active layers of reinforcement.

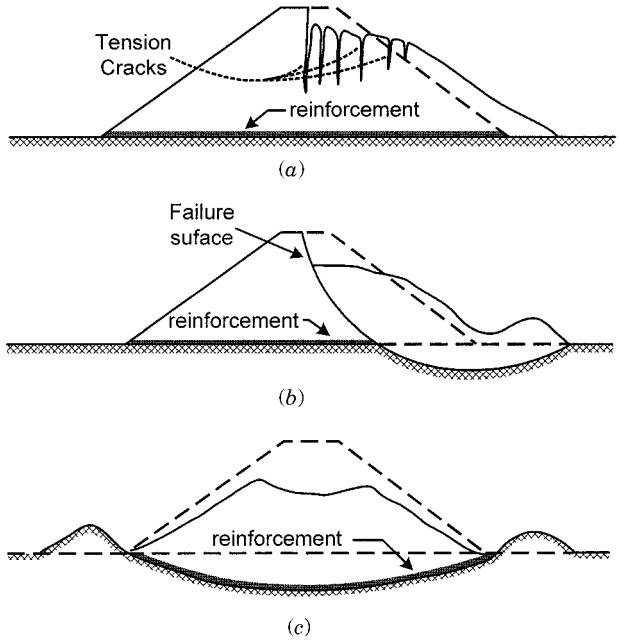


Figure 8.6 Potential modes of failure of reinforced embankments: (*a*) block sliding outward along reinforcement with slumping of the crest; (*b*) foundation failure with rotational sliding through embankment; (*c*) excessive elongation of reinforcement. (Modified from Haliburton et al., 1978).

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5. From Figure 8.5*b* determine $L_B/H' = 0.88$, $L_T/H' = 0.55$.
6. Compute $L_B = (0.88)(24 \text{ ft}) = 21 \text{ ft}$ and $L_T = (0.55)(24 \text{ ft}) = 13 \text{ ft}$.

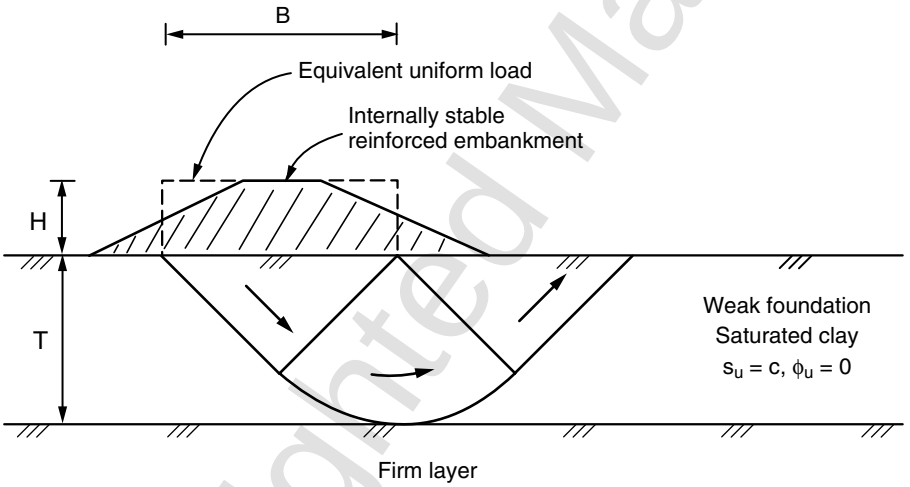
In Figure 7.27, $L_T = L_B = 20$ ft. The results from Figure 8.5*b* indicate that the reinforcement could be somewhat shorter at the top of the slope.

EMBANKMENTS ON WEAK FOUNDATIONS

Reinforcement near the base of an embankment can be used to improve stability with regard to spreading of the embankment and with regard to shear failure through the embankment and foundation. With reinforcement at the bottom of the embankment, the slopes can be made as steep as for an embankment constructed on a firm foundation. The volume of the embankment and the total load it imposes on the foundation can be reduced and its height can be increased.

Reinforced embankments have been used at a number of sites where weak foundations posed difficult stability problems, including Almere in the Netherlands (Rowe and Soderman, 1985); Mohicanville Dike 2 in Ohio (Duncan et al., 1988; Franks et al., 1988, 1991); St. Alban in Canada (Busbridge et al., 1985; Schaefer and Duncan, 1986, 1987); Hubrey Road in Ontario, Canada (Rowe and Mylleville, 1996); and Sackville, New Brunswick, Canada (Rowe et al., 1996).

Modes of failure. Potential modes of failure of reinforced embankments on weak foundations have been discussed by Haliburton et al. (1978) and by Bonaparte and Christopher (1987). Three possible modes of failure are shown in Figure 8.6.



$$F = \frac{q_{ult}}{q}$$

$q_{ult} = cN_C$ (F/L^2)

c = average undrained strength of foundation (F/L^2)

$q = \gamma H$ (F/L^2)

γ = total unit weight of embankment fill (F/L^6)

H = embankment height (L)

Ratio B/T	Value of N_c (NAVFAC, 1986)
≤ 1.4	5.1
2.0	6.0
3.0	7.0
5.0	10.0
10.0	17.0
20.0	30.0

Figure 8.7 Bearing capacity mode of failure of a strongly reinforced embankment on a weak foundation.

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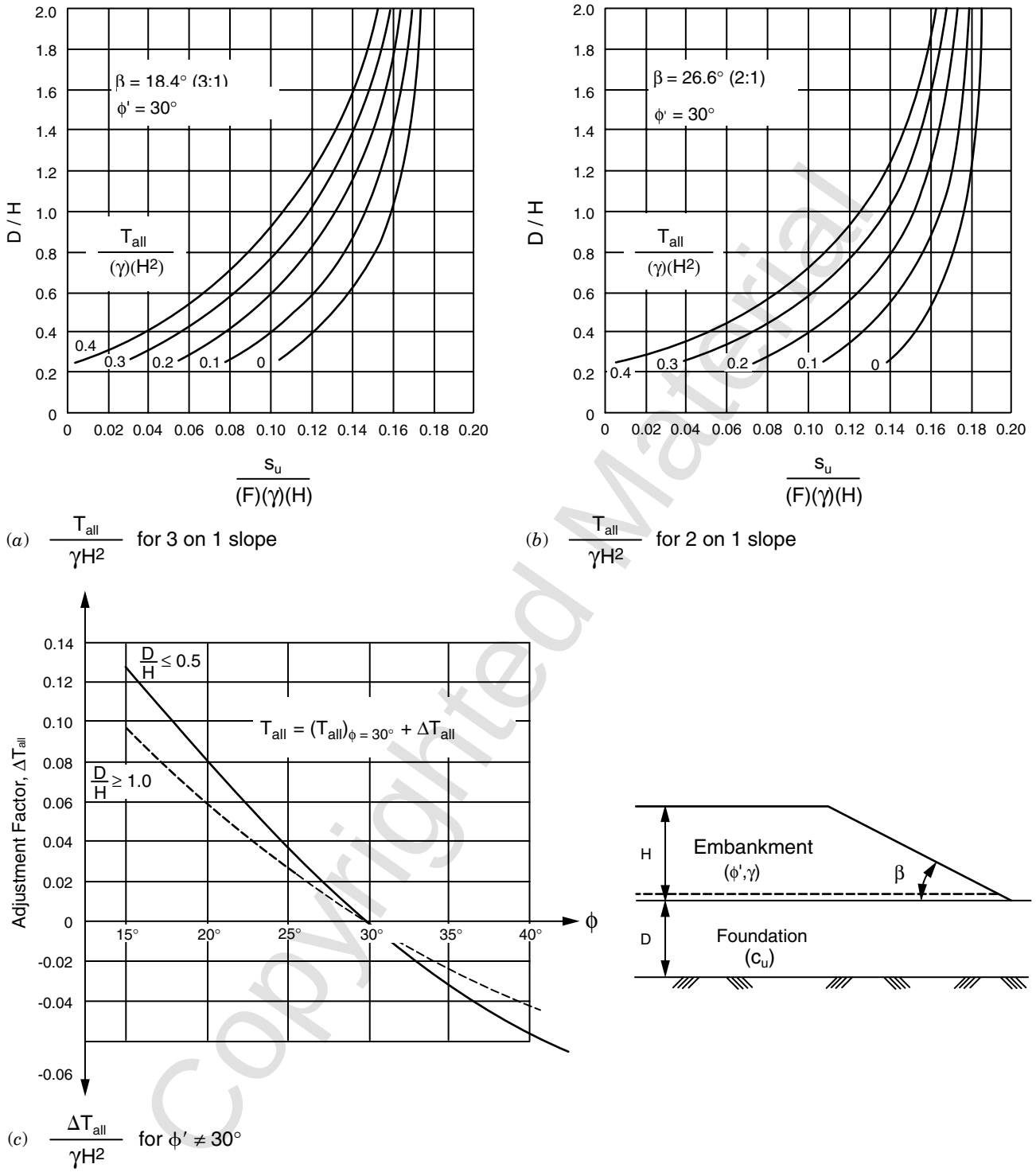


Figure 8.8 Stability charts for embankments on weak foundations. (After Bonaparte and Christopher, 1987.)

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Figure 8.6*a* shows the embankment sliding across the top of the reinforcing. This mode of failure is most likely if the interface friction angle between the embankment and the reinforcement is low, as it may be with geotextile reinforcement. A wedge analysis can be used to assess the safety of the embankment with regard to this mode of failure.

Figure 8.6*b* shows a shear surface cutting across the reinforcement and into the weak foundation. This mode of failure can occur only if the reinforcement ruptures or pulls out. Safety with regard to this mode of failure can be evaluated using circular, wedge, or noncircular slip surfaces, including reinforcement forces in the analysis as discussed previously.

Figure 8.6*c* shows large settlement of the embankment resulting from excessive elongation of the reinforcement. This mode of failure can occur if the strain in the reinforcement required to mobilize the reinforcement load is too large. Satisfying limit load criterion 3, discussed previously, will prevent this type of failure.

Even if an embankment is completely stable internally, it may still be subject to bearing capacity failure, as shown in Figure 8.7. This mode of failure can be analyzed using bearing capacity theory. If the foundation thickness (T) is small compared to the width of the equivalent uniform embankment load (B), the value of the bearing capacity factor N_c increases, as shown in the tabulated values in Figure 8.7 and the factor of safety with respect to bearing capacity failure also increases. Therefore, the shallower is the weak foundation, the less likely is the bearing capacity mode of failure.

Bonaparte and Christopher (1987) charts. Preliminary estimates of the reinforcement force required for a given factor of safety can be made using the stability charts shown in Figure 8.8, which were developed by Bonaparte and Christopher (1987). The terminology used in these charts is:

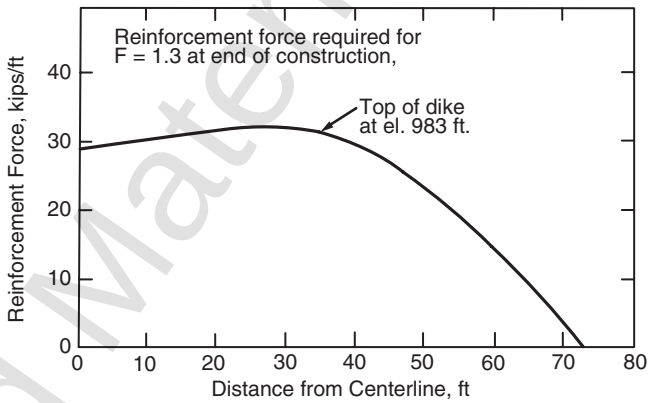


Figure 8.10 Required reinforcement force for Mohicanville Dike 2. (After Fowler et al., 1983.)

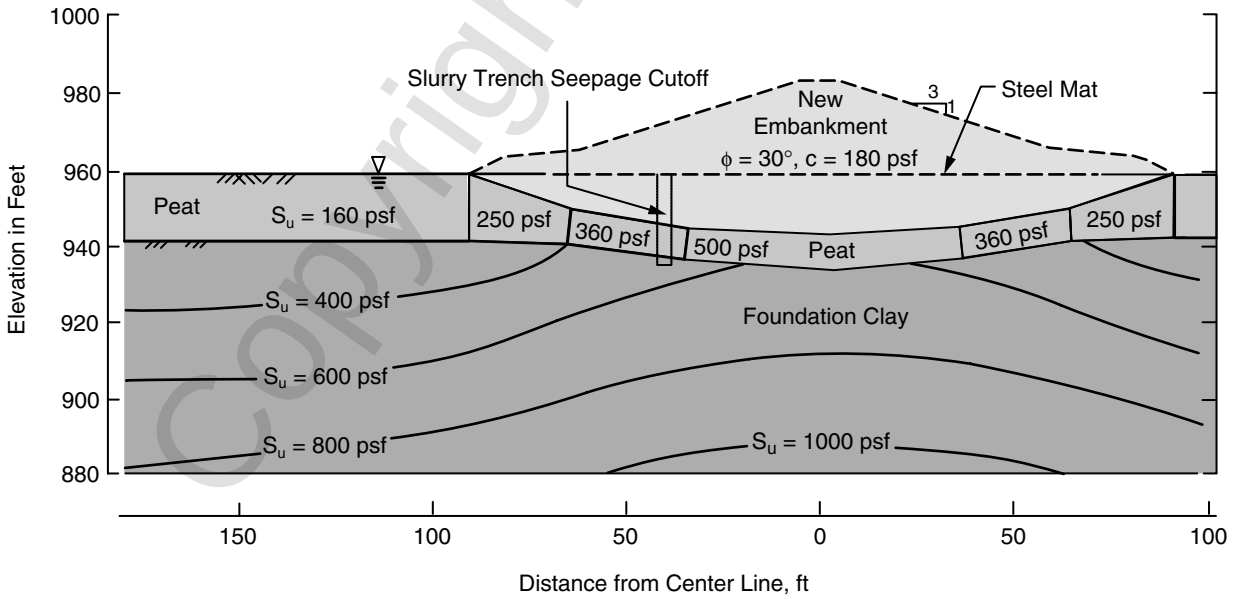


Figure 8.9 Cross section through Mohicanville Dike 2. (After Collins et al., 1982.)

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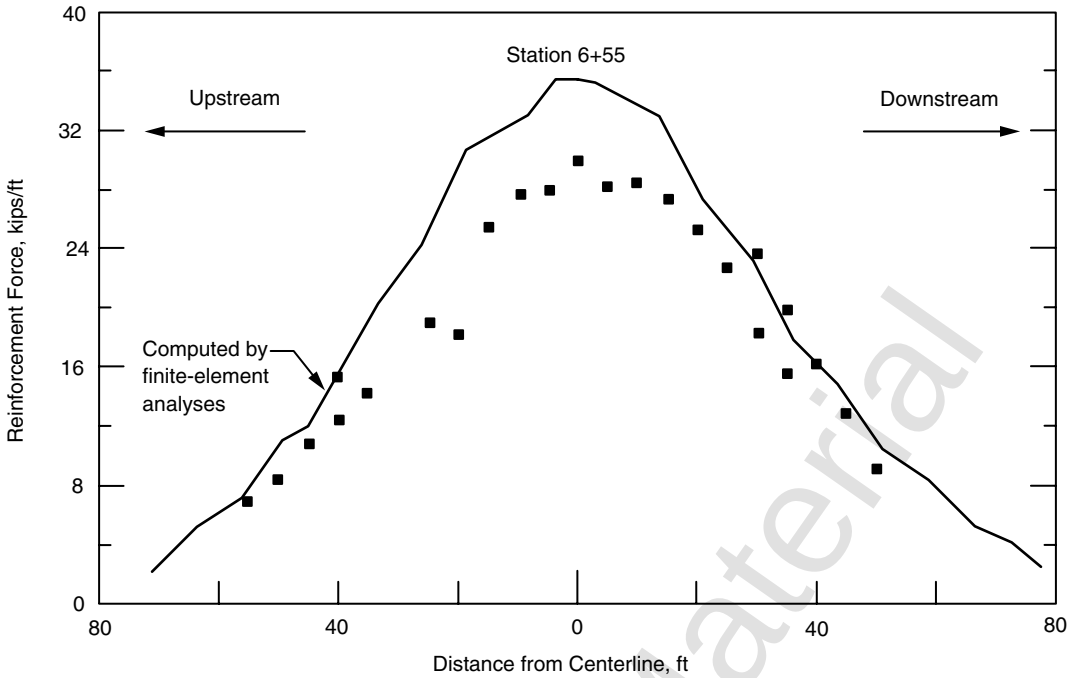


Figure 8.11 Reinforcement force distribution about the centerline for Station 6+55. (After Duncan et al., 1988.)

$\frac{T_{all}}{\gamma H^2}$ = dimensionless reinforcement force coefficient (8.15)

$\frac{s_u}{F\gamma H}$ = dimensionless factored stability coefficient (8.16)

T_{all} = allowable reinforcement force (F/L)

ΔT_{all} = change in T_{all} for embankment $\phi \neq 30°$

γ = total unit weight of embankment (F/L^3)

H = height of embankment (L)

s_u = undrained shear strength of foundation soil (foundation $\phi = 0$)

D = foundation depth (L)

β = slope angle (degrees)

Example. As an example of the use of these charts, consider the Mohicanville Dike No. 2 embankment shown in Figure 8.9. The Mohicanville project is described in more detail in the next section. Pertinent parameter values for the embankment and foundation are $H = 24$ ft, $\gamma = 136$ pcf, $\phi' = 32°$ for the embankment fill, embankment slope angle $\beta = \arctan(0.33) = 18°$, average foundation shear strength = 700 psf, $D = 80$ ft, $s_u/F\gamma H = 700/(1.3)(136)(24) = 0.16$ for factor of safety on soil shear strength $F = 1.3$.

The cohesion of the embankment fill ($c = 200$ psf) is neglected because the charts in Figure 8.8 were developed for cohesionless embankment fill. The charts assume that the reinforcement is placed at the bottom of the embankment, where it is most effective in improving stability. The charts can be used to determine the magnitude of the reinforcement force required for a given factor of safety using the following steps:

1. Compute $D/H = (80 \text{ ft})/(24 \text{ ft}) = 3.3$.
2. Compute

$$\frac{s_u}{F\gamma H} = \frac{700 \text{ psf}}{(1.3)(136 \text{ pcf})(24 \text{ ft})} = 0.16$$

3. From Figure 8.8*a*, estimate $T_{all}/\gamma H^2 = 0.4$ (the value of D/H is above the top of the chart, and $T_{all}/\gamma H^2 = 0.4$ was estimated by extrapolation).
4. From Figure 8.8*c*, determine $\Delta T_{all}/\gamma H^2 = -0.01$.
5. Compute $T_{all}/\gamma H^2 = 0.40 - 0.01 = 0.39$.

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6. Compute $T_{all} = (0.39)(136)(24)^2 = 31{,}000$ lb/ft.
7. Compute $T_{lim} = T_{all}F_R = (31{,}000 \text{ lb/ft})(1.5) = 47{,}000$ lb/ft for $F_R = 1.5$.

As shown in the next section, this result is in reasonable agreement with the results of detailed studies, indicating that the charts in Figure 8.8 can be used for preliminary assessment of reinforcement force.

Case history. Mohicanville Dike No. 2 is a rim dike on the Mohicanville Reservoir in Wayne County, Ohio (Duncan et al., 1988; Franks et al., 1988, 1991). Constructed on a weak peat and clay foundation, the dike failed during construction, and for many years the crest was 22 ft below its design elevation. A cross section through the dike is shown in Figure 8.9.

After evaluation of a number of alternatives for raising the dike to its design height, it was concluded that construction of a reinforced embankment afforded the best combination of cost and reliability. Limit equilibrium analyses and finite element analyses were preformed to determine the reinforcing force required for stability of the embankment. It was found that to achieve a factor of safety $F = 1.3$, a reinforcing force of 30,000 lb/ft was required. The results of equilibrium analyses are shown in Figure 8.10.

A heavy steel mesh was selected for reinforcement. This mesh has No. 3 mild steel bars spaced 2 in. apart perpendicular to the axis of the dike, welded into a mesh with No. 2 bars spaced 6 in. apart parallel to the axis of the dike. This mesh provided a cross-sectional area of about 1 in^2 of steel per foot of embankment length and a yield force (T_{lim}) equal to 48,000 lb/ft. This provides a factor of safety on reinforcement capacity, $F_R = T_{lim}/T_{all} = 48{,}000/30{,}000 = 1.6$.

The steel mesh was rolled up after fabrication into rolls containing strips 8 ft wide and 320 ft long. The steel yielded in bending, deformed plastically as it was rolled up, and stayed rolled up without restraint. The rolled strips of mesh were transported on trucks and were unrolled at the project site using the same equipment as that used to roll up the mesh in the fabricating plant. Each strip was cut into two 160-ft-long pieces that reached across the full width of the embankment, from upstream to downstream. The strips were dragged into position on the embankment using a front-end loader and a bulldozer. They were laid on, and were covered by, about 1 ft of clean sand.

The reinforcing mat was placed at elevation 960 ft, approximately 4 ft above the original ground elevation. In most areas, about 6 to 8 ft of old embankment fill was excavated to reach elevation 960 ft. In one 100-ft-long section of the embankment, where the foundation soils were thought to be exceptionally weak, a second layer of reinforcing was placed at elevation 961 ft.

The steel mat was not galvanized or otherwise protected against corrosion. Although the steel reinforcement will probably corrode in time, it is needed only for the first few years of the embankment's life. After the foundation gains strength through consolidation, the reinforcement will no longer be required for stability.

The embankment was designed using limit equilibrium analyses and finite element analyses that modeled consolidation of the foundation soils as well as interaction between the embankment and the steel reinforcing. The embankment was instrumented to measure reinforcement forces, settlements, horizontal movements, and pore pressures. Computed and measured reinforcement forces at the end of construction are shown in Figure 8.11. It can be seen that the calculated values agree quite well with the measured values. It is worthwhile to note that the finite element analyses were performed before the embankment was constructed, and the results shown in Figure 8.11 therefore constitute a true prediction of performance, not an after-the-fact matching of analytical results and field measurements.

This case history indicates that both limit equilibrium analyses and finite element analyses can be used to design reinforced embankments on weak foundations and to anticipate their performance. In most cases limit equilibrium analyses can be used as the sole design tool. However, in precedent-setting cases, as Mohicanville Dike No. 2 was in the mid-1980s, it is prudent to perform more thorough analyses using the finite element method.

Recapitulation

- Reinforcement can be used to improve the stability of slopes and embankments, making it possible to construct slopes and embankments steeper and higher than would otherwise be possible.
- Reinforced slopes and embankments can be analyzed using the procedures described in Chapter 6 by including the reinforcement forces as known forces in the analyses. The amount of force required to achieve a target value of factor of safety can be determined using repeated trials.
- Two methods have been used for limit equilibrium analyses of reinforced slopes: *method A,* in which allowable reinforcing forces are specified, and *method B,* in which *ultimate* reinforcing forces are specified. Method A is preferable, because it provides a means of applying different factors of safety to soil strength and reinforcing force, which have different sources of uncertainty and different amounts of uncertainty associated with their values.

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- The principal types of reinforcing materials that have been used for slopes and embankments are geotextile fabrics, geogrids, steel strips, steel grids, and high-strength steel tendons.
- The long-term capacity of reinforcement, denoted here as T_{lim}, depends on tensile strength, creep characteristics, installation damage, durability, pullout resistance, and stiffness.
- The allowable load assigned to reinforcing materials should include a factor of safety, as indicated by the expression $T_{all} = T_{lim}/F_R$, where T_{all} is the allowable force, T_{lim} is the capacity of the reinforcement to carry long-term loads, and F_R is the reinforcement factor of safety. The value of F_R should reflect the level of uncertainty in the analyses and the consequences of failure.
- Designing reinforced slopes is facilitated greatly by slope stability charts of the type developed by Schmertmann et al. (1987), which are shown in Figure 8.5*a* and *b*.
- Potential modes of failure of reinforced embankments on weak foundations include sliding across the top of the reinforcing, shear through the reinforcement and into the weak foundation, large settlement of the embankment resulting from excessive elongation of the reinforcement, and bearing capacity failure.
- Preliminary estimates of the reinforcement force required for a given factor of safety can be made using the stability charts shown in Figure 8.8.
- The Mohicanville Dike No. 2 case history shows that both limit equilibrium analyses and finite element analyses can be used to design reinforced embankments on weak foundations and to anticipate their performance. Finite element analyses should be performed for precedent-setting applications.

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