CONSTRUCTION EXCAVATIONS AND REINFORCED EMBANKMENTS ON SOFT GROUND

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- **STABILITY OF OPEN CUTS**
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CONSTRUCTION EXCAVATIONS

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In general, sides of construction excavations may be (i) unsupported (vertical or sloped) or (ii) braced or structurally supported. The choice in the use of these approaches depends, among other factors, on excavation depth, available space around excavation, soil conditions (type, strength, weight), ground water conditions, presence of nearby buildings and temporary loads, durations for which excavation will be kept open, economics and safety requirements. Herein, we will discuss only the salient aspects of those unsupported excavations or so-called open cuts.

The forces which tend to cause caving or gross distortion and settlement of the side of an excavation include: (i) gravity in the form of the weight of the soil mass including soil-water, (ii) water pressures (seepage forces), (iii) external loads (buildings, snow, water, excavation equipment, piled materials, etc.), and (iv) vibrations (blasting, earthquakes, etc.). Resistance to caving is provided in open excavations primarily by the shearing resistance of the soil. In those cases where the shearing resistance offered by the soil is not adequate for safe excavation, structural support in terms of sheeting, shoring, bracing, and tie-back systems is provided. Conversely, shearing resistance can be increased by ground freezing or dewatering. Since the failure of open cut results only when shear failure has occured at enough points to define a surface along which movement can take place, basic principles that govern the shear strength of soil must be clearly understood.

STABILITY OF OPEN CUTS

Open cuts are excavations in which no bracing is used to support the soil. They are used in excavations when hard soil is encountered, that needs no support and in highway, railroad, and canal cuts where the cost of long lines of bracing would be great. Cuts less than 50 feet deep whose failure would not endanger life are ordinarily designed on the basis of experience. Railroad design manuals and extended highway specifications give 1.5(H) to 1(V) as a standard slope for most conditions, and 2 to 1 for weak soils. This is based upon both soil strength and observation that it is difficult to maintain protective vegetation on slopes steeper than 1.5 (H) to 1 (V) or 1.75 (H) to 1 (V) in erosion susceptible silts.

Deep cuts are investigated on the basis of geologic and soil study and a slope stability analysis utilizing the drained shear strength. If the soil is a swelling or fissured clay or subject to unusual seepage, further investigation is necessary. The condition of nearby cuts in similar soil is a guide to cut performance. In critical or unusual situations, a trial cut excavated at a slope steep enough to cause failure. The soil with the laboratory data on the soils in the cut, is used to determine the safe slope. In extreme cases, where accurate analysis is impossible because of erratic soils, it may be prudent to place bench marks at the top of the finished slopes to warn of any unusual movements that could lead to failure.

According to OSHA Construction Safety and Health Standards, a trench is referred to as a narrow excavation in which the depth is greater than the width, although the width is not greater than 15 feet. This can include excavations for anything from cellars to highways.

Vertical Cuts

OSHA requires that all excavations over 5 feet deep be sloped, shored, sheeted, braced or otherwise supported. When soil conditions are unstable, excavations lower than 5 feet also must be sloped, supported, or shored. A theoretical analysis of the stability of a vertical cut in a cohesive soil indicates that the critical depth, H_c, at which failure takes place is given by the equation

$$H_c \leq \frac{4c_u}{\gamma} \text{ for } \phi_u = 0; \text{ or } \quad H_c \leq \frac{4c}{\gamma} \tan(45^o + \phi/2)$$

This simple expression explains why a vertical cut can be excavated in saturated soil without failure. This implication, however, is misleading and dangerous. First, clay cannot support tensile stresses very long; a vertical crack will soon develop in the tension zone destroying much (or all), of the tension force about the theoretical level of zero pressure (Figure 1). Thus, the safe vertical bank height in a saturated clay in the undrained state, that is, for short term, will be less than H_c [suggested to be about $(2.67c/\gamma)\tan(45^o+\phi/2)$]. A more insidious complication is surface water accumulation in the crack. It adds a positive pressure within the soil mass that changes the total force system drastically. This accounts for many failures of unsupported clay banks during rainy weather. A final complication is that total stresses in undrained shear approximate the failure conditions only so long as the water content and void ratio of the clay remain unchanged. After the cut, in the new stress environment, there is a redistribution of pore water pressures and the drained shear strength of clay becomes critical. The stability of vertical cuts in saturated clay, therefore, changes with time and the environment. Its calculation involves uncertainties that must be considered in design.

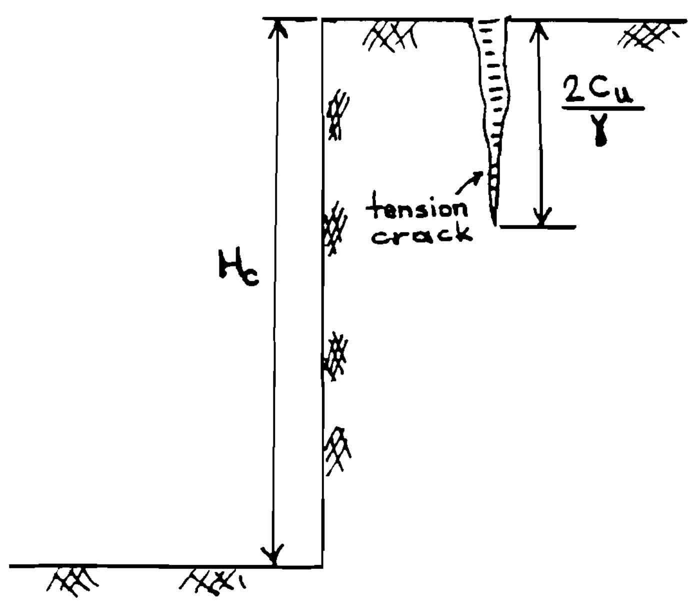

Figure 1 Tension Crack in a Vertical Cut

Sloped cuts

One method of insuring the safety and health of workers in a trench or excavation is to slope the sides of the cut to the "angle of repose", the angle closest to the perpendicular at which the soil will remain at rest. The angle of repose varies with different kinds of soil, and must be determined on each individual project. When an excavation has water conditions, silty material, or loose boulders, or when it is being dug in areas where erosion, deep frost, or slide plains are apparent, the angle of repose must be flattened. (See Table B-1 for the OSHA guidelines for approximate angle of repose.)

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TABLE B-1

MAXIMUM ALLOWABLE SLOPES

SOIL OR ROCK TYPE	MAXIMUM ALLOWABLE SLOPES (H:V)[1] FOR EXCAVATIONS LESS THAN 20 FEET DEEP [3]
STABLE ROCK	VERTICAL (90°)
TYPE A ($q_u \geq 1.5$ tsf except fissured)	3/4 : 1 (53°)
TYPE B ($0.5 < q_u < 1.5$ tsf or silty/loamy)	1:1 (45°)
TYPE C ($q_u \leq 0.5$ tsf or gravel/sand)	1½: 1 (34)

NOTES:

1. Numbers shown in parentheses next to maximum allowable slopes are angles expressed in degrees from the horizontal. Angles have been rounded off.
2. A short-term maximum allowable slope of 1/2H:1V (63°) is allowed in excavations in Type A soil that are 12 feet (3.67 m) or less in depth. Short-term maximum allowable slopes for excavations greater than 12 feet (3.67 m) in depth shall be 3/4H:1V (53°).
3. Sloping or benching for excavations greater than 20 feet deep shall be designed by a registered professional engineer.

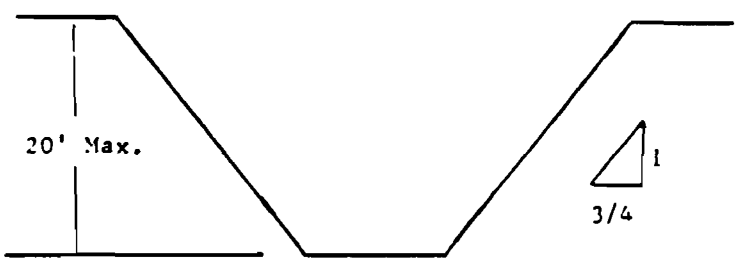
Figure B-.

Slope Configurations

(All slopes stated below are in the horizontal to vertical ratio)

B-1.1 Excavations made in Type A soil.

1. All simple slope excavation 20 feet or less in depth shall have a maximum allowable slope of ¾:1.



Simple Slope—General

Exception: Simple slope excavations which are open 24 hours or less (short term) and which are 12 feet or less in depth shall have a maximum allowable slope of ½:1.

The stability of excavation slopes depends on: (a) the strength of the soil in which the slope is excavated, as characterized by the strength parameters c_u and c' and ϕ', (b) the unit weight of the soil, γ, (c) the height of the slope, H, (d) the slope angle, β, and (e) the pore pressures. The critical failure mechanism is usually a deep surface in homogeneous cohesive soils, and surface sloughing and shallow sliding in homogeneous cohesionless soils. In non-homogeneous slopes, the critical shear surface may be either shallow or deep, depending on the shear strength characteristics and distributions of the soils within the slope.

The long-term stability of excavation slopes in cohesive soils is usually more critical than short-term stability, because the soil around the excavation swells under the reduced stresses and becomes weaker over a period of time. It may, however, be necessary to analyze the stability of excavated slopes for a number of pore pressure conditions.

The objective of a slope stability analysis is to determine the safety factor for a cut of given depth and slope. The safety factor can be defined as the ratio of shearing resistance of the soil available to the shear stress induced as a result of excavation along the most critical surface of potential failure in the slope. A safety factor equal to 1 indicates impending failure. Safety factors less than 1 indicate unsafe conditions; 1.0 to 1.2, questionable safety, and 1.3 to 1.4 satisfactory for cuts and fills. The safety factors for use in design include an allowance for the differences between laboratory test results and the real shear strength of the soil. A set of slope stability charts is appended to these notes including examples to compute the safety factor.

IMPROVEMENT OF CUT STABILITY

The stability of the cut can be improved by decreasing the soil stress or by increasing its strength. Soil stress can be reduced in most cases by making the slope flatter. If the sections that require improved stability are short, the slope can be partially supported by a small retaining wall or by cribbing. Water pressure in cracks in cohesive soils can be relieved by surface drains about the slope to intercept water and by horizontal drains driven into the face of the soil. Piles driven through the potential shear plane can increase the resisting moment; however; the additional shear resistance offered by piles is usually small compared to the shear forces or overturning moments. Better still, piles driven near the top of the slope can support part of the potentially sliding mass. Piles are most effective in increasing marginal stability; if the slope is very unsafe, enough piles to be effective are usually uneconomical.

Strength of cohesionless, slightly cohesive, or fissured soils and fractured rocks can be increased by relieving pore water pressures with surface drains and horizontal drains in the face of the slope. Good drainage has always been the most effective measure for improving slope stability where water is effective in instability. The strength of cohesive soils is difficult to improve quickly and permanently. In some cases, large drainage ducts, driven into the slope, have been able to reduce the soils water content and increase its strength, but the method is expensive.

Certain cemented soils such as loess and tuff, have high shear strength in spite of their loose structure. They will stand vertically in cuts as deep as 65 feet. Sloping cuts are stable only until rain falls. The bare porous soil absorbs the water, which seeps down rapidly because of the high vertical permeability. The cementing of the soil breaks down in water and the slope disintegrates by gullying and slumping until it becomes vertical. Vertical cuts will stand for many years with only occasional slumping or scaling along the vertical cleavage planes. Cuts are made wider than necessary in order to allow room for the debris that collects at the toe.

TIEBACK SUPPORTED SLOPES

Introduction

Permanent tied-back walls have been used to stabilize land slides and supporting structures in the United States since the early 1970's. The tied-back wall system contains elements which must perform individually and as a unit if a satisfactory design is to be achieved.

Listed here are three factors of safety (FS) that should be given special attention when designing tied-back walls. These FS should not be confused with each other because each controls a separate part of the design. They are: (1) an FS against failure of the steel tendons; (2) the FS against failure in the anchor zone; and (3) the overall external FS against wall failure. This FS is necessary for stability of the wall and its surroundings. This becomes very important if a slope which is protected by a wall is unstable. In case of slope failure, soil pressure acting against the wall will not be the same as the active zone soil pressure. At the time of the slope failure, a much larger soil mass will act on the wall. For this reason, the anchor load based on slope stability analysis controls the design. The geometry of the wall, soil parameter, location of the water table, number and angle of inclination of anchors, and their free lengths should be inputted to the analysis. The anchor load that results in an acceptable FS should be determined. The determined anchor load using slope stability analysis should be considered as design load.

As a result, the need for a method of analyzing the overall stability of slopes and retaining walls subjected to horizontal or inclined concentrated loads has become more evident. Until now, the input of horizontal or inclined concentrated loads acting on a near vertical slope was somewhat difficult. In addition the factor of safety was not formulated for this type of loading and thus, did not fully account for the distribution of force to the failure surface caused by concentrated boundary loads. A typical approach is to utilize Flamant's Formulas as proposed by Tenier and Morlier (1982) and the modified Bishop method of analysis for circular failure surfaces, and the simplified Janbu method of analysis for non-circular failure surfaces. The tie-back option may be used with either random or specific failure surface generation methods for irregular, block or circular failure surfaces. The word "tieback" is used to mean tie-back or other types of concentrated loads applied to the ground surface.

The force from a concentrated load is distributed through the soil mass to the failure surface and hence to all slices of the sliding mass. Some slope stability programs on the other hand, take a concentrated load into account only on the slice on which it acts. This distribution of load throughout the soil mass is an important feature.

First an equivalent line load is calculated for a row of tie-backs by dividing the specified tie-back load (point load) by the corresponding horizontal spacing between tieback loads. The resulting line load is called TLOAD, Figure 1, and is inclined from the horizontal by an angle INCLIN. The radial stress on the midpoint of a slice is calculated using Flamant's Formula (Tenier and Morlier, 1982):

$$\sigma_r = \frac{2(\text{TLOAD})\cos(\text{TTHETA})}{(\pi)(\text{DIST})}$$

where

σ_r	=	Radial stress
TLOAD	=	Equivalent tieback line load
TTHETA	=	Angle between the line of action of the tie-back and the line between the point of application of the tie-back on the ground surface and the midpoint of a slice
π	=	pi
DIST	=	Distance between the point of application of the tie-back on the ground surface and the midpoint of a slice.

The radial force, PRAD, at the midpoint of the base of the slice due to the concentrated load is calculated by multiplying the radial stress by the length of the base of the slice:

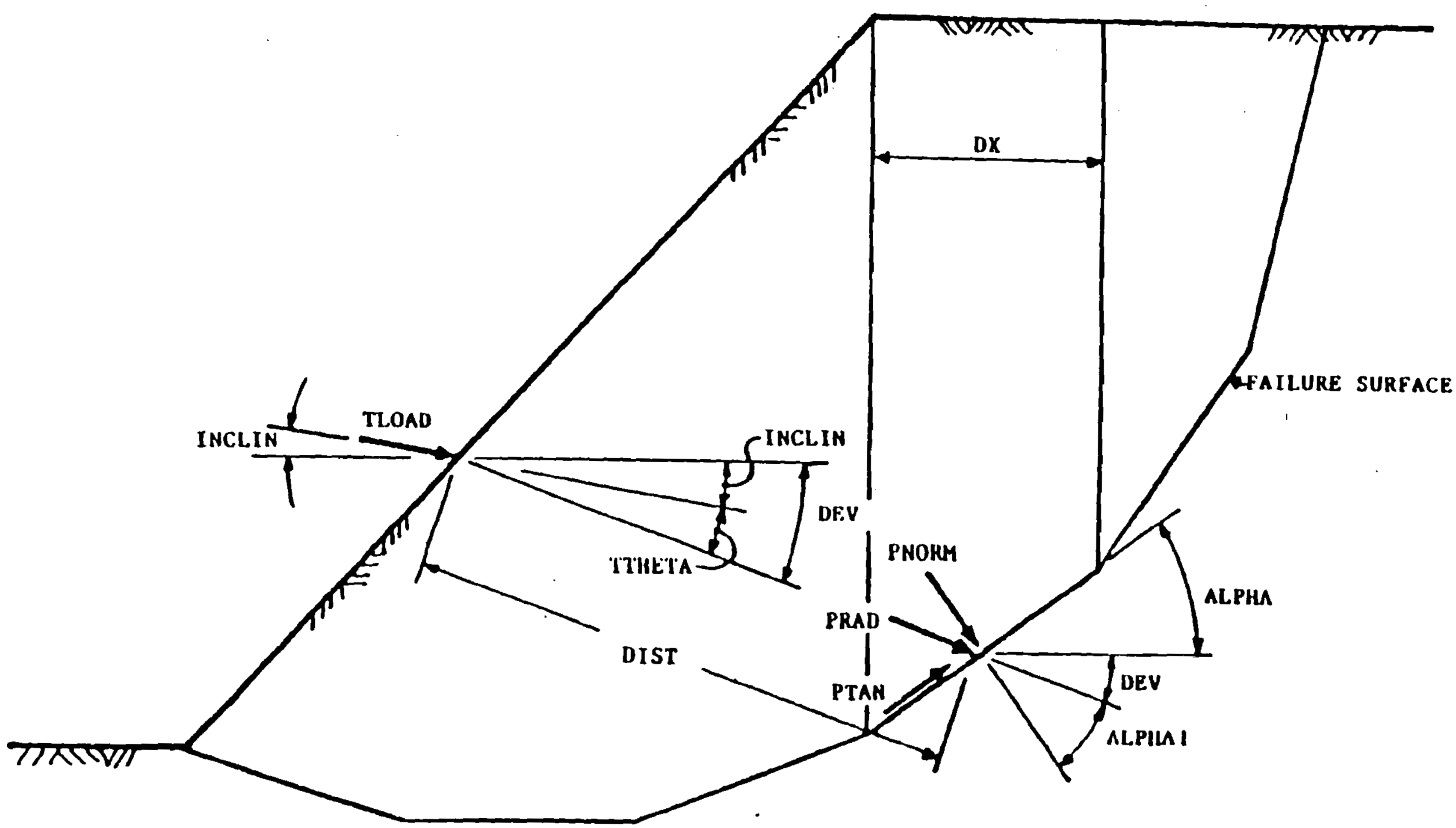


FIGURE 1. - TRANSFER OF CONCENTRATED LOAD TO FAILURE SURFACE

$$PRAD = \frac{2(TLOAD)\cos(TTHETA)}{(\pi)\,(DIST)} \bullet \frac{(DX)}{\cos(ALPHA)}$$

where

PRAD	=	Radial force on base of slice due to concentrated load
ALPHA	=	Inclination of base of slice
DX	=	Slice width

The location of bond length is very important to determine the anchor design load required for stability. This location will be evident dependent on the amount of free length of the tie-back. If the free length is larger than the distance of the failure surface to the tie-back load all the load will be distributed onto the potential failure surface. If free length is smaller than the distance from the failure surface to the tie-back load, a portion of the load will be distributed over the failure surface. If the free length is too short, then the tie-back load will not have any effect on stability (if the anchor zone is in the active wedge).

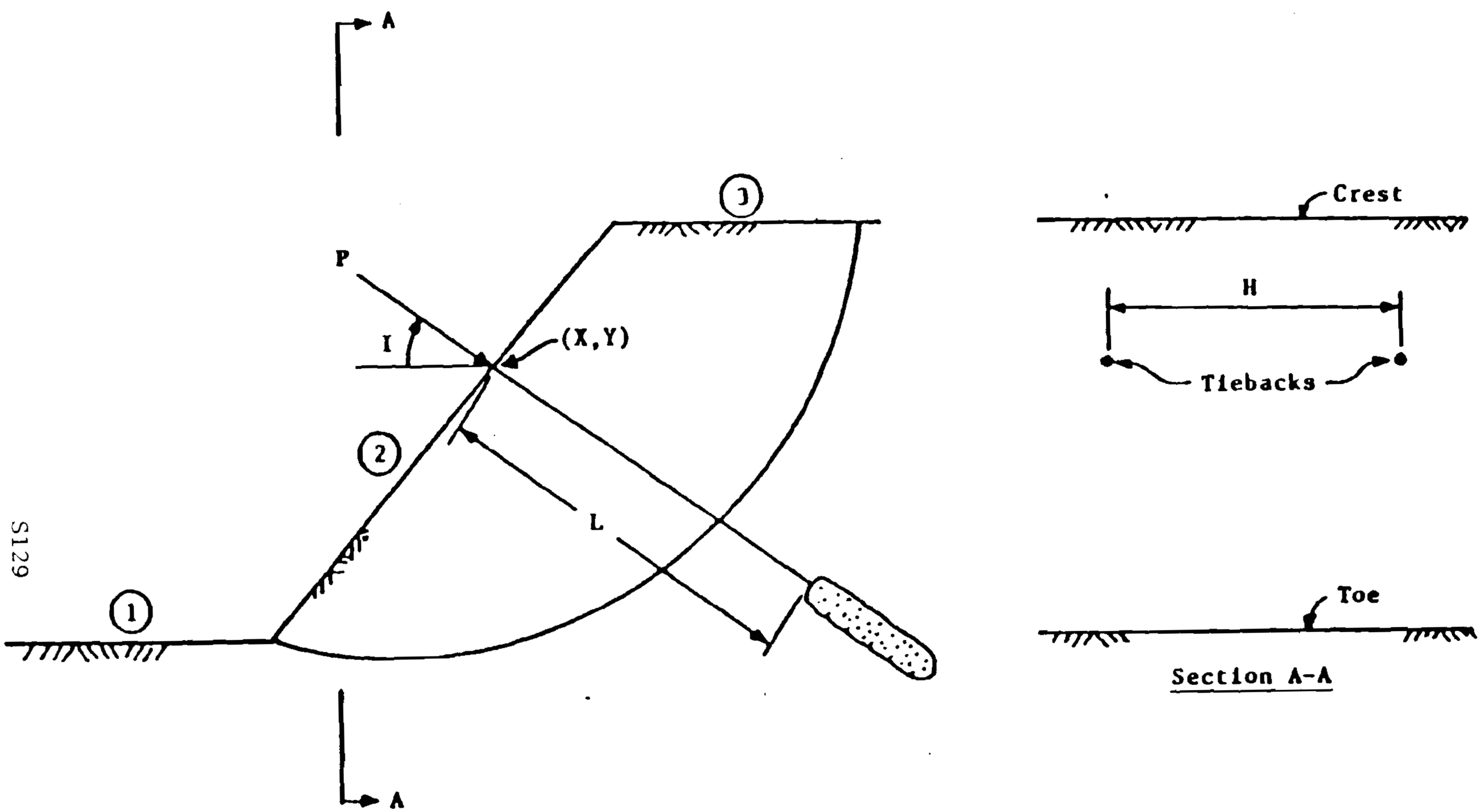
The minimum free length is 15 ft., according to FHWA. A procedure to determine the amount of free length at different locations in the wall is given in FHWA Publication DP-68-IR. Criteria for determining free length are: (1) Location from the wall of the furthest critical failure surface; (2) long-term deformation of ground or tendon; and (3) depth of adequate bearing layer for anchor zone.

The tie-back input parameters are summarized in Figure 2.

References

Boghrat, A. R. (1989), "Use of STABL Program in Tied-Back Wall Design," *ASCE Journal of Geotechnical Engineering*, Vol. 115, No. 4, pp. 546-552.

Tenier, P. and Morlier, P. (1982), "Influence of Concentrated Loads on Slope Stability," *Canadian Geotechnical Journal*, Vol. 19, pp. 396-400.



Tieback Input Parameters

- (2) Ground surface boundary number where tieback load is applied
- Y Y coordinate of point of application (ft) or (m)
- P Magnitude of load per tieback (lbs) or (kg)
- H Horizontal spacing between tiebacks (ft) or (m)
- I Inclination of tieback load (deg)
- L Free length of tieback (ft) or (m)

FIGURE 2: - TIEBACK INPUT PARAMETERS

The following information is extracted from the Geotextile Engineering Manual by Christopher and Holtz prepared for the Federal Highway Administration in 1984.

5.3.4.1 Types of Foundation Failures

In order to develop design concepts and criteria, possible failure modes, both for conventional and reinforced embankments, must be considered. We have found the discussion on this subject by Terzaghi and Peck (1967, pp 459-471) very useful. They state that the instability of embankment foundations are mainly of two types: a) where the embankment sinks into the foundation soils, and b) failure by spreading. There are other ways, too, that embankments can fail. If the embankment has to retain water, failure could occur due to piping. Geotextiles can protect against piping, as discussed in Chapter 3. Liquifaction due to vibratory or earthquake loads can be a problem, but it is difficult to see how geotextiles can guard against this eventuality. In this section we will concentrate on sinking and spreading failures.

Failure by sinking occurs primarily on (1) very soft, marshy ground containing organic silts and clays, and on (2) deep deposits of soft, fairly homogeneous clays. In (1), without reinforcement, the fill is usually designed to sink into the soft ground (displacement method in Table 5-1), as illustrated in Figure 5-7a. In (2), a berm may be used as a counterweight as shown in Figure 5-7b.

Failure by spreading, on the other hand, occurs only on stratified deposits containing fairly homogeneous and relatively thin layers of soft clay and in fills on clay strata containing thin sand or silt seams and partings. In the case of spreading, again there are basically two types of failures (Figures 5-8 and 5-9). One is characterized by relatively slow subsidence at the crest of the fill, and the other occurs very rapidly with the soils moving quite a distance downslope. The latter type of failure is catastrophic, and occurs when the clay stratum contains continuous layers or extensive lenses of coarse silt and sand.

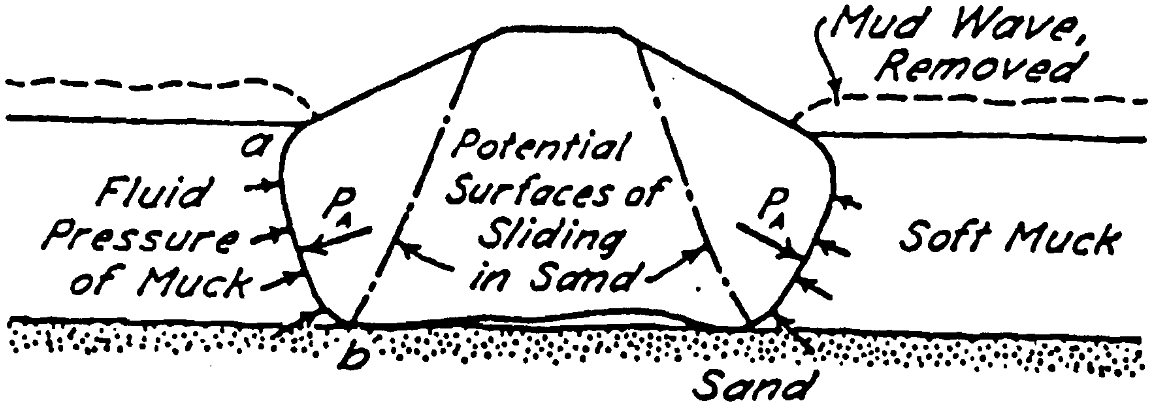


Figure 5-7a. Diagram Showing Forces Acting on Soil Adjoining Buried Part of Fill Constructed by the Displacement Method (Terzaghi and Peck, 1967)

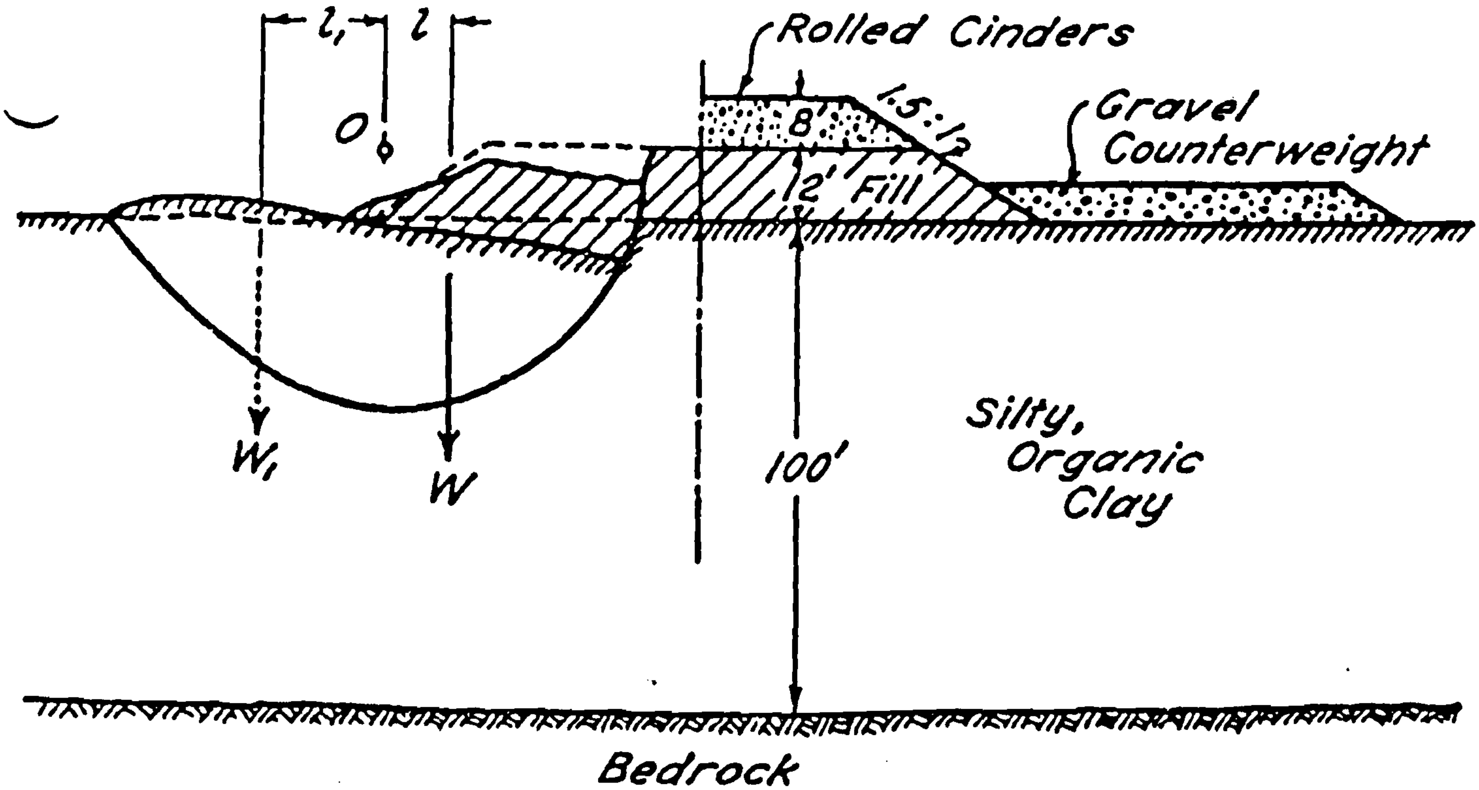


Figure 5- 7b. Section Through Gravel Fill on Deposit of Uniform Soft Clay Left side shows principal features of failure during construction. Right side shows reconstructed fill stabilized by using lightweight fill and a gravel counterweight (after Gottstein, 1936, in Terzaghi and Peck, 1967)

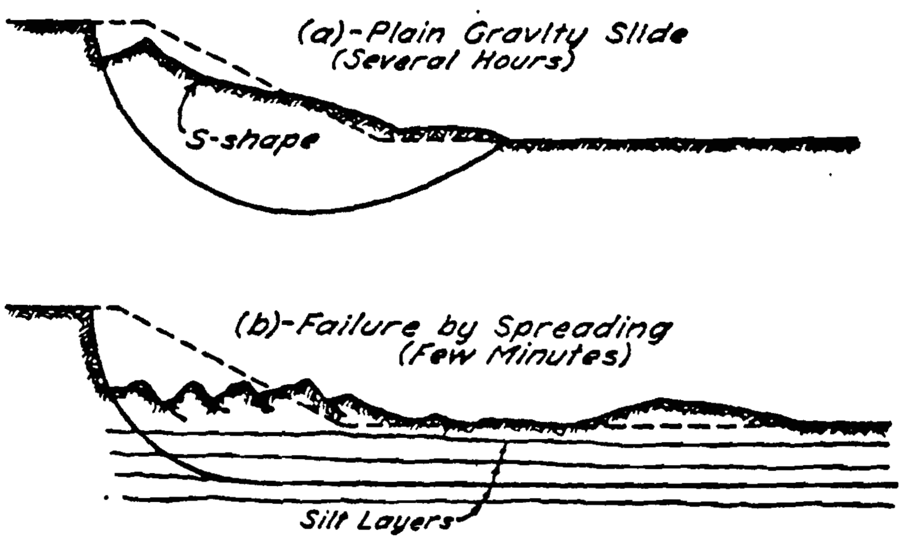


Figure 5-8. Cross-Section Through Typical Slide in Varved Clay
(a) If pore water pressure in silt layers is inconsequential
(b) If pore water pressure in silt layers is almost equal to overburden pressure
(Terzaghi and Peck, 1967)

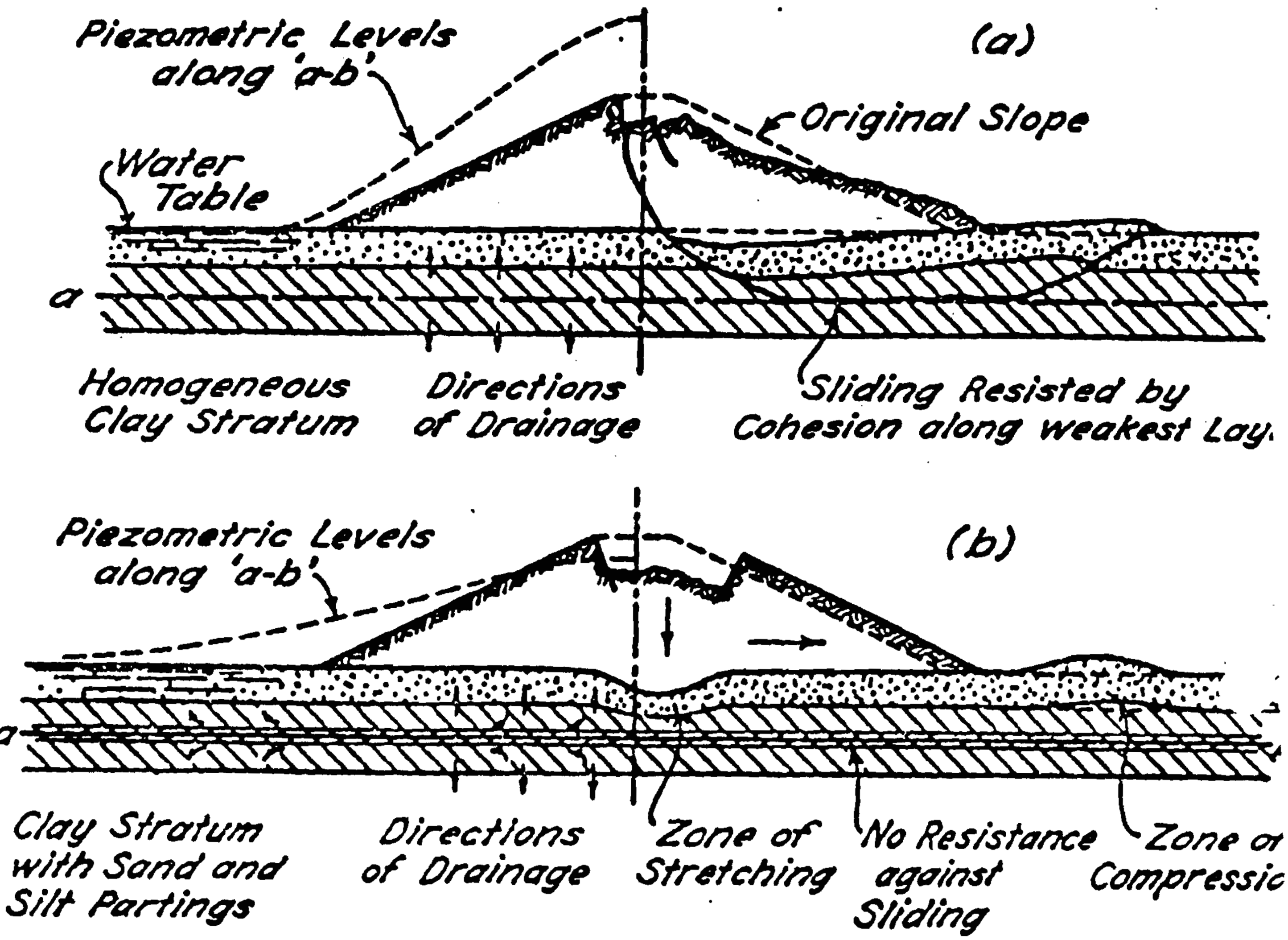


Figure 5-9. Type of Failure of Base of Fill Containing Thin Clay Stratum
(a) If clay stratum contains no horizontal pervious partings.
(b) If clay stratum contains pervious sand or silt partings.
(Terzaghi and Peck, 1967)

Haliburton, Anglin, and Lawmaster (1978 a and b) and Fowler (1981) proposed the modes of embankment failure shown in Figure 5-10, which are basically the same as the two Terzaghi and Peck (1967) failure types. Figure 5-10a shows the case of failure by spreading; Figures 5-10b and c are similar to those shown in Figures 5-7b and 7a, respectively.

5.3.4.2 Stability Analyses for Embankments with Geotextile Reinforcement

Johnson (1974) presents an excellent discussion of all aspects of stability analyses for the design of embankments, and we recommend you read this before you begin the design of a major embankment project, whether reinforced or not.

Although a number of analysis procedures utilizing such techniques as the finite element method have been proposed (see, for example, Rowe, 1982; Andrawes, et al. (1982); Petrik, et al. (1982); and Boutrup and Holtz, 1982), none has yet been developed to the point where it is a practical design tool for the average highway engineer. When non-linear soil models and large deformations are considered, the numerical problems associated with finite element analyses become very complex and subject to considerable error. Consequently, at present, only limited equilibrium type analyses are used for the design of geotextile-reinforced embankments. These analyses are similar to conventional bearing capacity or slope stability analyses. The bearing capacity analysis assumes the embankment to be an infinitely long strip footing. The slope stability analysis involves calculations for stability of a series of assumed sliding surfaces in which the reinforcement acts as a horizontal force to increase the resisting moment.

There are difficulties with analyzing composite soil-geotextile systems using limiting equilibrium, which leads to some doubt as to the validity of the general approach. For one thing, the techniques assume rigid-plastic stress-strain behavior, and they do not take into account the effect of system deformation on the embankment-geotextile interaction. Another difficulty is that these procedures do not consider any redistribution of stresses in the embankment and the foundation caused by the reinforcement. We know from the performance of fabric-reinforced retaining walls (Section 5.7) that the stress distribution

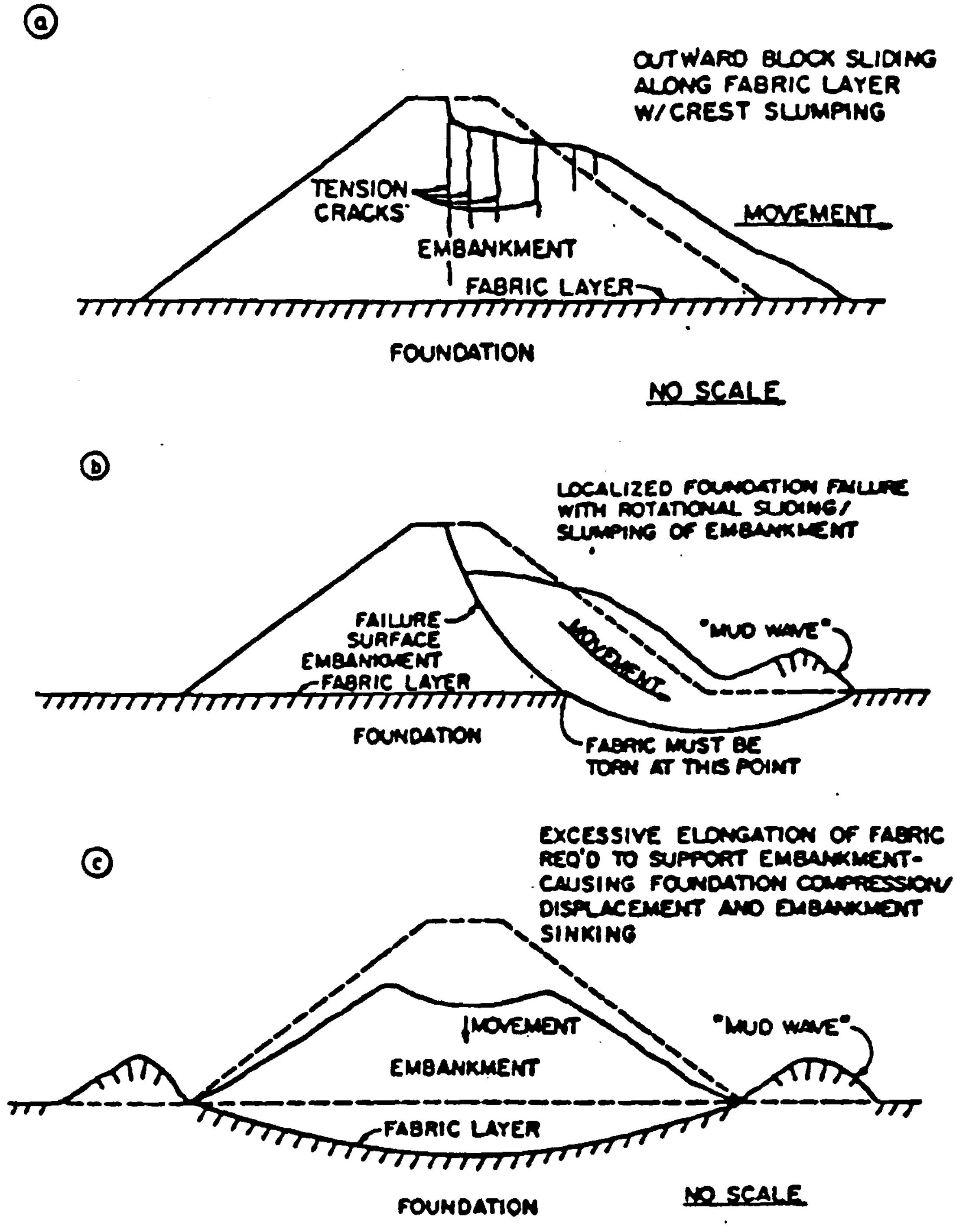


Figure 5-10. Potential Unsatisfactory Behavior That Might Occur for Fabric-Reinforced Embankments (Haliburton, Anglin, and Lawmaster, 1978 a and b; Fowler, 1981)

behind the wall must be very different from that assumed by classical earth pressure theory. Consequently, simple design procedures based on such assumptions are probably not justified, although, as we shall see in Section 5.7, at present we have no other choice. Fortunately, in the case of retaining structures, the assumption of a classical stress distribution is on the conservative side. The analogous situation with regard to geotextile-reinforced embankments is at present unknown because of the lack of field verification of the design procedures. (The effect of geotextile reinforcement on the redistribution of stresses and its concurrent effect on limiting equilibrium type stability analyses is the object of research currently underway at Purdue University.)

Even given these problems, the use of extensions of the common stability analysis procedures is attractive because of their simplicity, and we use them because they are all we have available at present.

The following calculation steps must be taken in the analysis of the stability of a geotextile-reinforced embankment:

1. Check overall bearing capacity (Section 5.3.4.2a).

2. Check edge bearing capacity or slope stability (Section 5.3.4.2b).

3. Conduct a sliding wedge analysis for embankment spreading (Section 5.3.4.2c).

4. Perform an analysis to limit geotextile deformations (Section 5.3.4.2d).

5. Determine fabric strength requirements in the longitudinal direction (Section 5.3.4.2e).

Although not strictly a part of stability, embankment settlement (Section 5.3.4.2f) and creep (Section 5.3.4.2g) must also be considered.

5.3.4.2a Overall Bearing Capacity

The overall bearing capacity of the embankment must be satisfactory, with or without geotextile reinforcement. If the overall stability of the embankment is not satisfied, there is no point in trying to reinforce the embankment. Procedures for calculating the overall bearing capacity are given in standard foundation engineering textbooks such as Terzaghi and Peck (1967), Vesic (1975), Perloff and Baron (1976), and U.S. Navy (1982), DM-7. Analyses for bearing capacity follow classical limiting equilibrium type analyses for strip footings using assumed logarithmic spiral or circular failure surfaces. A trial and error solution to determine the minimum factor of safety is carried out. Usually, an analysis using the logarithmic spiral results in a slightly lower calculated factor of safety.

A second bearing capacity consideration is the possibility of lateral squeeze of the underlying soils. Therefore, the lateral stress and corresponding shear forces developed under the embankment should be evaluated with respect to resisting passive forces and shear strength of the soil.

If an overall bearing capacity analysis indicates an unsafe condition, stability can be improved by adding berms or by extending the base of the embankment to provide a mat and thus spread the load to a greater area. The berm or mat can be reinforced by a properly designed geotextile to maintain continuity with the embankment and reduce the risk of lateral spreading (splitting) of the embankment (to be covered in Section 5.3.4.2c).

One interesting approach to increasing the overall bearing capacity utilizing reinforcement has been developed by Tensar Corporation (1982). Even though the method refers to geogrids, the method is applicable to geotextiles.

See the figure on the next page for bearing capacity analysis.

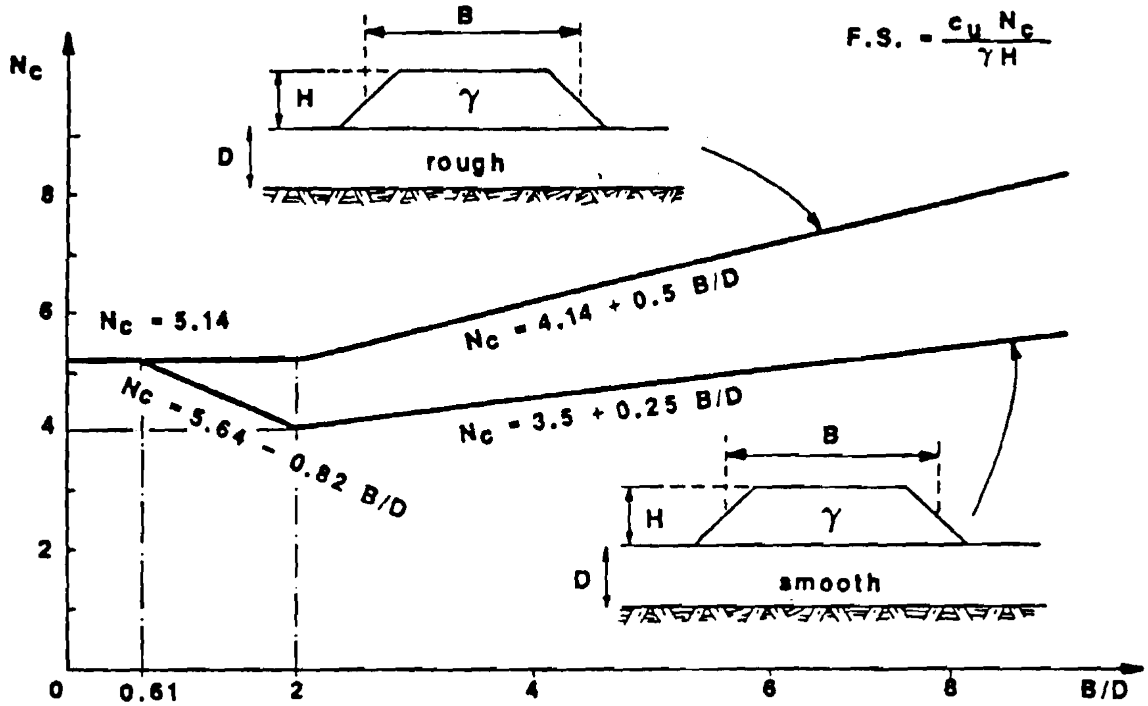


Fig. A Bearing capacity failure of embankment. Simplified design chart for calculating the factor of safety using bearing capacity factors from Mandel and Salencon (29, 30). Chart originally published in (3).

5.3.4.2b Edge Bearing Capacity Using a Modified Circular Arc Stability Analysis

If the overall bearing capacity of the embankment is satisfactory, then the stability of the edge of the embankment should be analyzed. The potential failure modes were shown in Figures 5-7 through 5-10. For stability, the tensile strength of the reinforcement must be sufficiently high to resist tearing at the intersection of the potential failure surface. The analysis basically consists of determining the most critical failure surface(s), then adding one or more layers of a geotextile at the base of the embankment with sufficient strength at tolerable strains to provide the resistance necessary to prevent failure plus a factor of safety.

The critical failure surface(s) can be obtained from conventional geotechnical limit equilibrium analysis methods (e.g., Bishop's and Morgenstern's circular arc methods). The driving moment (M_D) and the soil resisting moment (M_R) are determined for each critical surface (Figure 5-11). The additional resisting moment (ΔM_R) for the required factor of safety (S.F.) is then calculated from:

$$\Delta M_R = S.F. (M_D) - M_R \qquad (5\text{-}1)$$

The geotextile reinforcement can then be determined to provide the required additional reinforcing moment.

A number of analysis procedures have been proposed for determining the required reinforcement and several of these methods are included in Appendix C of this manual for your consideration. We believe it is best that you check your design by two or three independent procedures to give you confidence in your design calculations.

Basically, the difference between the methods is how the effect of the reinforcing is considered in the stability analysis. Broms (1977), Tensar (1982), and Jewell (1982) all consider the reinforcing to provide an additional resisting moment equal to the fabric

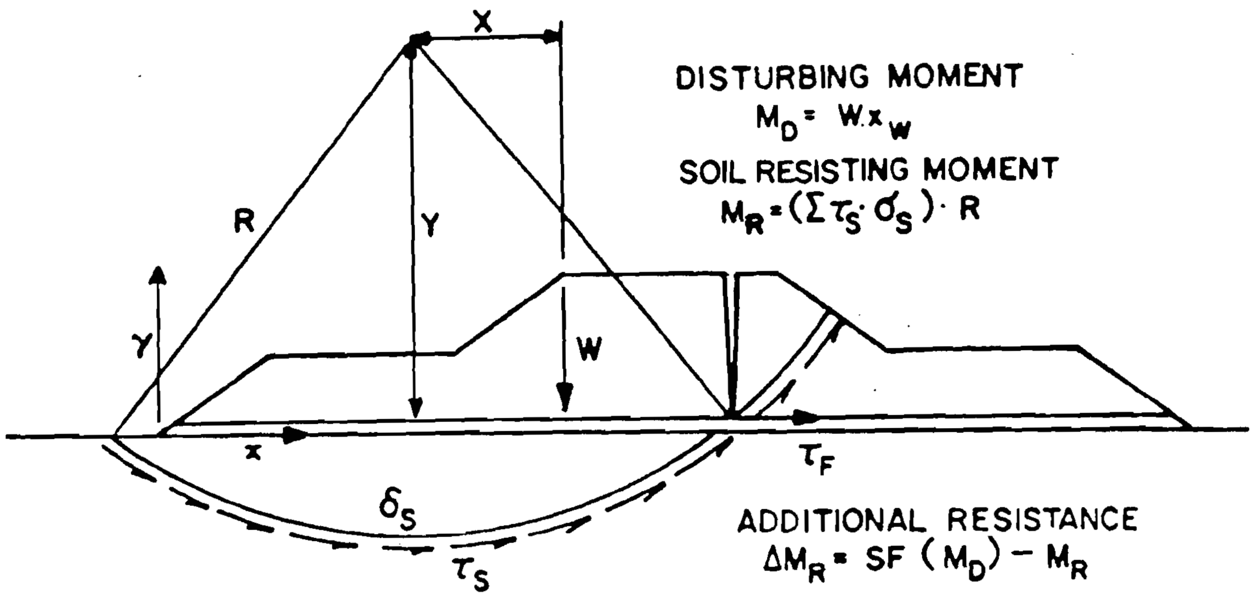


FIGURE 5-11. DEFINITIONS AND FORCES FOR CIRCULAR ARC ANALYSIS.

strength (T_F) times then vertical distance (Y) from the plane of the fabric to the center rotation, or

$$\Delta M_R = T_F \cdot Y \tag{5-2}$$

Wager (1981) includes this term plus an additional resisting moment contribution calculated from the vertical component (X) due to the soil-fabric interaction and corresponding increase in strength of the embankment. Thus:

$$\Delta M_R = T_F Y + X T_F \tan \phi \tag{5-3}$$

Note that all these methods require the depth of the critical failure circle to be relatively shallow (i.e., y is large), otherwise the contribution of the fabric to increase the resisting moment will be small.

Fowler (1981) assumes the reinforcement would be equivalent to the strength of a thin cohesive layer uniformly distributed along the failure plane or arc where:

$$\Delta M_R = T_F R \tag{5-4}$$

He also developed simple design charts for specific slope angles and a factor of safety of 1.3 which is included in Appendix C.

Ingold (1982) presented two design approaches. One involved a type of "infinte slope" analysis and the other was a modification of Bishop's circular arc analysis in which the reinforcement strength is mobilized in a manner similar to Broms, Wager, Tensar, and Jewel.

Even though it might appear that several of these methods are similar, we have included them in Appendix C because each contains additional pointers on the analysis, design, and performance of geotextile-reinforced embankments.

See the stability chart given in the figure on the next page for reinforced embankments.

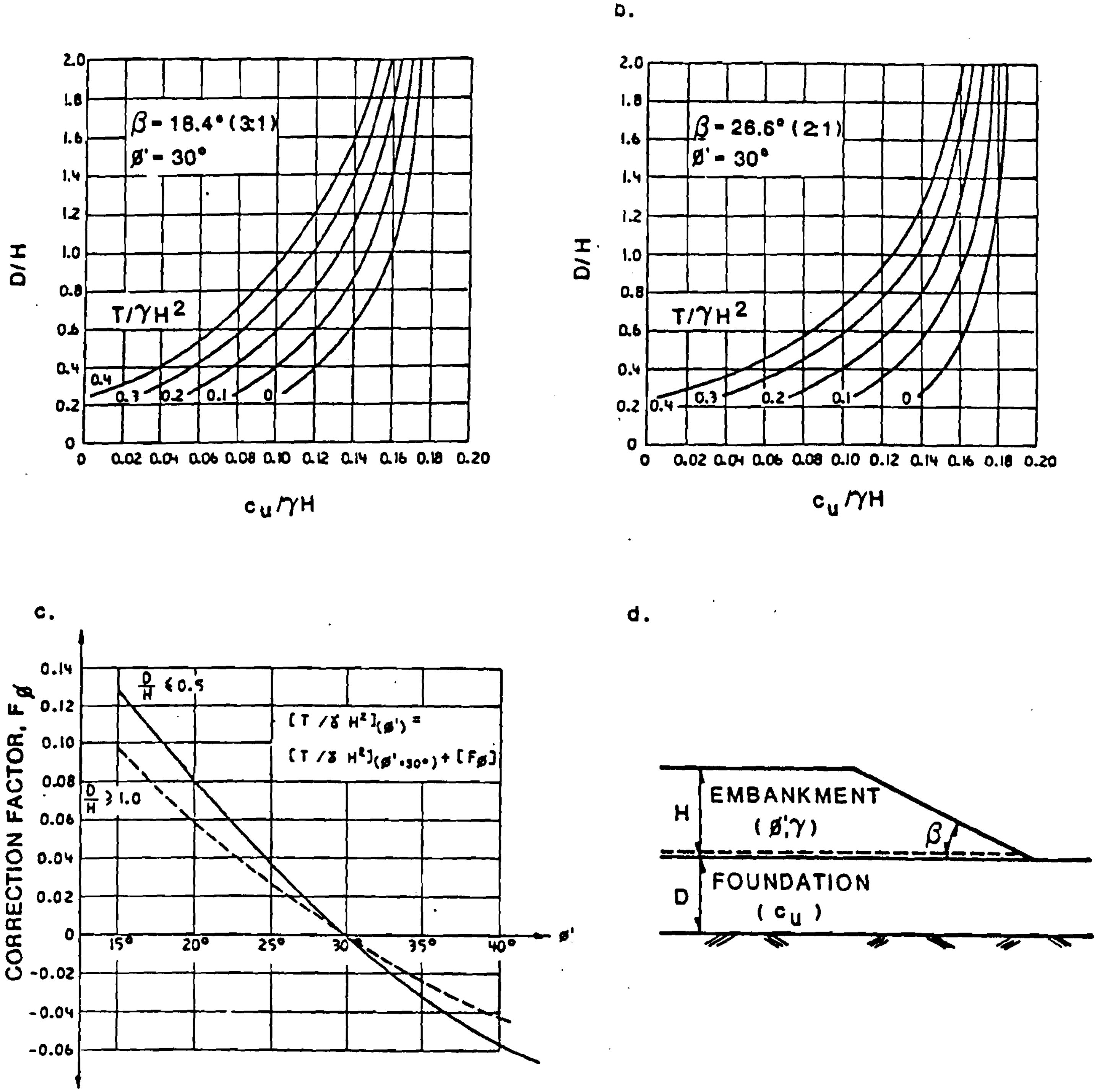


Fig. B Stability charts for the design of embankments on weak foundations. Charts from Milligan and Busbridge (31) give the required reinforcement force per unit width, T(kN/m), to obtain a state of limit equilibrium (F.S.=1). To obtain larger factors of safety use charts with factored soil strengths ($\tan \phi'_f = \tan \phi'/F.S.$); $C_{uf} = C_u/F.S.$).

5.3.4.2c Sliding Wedge Analysis (Lateral Embankment Spreading)

Conventional sliding wedge analyses are illustrated in: Terzaghi and Peck (1967), Perloff and Baron (1976), and U.S. Navy (1982). For unreinforced as well as reinforced embankments, the driving forces result from the lateral earth pressures exerted by the embankment which must, for equilibrium, be transferred to the foundation soils by shearing stresses. Instability occurs in reinforced embankments when either a) the frictional forces between the reinforcement and the embankment or the reinforcement and the subgrade are insufficient to resist the applied shearing stresses, or b) the shearing resistance of the foundation soils just below the embankment are insufficient to maintain equilibrium. Thus, the reinforcement must have sufficient frictional resistance to resist sliding on the plane of the reinforcement, and the tensile strength of the geotextile must be sufficient to resist rupture and tearing.

The forces involved in an analysis of embankment sliding are shown in Figure 5-12a. The failure actuating force is the lateral earth pressure in the embankment. The resisting force is developed by frictional resistance between the embankment base and the geotextile. Lateral earth pressures are a maximum at the crest of the embankment. The resultant of the active earth pressure per unit length of the embankment for a given cross section, P_A, may be calculated as follows:

$$P_A = 0.5\ \gamma\ H^2 K_A \qquad (5\text{-}5)$$

where:

γ	=	embankment fill compacted unit weight (units: force per length^3)
H	=	maximum embankment height (units: length)
K_A	=	coefficient of active earth pressure (dimensionless)

For cohesionless embankment fill, this equation becomes:

$$P_A = 0.5\ \gamma\ H^2 \tan^2 (45^\circ - \phi/2) \qquad (5\text{-}6)$$

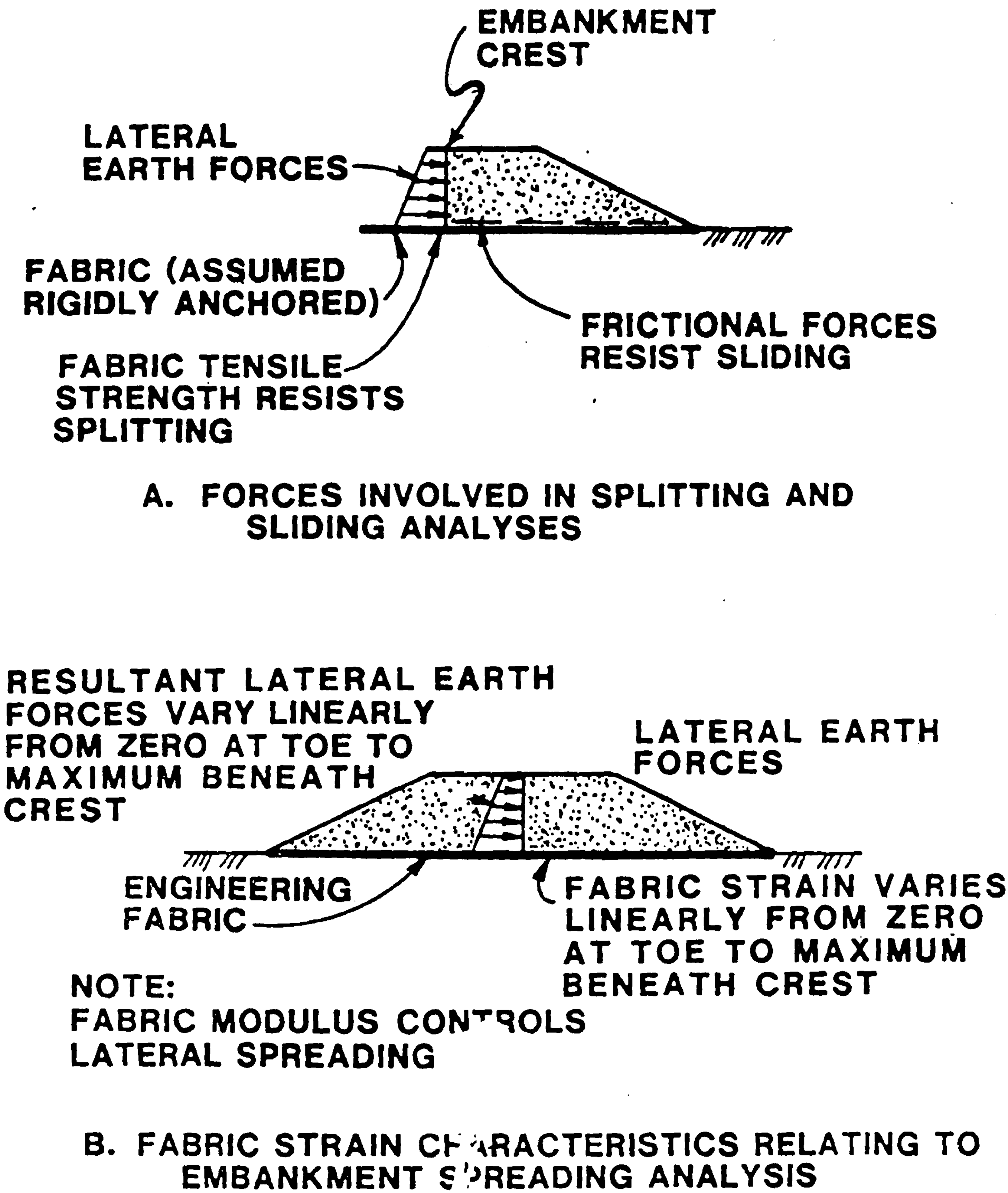


Figure 5-12 Assumed Stresses and Strains Related to Lateral Earth Pressures

Resistance to sliding results from the development of frictional forces along the interface between the fill material and the geotextile. This resistance force may be calculated as:

$$P_R = 0.5 \ \gamma b H \tan \phi_{SF} \tag{5-7}$$

where

P_R = resultant of resistant forces (units: force/length)

ϕ_{SF} = the soil-fabric friction angle (degrees)

b = distance from crest to toe of embankment (units: length)

A factor of safety against embankment sliding may be found by taking the ratio of the resistant forces to the actuating forces. For a given embankment geometry, it is controlled by the soil-geotextile friction. Fowler (1981) recommends a minimum factor of safety of 2 against sliding failure.

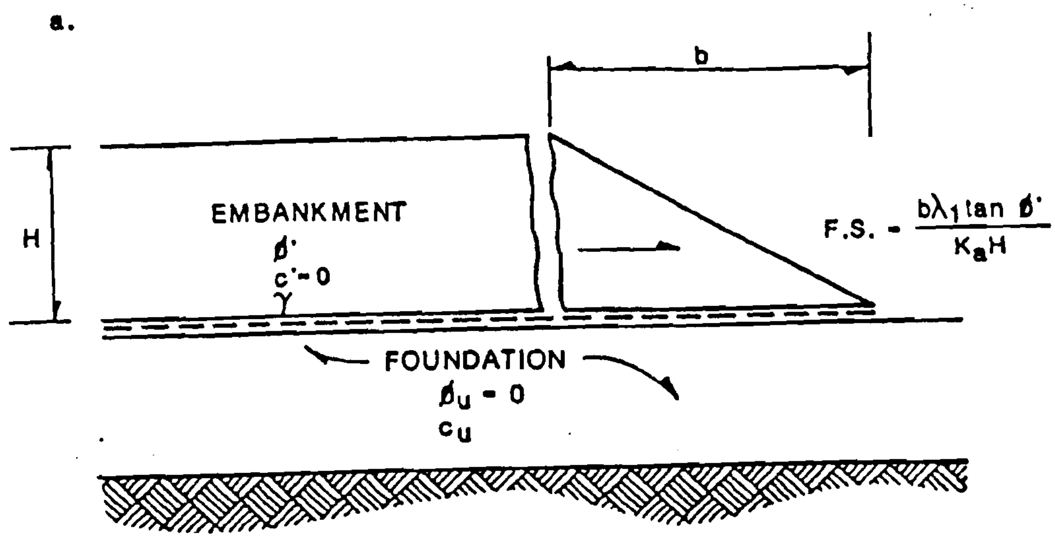
How to determine or estimate the soil-fabric friction will be discussed later in this section.

If you find that the required soil-fabric friction angle is greater than what might b reasonably achieved with your choice of fabric and the particular soils in the embankment and subgrade, then the embankment slope must be flattened, or alternatively, berms might prove feasible. Generally, however, more than sufficient frictional resistance exists with the geotextiles commonly used for reinforcement.

Assuming that sufficient friction is mobilized between the subgrade and the reinforcing geotextile, then the resultant lateral earth pressures must be resisted by the tension in the geotextile reinforcement. Fowler (1981) recommends a minimum factor of safety of 1.5 be used for the geotextile to prevent embankment splitting and tearing. Thus, the minimum required fabric tension, T_F, is:

$$T_F = 1.5 \, P_A \tag{5-8}$$

See the figure on the next page for lateral sliding analysis.



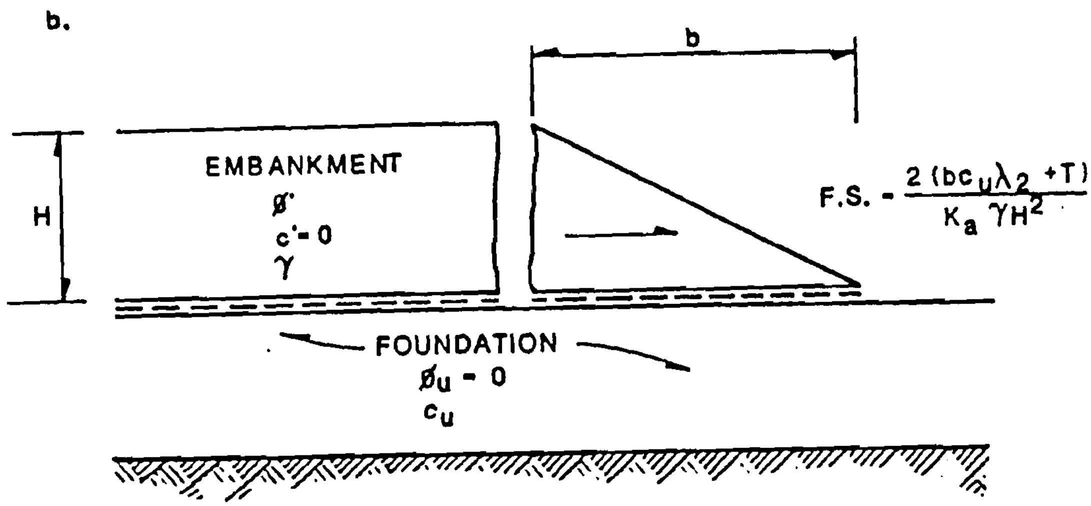


Fig. C Lateral sliding of embankment. Simplified analyses for: (a) embankment sliding over reinforcement; (b) reinforcement tensile rupture and embankment sliding over the foundation soil. T(kN/m) is the tensile force in the reinforcement at rupture; $\lambda_1 \cdot \tan\phi'$ is the embankment fill-reinforcement interface friction; $\lambda_2 C_u$ is the reinforcement-foundation interface adhesion.

5.3.4.2d Analysis to Limit Fabric Deformation

Because the tensile forces in the geotextile required to prevent a failure by lateral spreading are not developed without some strain in the geotextile, some lateral movement must be expected. Consequently, this movement is controlled by the modulus of the geotextile. As shown in Figure 5-12b, the distribution of strain (which will occur from incipient spreading) is assumed to vary linearly from zero at the toe to a maximum value beneath the embankment crest. This is because the lateral earth pressure also varies in a similar manner and the resultant earth pressure forces are transmitted to the reinforcement by shearing stresses at the base of the embankment. This assumption is somewhat unconservative because most geotextiles have stress-strain curves which are concave upward rather than linear. Therefore, a factor of safety of 1.5 is recommended in determining the minimum required geotextile tensile modulus. If the tensile strength, T_F, as determined from Equation 5-8 is used to calculate the required modulus, E_F, then the factor of safety of 1.5 will be automatically taken into account. Thus, the minimum required geotextile tensile modulus is:

$$E_F = \frac{T_F}{\epsilon_{max}} \qquad (5\text{-}9)$$

where

ϵ_{max} = maximum strain which the geotextile is expected to undergo at the centerline of the embankment

If the strain distribution shown in Figure 5-12b is assumed, the maximum strain in the geotextile is equal to twice the average strain over the embankment width. An average lateral spreading of 5% has been found to be a reasonable limiting value, both from a construction and fabric property standpoint (Fowler and Haliburton, 1980; Fowler, 1981), and this value is recommended for design. Using 5% as the average strain, the maximum strain which would be expected is therefore 10%, and thus from Equation 5-9, the required fabric tensile modulus is:

$$E_F = 10\,T_F \qquad (5\text{-}10)$$

The following pages are from:
Holtz, R.D., Christopher, B.R., and Berg, R.R. (1997), Geosynthetic Engineering, B.Tech Publishers, Vancouver, B.C., 452 pp.

Most reinforcement analyses assume that the fill material is granular. In fact, in the past the use of cohesive soils together with geosynthetic reinforcement has been discouraged. This may be an unrealistic restriction, although there are problems with placing and compacting cohesive earth fills on especially soft subsoils. Furthermore, the frictional resistance between geosynthetics and cohesive soils is problematic. It may be possible to use composite embankments. Cohesionless fill could be used for the first 0.5 to 1 m; then the rest of the embankment could be constructed to grade with locally available materials.

7.5 DESIGN EXAMPLE

DEFINITION OF DESIGN EXAMPLE

- Project Description: A 4-lane highway is to be constructed over a peat bog. Alignment and anticipated settlement require construction of an embankment with an average height of 2 m. See project cross section figure.

- Type of Structure: embankment supporting a permanent paved road

- Type of Application: geosynthetic reinforcement

- Alternatives:
  i) excavate and replace - wetlands do not allow;
  ii) lightweight fill - high cost;
  iii) stone columns - soils too soft;
  iv) drainage and surcharge - yes; or
  v) very flat (8H:1V) slope - right-of-way restriction

GIVEN DATA

- Geometry - as shown in project cross section figure

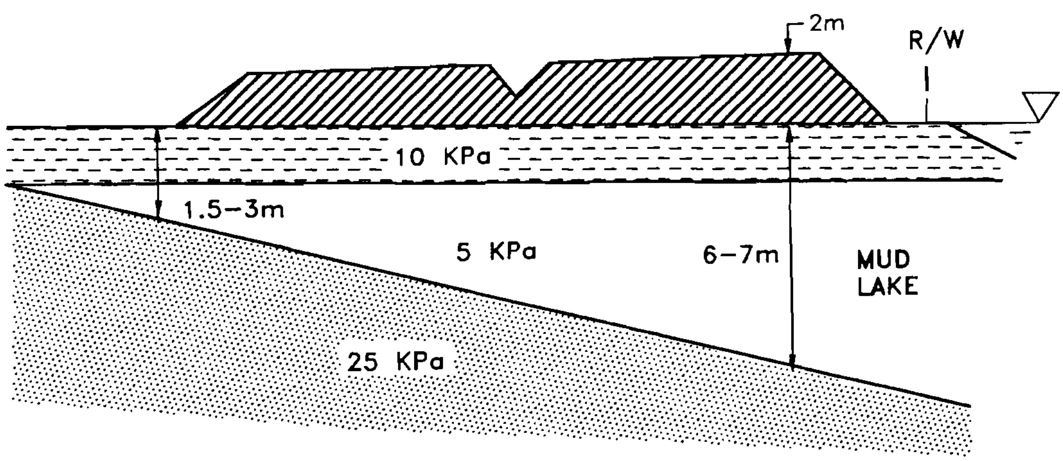
- Soils
  - subsurface exploration indicates $c_u = 5$ kPa in weakest areas
  - soft soils are underlain by firmer soils of $c_u = 25$ kPa
  - embankment fill soil will be sands and gravel
  - lightweight fill costs $250,000 more than sand/gravel

- Stability - Stability analyses of the unreinforced embankment were conducted with the STABL computer program. The most critical condition for embankments on soft soils is end-of-construction case; therefore, UU (unconsolidated, undrained) soil shear strength values are used in analyses.

The following pages are from:
Holtz, R.D., Christopher, B.R., and Berg, R.R. (1997), Geosynthetic Engineering, B.Tech Publishers, Vancouver, B.C., 452 pp.

- Results of the analyses:
  a) With 4:1 side slopes and sand/gravel fill ($\gamma = 21.7$ kN/m^3), FS $\approx$ 0.72.
  b) Since FS was substantially less than 1 for 4H:1V slopes, flatter slopes were evaluated, even though additional right-of-way would be required. With 8:1 side slopes and sand/gravel fill ($\gamma = 21.7$ kN/m^3), a FS $\approx$ 0.87 was computed.
  c) Light-weight fill ($\gamma = 15.7$ kN/m^3) was also considered, with it, the FS varied between $\approx$0.90 to 1.15

- Transportation Department required safety factors are:
  Fs_{min} > 1.5 for long-term conditions
  FS_{allow} $\approx$ 1.3 for short-term conditions



Project Cross Section

REQUIRED

Design geotextile reinforcement to provide a stable embankment.

DEFINE

A. Geotextile function(s)

The following pages are from:
Holtz, R.D., Christopher, B.R., and Berg, R.R. (1997), Geosynthetic Engineering, B.Tech Publishers, Vancouver, B.C., 452 pp.

B. Geotextile properties required

C. Geotextile specification

SOLUTION

A. Geotextile function(s):
Primary - reinforcement (for short-term conditions)
Secondary - separation and filtration

B. Geotextile properties required:
tensile characteristics
interface shear strength
survivability
apparent opening size (AOS)

DESIGN

Design embankment with geotextile reinforcement to meet short-term stability requirements.

STEP 1. DEFINE DIMENSIONS AND LOADING CONDITIONS

See project cross section figure.

STEP 2. SUBSURFACE CONDITIONS AND PROPERTIES

Undrained shear strength provided in given data. Design for end-of-construction. Long-term design with drained shear strength parameters not covered within this example.

STEP 3. EMBANKMENT FILL PROPERTIES

sand and gravel, with
$\gamma_m = 21.7 \text{ kN/m}^3$ $\phi' = 35°$

STEP 4. ESTABLISH DESIGN REQUIREMENTS

Transportation Department required safety factors are:
$FS_{min} > 1.5$ for *long-term* conditions
$FS_{min} \approx 1.3$ for *short-term* conditions

settlement -

Primary consolidation must be completed prior to paving roadway. A total fill height of 2 m is anticipated to reach design elevation. This height includes the additional fill material thickness to compensate for anticipated settlements.

The following pages are from:
Holtz, R.D., Christopher, B.R., and Berg, R.R. (1997), Geosynthetic Engineering, B.Tech Publishers, Vancouver, B.C., 452 pp.

STEP 5. CHECK OVERALL BEARING CAPACITY

Recommended minimum safety factor (section 7.3-2) is 2.

A. Overall bearing capacity of soil, ignoring footing size is

$q_{ult} = c\, N_c$

$q_{ult} = 5 \text{ kPa} \times 5.14 = 25.7 \text{ kPa}$

Considering depth of embedment (*i.e.*, shearing will have to occur through the embankment for a bearing capacity failure) the bearing capacity is more accurately computed (see Meyerhof) as follows.

$N_c = 4.14 + 0.5\, B/D$ where, B = the base width of the embankment (~ 31 m), and D = the average depth of soft soil (~ 4.5 m)

$N_c = 4.14 + 0.5\,(31 \text{ m} / 4.5 \text{ m}) = 7.6$

$q_{ult} = 5 \text{ kPa} \times 7.6 = 38 \text{ kPa}$

maximum load, $P_{max} = \gamma_m\, H$

w/o a geotextile -

$P_{max} = 21.7 \text{ kN/m}^3 \times 2 \text{ m} = 43.4 \text{ kPa}$

implies $FS = 38 \div 43.4 = 0.88$ $\therefore$ NO GOOD

with a geotextile, and assuming that the geotextile will result in an even distribution of the embankment load over the width of the geotextile (*i.e.*, account for the slopes at the embankment edges),

$P_{avg} = A_E\, \gamma_m / B$ where, A = cross section area of embankment, B = base width of the embankment

$P_{avg} = \{[\tfrac{1}{2}\,(31 \text{ m} + 15 \text{ m})\, 2 \text{ m}]\; 21.7 \text{ kN/m}^3\} / 31 \text{ m}$

$P_{avg} = 32.2 \text{ kPa} < q_{ult}$ worst case Safety Factor Marginal

Add berms to increase bearing capacity. Berms, 3 m wide, can be added within the existing right-of-way, increasing the base width to 37 m. With this increase in width,

$N_c = 4.14 + 0.5\,(37 \text{ m} / 4.5 \text{ m}) = 8.3$

$q_{ult} = 5 \text{ kPa} \times 8.3 = 41.5 \text{ kPa}$

and,

$P_{avg} = 32.2 \text{ kPa}\,(31 / 37) = 27.0 \text{ kPa}$

$FS = 41.5 \text{ kPa} / 27.0 \text{ kPa} = 1.54$ Safety Factor O.K.

B. Lateral squeeze

From FHWA Foundation Manual (Cheney and Chassie, 1993) -

If $\gamma_{fill} \times H_{fill} > 3c$, then lateral squeeze of the foundation soil can occur. Since $P_{max} = 43.4$ kPa is much greater than 3c, even considering the crust layer (c = 10 kPa), a rigorous lateral squeeze analysis was performed using the method by Jürgeson (1934). In this method, the lateral stress beneath the toe of the

The following pages are from:
Holtz, R.D., Christopher, B.R., and Berg, R.R. (1997), Geosynthetic Engineering, B.Tech Publishers, Vancouver, B.C., 452 pp.

embankment is determined through charts or finite element analysis and compared to the shear strength of the soil. This method indicated a safety factor of approximately 1 for the 31 m base width. Adding the berm and extending the reinforcement to the toe of the berm decreases the potential for lateral squeeze as the lateral stress is reduced at the toe of the berm. The berms increased $FS_{SQUEEZZE}$ to greater than 1.5.

Also, comparing the reinforced design with Figure 7-5 indicates that the reinforced structure should be stable.

STEP 6. PERFORM ROTATIONAL SHEAR STABILITY ANALYSIS

Recommended minimum safety factor at end of construction (section 7.3-2) is 1.3.

The critical unreinforced failure surface is found through rotational stability methods. For this project, STABL4M was used and the critical, unreinforced surface FS = 0.72. As the soil supporting the embankment was highly compressible peat, the reinforcement was assumed to rotate such that $\beta = \theta$ (Figure 7-3 and Eq. 7-4b). Thus,

$$FS_{req} = \frac{M_R + T_g R}{M_D} \geq 1.3$$

$$T_r = \frac{1.3 M_D - M_R}{R}$$

therefore, $T_g \approx 263$ kN/m

Feasible - yes. Geosynthetics are available which exceed this strength requirement, especially if multiple layers are used. For this project, an installation damage factor of approximately equal to 0, and 2 layers were used:

Bottom:	90 kN/m
Top:	180 kN/m

The use of 2 layers allowed the lower cost bottom material to be used over the full embankment plus berm width, while the higher strength and more expensive geotextile was only placed under the embankment section where it was required.

STEP 7. CHECK LATERAL SPREADING (SLIDING) STABILITY

Recommended minimum safety factor (section 7.3-2) is 1.5.

The following pages are from:
Holtz, R.D., Christopher, B.R., and Berg, R.R. (1997), Geosynthetic Engineering, B.Tech Publishers, Vancouver, B.C., 452 pp.

A. from Figure 7-4b:

$T = FS \times P_A = FS \times 0.5\, K_a\, \gamma_m\, H^2$

$T = 1.5\,(0.5)\,[\tan^2(45 - 35/2)]\,(21.7\ kN/m^3)\,(2\ m)^2$

$T = 17.6\ kN/m$

Use FS = 3 for creep and installation damage
therefore, $T_{ls} = 53\ kN/m$

$T_{ls} < T_g$, therefore $T_{design} = T_g = 263\ kN/m$

B. check sliding:

FS > 6, OK

$$FS = \frac{b \tan \phi_{sg}}{K_a\, H}$$

$$FS = \frac{8m \times \tan 23}{0.27 \times 2m}$$

STEP 8. ESTABLISH TOLERABLE DEFORMATION (LIMIT STRAIN) REQUIREMENTS

For cohesionless sand and gravel over deformable peat use $\varepsilon = 10\%$

STEP 9. EVALUATE GEOSYNTHETIC STRENGTH REQUIRED IN LONGITUDINAL DIRECTION

From Step 7,
use $T_L = T_{ls} = 53\ kN/m$ for reinforcement and seams in the cross machine (X-MD) direction

STEP 10. ESTABLISH GEOSYNTHETIC PROPERTIES

A. Design strength and elongation based upon ASTM D 4595
Ultimate tensile strength

$T_{d1} = T_{ult} \geq 90\ kN/m$ in MD - Layer 1

$T_{d2} = T_{ult} \geq 180\ kN/m$ in MD - Layer 2

$T_{ult} \geq 53\ kN/m$ in X-MD - both layers

The following pages are from:
Holtz, R.D., Christopher, B.R., and Berg, R.R. (1997), Geosynthetic Engineering, B.Tech Publishers, Vancouver, B.C., 452 pp.

Reinforcement Modulus, J

$J = T_{ls} / 0.10 = 530$ kN/m for limit strain of 10%

$J \geq 530$ kN/m - MD and X-MD, both directions

B. seam strength

$T_{seam} \geq 53$ kN/m with controlled fill placement

C. soil-geosynthetic adhesion

from testing, per ASTM D 5321, $\phi_{sg} \geq 23°$

D. geotextile stiffness based upon site conditions and experience

E. survivability and constructability requirements

Assume:
1. medium ground pressure equipment
2. 300 mm first lift
3. uncleared subgrade

Use a High Survivability geotextile with elongation < 50% (from Tables 7-2 and 7-3):

Geotextile separator shall meet or exceed the minimum average roll values of:

Property	Test Method	Value
Grab Strength	ASTM D 4632	1400 N
Puncture Resistance	ASTM D 4833	500 N
Tear Resistance	ASTM D 4533	500 N

Drainage and filtration requirements -

Use Table 5-4

Need grain size distribution of subgrade soils

Determine: maximum AOS for retention

minimum $k_g > k_s$

minimum AOS for clogging resistance

Complete Steps 11 through 15 to finish design.

STEP 11. PERFORM SETTLEMENT ANALYSIS

STEP 12. ESTABLISH CONSTRUCTION SEQUENCE REQUIREMENTS

STEP 13. ESTABLISH CONSTRUCTION OBSERVATION REQUIREMENTS

STEP 14. HOLD PRECONSTRUCTION MEETING

STEP 15. OBSERVE CONSTRUCTION

The following pages are from:
Holtz, R.D., Christopher, B.R., and Berg, R.R. (1997), Geosynthetic Engineering, B.Tech Publishers, Vancouver, B.C., 452 pp.

7.6 SPECIFICATIONS

Because the reinforcement requirements for soft-ground embankment construction will be project and site specific, standard specifications, which include suggested geosynthetic properties, are not appropriate, and special provisions or a separate project specification must be used. The following example includes most of the items that should be considered in a reinforced embankment project.

HIGH STRENGTH GEOTEXTILE FOR EMBANKMENT REINFORCEMENT
(from Washington Department of Transportation, November 1994)

Description

This work shall consist of furnishing and placing construction geotextile in accordance with the details shown in the plans.

Materials

Geotextile and Thread for Sewing

The material shall be a woven geotextile consisting only of long chain polymeric filaments or yarns formed into a stable network such that the filaments or yarns retain their position relative to each other during handling, placement, and design service life. At least 85 percent by weight of the of the material shall be polyolefins or polyesters. The material shall be free from defects or tears. The geotextile shall be free of any treatment or coating which might adversely alter its hydraulic or physical properties after installation. The geotextile shall conform to the properties as indicated in Table 1.

Thread used shall be high strength polypropylene, polyester, or Kevlar thread. Nylon threads will not be allowed.

The following pages are from:
Holtz, R.D., Christopher, B.R., and Berg, R.R. (1997), Geosynthetic Engineering, B.Tech Publishers, Vancouver, B.C., 452 pp.

Geotextile Properties

Table 1.
Properties for high strength geotextile for embankment reinforcement.

Property	Test Method[1]	Geotextile Property Requirements[2]
AOS	ASTM D4751	.84 mm max. (#20 sieve)
Water Permittivity	ASTM D4491	0.02/sec. min.
Tensile Strength, min. in machine direction	ASTM D4595	(to be based on project specific design)
Tensile Strength, min. in x-machine direction	ASTM D4595	(to be based on project specific design)
Secant Modulus at 5% strain	ASTM D4595	(to be based on project specific design)
Seam Breaking Strength	ASTM D4884	(to be based on project specific design)
Puncture Resistance	ASTM D4833	330 N min.
Tear Strength, min. in machine and x-machine direction	ASTM D4533	330 N min.
Ultraviolet (UV) Radiation Stability	ASTM D4355	70% Strength Retained min., after 500 hrs in weatherometer

[1] The test procedures are essentially in conformance with the most recently approved ASTM geotextile test procedures, except geotextile sampling and specimen conditioning, which are in accordance with WSDOT Test Methods 914 and 915, respectively. Copies of these test methods are available at the Headquarters Materials Laboratory in Tumwater, Washington.
[2]All geotextile properties listed above are minimum average roll values (i.e., the test result for any sampled roll in a lot shall meet or exceed the values listed).

Geotextile Approval

Source Approval

The Contractor shall submit to the Engineer the following information regarding each geotextile proposed for use:

Manufacturer's name and current address,
Full Product name,
Geotextile structure, including fiber/yarn type, and
Geotextile polymer type(s).

If the geotextile source has not been previously evaluated, a sample of each proposed geotextile shall be submitted to the Headquarters Materials Laboratory in Tumwater for evaluation. After the sample and required information for each geotextile type have arrived at the Headquarters Materials Laboratory in Tumwater, a maximum of 14 calendar days will be required for this testing. Source approval will be based on conformance to the applicable values from Table 1. Source approval shall not be the basis of acceptance of specific lots of material unless the lot sampled can be clearly identified, and the number of samples tested and approved meet the requirements of WSDOT Test Method 914.

Geotextile Samples for Source Approval
Each sample shall have minimum dimensions of 1.5 meters by the full roll width of the geotextile. A minimum of 6 square meters of geotextile shall be submitted to the Engineer for testing. The geotextile machine direction shall be marked clearly on each sample submitted for testing. The machine direction is defined as the direction perpendicular to the axis of the geotextile roll.

The geotextile samples shall be cut from the geotextile roll with scissors, sharp knife, or other suitable method which produces a smooth geotextile edge and does not cause geotextile ripping or tearing. The samples shall not be taken from the outer wrap of the geotextile nor the inner wrap of the core.

Acceptance Samples
Samples will be randomly taken by the Engineer at the jobsite to confirm that the geotextile meets the property values specified.

Approval will be based on testing of samples from each lot. A "lot" shall be defined for the purposes of this specification as all geotextile rolls within the consignment (i.e., all rolls sent to the project site) which were produced by the same manufacturer during a continuous period of production at the same manufacturing plant and have the same product name. After the samples and manufacturer's certificate of compliance have arrived at the Headquarters Materials Laboratory in Tumwater, a maximum of 14 calendar days will be required for this testing. If the results of the testing show that a geotextile lot, as defined, does not meet the properties required in Table 1, the roll or rolls which were sampled will be rejected. Two additional rolls for each roll tested which failed from the lot previously tested will then be selected at random by the Engineer for sampling and retesting. If the retesting shows that any of the additional rolls tested do not meet the required properties, the entire lot will be rejected. If the test results from all the rolls retested meet the required properties, the entire lot minus the roll(s) which failed will be accepted. All geotextile which has defects, deterioration, or damage, as determined by the Engineer, will also be rejected. All rejected geotextile shall be replaced at no expense to the Contracting Agency.

Certificate of Compliance
The Contractor shall provide a manufacturer's certificate of compliance to the Engineer which includes the following information about each geotextile roll to be used:

Manufacturer's name and current address,
Full product name,

Geotextile structure, including fiber/yarn type,
Geotextile polymer type(s),
Geotextile roll number, and
Certified test results.

Approval of Seams

If the geotextile seams are to be sewn in the field, the Contractor shall provide a section of sewn seam before the geotextile is installed which can be sampled by the Engineer.

The seam sewn for sampling shall be sewn using the same equipment and procedures as will be used to sew the production seams. The seam sewn for sampling must be at least 2 meters in length. If the seams are sewn in the factory, the Engineer will obtain samples of the factory seam at random from any of the rolls to be used. The seam assembly description shall be submitted by the Contractor to the Engineer and will be included with the seam sample obtained for testing. This description shall include the seam type, stitch type, sewing thread type(s), and stitch density.

Construction Requirements

Geotextile Roll Identification, Storage, and Handling

Geotextile roll identification, storage, and handling of the geotextile shall be in conformance to ASTM D 4873. During periods of shipment and storage, the geotextile shall be kept dry at all times and shall be stored off the ground. Under no circumstances, either during shipment or storage, shall the materials be exposed to sunlight, or other form of light which contains ultraviolet rays, for more than five calendar days.

Preparation and Placement of the Geotextile Reinforcement

The area to be covered by the geotextile shall be graded to a smooth, uniform condition free from ruts, potholes, and protruding objects such as rocks or sticks. The Contractor may construct a working platform, up to 0.6 meters in thickness, in lieu of grading the existing ground surface. A working platform is required where stumps or other protruding objects which cannot be removed without excessively disturbing the subgrade are present. All stumps shall be cut flush with the ground surface and covered with at least 150 mm of fill before placement of the first geotextile layer. The geotextile shall be spread immediately ahead of the covering operation. The geotextile shall be laid with the machine direction perpendicular or parallel to centerline as shown in Plans. Perpendicular and parallel directions shall alternate. All seams shall be sewn. Seams to connect the geotextile strips end to end will not be allowed, as shown in the Plans. The geotextile shall not be left exposed to sunlight during installation for a total of more than 10 calendar days. The geotextile shall be laid smooth without excessive wrinkles. Under no circumstances shall the geotextile be dragged through mud or over sharp objects which could damage the geotextile. The cover material shall be placed on the geotextile in such a manner that a minimum of 200 mm of material will be between the equipment tires or tracks and the geotextile at all times. Construction vehicles shall be limited in size and weight such that rutting in the initial lift above the geotextile is not greater than 75 mm deep, to prevent overstressing the geotextile. Turning of vehicles on the first lift above the geotextile will not be permitted. Compaction of the first lift above the geotextile shall be limited to routing of placement and spreading equipment only. No

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Holtz, R.D., Christopher, B.R., and Berg, R.R. (1997), Geosynthetic Engineering, B.Tech Publishers, Vancouver, B.C., 452 pp.

vibratory compaction will be allowed on the first lift.
Small soil piles or the manufacturer's recommended method shall be used as needed to hold the geotextile in place until the specified cover material is placed.

Should the geotextile be torn or punctured or the sewn joints disturbed, as evidenced by visible geotextile damage, subgrade pumping, intrusion, or roadbed distortion, the backfill around the damaged or displaced area shall be removed and the damaged area repaired or replaced by the Contractor at no expense to the Contracting Agency. The repair shall consist of a patch of the same type of geotextile placed over the damaged area. The patch shall be sewn at all edges.

If geotextile seams are to be sewn in the field or at the factory, the seams shall consist of two parallel rows of stitching, or shall consist of a J-seam, Type Ssn-1, using a single row of stitching. The two rows of stitching shall be 25 mm apart with a tolerance of plus or minus 13 mm and shall not cross, except for restitching. The stitching shall be a lock-type stitch. The minimum seam allowance, i.e., the minimum distance from the geotextile edge to the stitch line nearest to that edge, shall be 40 mm if a flat or prayer seam, Type SSa-2, is used. The minimum seam allowance for all other seam types shall be 25 mm. The seam, stitch type, and the equipment used to perform the stitching shall be as recommended by the manufacturer of the geotextile and as approved by the Engineer.

The seams shall be sewn in such a manner that the seam can be inspected readily by the Engineer or his representative. The seam strength will be tested and shall meet the requirements stated in this Specification.

Embankment construction shall be kept symmetrical at all times to prevent localized bearing capacity failures beneath the embankment or lateral tipping or sliding of the embankment. Any fill placed directly on the geotextile shall be spread immediately. Stockpiling of fill on the geotextile will not be allowed.

The embankment shall be compacted using Method B of Section 2-03.3(14)C. Vibratory or sheepsfoot rollers shall not be used to compact the fill until at least 0.5 meters of fill is covering the bottom geotextile layer and until at least 0.3 meters of fill is covering each subsequent geotextile layer above the bottom layer.

The geotextile shall be pretensioned during installation using either Method 1 or Method 2 as described herein. The method selected will depend on whether or not a mudwave forms during placement of the first one or two lifts. If a mudwave forms as fill is pushed onto the first layer of geotextile, Method 1 shall be used. Method 1 shall continue to be used until the mudwave ceases to form as fill is placed and spread. Once mudwave formation ceases, Method 2 shall be used until the uppermost geotextile layer is covered with a minimum of 0.3 meters of fill. These special construction methods are not needed for fill construction above this level. If a mudwave does not form as fill is pushed onto the first layer of geotextile, then Method 2 shall be used initially and until the uppermost geotextile layer is covered with at least 0.3 meters of fill.

The following pages are from:
Holtz, R.D., Christopher, B.R., and Berg, R.R. (1997), Geosynthetic Engineering, B.Tech Publishers, Vancouver, B.C., 452 pp.

Method 1

After the working platform, if needed, has been constructed, the first layer of geotextile shall be laid in continuous transverse strips and the joints sewn together. The geotextile shall be stretched manually to ensure that no wrinkles are present in the geotextile. The fill shall be end-dumped and spread from the edge of the geotextile. The fill shall first be placed along the outside edges of the geotextile to form access roads. These access roads will serve three purposes: to lock the edges of the geotextile in place, to contain the mudwave, and to provide access as needed to place fill in the center of the embankment. These access roads shall be approximately 5 meters wide. The access roads at the edges of the geotextile shall have a minimum height of 0.6 meters when completed. Once the access roads are approximately 15 meters in length, fill shall be kept ahead of the filling operation, and the access roads shall be kept approximately 15 meters ahead of this filling operation as shown in the Plans. Keeping the mudwave ahead of this filling operation and keeping the edges of the geotextile from moving by use of the access roads will effectively pre-tension the geotextile. The geotextile shall be laid out no more than 6 meters ahead of the end of the access roads at any time to prevent overstressing of the geotextile seams.

Method 2

After the working platform, if needed, has been constructed, the first layer of geotextile shall be laid and sewn as in Method 1. The first lift of material shall be spread from the edge of the geotextile, keeping the center of the advancing fill lift ahead of the outside edges of the lift as shown in the Plans. The geotextile shall be manually pulled taut prior to fill placement. Embankment construction shall continue in this manner for subsequent lifts until the uppermost geotextile layer is completely covered with 0.3 meters of compacted fill.

Measurement

High strength geotextile for embankment reinforcement will be measured by the square meter for the ground surface area actually covered.

Payment

The unit contract price per square meter for "High Strength Geotextile For Embankment Reinforcement," shall be full pay to complete the work as specified.

7.7 COST CONSIDERATIONS

The cost analysis for a geosynthetic reinforced embankment includes:

1. Geosynthetic cost: including purchase price, factory prefabrication, and shipping.
2. Site preparation: including clearing and grubbing, and working table preparation.
3. Geosynthetic placement: related to field workability (see Christopher and Holtz, 1989),
   a) with no working table, or
   b) with a working table.