ENCE 3610
Soil Mechanics

Lecture 13
Slope Stability

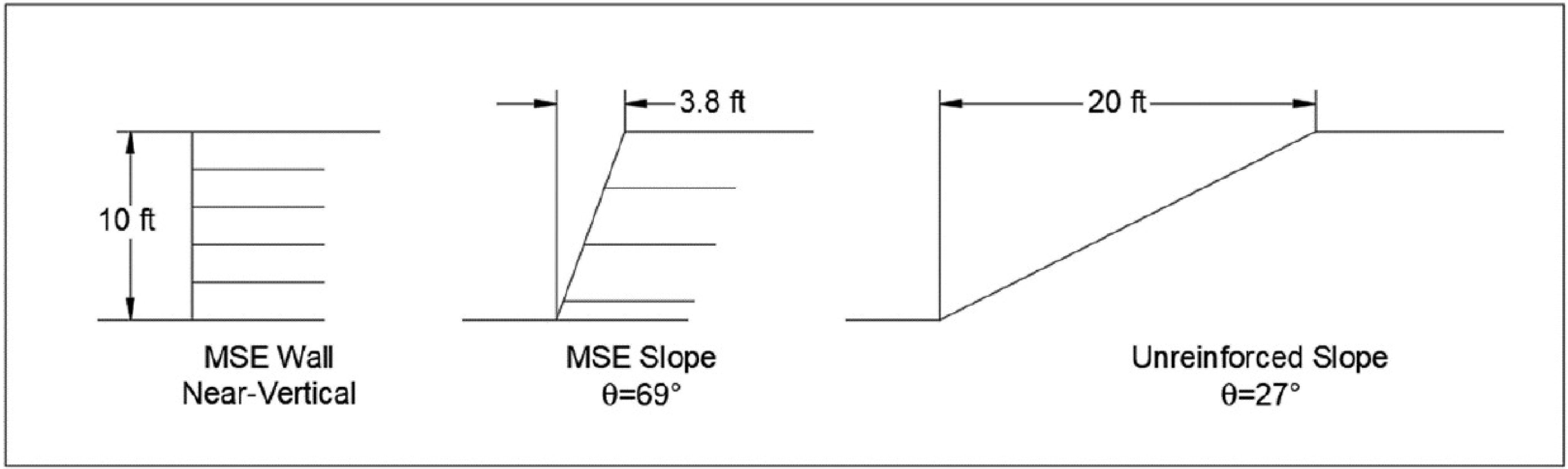

Figure 7-12 Difference in Usable Land for Walls and Slopes

Overview of Slope Stability
Types of Slope Failure

- Slope failure is one of the most important types of failure in geotechnical engineering
- As opposed to other types of failure (especially settlement,) slope failures are generally catastrophic
- Slope failures in Sweden in early 1900's led to development of one of the first analytic methods of geotechnical analysis—rotational failure analysis using slip circle and method of slices
- Falls (rock falls)—generally by surface rock
- Topples—rotation of rock away from a vertically inclined joint or fissure
- Slides
  - Rotational slides—the "classic" mode of slope failure, rotational failure along a circular (or nearly so) surface
  - Translational slides—failure along a planar surface
- Spreads—like translational slide, except material separates and moves apart as it moves downward
- Flows—material comes down in nearly liquid form—includes avalanches

Aspects of Slope Stability Analysis

- Limit equilibrium analysis
  - Evaluate the slope as if it were about to fail and determine the shear strength along the surface
  - Computed stresses are compared to the shear strength to determine the factor of safety

$$F = \frac{\text{strength}}{\text{load}}$$

  - Same concept used in bearing capacity analysis of shallow and deep foundations
- Effective Stress vs. Total Stress Analyses
  - Generally, effective stress analysis is used in slope stability
  - Total stress analysis used when excess pore water pressures are present
- Critical failure surface
  - Validity of analysis depends upon choosing correct failure surface (surface of lowest factor of safety)
  - Trial and error problem; computer program makes search much more effective
- Soil Strength Models

Examples of Limit Equilibrium Analysis

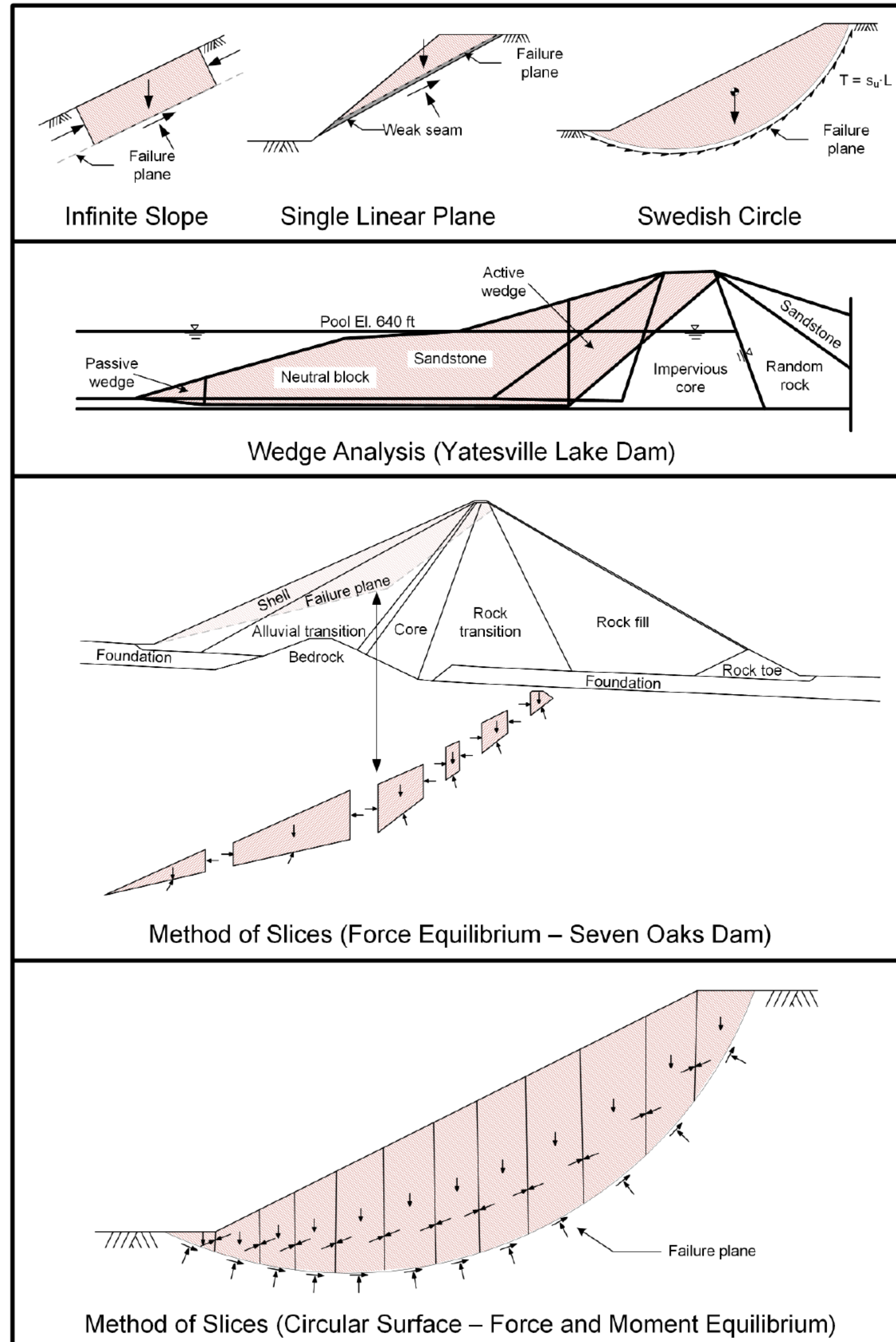

Figure 7-3 Examples of Limit Equilibrium Analysis

Soil Strength Models for Different Soil Types and Drainage Conditions

Table 7-1 Strength Models for Different Soil Types and Drainage Conditions

Soil type	Drainage conditions	s, strength	Comments
Coarse-grained soils	Drained	$s = \sigma'_{ff} \cdot \tan(\phi')$	Drained conditions are often assumed for coarse-grained soils, such as sands and gravels under static loading conditions. Non-linear envelopes can be used as well.
Coarse-grained soils	Undrained	$s = s_{su}$	Used for dynamic loading and certain conditions of static loading. Can be used for clays and silts as well for dynamic or cyclic loading. For normal undrained analysis, the effective stress strength parameters should be used for coarse-grained soils.
Overconsolidated fine-grained soils	Drained	$s = c' + \sigma'_{ff} \cdot \tan(\phi')$	A non-linear envelope can be used for this case as well. Some engineers do not like to use effective stress cohesion for any soil.
Normally consolidated fine-grained soils	Drained	$s = \sigma'_{ff} \cdot \tan(\phi')$	The effective stress friction angle should correspond to the soil in a normally consolidated state. This is equal to the fully softened friction angle.
Fine-grained soils (saturated)	Undrained	$s = s_u$	The undrained shear strength can be determined using a variety of laboratory or *in situ* tests. The magnitude of s_u can vary with depth.
Fine-grained soils (partially saturated)	Undrained	$s = c + \sigma_{ff} \cdot \tan(\phi)$	This strength model is used for partially saturated soils like compacted clays. The shear strength parameters should be measured using Unconsolidated Undrained triaxial tests. Only use a linear envelope for the range of stress where it appears to be appropriate.

Note: σ'_{ff} = effective stress on failure plane, ϕ' = effective stress friction angle, c' = effective cohesion,
σ_{ff} = total normal stress on failure plane, ϕ = total stress friction angle, c = total stress cohesion,
s_{su} = undrained steady state shear strength, and s_u = undrained shear strength

Plasticity Theory: Upper and Lower Bound

Since slope stability inevitably involves plasticity, it is necessary to review the theory of plasticity.

In formulating the basic theorems of the theory of plasticity two types of fields are being used, which can be defined as follows.

1. An *equilibrium system*, or a *statically admissible field* of stresses is a distribution of stresses that satisfies the following conditions :a. it satisfies the conditions of equilibrium in each point of the body,b. it satisfies the boundary conditions for the stresses,c. the yield condition is not exceeded in any point of the body.

2. A *mechanism*, or a *kinematically admissible field* of displacements is a distribution of displacements and deformations that satisfies the following conditions :a. the displacement field is compatible, i.e. no gaps or overlaps are produced in the body (sliding of one part along another part is allowed),b. it satisfies the boundary conditions for the displacements,c. wherever deformations occur the stresses satisfy the yield condition.

The basic theorems of the plasticity theory are,

1. *Lower bound theorem.*The true failure load is larger than the load corresponding to an equilibrium system.

2. *Upper bound theorem.*The true failure load is smaller than the load corresponding to a mechanism, if that load is determined using the virtual work principle.

The first theorem states that if for a certain load an equilibrium system can be found (ignoring compatibility), then that load can certainly be carried. The second theorem states that if a mechanism can be found corresponding to a certain load (where equilibrium is taken into account only insofar as it corresponds to the chosen deformation), then this load can certainly *not* be carried.

Vertical Slope in Cohesive Soils Lower Bound Solution

A simple equilibrium system is shown in Figure 44.2, consisting of three zones. On the interfaces between the zones the normal stresses parallel to these interfaces may be discontinuous, without disturbing equilibrium, see Chapter 40. The boundary conditions for the stresses are that the normal stresses and the shear stresses are zero, all along the upper surface. These conditions are exactly satisfied by the stress fields indicated in Figure 44.2. This field can be constructed by starting to assume that in the entire field the shear stress $\sigma_{xz} = 0$, because this shear stress must be zero on the two horizontal boundaries, and on the vertical slope. In order to satisfy the condition that on the vertical slope the horizontal stress $\sigma_{xx} = 0$, it follows from the equation of horizontal equilibrium, eq. (44.1) that this stress must be zero throughout zone I. The expressions for the vertical normal stress σ_{zz} follow immediately from the equation of vertical equilibrium (44.2) by putting $\sigma_{zx} = 0$, and using the boundary conditions at the top of the soil. The expressions for the horizontal stress σ_{xx} in zones II and III can be chosen arbitrarily, but they must be constant in x-direction (to satisfy horizontal equilibrium), and preferably as close to σ_{zz} as possible, to keep the maximum shear stress as small as possible. By choosing $\sigma_{xx} = \gamma z$ the Mohr circle in zone II, in the lower left part, reduces to a point. This seems to be very attractive, but the consequence is that in zone III, the lower right part, the difference of the stresses σ_{xx} and σ_{zz} is rather large, $\sigma_{zz} - \sigma_{xx} = \gamma h$.

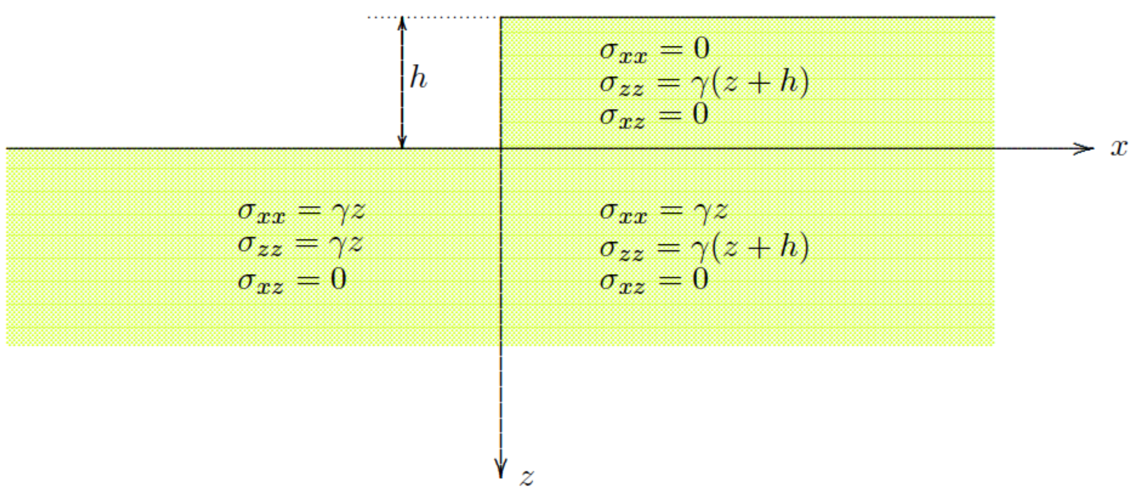

Figure 44.2: Equilibrium system.

The vertical and horizontal stresses are principal stresses in this case, because everywhere $\sigma_{xz} = 0$. Therefore, the Mohr-Coulomb criterion now is $|\, \sigma_{xx} - \sigma_{zz} \,| \leq 2c$. As the largest value of $\sigma_{xx} - \sigma_{zz}$ occurs in the lower right part, it follows that the critical value of the height $h = 2c/\gamma$. This is a lower bound, i.e.

$$h_c \geq \frac{2c}{\gamma}. \tag{44.3}$$

Vertical Slope in Cohesive Soils
Upper Bound Solution

A simple upper bound can be found by considering a mechanism consisting of a single straight slip surface, at an angle α with the vertical direction, see Figure 44.3.

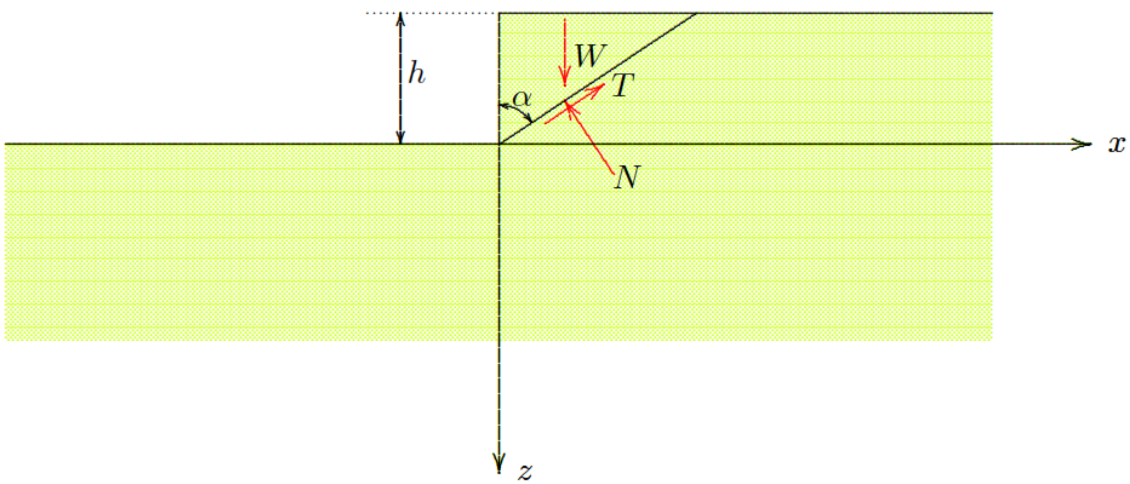

Figure 44.3: Mechanism with straight slip surface.

The weight of the sliding wedge is $W = \frac{1}{2}\gamma h^2 \tan\alpha$, and it follows from the condition of equilibrium in the direction of sliding (that is equivalent with the virtual work principle for the deformation mode of the mechanism) that

$$T = W\cos\alpha = \tfrac{1}{2}\gamma h^2 \sin\alpha.$$

Because the length of the slip plane is $h/\cos\alpha$ it follows that

$$T = \frac{c\,h}{\cos\alpha}.$$

Combination of these two equations gives

$$h = \frac{4c}{\gamma}\,\frac{1}{\sin 2\alpha}. \tag{44.7}$$

The height of the excavation appears to depend upon the angle α. The critical sliding plane is the one for which h is a minimum. This minimum occurs if $\sin 2\alpha$ has its maximum value, i.e. $2\alpha = \frac{1}{2}\pi$, or $\alpha = 45°$. Because this is an upper bound it follows that

$$h_c \le \frac{4c}{\gamma}. \tag{44.8}$$

This is the upper bound for straight slip surfaces.

Slopes of Purely Cohesionless Soils vs. Purely Cohesive Ones

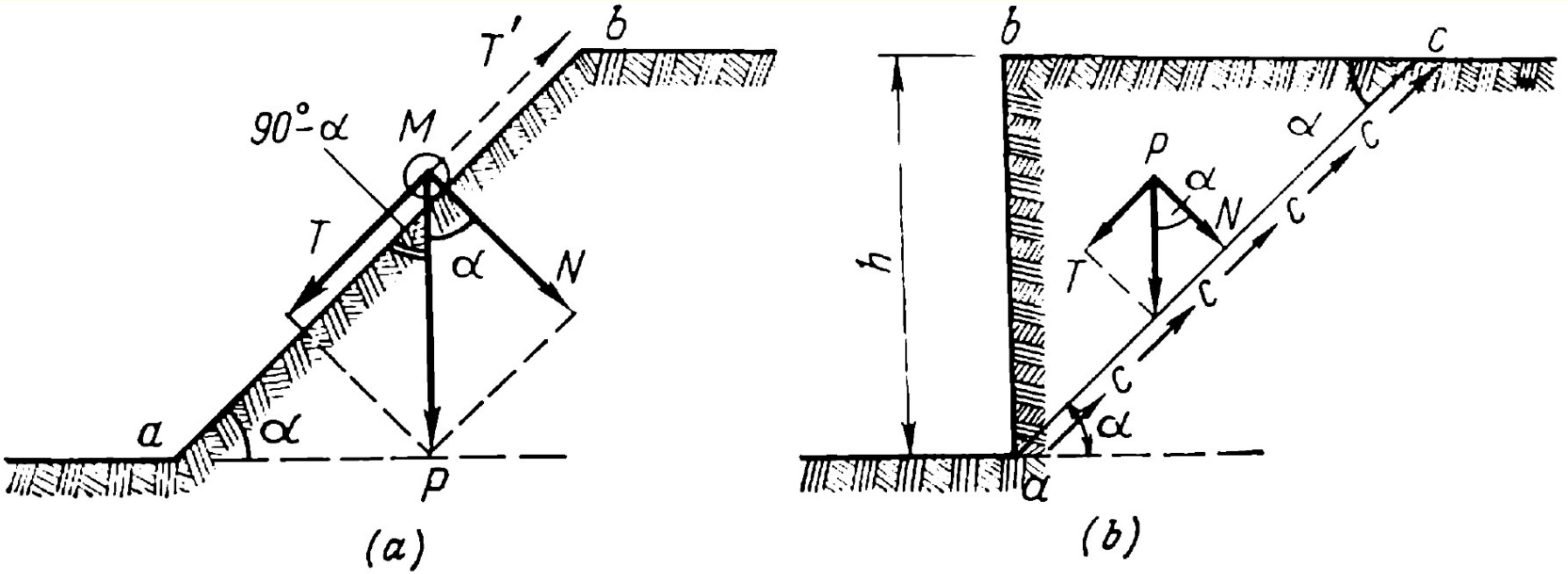

Fig. 75. Forces acting on a particle in a slope of perfectly loose soil (*a*) and on the vertical dip of a cohesive soil (*b*)

Critical Height and Slope Stability Number

- Critical Height
  - Critical height is found between upper and lower bounds $\frac{2c}{\gamma} \leq h_{cr} < \frac{4c}{\gamma}$
  - Values have also been determined between these two
  - Obviously the lower bound is the most conservative

Slope Stability Number

Rearranging and redefining the coefficient

$$N_o = \frac{\gamma h}{c}$$

Adding a factor of safety

$$N_o = F_s \frac{\gamma h}{c}$$

Planar Failure Surfaces

- Most slope stability analyses (including circular) are two dimensional, i.e, they assume the slope is infinitely long with the same profile
- Infinite Slope Analysis
  - Failure surface is under the slope and also parallel to it
  - Usually one when a weak layer is above a hard (bedrock) layer
- Planar failure analysis
  - Failure surface is under the slope but not parallel to it

Infinite Slope

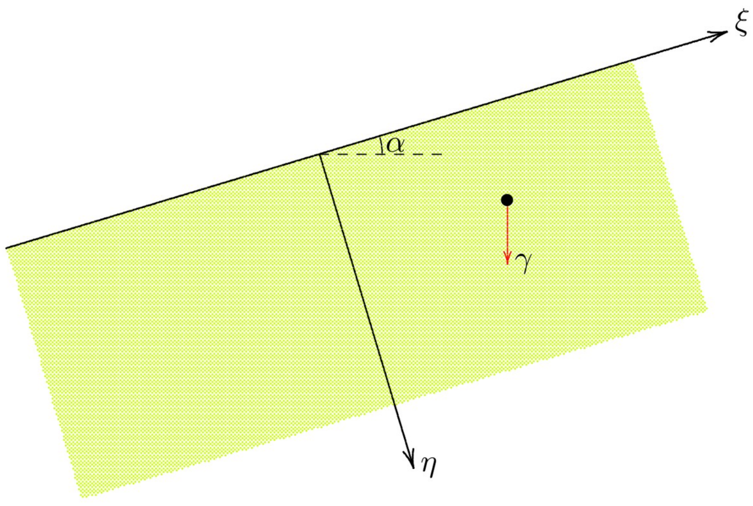

Figure 45.1: Infinite slope in dry sand.

$$\sigma'_{\eta\xi} = -\gamma\eta\sin\alpha,$$
$$\sigma'_{\eta\eta} = +\gamma\eta\cos\alpha.$$

$$\frac{|\sigma'_{\eta\xi}|}{|\sigma'_{\eta\eta}|} = \tan\alpha$$

$$F = \frac{|\sigma'_{\eta\xi}/\sigma'_{\eta\eta}|_{\max}}{|\sigma'_{\eta\xi}/\sigma'_{\eta\eta}|} \quad F = \frac{\tan\phi}{\tan\alpha}$$

- Only valid for purely cohesionless soils
- Only valid for the case where the slope and the failure surface are parallel
- Only valid when water table is not significant
- Result independent of unit weight
- Slope stability degraded in the case when water is flowing down the slope or in a purely horizontal direction (earth dams)

Soil With Steady-State Seepage

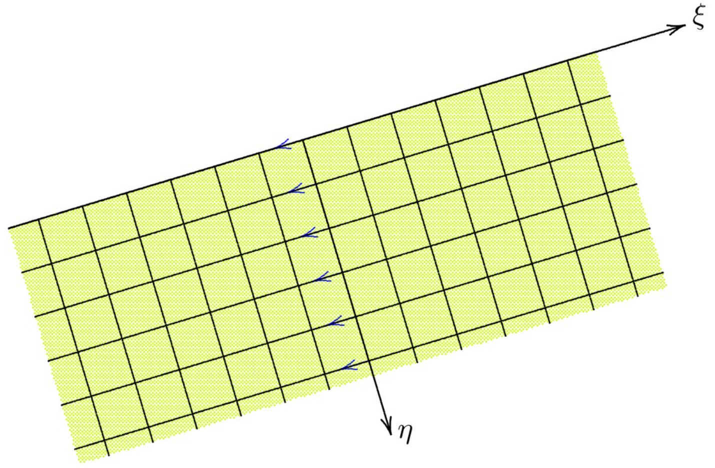

Figure 45.3: Parallel groundwater flow.

$$h = z + \frac{p}{\gamma_w} = A\frac{\eta}{\gamma_w} - \eta\cos\alpha + \xi\sin\alpha$$

$$A = \gamma_w \cos\alpha \qquad p = \gamma_w \eta \cos\alpha$$

$$\frac{\partial \sigma'_{\xi\xi}}{\partial \xi} + \frac{\partial \sigma'_{\eta\xi}}{\partial \eta} + \gamma \sin\alpha = 0,$$

$$\frac{\partial \sigma'_{\xi\eta}}{\partial \xi} + \frac{\partial \sigma'_{\eta\eta}}{\partial \eta} - (\gamma - \gamma_w)\cos\alpha = 0.$$

$$\sigma'_{\eta\xi} = -\gamma\eta\sin\alpha,$$

$$\sigma'_{\eta\eta} = (\gamma - \gamma_w)\eta\cos\alpha.$$

$$F = \frac{\gamma - \gamma_w}{\gamma}\,\frac{\tan\phi}{\tan\alpha}$$

Infinite slopes, with seepage and cohesion:

$$F = \frac{c'}{\gamma H sin\alpha cos\alpha} + \frac{(\gamma - \gamma_w)\, tan\phi'}{\gamma tan\alpha}$$

where H =vertical distance from surface

Infinite Slope Example

- Given
  - Infinite Slope, H =15’, α = 20 deg.
  - Soil, φ= 10 deg., c = 500 psf, γ = 110 pcf, saturated w/seepage
- Find
  - Factor of safety for translational failure
- Solution
  - Substituting into the equation below, FS = 1.15

Infinite slopes, with seepage and cohesion:

$$F = \frac{c'}{\gamma H sin\alpha cos\alpha} + \frac{(\gamma - \gamma_w)\, tan\phi'}{\gamma tan\alpha}$$

where H =vertical distance from surface

Methods of Failure Analysis for Rotational Failure

- Friction Circle Method
- Chart and Table Solutions
  - Taylor's Stability Number
  - Janbu Charts
  - A good way for preliminary calculations
- Non-circular failure surfaces
- Vertical Slopes
- Methods of Slices
  - Fellenius Method (Ordinary Method)
  - Bishop Method (Simplified)
  - Spencer
  - Morganstern-Price
  - GLE
  - The "classic" way to analyse slope stability, but computationally intensive by hand
- Finite Element Methods

Limit Equalibrium and Circular Slope Failure

Most methods assume that the soil fails along a circular slip surface, see Figure 46.1. The soil above the slip surface is subdivided into a number of *slices*, bounded by vertical interfaces. At the slip surface the shear stress is τ, which is assumed to be a factor F smaller than the maximum possible shear stress, i.e.

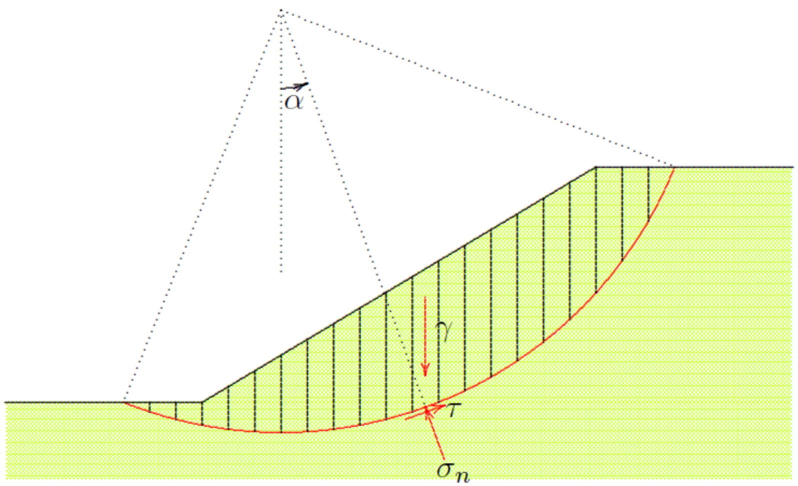

Figure 46.1: Circular slip surface.

$$\tau = \frac{1}{F}\,(c + \sigma_n' \tan\phi). \tag{46.1}$$

The factor F is assumed to be the same for all slices, an assumption that is common to all methods.

The equilibrium equation to be used in conjunction with a circular slip surface is the equation of equilibrium of moments with respect to the center of the circle. This equation gives

$$\sum \gamma h b R \sin\alpha = \sum \frac{\tau b R}{\cos\alpha}. \tag{46.2}$$

Here h is the height of a slice, b its width, γ the volumetric weight of the soil in the slice, and R is the radius of the circle. More generally it can be defined that γbh is the weight of the slice, possibly consisting of a sum of parts with different unit weight.

If all slices have the same width, it now follows from (46.1) and (46.2) that

$$F = \frac{\sum[(c + \sigma_n' \tan\phi)/\cos\alpha]}{\sum \gamma h \sin\alpha}. \tag{46.3}$$

This is the basic formula for many computation methods. The various methods usually differ in the method of calculating the normal effective stress σ_n'.

Fellenius Method

In Fellenius' method, the oldest method for the analysis of slope stability, it is assumed that there are no forces between the slices. The only remaining forces acting on a slice, see Figure 46.2, then are the weight γhb, a normal stress σ_n and a shear stress τ at the bottom of the slice. The normal stress σ_n can most conveniently be expressed into the known weight by considering the equilibrium of the slice in the direction perpendicular to the slip surface. This gives

$$\sigma_n = \gamma h \cos^2 \alpha, \tag{46.4}$$

and, because $\sigma_n = \sigma'_n + p$,

$$\sigma'_n = \gamma h \cos^2 \alpha - p. \tag{46.5}$$

Substitution into (46.3) finally gives

$$F = \frac{\sum\{[c + (\gamma h \cos^2 \alpha - p) \tan \phi]/\cos \alpha\}}{\sum \gamma h \sin \alpha}. \tag{46.6}$$

This is the Fellenius formula.

For a slope in homogeneous soil the computation can be executed by assuming a certain location of the circle, and subdividing the sliding soil wedge into 10 or 20 slices. By measuring the values of α and h for each slice the value of the stability factor F can be determined. This must be repeated for a large number of circles, to determine the smallest value of F. In non-homogeneous soil the computation is somewhat more complicated because for each slice the value of γh must be determined as the sum of the contributions of a number of layers in the slice.

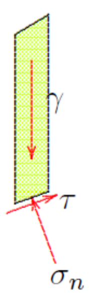

Figure 46.2: Fellenius.

Several objections can be made against this method. To begin with, a sound fundamental base lacks for all slip surface methods for materials with internal friction, as seen before (see Chapter 42). But there are other objections as well. Disregarding the forces transmitted between the slices is a severe approximation, and vertical equilibrium is violated. Furthermore, there is an internal inconsistency in stating on the one hand that sliding occurs along the circle, and on the other hand stating that the

Bishop's Method

A method that is frequently used in engineering practice is Bishop's method. In this method the forces between the slices are not neglected, but it is assumed that the resultant force is horizontal, see Figure 46.3. By considering the vertical equilibrium of each slice only, the horizontal forces do not enter into the computations, however.

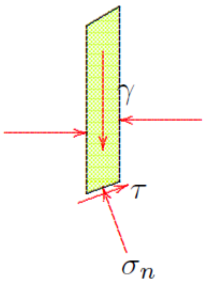

Figure 46.3: Bishop.

The basic equation again is the equation of moment equilibrium, eq. (46.3). Vertical equilibrium of a slice now requires that

$$\gamma h = \sigma_n + \tau \frac{\sin\alpha}{\cos\alpha} = \sigma'_n + p + \tau \frac{\sin\alpha}{\cos\alpha}.$$

If in this equation the value of τ is written, in agreement with (46.1), as $\tau = (c + \sigma'_n \tan\phi)/F$, the result is

$$\sigma'_n\left(1 + \frac{\tan\alpha\,\tan\phi}{F}\right) = \gamma h - p - \frac{c}{F}\tan\alpha. \qquad (46.7)$$

Substitution of σ'_n into (46.3) now leads to the final equation for Bishop's method,

$$F = \frac{\sum \dfrac{c + (\gamma h - p)\tan\phi}{\cos\alpha(1 + \tan\alpha\,\tan\phi/F)}}{\sum \gamma h \sin\alpha}. \qquad (46.8)$$

Because the stability factor F also appears in the right hand side, it must be determined iteratively, by starting from an initial estimate (for instance $F = 1$), and then calculating an updated value using (46.8). This must be repeated until the value of F no longer changes. In general the procedure converges rather fast. As the computations must be executed by a computer program anyhow (many circles have to be investigated) the iterations can easily be incorporated into the program. Computer programs are available on the internet (search for *geotechnical software*).

If $\phi = 0$ the Bishop and Fellenius methods are identical. If $\phi > 0$ Bishop's method usually gives somewhat smaller values. Because Bishop's method is more consistent (vertical equilibrium is satisfied), and it confirms known results for special cases, it is often used in geotechnical engineering. Various other methods have been developed, but the results often differ only slightly from those obtained by Bishop's method. That may explain its popularity.

Method of Slices

46.1 Circular slip surface

Most methods assume that the soil fails along a circular slip surface, see Figure 46.1. The soil above the slip surface is subdivided into a number of *slices*, bounded by vertical interfaces. At the slip surface the shear stress is τ, which is assumed to be a factor F smaller than the maximum possible shear stress, i.e.

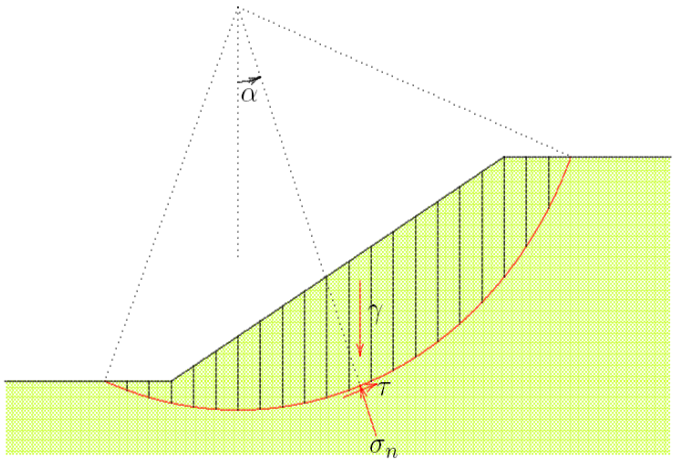

Figure 46.1: Circular slip surface.

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The equilibrium equation to be used in conjunction with a circular slip surface is the equation of equilibrium of moments with respect to the center of the circle. This equation gives

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Methods of Slip Circle Analysis

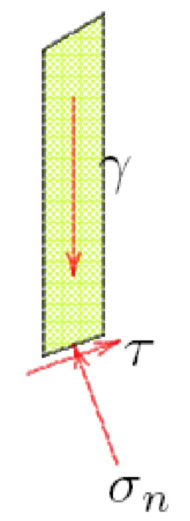


Figure 46.2: Fellenius.

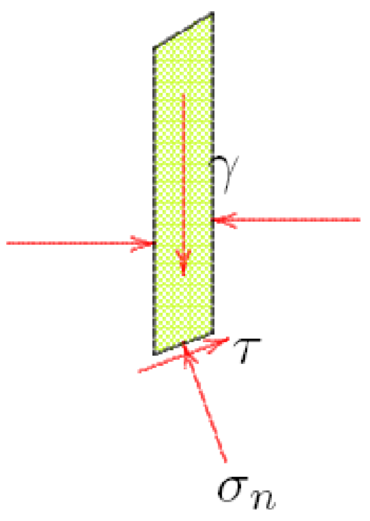


Figure 46.3: Bishop.

Use of Computer Software

- Automates many of the processes that are required for slope analysis
- Eliminates the need for iterative solutions
- Enables running multiple cases and varying parameters without difficulty
- For our purposes we will use the SLOPE software program from Verruijt

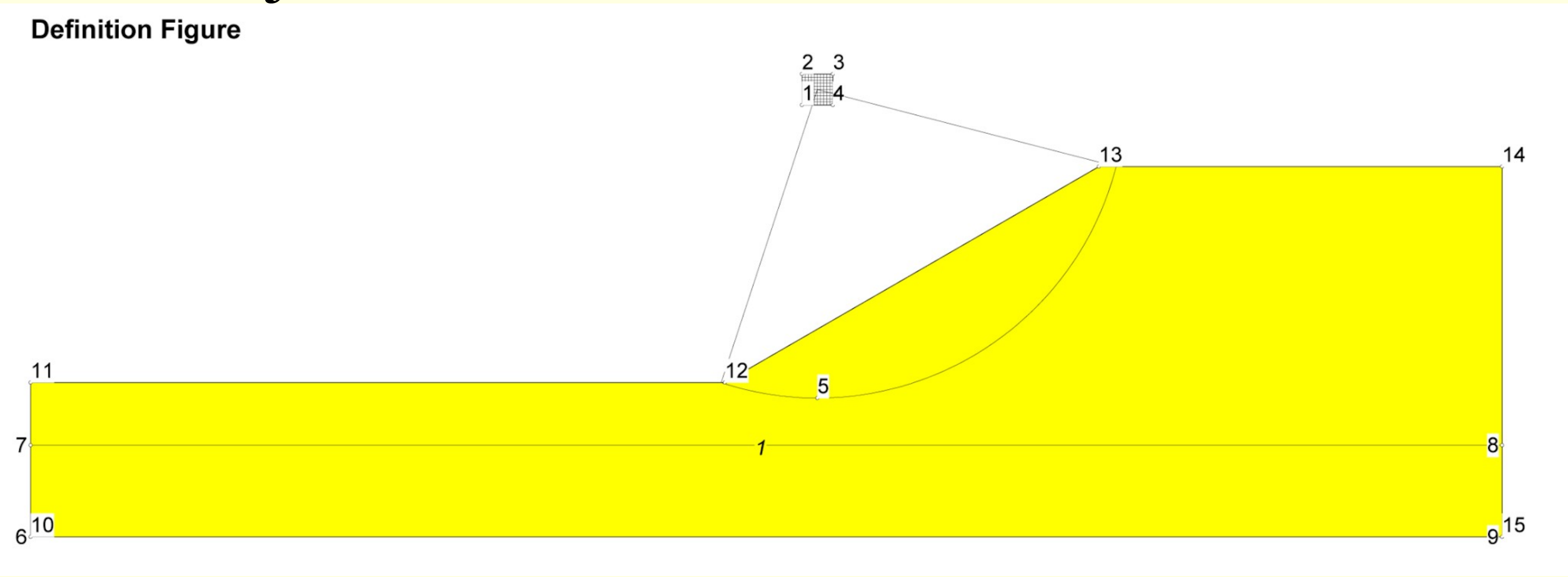


Table Solution (from Tsytovich)

- A “quick and dirty” method to estimate the stability of the slope
- Equation: $$FS = A\tan\phi + B\frac{c}{\gamma H}$$
  - FS = factor of safety
  - ϕ = internal friction angle of soil
  - c = cohesion of the soil
  - γ = unit weight of soil
  - A,B = coefficients given in the next slide, functions of e and h
  - e = distance from slope toe to hard underground layer (see diagram at right
  - h or H = height of slope from toe to top
  - m = second number of slope ratio rise:run, assuming rise = 1
- Solving for H,

$$H = \frac{Bc}{\gamma(FS - A\tan\phi)}$$

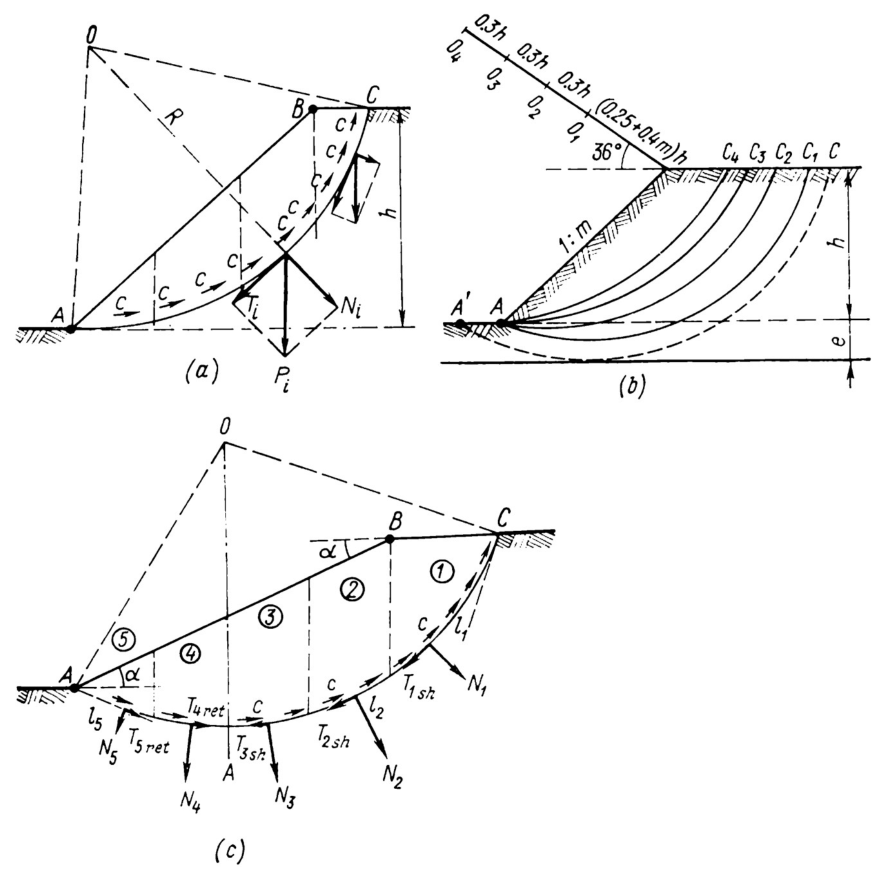


Fig. 78. To prediction of stability of a slope by circular-cylindrical slip surfaces

a) diagram of acting forces; (*b*) position of critical slip arcs; (c) diagram of forces acting along the slip surface

Chart Solution (from Tsytovich)

Coefficients *A* and *B* for Approximate Prediction of Stability of Slopes

Slope ratio $1 : m$	Slip surface passes through lower edge of slope		Slip surface passes through the base and has a horizontal tangent at a depth of							
			$e = \frac{1}{4} h$		$e = \frac{1}{2} h$		$e = h$		$e = 1\frac{1}{2} h$	
	A	*B*	*A*	*B*	*A*	*B*	*A*	*B*	*A*	*B*
1 : 1.00	2.34	5.79	2.56	6.10	3.17	5.92	4.32	5.80	5.78	5.75
1 : 1.25	2.64	6.05	2.66	6.32	3.24	6.62	4.43	5.86	5.86	5.80
1 : 1.50	2.64	6.50	2.80	6.53	3.32	6.13	4.54	5.93	5.94	5.85
1 : 1.75	2.87	6.58	2.93	6.72	3.41	6.26	4.66	6.00	6.02	5.90
1 : 2.00	3.23	6.70	3.10	6.87	3.53	6.40	4.78	6.08	6.10	5.95
1 : 2.25	3.19	7.27	3.26	7.23	3.66	6.56	4.90	6.16	6.18	5.98
1 : 2.50	3.53	7.30	3.46	7.62	3.82	6.74	5.08	6.26	6.26	6.02
1 : 2.75	3.59	8.02	3.68	8.00	4.02	6.95	5.17	6.36	6.34	6.05
1 : 3.00	3.59	8.81	3.93	8.40	4.24	7.20	5.31	6.47	6.44	6.09

Example of Software Solution for Circular Slip Surface Failure

- Given
  - Problem Below, water table at base of slope
  - Slope: γ_t =18.9 kN/m^3, c_u =23.9 kPa, h = 10.7 m
    - Since slope is 1.5H:1V, length of slope l = 10.7*1.5 = 16 m
  - Foundation: γ_t =18.9 kN/m^3, c_u =47.9 kPa, h = 7.6 m
- Find
  - Factor of safety for slope stability using SLOPE w/Bishop's Method and Tsytovich's Quick and Dirty Method
  - Neutral stress = 1 – sinφ

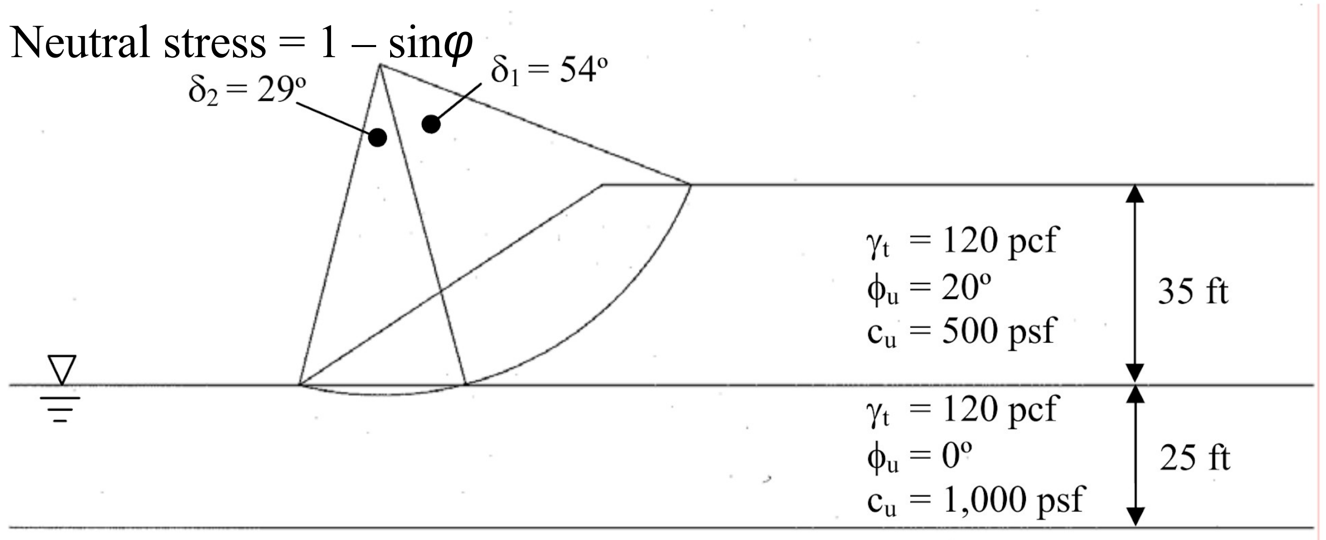
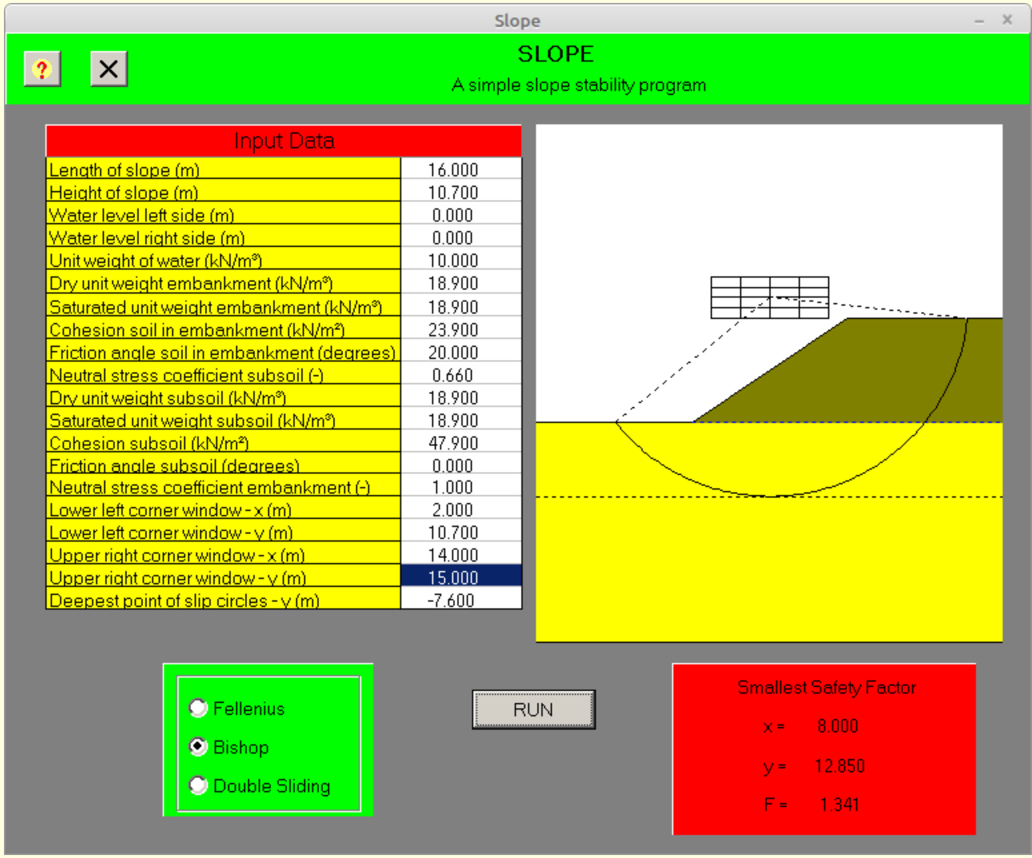


Figure 6-20. Data for Example 6-2.

Example of Software Solution for Circular Slip Surface Failure

- Tsytovich "Quick and Dirty Method"
  - Slope given as 1:1.5
  - e = 25' and H = 35', thus e/H = 5/7 = 0.71, we will use e/H = 0.75, which will simplify the interpolation
  - The method is too crude to consider different properties above and below the toe of the slope, so we'll try both of them
  - By interpolation, A=3.93 and B = 6.03
  - Properties above:Substituting, FS = (3.93) tan(20) + (6.03) (500)/ ((120)(35) = 2.15
  - Properties below: Substituting, FS = (6.03)(1000)/((120)(35)) = 1.43
- SLOPE solution, assuming slip circle touches lower hard layer



What this shows is that the soil below the toe of the slope is the primary driver of the design. Its lack of internal friction makes it unable to take advantage of the overburden of the slope.

Other Types of Rotational Analysis

- Spencer
  - Assumes forces on the sides of the slices are parallel
  - Solves for both moment and force equation on slices
- Morganstern-Price
  - Imposes normal and shear forces on the sides of the slices
  - Includes water pressure effects
- Both methods require equilibrium of forces on each slice
- Morganstern Rapid Drawdown Method

Morganstern Rapid Drawdown Method

- Given
  - Problem as before, except right water table at top of slope; left water table varies
- Find
  - Factors of safety as water is drawn down
  - Classic Morganstern rapid drawdown assumes toe circle

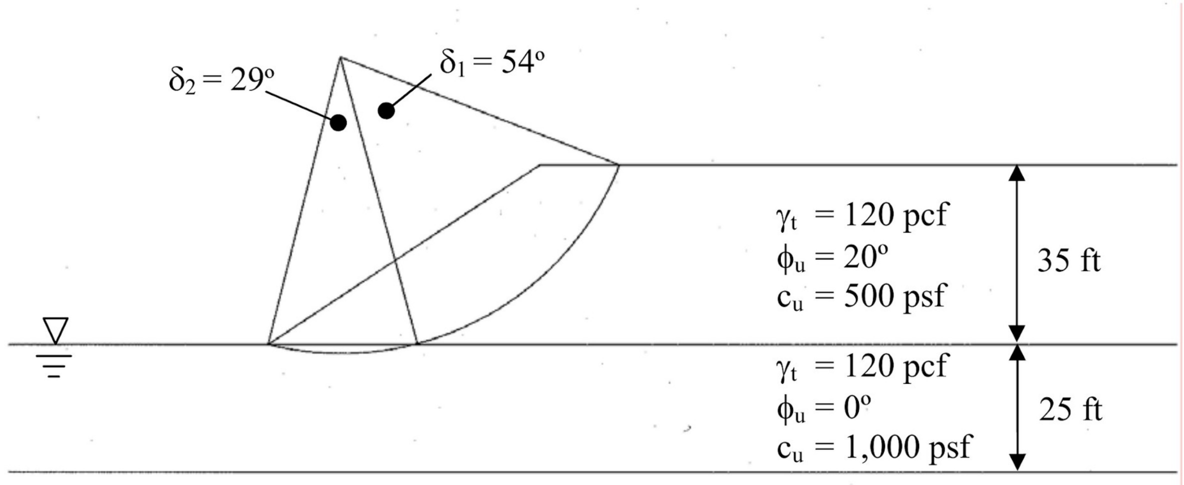
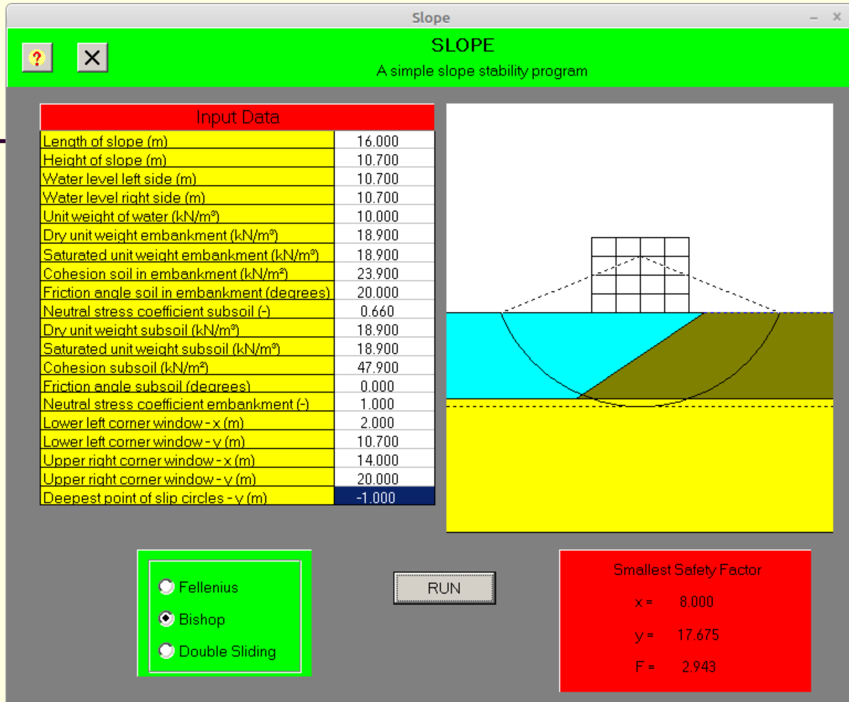
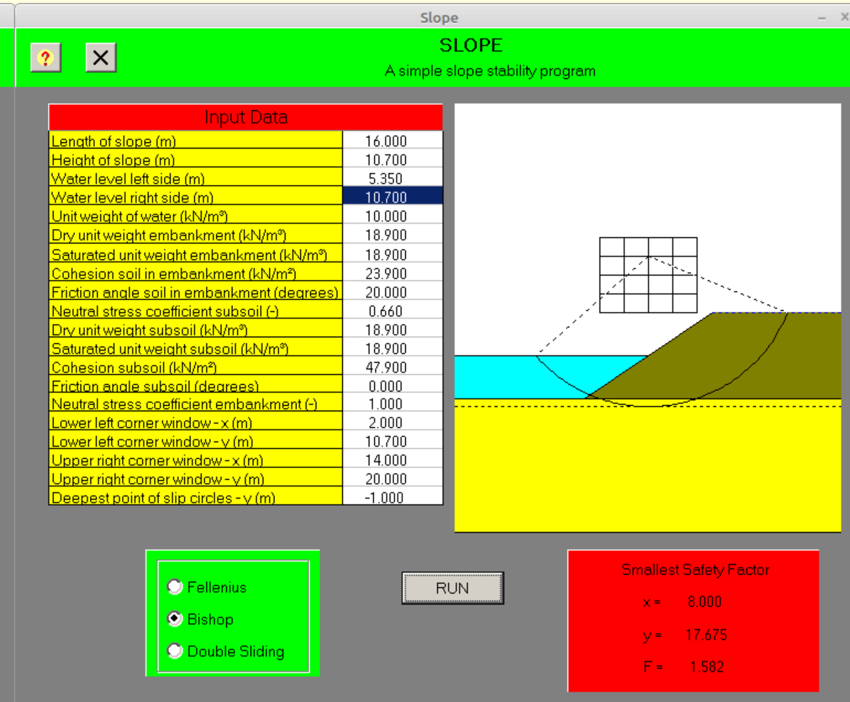
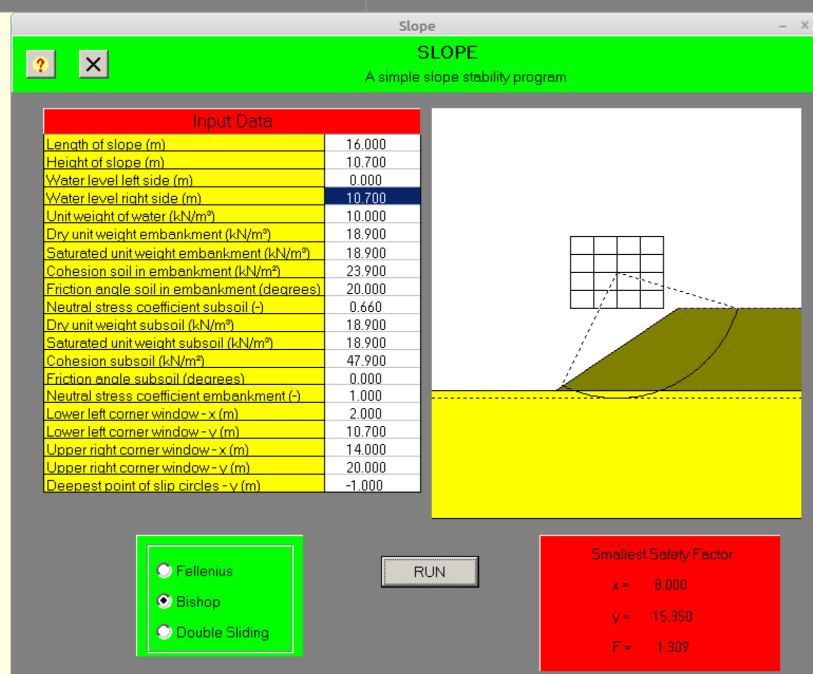


Figure 6-20. Data for Example 6-2.

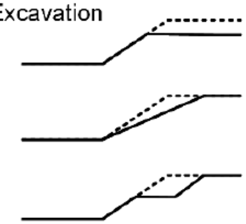
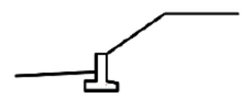
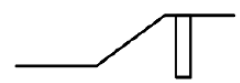
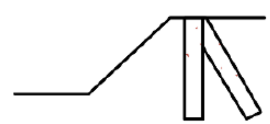
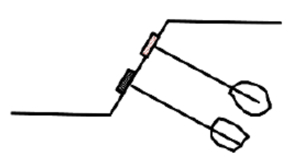
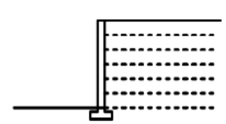
Morganstern Rapid Drawdown Method







Solutions to Slope Stability Problems

Scheme	Applicable Methods	Comments
1. Changing Geometry - Excavation 	1. Reduce slope height by excavation at the top of slope. 2. Flatten the slope. 3. Excavate a bench in the top of the slope.	1. Area has to be accessible to construction equipment. Disposal site needed for excavated soil. Drainage sometimes incorporated into this method.
2. Toe Berm Fill 	1. Compared earth or rock berm placed at the toe. Drainage may be required behind berm.	1. Sufficient width and thickness of berm required so failure will not occur below or through berm. Borrow soils required.
3. Retaining Structure 	1. Retaining structure – Crib, Cantilever, or MSE.	1. Usually expensive. Cantilever walls may have to be tied back.
	2. Drilled, cast-in-place vertical piles founded well below the slide plane. Generally, 18 to 30 inches in diameter at 4 to 8 foot spacing. Larger diameter piles at closer spacing may be required in some cases to mitigate failures of cuts in highly fissured clays.	2. Spacing should be such that soil can arch between piles. Grade beam can be used to tie piles together. Very large diameter piles (6+ feet) have been used for deep slides.
	3. Drilled, cast-in-place vertical piles tied back with battered piles or a deadman. Piles founded well below the slide plane. Generally, 12 to 30 inches in diameter at 4 to 8 foot spacing. Larger diameter piles at closer spacing may be required in some cases to mitigate failures of cuts in highly fissured clays.	3. Space close enough so soil will arch between piles. Piles can be tied together with grade beam.
	4. Earth and rock anchors and rock bolts with anchor plates.	4. Can be used for high slopes in restricted areas. Conservative design should be used, especially for permanent support. Use may be essential for slopes in rocks where joints dip toward excavation and daylight in the slope.
	5. MSE Wall	5. Various types of geosynthetics and wall facings are available.
4. Other Methods	See DM 7.2.	

Slope Failure in Fissured Rock

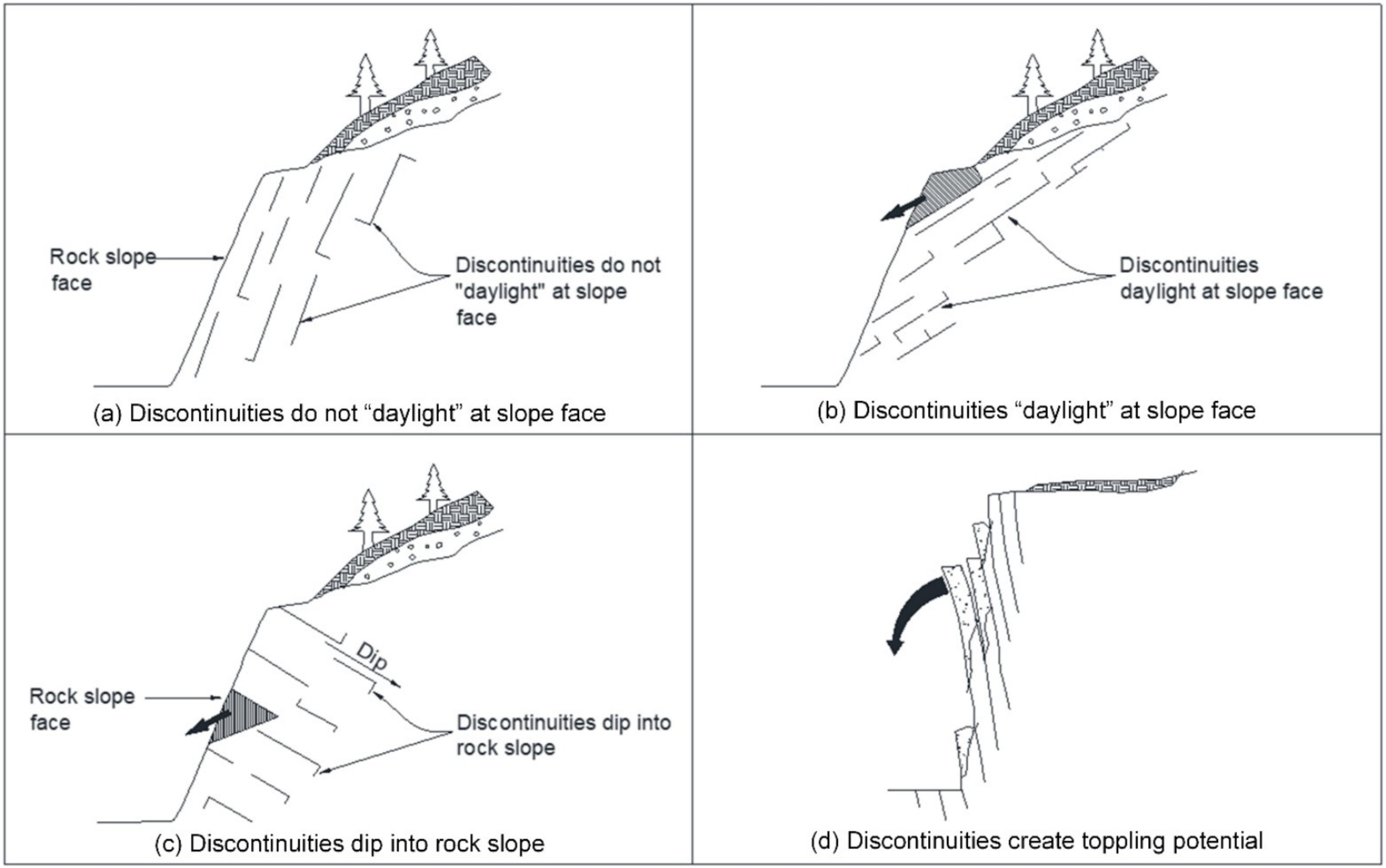


Figure 7-13 Rock Discontinuity Conditions (after FHWA 1998)

Slope Failure in Fissured Rock

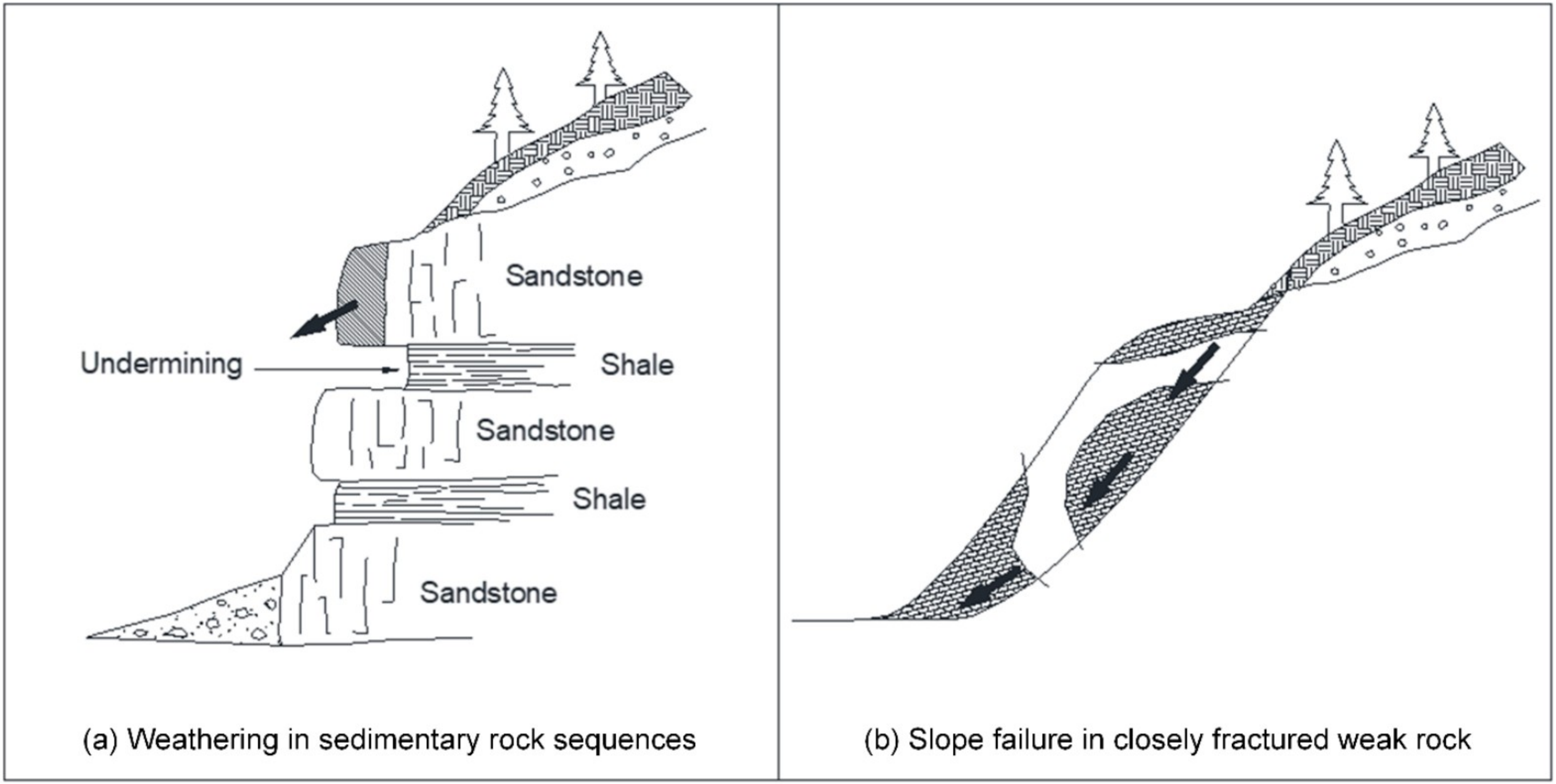


Figure 7-14 Weathering and Weak Rock Conditions (after FHWA 1998)

Other Solutions

- Lightweight fill
- Retaining Walls
  - Especially useful when space for a slope is limited
- Tieback anchors and soil nailing
- Improvement of drainage
- Geogrids and Mechanically Stabilized Earth (MSE) walls

Questions?