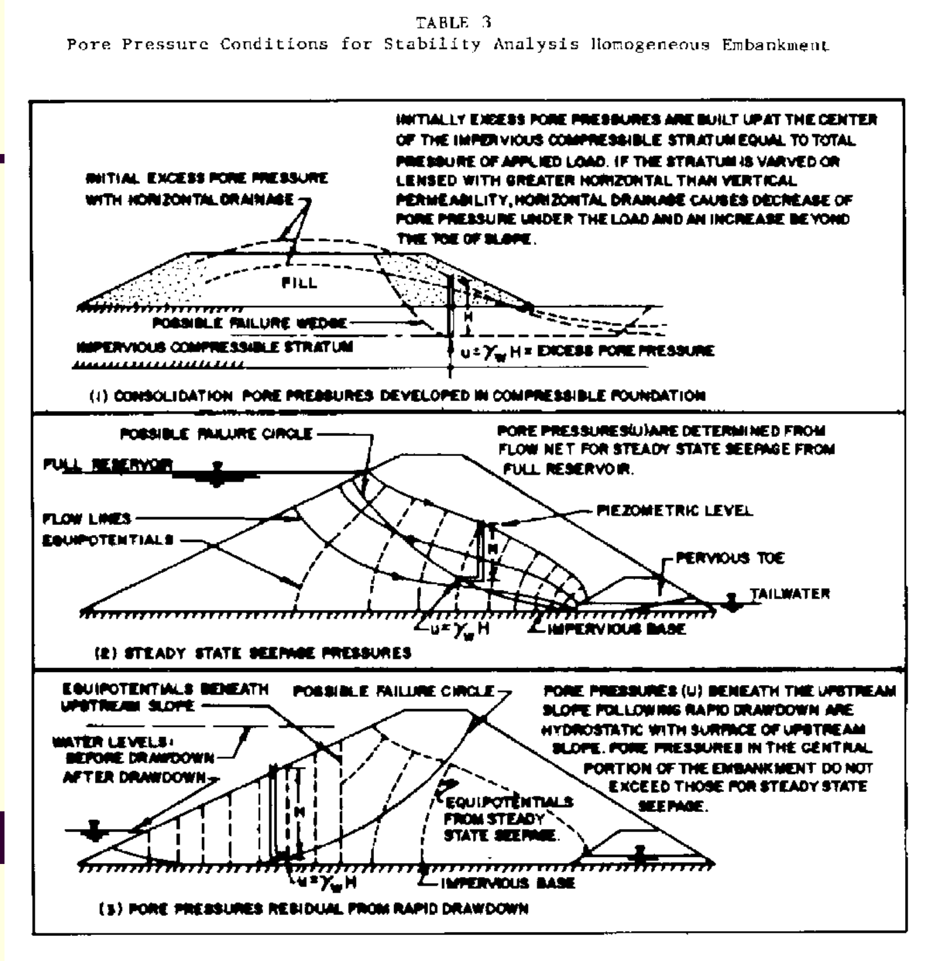
ENCE 3610 Soil Mechanics



Lecture 13
Slope Stability

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

Plasticity Theory: Upper and Lower Bound

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, $\sigma_{zz} - \sigma_{xx} = \gamma h$.

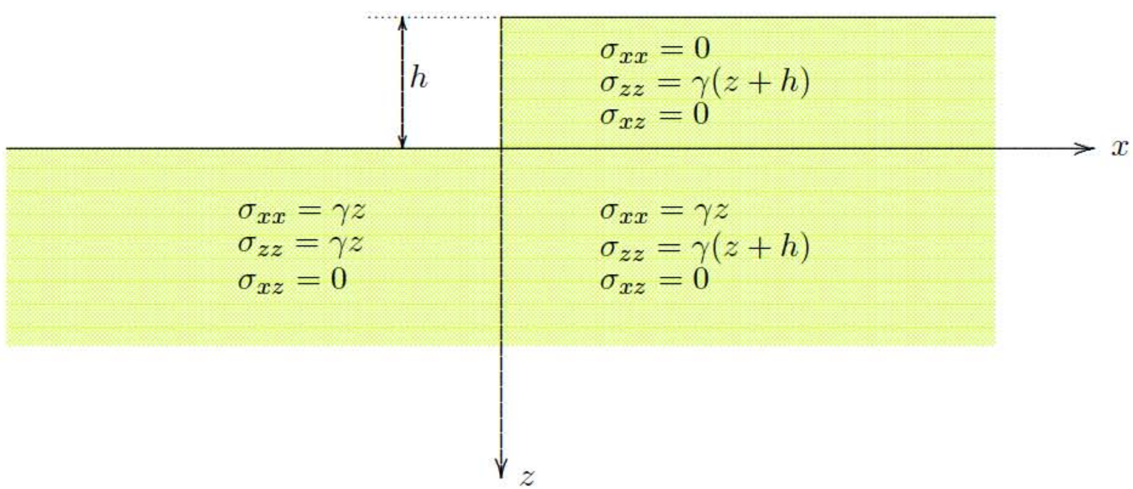


Figure 44.2: Equilibrium system.

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.

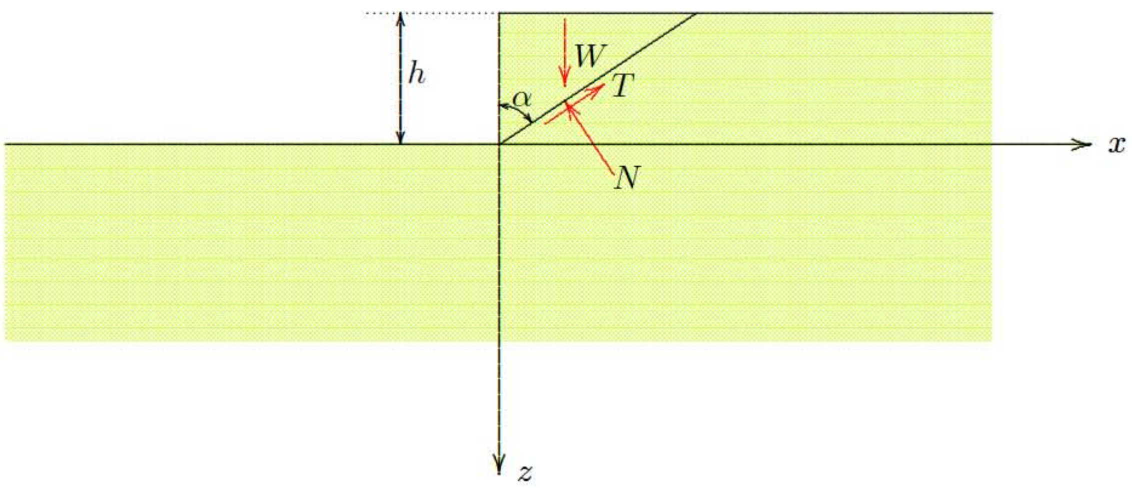


Figure 44.3: Mechanism with straight slip surface.

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 $\alpha = 45^\circ$. Because this is an upper bound it follows that

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

This is the upper bound for straight slip surfaces.

Critical Height and Slope Stability Number

- Critical Height
  - Critical height is found between upper and lower bounds

$$\frac{2c}{\gamma} < h_{cr} < \frac{4c}{\gamma}$$

  - Values have also been determined between these two
  - Obviously the lower bound is the most conservative

Slope Stability Number

Rearranging and redefining the coefficient

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

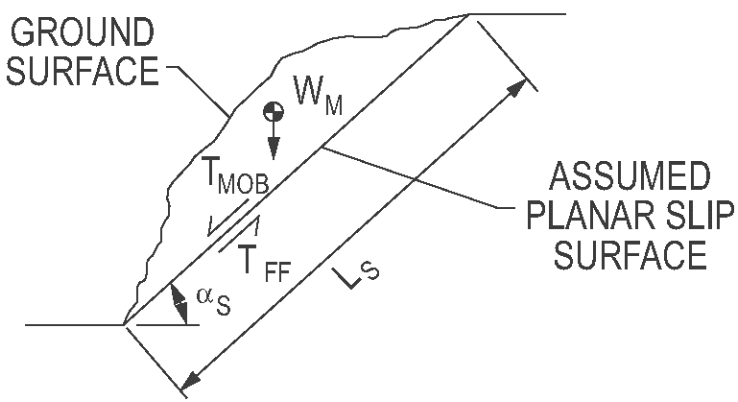
Adding a factor of safety

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

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

General Case of Planar Slope Failure



SLOPE FAILURE ALONG PLANAR SURFACE

FS	=	factor of safety against slope instability
	=	T_{FF}/T_{MOB}
T_{FF}	=	available shearing resistance along slip surface
	=	$cL_S + W_M \cos\alpha_S \tan\phi$
T_{MOB}	=	mobilized shear force along slip surface
	=	$W_M \sin\alpha_S$
L_S	=	length of assumed planar slip surface
W_M	=	weight of soil above slip surface
α_S	=	angle of assumed slip surface with respect to horizontal

Infinite Slope

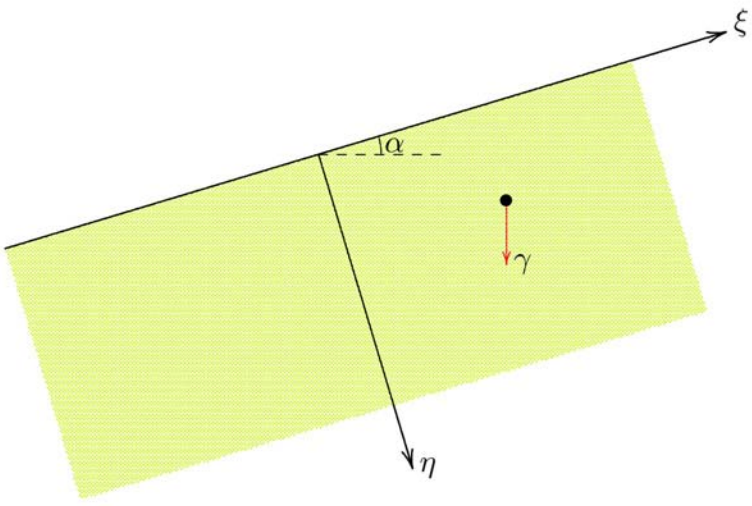


Figure 45.1: Infinite slope in dry sand.

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

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

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

- 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

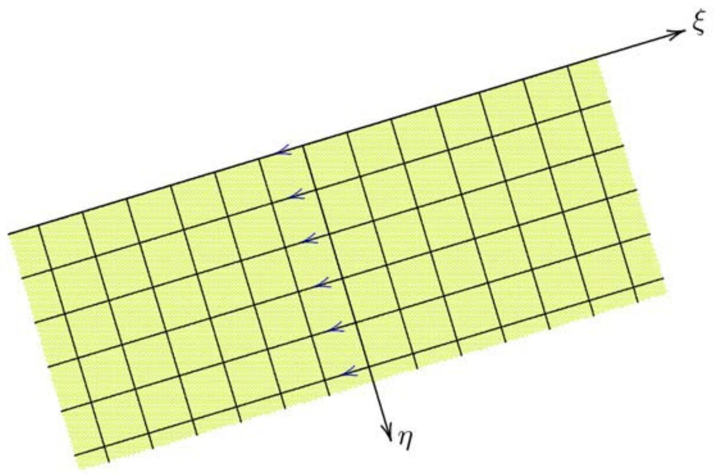


Figure 45.3: Parallel groundwater flow.

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

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

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

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

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

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

Infinite slopes, with seepage and cohesion:

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

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

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 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 Equilibrium Method (Method of Slices)

1. TYPES OF ANALYSIS. For slopes in relatively homogeneous soil, the failure surface is approximated by a circular arc, along which the resisting and rupturing forces can be analyzed. Various techniques of slope stability analysis may be classified into three broad categories.

a. Limit Equilibrium Method. Most limit equilibrium methods used in geotechnical practice assume the validity of Coulomb's failure criterion along an assumed failure surface. A free body of the slope is considered to be acted upon by known or assumed forces. Shear stresses induced on the assumed failure surface by the body and external forces are compared with the available shear strength of the material. This method does not account for the load deformation characteristics of the materials in question. Most of the methods of stability analysis currently in use fall in this category.

The method of slices, which is a rotational failure analysis, is most commonly used in limit equilibrium solutions. The minimum factor of safety is computed by trying several circles. The difference between various approaches stems from (a) the assumptions that make the problem determinate, and (b) the equilibrium conditions that are satisfied. The soil mass within the assumed slip surface is divided into several slices, and the forces acting on each slice are considered. The effect of an earthquake may be considered by applying appropriate horizontal force on the slices. Figure 1 (Reference 2, Soil Mechanics, by Lambe and Whitman) illustrates this method of analysis applied to a slope of homogeneous sandy soil subjected to the forces of water seeping laterally toward a drain at the toe.

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.

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

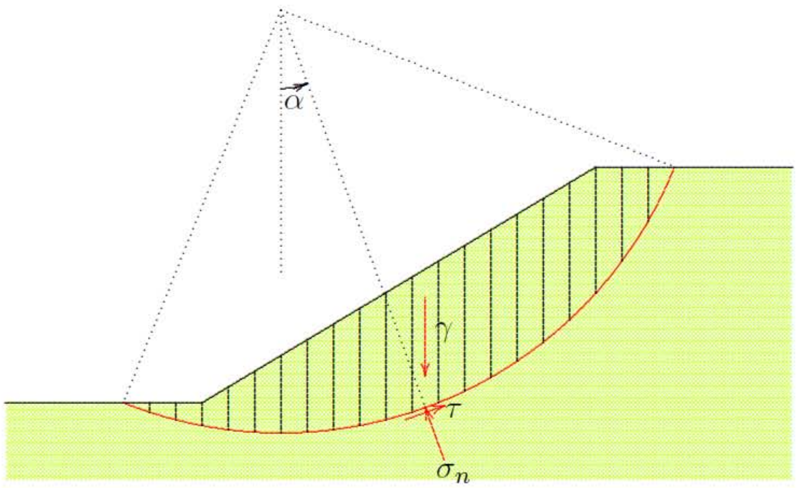


Figure 46.1: Circular slip surface.

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.

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

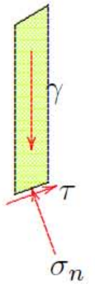


Figure 46.2: Fellenius.

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.

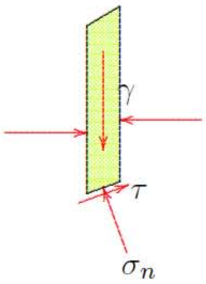


Figure 46.3: Bishop.

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

$$\sigma_n'(1 + \frac{\tan\alpha\,\tan\phi}{F}) = \gamma h - p - \frac{c}{F}\tan\alpha. \tag{46.7}$$

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

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

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.

Slope Stability Charts (Janbu)

3. SLOPE STABILITY CHARTS.

a. Rotational Failure in Cohesive Soils (Ø = 0)

(1) For slopes in cohesive soils having approximately constant strength with depth use Figure 2 (Reference 4, Stability Analysis of Slopes with Dimensionless Parameters, by Janbu) to determine the factor of safety.

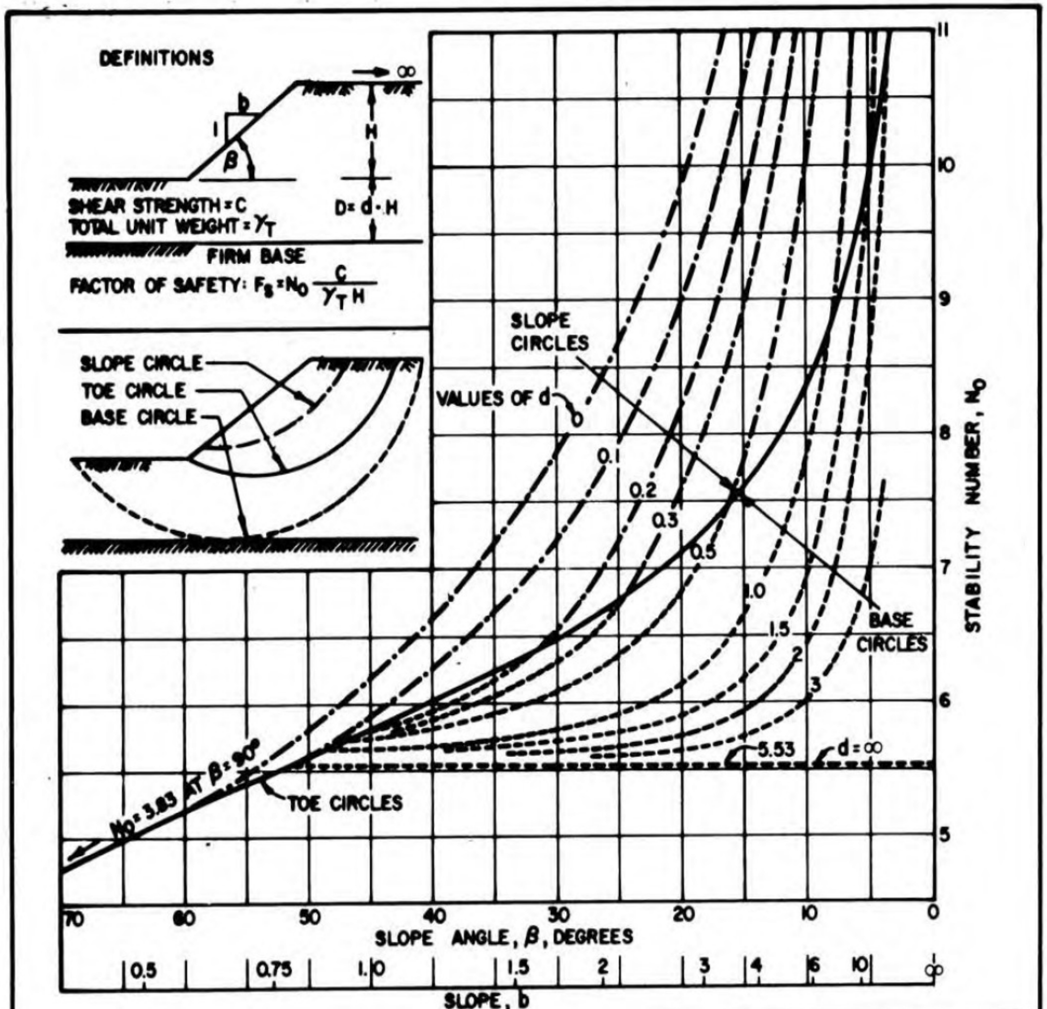
(2) For slope in cohesive soil with more than one soil layer, determine centers of potentially critical circles from Figure 3 (Reference 4). Use the appropriate shear strength of sections of the arc in each stratum. Use the following guide for positioning the circle.

(a) If the lower soil layer is weaker, a circle tangent to the base of the weaker layer will be critical.

(b) If the lower soil layer is stronger, two circles, one tangent to the base of the upper weaker layer and the other tangent to the base of the lower stronger layer, should be investigated.

(3) With surcharge, tension cracks, or submergence of slope, apply corrections of Figure 4 to determine safety factor.

(4) Embankments on Soft Clay. See Figure 5 (Reference 5, The Design of Embankments on Soft Clays, by Jakobsen) for approximate analysis of embankment with stabilizing berms on foundations of constant strength. Determine the probable form of failure from relationship of berm and embankment widths and foundation thickness in top left panel of Figure 5.



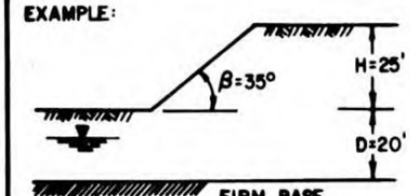
ASSUMED CONDITIONS:

1. NO OPEN WATER OUTSIDE OF SLOPE.
2. NO SURCHARGE OR TENSION CRACKS.
3. SOIL IS HOMOGENEOUS TO DEPTH D.
4. SHEAR STRENGTH IS DERIVED FROM COHESION ONLY AND IS CONSTANT WITH DEPTH.
5. FAILURE TAKES PLACE AS ROTATION ON CIRCULAR ARC.

NOTES:

1. FOR LOCATION OF CENTER OF CRITICAL CIRCLE SEE FIG. 3.
2. FOR REDUCTION FACTORS FOR SUBMERGENCE, SURCHARGE OR TENSION CRACKS, SEE FIG. 4.

EXAMPLE:



γ_T = 115 PCF C = 600 PSF ϕ = 0

d = D/H = 20/25 = 0.80, β = 35°

$F_S = N_O \frac{C}{\gamma_T H} = 5.8 \frac{(600)}{(115)(25)} = \underline{1.21}$

FAILURE SURFACE IS BASE CIRCLE.

FIGURE 2
Stability Analysis for Slopes in Cohesive Soils, Undrained Conditions, i.e., Assumed Ø = 0

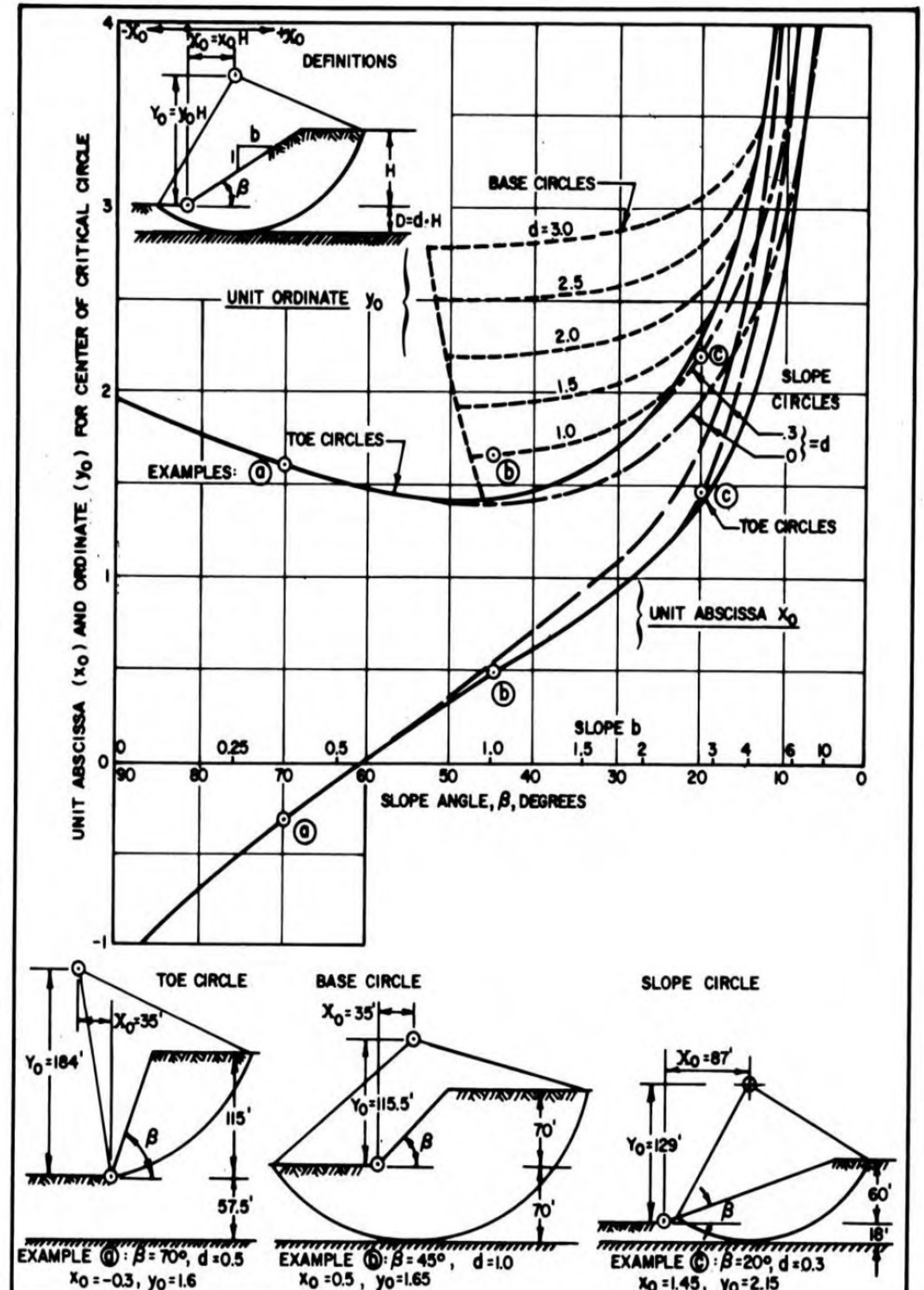


FIGURE 3
Center of Critical Circle, Slope in Cohesive Soil

Example of Chart Solution

- Given: Slope
  - $c_u = 40$ kPa
  - $\gamma = 17.5$ kN/m^3
  - $\beta = 60°$
- Find
  - Maximum depth of excavation without slope failure
  - Radius of critical circle when factor of safety is unity
  - Distance from crown of slope to slip circle at top of slope

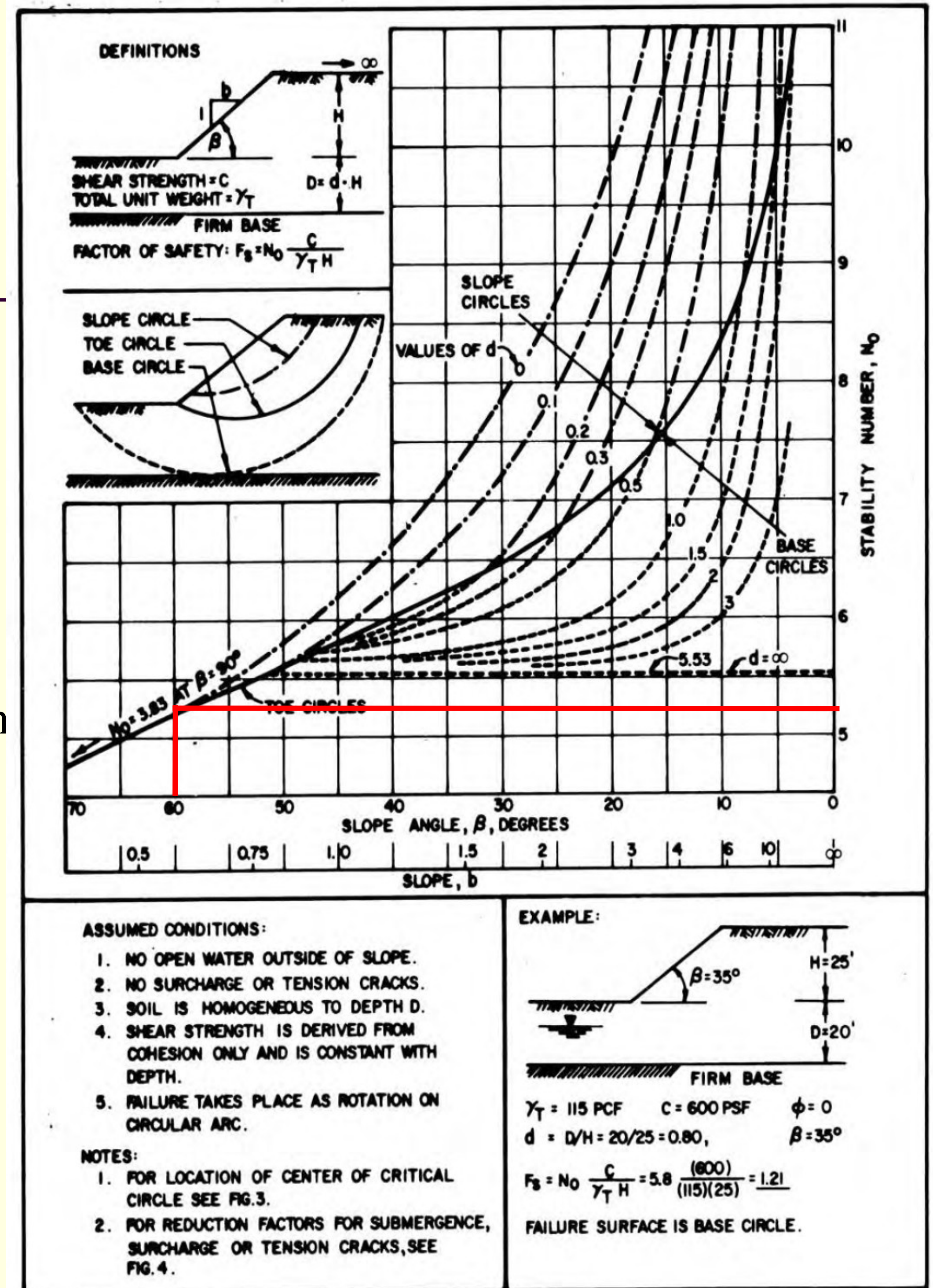


FIGURE 2
Stability Analysis for Slopes in Cohesive Soils, Undrained Conditions, i.e., Assumed Ø = 0

Example of Chart Solution

- Solution
  - From chart, Stability Number $N_o = 5.2$
  - Assume FS = 1 (for maximum possible height)
  - Solving for H, $H = cN_o/(\gamma F_s) = (40)(5.2)/((17.5)(1)) = 11.89$ m
  - Note that, from portion of chart used, we have a toe circle

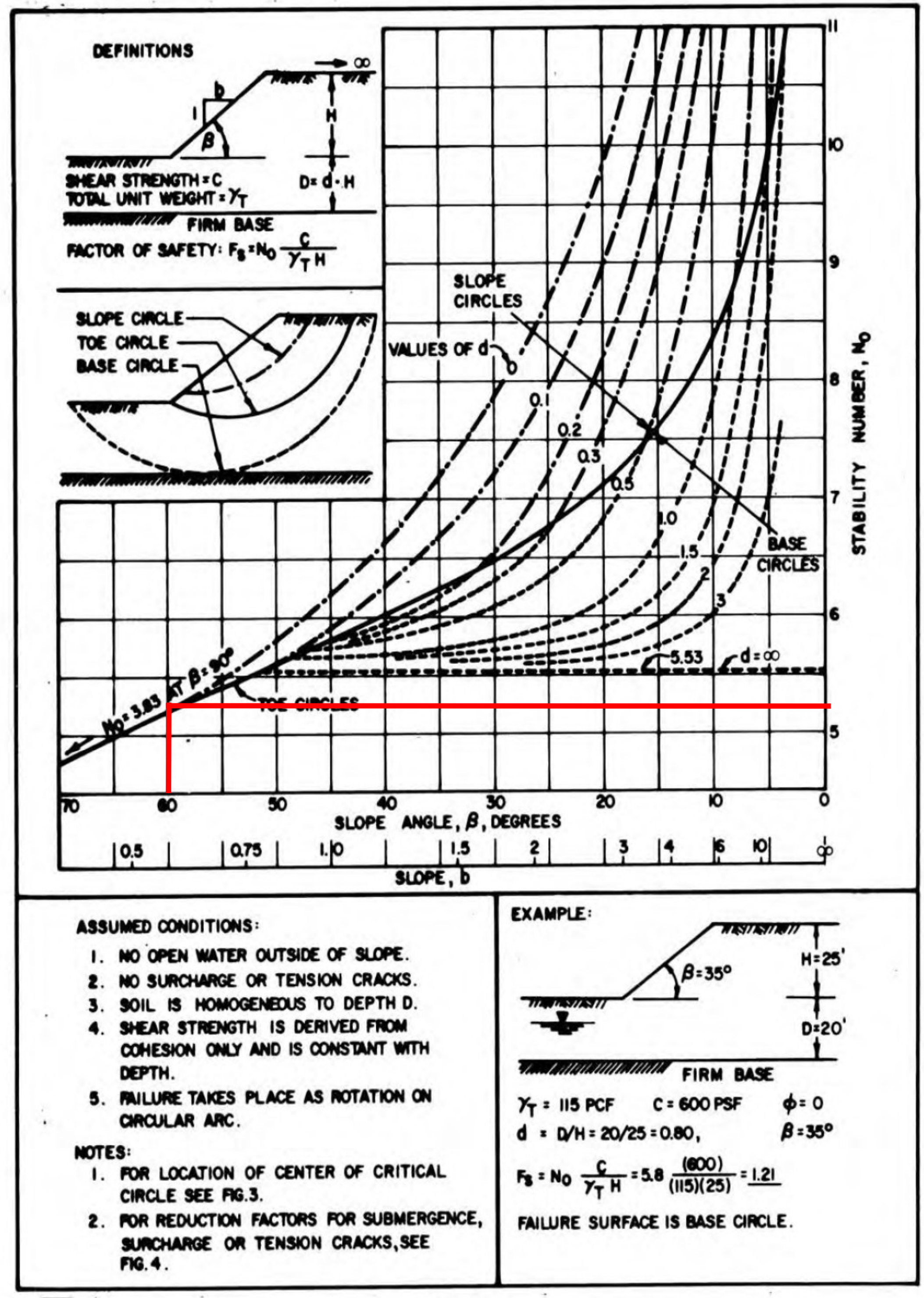


FIGURE 2
Stability Analysis for Slopes in Cohesive Soils, Undrained Conditions, i.e., Assumed Ø = 0

Example of Chart

- Solution
  - H = 11.89 m
  - From “Unit Absicca” curve, $x_o = 0$
  - From “Unit Ordinate” curve, $y_o = 1.5H = (1.5)(11.89) = 17.8m$

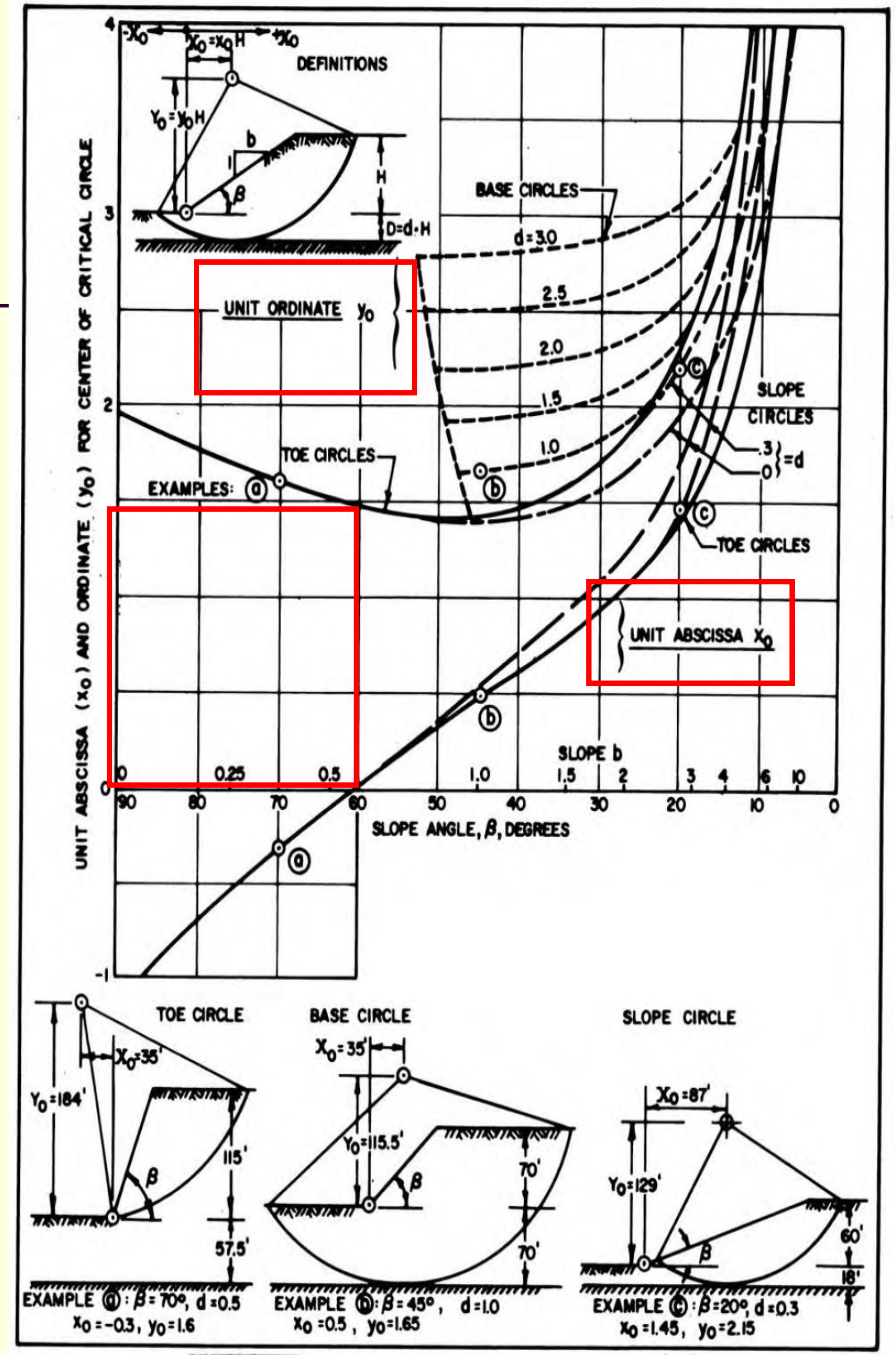


FIGURE 3
Center of Critical Circle, Slope in Cohesive Soil

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.

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

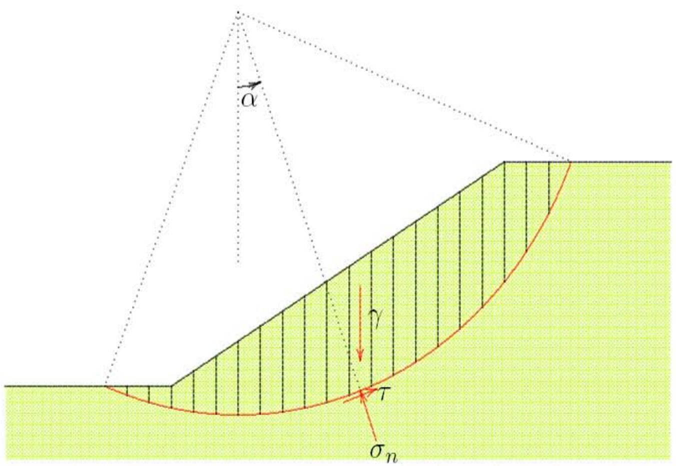


Figure 46.1: Circular slip surface.

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

$$\sum \gamma hbR \sin\alpha = \sum \frac{\tau bR}{\cos\alpha}. \tag{46.2}$$

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.

Methods of Slip Circle Analysis

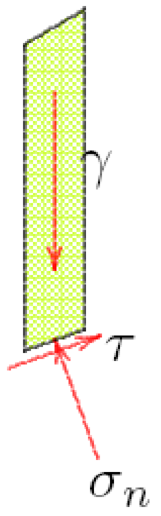


Figure 46.2: Fellenius.

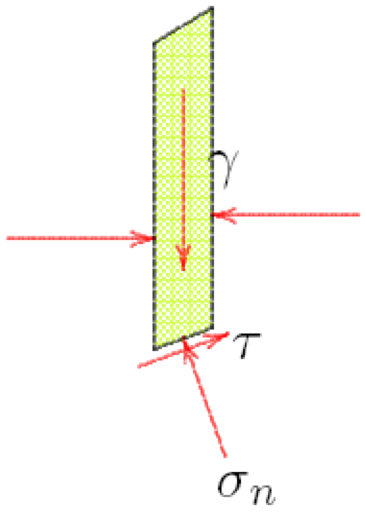
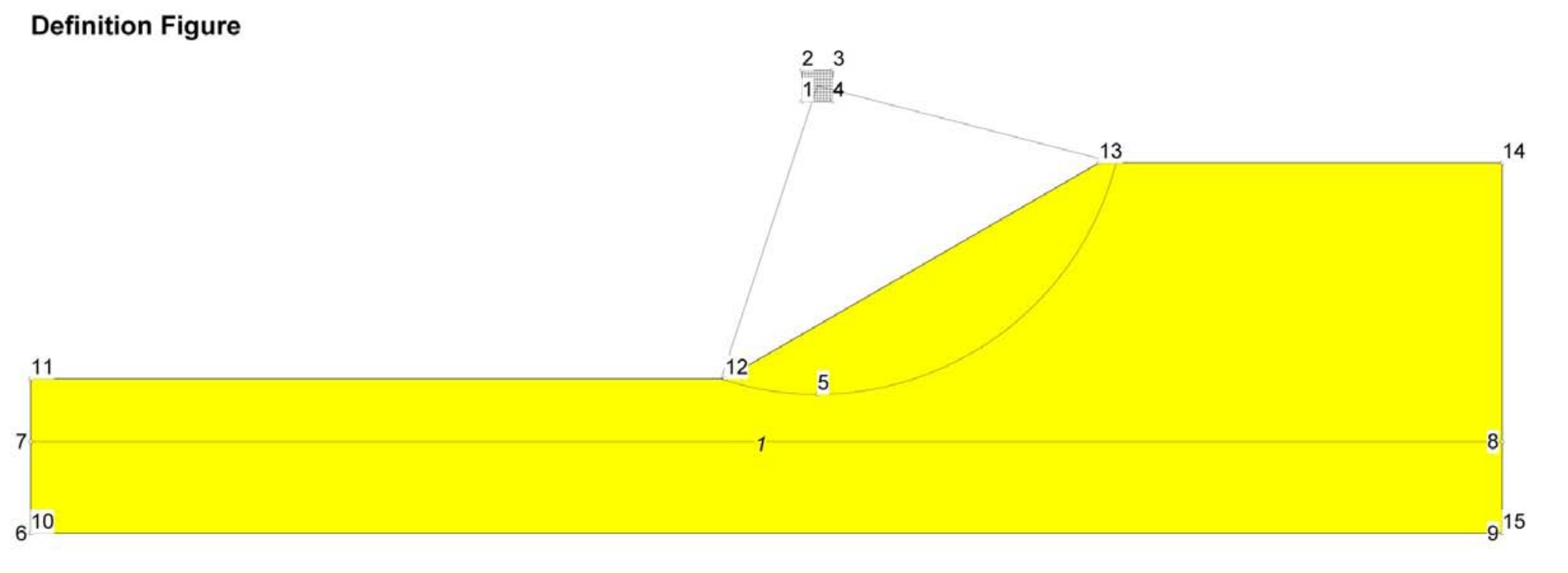


Figure 46.3: Bishop.

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



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 and Bishop's Method
  - Neutral stress = 1 – sinφ

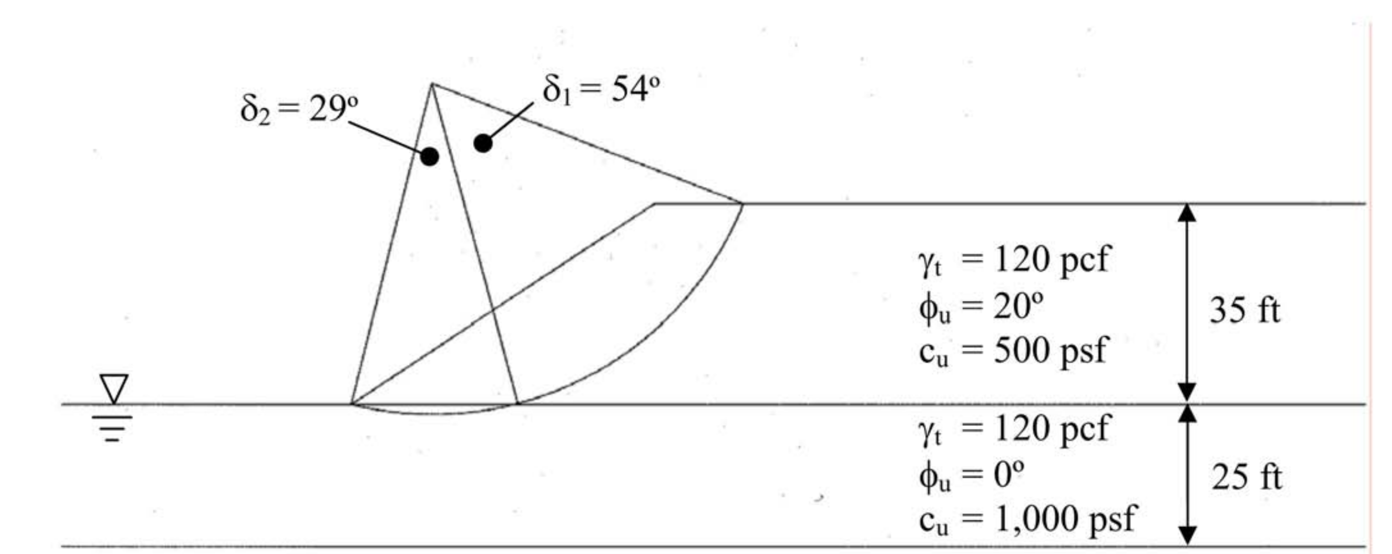


Figure 6-20. Data for Example 6-2.

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

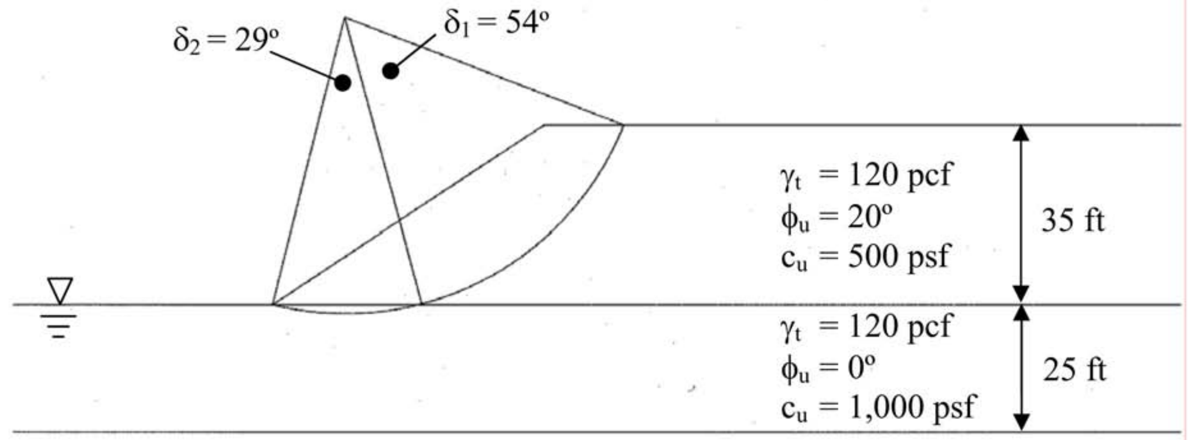


Figure 6-20. Data for Example 6-2.

Solutions to Slope Stability Problems

TABLE 4
Methods of Stabilizing Excavation Slopes

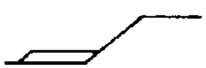
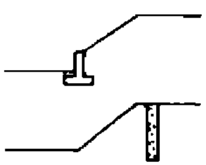
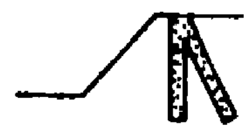
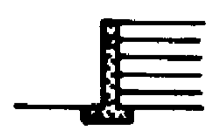
Scheme	Applicable Methods	Comments
1. Changing Geometry EXCAVATION 	1. Reduce slope height by excavation at top of slope. 2. Flatten the slope angle. 3. Excavate a bench in upper part of slope.	1. Area has to be accessible to construction equipment. Disposal site needed for excavated soil. Drainage sometimes incorporated in this method.
2. Earth Berm Fill 	1. Compacted earth or rock berm placed at and beyond the toe. Drainage may be provided behind berm.	1. Sufficient width and thickness of berm required so failure will not occur below or through berm.
3. Retaining Structures RETAINING STRUCTURES 	1. Retaining wall – crib or cantilever type. 2. Drilled, cast-in-place vertical piles, founded well below bottom of slide plane. Generally 18 to 36 inches in diameter and 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.	1. Usually expensive. Cantilever walls might have to be tied back. 2. Spacing should be such that soil can arch between piles. Grade beam can be used to tie piles together. Very large diameter (6 feet ±) piles have been used for deep slides.

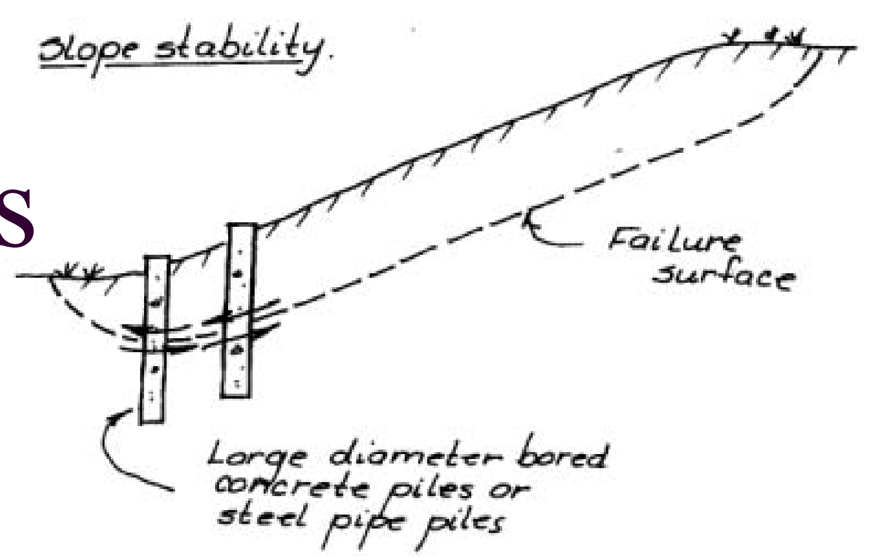
TABLE 4 (continued)
Methods of Stabilizing Excavation Slopes

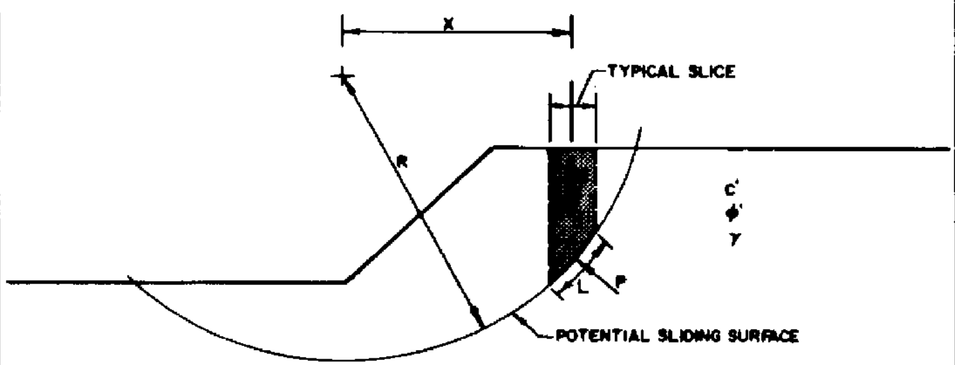
Scheme	Applicable Methods	Comments
	3. Drilled, cast-in-place vertical piles tied back with battered piles or a deadman. Piles founded well below slide plane. Generally, 12 to 30 inches in diameter and at 4- to 8-foot spacing.	3. Space close enough so soil will arch between piles. Piles can be tied together with grade beam.
	4. Earth and rock anchors and rock bolts.	4. Can be used for high slopes, and in very 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 such joints daylight in the slope.
	5. Reinforced earth.	5. Usually expensive.
4. Other Methods	See Table 7, DM-7.2, Chapter 1	

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

Pile Stabilised Slopes



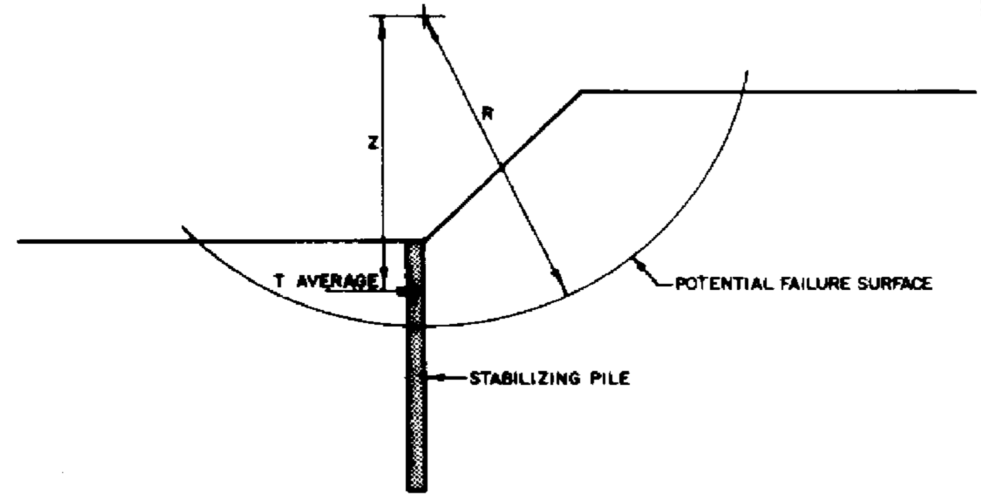


SAFETY FACTOR (F_s) FOR MOMENT EQUILIBRIUM FOR METHOD OF SLICES WITH CIRCULAR POTENTIAL SLIDING SURFACE (WITHOUT SEISMIC LOADS, SURCHARGE LOADS, OR PARTIAL SUBMERGENCE OF SLOPE) IS TYPICALLY DEFINED AS:

$$F_s = \frac{\Sigma c' L R + \Sigma (P - uL) R \tan \phi'}{\Sigma W X}$$

WHERE:
- c' = EFFECTIVE COHESION.
- ϕ' = EFFECTIVE FRICTION ANGLE.
- W = TOTAL WEIGHT OF SLICE.
- P = TOTAL NORMAL FORCE ON BASE OF SLICE.
- L = LENGTH OF POTENTIAL SLIDING SURFACE ACROSS SLICE.
- u = AVERAGE PORE WATER PRESSURE ON POTENTIAL SLIDING SURFACE ACROSS SLICE.
- R = RADIUS OF MOMENT ARM FOR POTENTIAL SLIDING SURFACE
- X = HORIZONTAL DISTANCE FROM CENTROID OF SLICE TO CENTER OF ROTATION.

FIGURE 11
Influence of Stabilizing Pile on Safety Factor



SAFETY FACTOR FOR MOMENT EQUILIBRIUM CONSIDERING THE SAME FORCES AS ABOVE PLUS THE EFFECT OF THE STABILIZING PILE IS EXPRESSED AS:

$$F_s = \frac{\Sigma c' L R + \Sigma (P - uL) R \tan \phi' + TZ}{\Sigma W X}$$

WHERE:
- T = AVERAGE TOTAL THRUST (PER LIN. FT., HORIZ.) RESISTING SOIL MOVEMENT.
- Z = DISTANCE FROM CENTROID OF RESISTING PRESSURE (THRUST) TO CENTER OF ROTATION.

FIGURE 11 (continued)
Influence of Stabilizing Pile on Safety Factor

Questions?