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# Introduction to Slope Stability Analysis

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# An Introduction To Slope Stability Analysis



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**1. GENERAL.** This publication is concerned with characteristics and critical aspects of the stability of excavation slopes; methods of designing slopes, including field observations and experience, slope stability charts, and detailed analyses; factors of safety; and methods of stabilizing slopes and slides. The emphasis in this publication is on simple, routine procedures. It does not deal with specialized problems, such as the stability of excavated slopes during earthquakes.

**2. SLOPE STABILITY PROBLEMS.** Excavation slope instability may result from failure to control seepage forces in and at the toe of the slope, too steep slopes for the shear strength of the material being excavated, and insufficient shear strength of subgrade soils. Slope instability may occur suddenly, as the slope is being excavated, or after the slope has been standing for some time. Slope stability analyses are useful in sands, silts, and normally consolidated and overconsolidated clays, but care must be taken to select the correct strength parameter. Failure surfaces are shallow in cohesionless materials and have an approximately circular or sliding wedge shape in clays.

**2.1 COHESIONLESS SLOPES RESTING ON FIRM SOIL OR ROCK.** The stability of slopes consisting of cohesionless soils depends on the angle of internal friction  $f'$ , the slope angle, the unit weight of soil, and pore pressures. Generally, a slope of 1 vertical (V) on 1 1/2 horizontal (H) is adequate; but if the slope is subjected to seepage or sudden drawdown, a slope of 1V on 3H is commonly employed. Failure normally occurs by surface raveling or shallow sliding. Where consequences of failure may be important, required slopes can be determined using simple infinite slope analysis. Values of  $f'$  for stability analyses are determined from laboratory tests or estimated from correlations. Pore pressure due to seepage reduces slope stability, but static water pressure, with the same water level inside and outside the slopes, has no effect. Benches, paved ditches, and planting on slopes can be used to reduce runoff velocities and to retard erosion. Saturated slopes in cohesionless materials may be susceptible to liquefaction and flow slides during earthquakes, while dry slopes are subject to settlement and raveling. Relative densities of 75 percent or larger are required to ensure seismic stability.

**2.2 COHESIVE SLOPES RESTING ON FIRM SOIL OR ROCK.** The stability of slopes consisting of cohesive soils depends on the strength of soil, its unit weight, the slope height, the slope angle, and pore pressures. Failure usually occurs by sliding on a deep surface tangent to the top of firm materials. For relatively high slopes that drain slowly, it may be necessary to analyze the stability for three limiting conditions:

**2.2.1 SHORT-TERM OR END-OF-CONSTRUCTION CONDITION.** Analyze this condition using total stress methods, with shear strengths determined from Q tests on undisturbed specimens. Shear strengths from unconfirmed compression tests may be used but generally may show more scatter. This case is often the only one analyzed for stability of excavated slopes. The possibility of progressive failure or large creep deformations exists for safety factors less than about 1.25 to 1.50.

**2.2.2 LONG-TERM CONDITION.** If the excavation is open for several years, it may be necessary to analyze this condition using effective stress methods, with strength parameters determined from S tests or R tests on undisturbed specimens. Pore pressures are governed by seepage conditions and can be determined using flow nets or other types of seepage analysis. Both internal pore pressures and external water pressures should be included in the analyses. This case generally does not have to be analyzed.

**2.2.3 SUDDEN DRAWDOWN CONDITION,** or other conditions where the slope is consolidated under one loading condition and is then subjected to a rapid change in loading, with insufficient time for drainage. Analyze this condition using total stress methods, with shear strengths measured in R and S tests. Shear strength shall be based on the minimum of the combined R and S envelopes. This case is not normally encountered in excavation slope stability.

**2.3 EFFECT OF SOFT FOUNDATION STRATA.** The critical failure mechanism is usually sliding on a deep surface tangent to the top of an underlying firm layer. Short-term stability is usually more critical than long-term stability. The strength of soft clay foundation strata should be expressed in terms of total stresses and determined using Q triaxial compression tests on undisturbed specimens or other recognized methods.

### **3. SLOPES IN SOILS PRESENTING SPECIAL PROBLEMS.**

**3.1 STIFF-FISSURED CLAYS AND SHALES.** The shearing resistance of most stiff-fissured clays and shales may be far less than suggested by the results of shear tests on undisturbed samples. This result is due, in part, to prior shearing displacements that are much larger than the displacement corresponding to peak strength. Slope failures may occur progressively, and over a long period of time the shearing resistance may be reduced to the residual value; the minimum value that is reached only at extremely large shear displacements. Temporary slopes in these materials may be stable at angles that are steeper than would be consistent with the mobilization of only residual shear strength. The use of local experience and empirical correlations are the most reliable design procedures for these soils.

**3.2 LOESS.** Vertical networks of interconnected channels formed by decayed plant roots result in a high vertical permeability in loess. Water percolating downward destroys the weakly cemented bonds between particles, causing rapid erosion and slope failure. Slopes in loess are frequently more stable when cut vertically to prevent infiltration. Benches at intervals can be used to reduce the effective slope angle. Horizontal surfaces on benches and at the top and bottom of the slope must be sloped slightly and paved or planted to prevent infiltration. Ponding at the toe of a slope must be prevented. Local experience and practice are the best guides for spacing benches and for protecting slopes against infiltration and erosion.

**3.3 RESIDUAL SOILS.** Depending on rock type and climate, residual soils may present special problems with respect to slope stability and erosion. Such soils may contain pronounced structural features characteristic of the parent rock or the weathering process, and their characteristics may vary significantly over short distances. It may be difficult to determine design shear strength parameters from laboratory tests. Representative shear strength parameters should be determined by back-analyzing slope failures and by using empirical design procedures based on local experience.

**3.4 HIGHLY SENSITIVE CLAYS.** Some marine clays exhibit dramatic loss of strength when disturbed and can actually flow like syrup when completely remolded. Because of disturbance during sampling, it may be difficult to obtain representative strengths for such soils from laboratory tests. Local experience is the best guide to the reliability of laboratory shear strength values for such clays.

## 4. SLOPE STABILITY CHARTS.

### 4.1 UNIFORM SOIL, CONSTANT SHEAR STRENGTH, $\Phi = 0$ , rotational failure.

**4.1.1 GROUNDWATER AT OR BELOW TOE OF SLOPE.** Determine shear strength from unconfined compression, or better, from Q triaxial compression tests. Use the upper diagram of Figure 1 to compute the safety factor. If the center and depth of the critical circle are desired, obtain them from the lower diagrams of Figure 1.

**4.1.2 PARTIAL SLOPE SUBMERGENCE, SEEPAGE SURCHARGE LOADING, TENSION CRACKS.** The effect of partial submergence of a slope is given by a factor  $\mu_w$  in Figure 2; seepage is given by a factor  $\mu_w'$  in Figure 2; surcharge loading is given by a factor  $\mu_q$  in Figure 2; and tension cracks is given by a factor  $\mu_t$  in Figure 3.

Compute safety factor from the following:

$$F = (\mu_w \mu_w' \mu_q \mu_t N_0 C) / (\gamma H + q - \gamma_w H_w') \quad (\text{Eq. 1})$$

where

$\gamma$  = total unit weight of soil

$q$  = surcharge loading

$N_0$  = stability number from Figure 1

If any of these conditions are absent, their corresponding  $i$  factor equals 1.0; if seepage out of the slope does not occur,  $H$  equals  $IH$ .

## **4.2 STRATIFIED SOIL LAYERS, $\phi = 0$ , rotational failure.**

**4.2.1 WHERE THE SLOPE AND FOUNDATION CONSIST OF A NUMBER OF STRATA**, each having a constant shear strength, the charts given in Figures 1 through 3 can be used by computing an equivalent average shear strength for the failure surface. However, a knowledge of the location of the failure surface is required. The coordinates of the center of the circular failure surface can be obtained from the lower diagrams of Figure 1. The failure surface can be constructed, and an average shear strength for the entire failure surface can be computed by using the length of arc in each stratum or the number of degrees intersected by each soil stratum as a weighing factor.

**4.2.2 IT MAY BE NECESSARY TO CALCULATE THE SAFETY FACTOR FOR FAILURE SURFACES** at more than one depth, as illustrated in Figure 4.

## **4.3 CHARTS FOR SLOPES IN UNIFORM SOILS with $\phi > 0$ .**

**4.3.1 A STABILITY CHART** for slopes in soils with  $\phi > 0$  is shown in Figure 5. Correction factors for surcharge loading at the top of the slope, submergence, and seepage are given in Figure 2; and for tension cracks, in Figure 3.

**4.3.2 THE LOCATION OF THE CRITICAL CIRCLE** can be obtained, if desired, from the plot on the right side of Figure 5. Because simple slopes in uniform soils with  $\phi > 0$  generally have critical circles passing through the toe of the slope, the stability numbers given in Figure 5 were developed by analyzing toe circles. Where subsoil conditions are not uniform and there is a weak layer beneath the toe of the slope, a circle passing beneath the toe may be more critical than a toe circle.

**4.4 INFINITE SLOPES.** Conditions that can be analyzed accurately using charts for infinite slope analyses shown in Figure 6 are:

**4.4.1 SLOPES IN COHESIONLESS MATERIALS** where the critical failure mechanism is shallow sliding or surface raveling.

**4.4.2 SLOPES WHERE A RELATIVELY THIN LAYER OF SOIL OVERLIES FIRMER SOIL OR ROCK** and the critical failure mechanism is sliding along a plane parallel to the slope, at the top of the firm layer.

**4.5 SHEAR STRENGTH INCREASING WITH DEPTH** and  $\Phi = 0$ . A chart for slopes in soils with shear strength increasing with depth and  $\Phi = 0$  is shown in Figure 7.

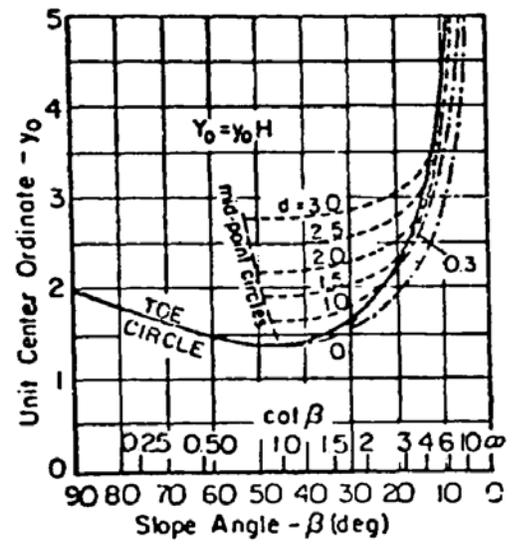
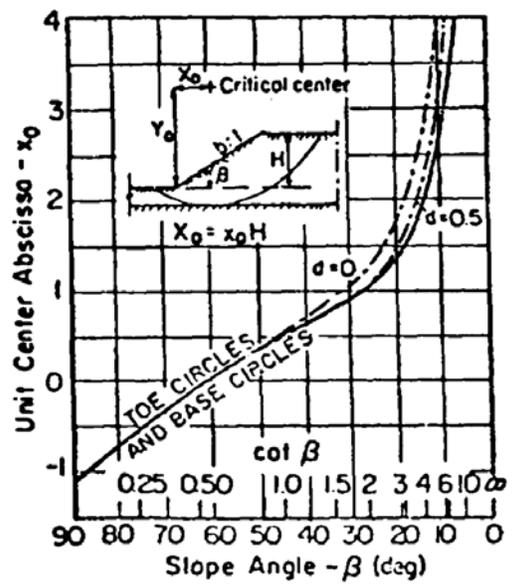
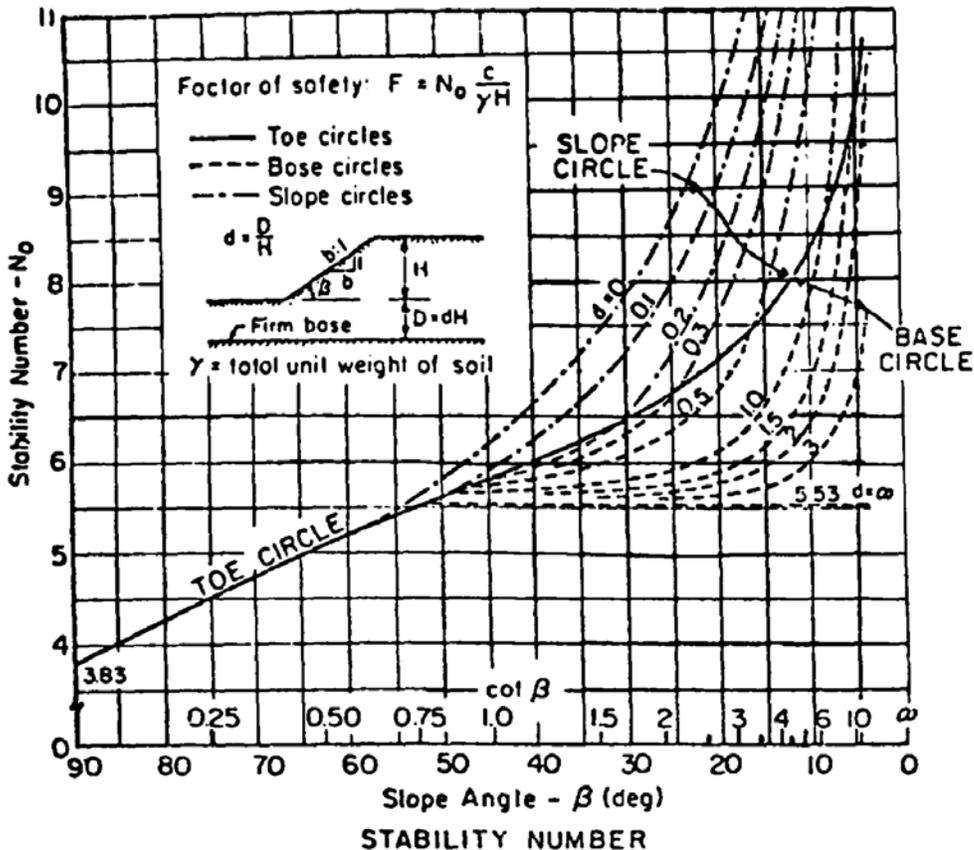
**5. DETAILED ANALYSES OF SLOPE STABILITY.** If the simple methods given for estimating slope stability do not apply and site conditions and shear strengths have been determined, more detailed stability analyses may be appropriate. Such methods are described in engineering literature, and simplified versions are presented below.

**5.1 THE METHOD OF MOMENTS** for  $\Phi = 0$ . This method is simple but useful for the analysis of circular slip surfaces in  $f = 0$  soils, as shown in Figure 8.

**5.2 THE ORDINARY METHOD OF SLICES.** This simple and conservative procedure for circular slip surfaces can be used in soils with  $f > 0$ . For flat slopes with high pore pressures and  $f > 0$ , the factors of safety calculated by this method may be much smaller than values calculated by more accurate methods. An example is presented in Figures 9 through 11. Various trial circles must be assumed to find the critical one. If  $\Phi$  is large and  $c$  is small, it may be desirable to replace the circular sliding surface by plane wedges at the active and passive extremities of the sliding mass.

**5.3 THE SIMPLIFIED WEDGE METHOD.** This method is a simple and conservative procedure for analyzing noncircular surfaces. An example is shown in Figure 12. Various trial failure surfaces with different locations for active and passive wedges must be assumed. The base of the central sliding wedge is generally at the bottom of a soft layer.

**6. STABILIZATION OF SLOPES.** If a slide is being stabilized by flattening the slope or by using a buttress or retaining structure, the shear strength at time of failure corresponding to a factor of safety of 1 should be calculated. This strength can be used to evaluate the safety factor of the slope after stabilization. For methods for stabilizing slopes and landslides, refer to W. J. Turnbull and M. J. Hvorslev, "Special Problems in Slope Stability," *Journal, Soil Mechanics and Foundation Division*, Vol. 93, No. SM4, 1967, pp 499-528, American Society of Civil Engineers, Reston, VA. Often one or more of these schemes may be used together. Schemes I through V are listed approximately in order of increasing cost.

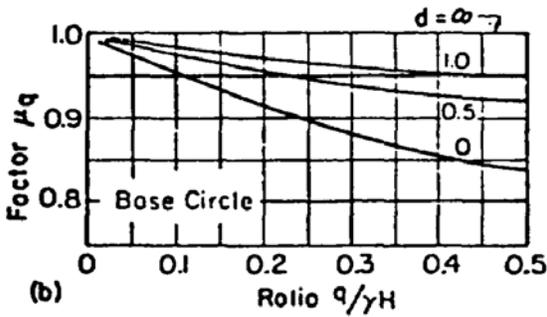
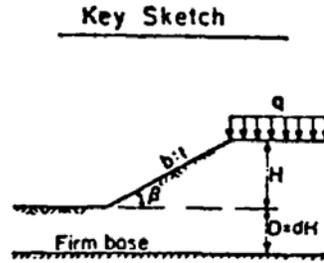
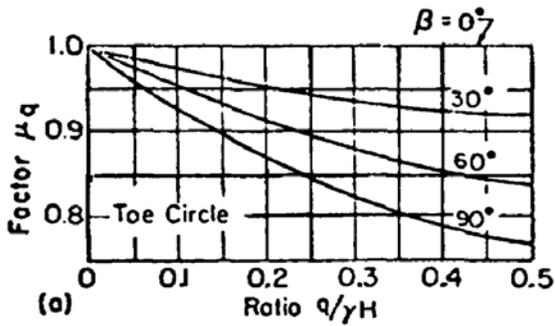


CENTER COORDINATES FOR CRITICAL CIRCLE

Figure 1

Slope stability charts for  $\Phi = 0$  soils

REDUCTION FACTORS FOR SURCHARGE LOADING ( $\mu_q$ )



REDUCTION FACTORS FOR SUBMERGENCE ( $\mu_w$ ) AND SEEPAGE ( $\mu'_w$ )

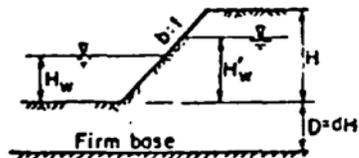
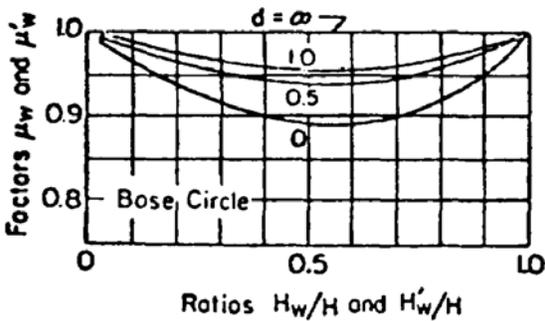
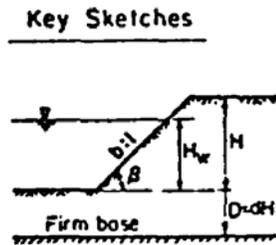
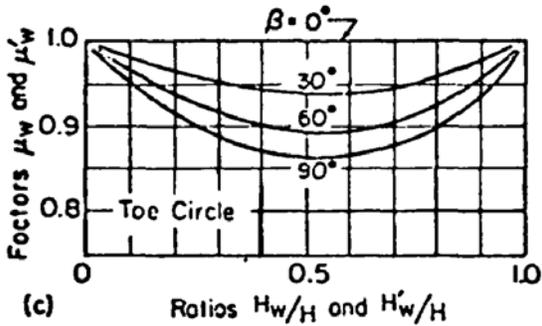
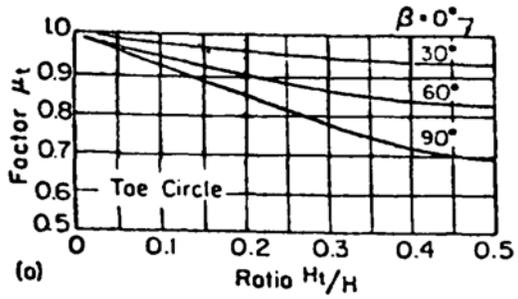


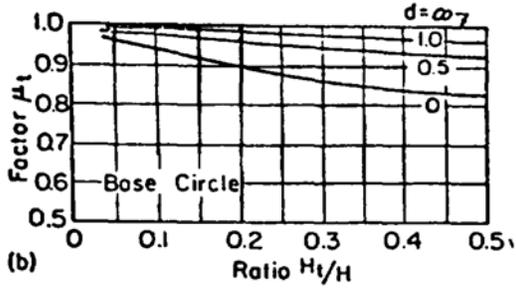
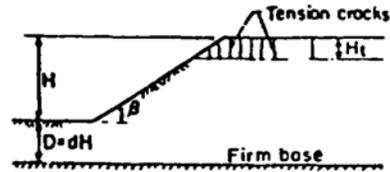
Figure 2

Reduction factors ( $\mu_q$ ,  $\mu_w$ ,  $\mu'_w$ ) for slope stability charts for  $\Phi = 0$  and  $\Phi > 0$  soils

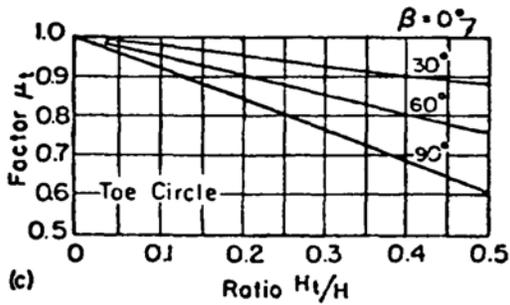
REDUCTION FACTOR FOR TENSION CRACK  
No Hydrostatic Pressure in Crack



Key Sketch



REDUCTION FACTOR FOR TENSION CRACK  
Full Hydrostatic Pressure in Crack



Key Sketch

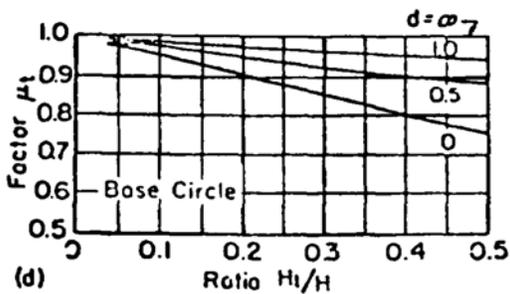
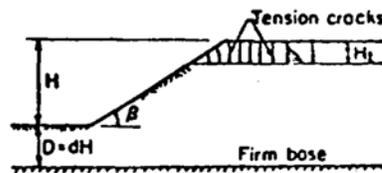
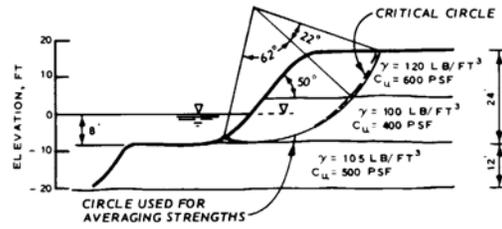
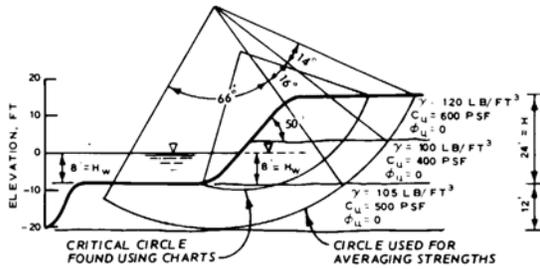


Figure 3

Reduction factors (tension cracks,  $\mu_t$ ) for slope stability charts for  $\Phi = 0$  and  $\Phi > 0$  soils



$$C_{ave} = \frac{(14)(600) + (16)(400) + (66)(500)}{96} = 498 \text{ psf}$$

$$\gamma_{ave} \approx \frac{120 + 100 + 105}{3} = 108 \text{ lb/ft}^3$$

$$d = \frac{12}{24} = 0.5 \quad H_w/H = \frac{8}{24} = 0.33$$

From figure 8-2,  $\mu_w = 0.95$  for  $d = 0.5$  and  $H_w/H = 0.33$

From figure 8-1,  $N_o = 5.6$  for  $d = 0.5$  and  $\beta = 50^\circ$

$$F = \frac{(0.95)(5.6)(498)}{(108)(24) - (62.4)(8)} = 1.27$$

From figure 8-1,  $x_o = 0.35$ ,  $y_o = 1.5$ , critical circle intersects near toe of slope

$$x_o = (0.35)(24) = 8.4 \text{ ft}, \quad y_o = (1.5)(24) = 36 \text{ ft}$$

$$C_{ave} = \frac{(22)(600) + (62)(400)}{84} = 452 \text{ psf}$$

$$\gamma_{ave} \approx \frac{120 + 100}{2} = 110 \text{ lb/ft}^3$$

$$d = 0 \quad H_w/H = \frac{8}{24} = 0.33$$

From figure 8-2,  $\mu_w = 0.93$  for  $\beta = 50^\circ$  and  $H_w/H = 0.33$

From figure 8-1,  $N_o = 5.8$  for  $\beta = 50^\circ$

$$F = \frac{(0.93)(5.8)(452)}{(110)(24) - (62.4)(8)} = 1.14$$

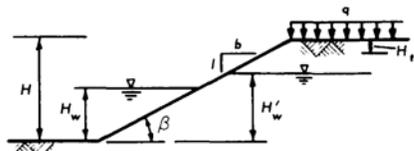
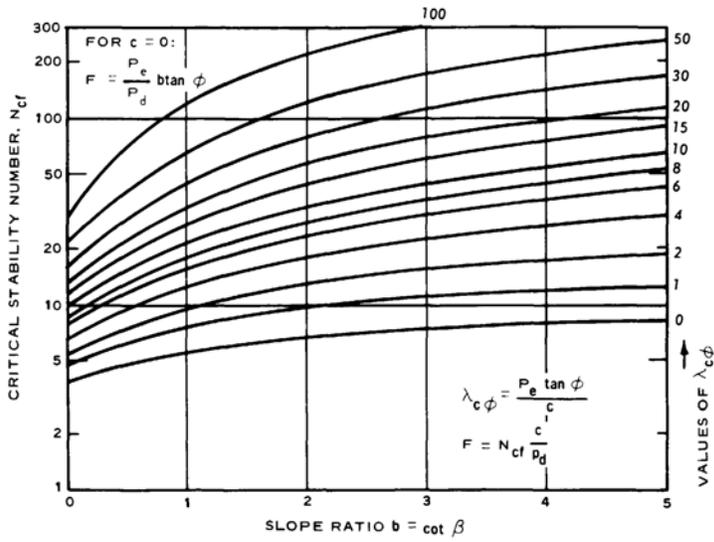
(more critical than circle tangent to elev. -20 ft)

From figure 8-3,  $x_o = 0.35$ ,  $y_o = 1.4$ , critical circle intersects near toe of slope

$$x_o = (0.35)(24) = 8.4 \text{ ft}, \quad y_o = (1.4)(24) = 33.6 \text{ ft}$$

Figure 4

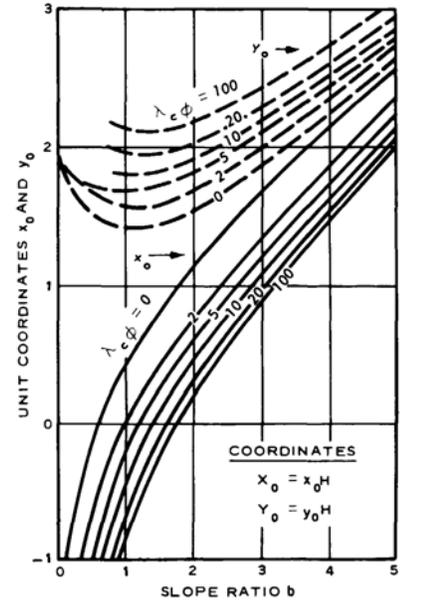
Example of use of charts for slopes in soils with uniform strength and  $\Phi = 0$



$$P_d = \frac{\gamma H + q - \gamma_w H_w}{\mu_q \mu_w \mu_t}$$

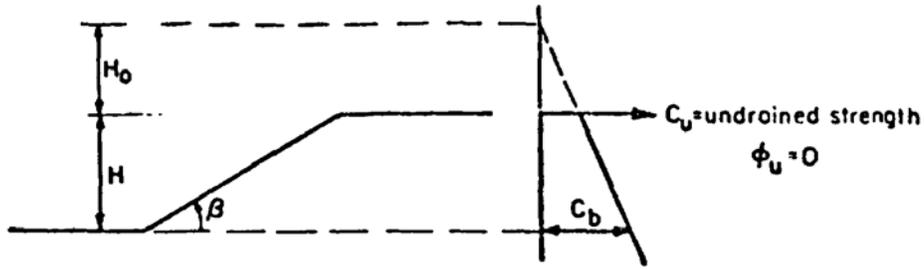
$$P_e = \frac{\gamma H + q - \gamma_w H'_w}{\mu_q \mu'_w}$$

(IN FORMULA FOR  $P_e$  TAKE  $q = 0$ ,  $\mu_q = 1$  FOR UNCONSOLIDATED CONDITION)



CENTER COORDINATES FOR CRITICAL CIRCLE

Figure 5  
Slope stability charts for  $\Phi > 0$  soils



Steps:

- ① Extrapolate strength profile upward to determine value of  $H_0$ , where strength profile intersects zero
- ② Calculate  $M = H_0/H$
- ③ Determine stability number  $N$  from chart below
- ④ Determine  $C_b$  = strength at bottom of slope
- ⑤ Calculate  $F = N \frac{C_b}{\gamma(H+H_0)}$

Use  $\gamma = \gamma_{\text{buoyant}}$  for submerged slope

Use  $\gamma = \gamma_m$  for no water outside slope

Use average  $\gamma$  for partly submerged slope

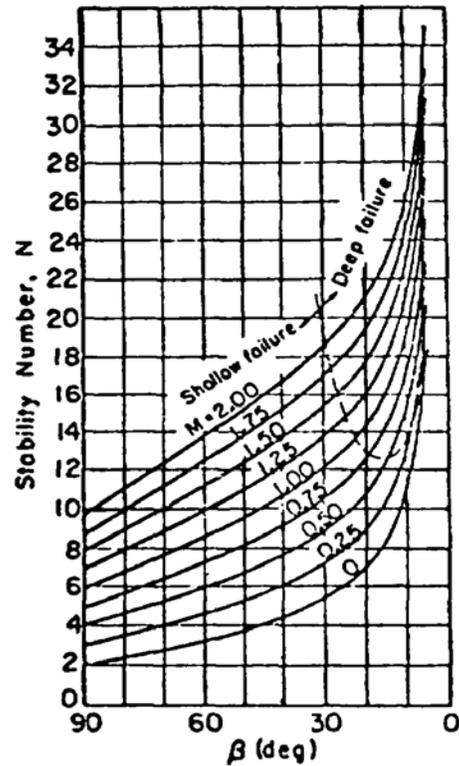
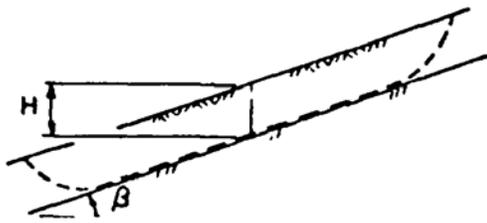
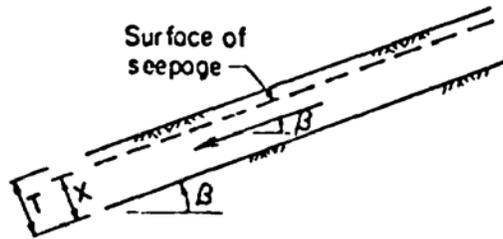


Figure 6

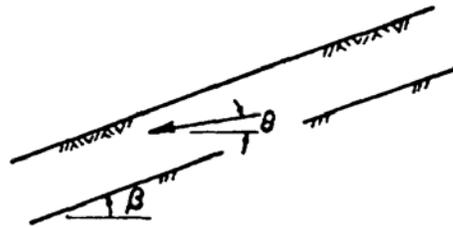
Stability charts for infinite slopes



$\gamma$  = total unit weight of soil  
 $\gamma_w$  = unit weight of water  
 $c'$  = cohesion intercept  
 $\phi'$  = friction angle } Effective Stress  
 $r_u$  = pore pressure ratio =  $\frac{u}{\gamma H}$   
 $u$  = pore pressure at depth  $H$



Seepage parallel to slope  
 $r_u = \frac{X}{T} \frac{\gamma_w}{\gamma} \cos^2 \beta$



Seepage emerging from slope at angle  $\theta$

$$r_u = \frac{\gamma_w}{\gamma} \frac{1}{1 + \tan \beta \tan \theta}$$

**Steps:**

- ① Determine  $r_u$  from measured pore pressures or formulas at right
- ② Determine A and B from charts below
- ③ Calculate  $F = A \frac{\tan \phi'}{\tan \beta} + B \frac{c'}{\gamma H}$

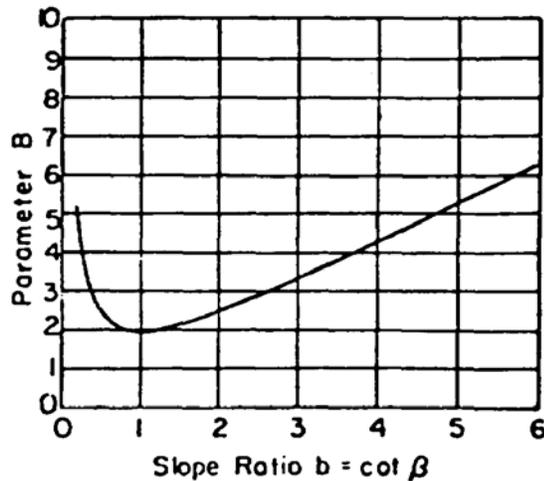
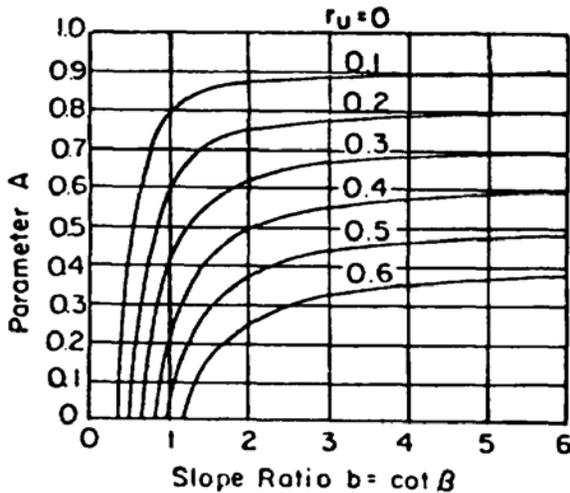
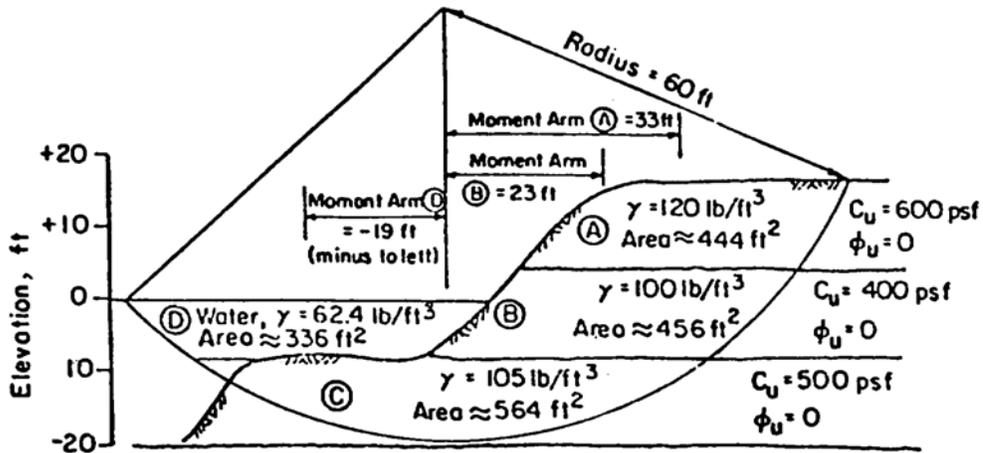


Figure 7

Slope stability charts for  $\Phi = 0$  and strength increasing with depth



Section	Area (ft <sup>2</sup> )	$\gamma$ (lb/ft <sup>3</sup> )	Weight (lb/ft)	Moment Arm (ft)	Moment (ft-lb/ft)
(A)	444	120	53,280	+ 33	+ 1.76 × 10 <sup>6</sup>
(B)	456	100	45,600	+ 23	+ 1.05 × 10 <sup>6</sup>
(C)	564	105	59,220	0	0.0
(D)	336	62.4	20,970	- 19	- 0.40 × 10 <sup>6</sup>
Total Overturning Moment =					+ 2.41 × 10 <sup>6</sup>

Section	Ave. Length (ft)	C <sub>u</sub> (psf)	Force (lb/ft)	Moment Arm = Radius (ft)	Moment (ft-lb/ft)
(A)	14	600	8,400	60	0.50 × 10 <sup>6</sup>
(B)	16.5	400	6,600	60	0.40 × 10 <sup>6</sup>
(C)	69	500	34,500	60	2.07 × 10 <sup>6</sup>
(D)	18	0	0	60	0.00
Total Resisting Moment =					2.97 × 10 <sup>6</sup>

$$\text{Factor of Safety, } F = \frac{\text{Resisting Moment}}{\text{Overturning Moment}} = \frac{2.97 \times 10^6}{2.41 \times 10^6} = 1.23$$

Figure 8

Method of moments for  $\Phi = 0$

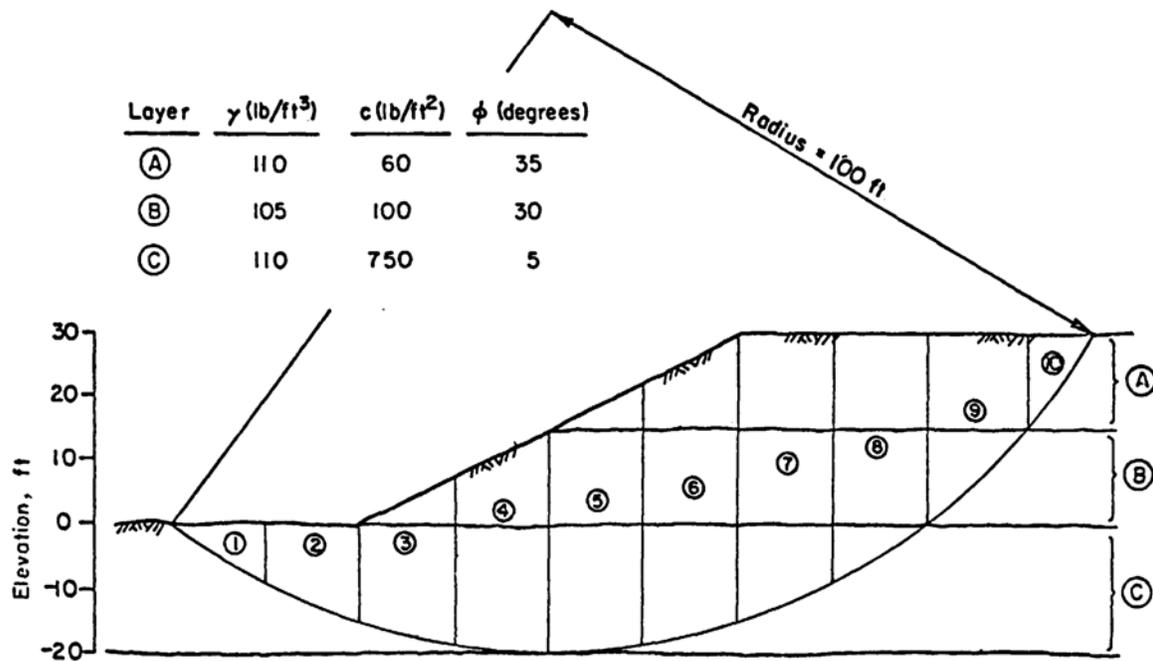
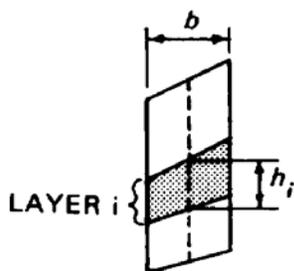


Figure 9

Example problem for ordinary method of slices



$\gamma_i$  = unit weight of layer i

$h_i$  = height of layer at center of slice

$W_i$  = partial weight =  $bh_i\gamma_i$

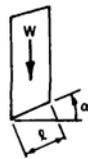
$\Sigma W_i$  = total weight of slice

Slice No.	b ft	$h_i$ ft	$\gamma_i$ lb/ft <sup>3</sup>	$W_i$ lb/ft	$\Sigma W_i$ lb/ft
1	15	5	110	8,100	8,200
2	15	13	110	21,400	21,400
3	15	4	105	6,300	35,200
		17.5	110	28,900	
4	15	11.5	105	18,100	50,300
		19.5	110	32,200	
5	15	4	110	6,600	62,400
		15	105	23,600	
		19.5	110	32,200	
6	15	11.5	110	19,000	71,500
		15	105	23,600	
		17.5	110	28,900	
7	15	15	110	24,800	69,800
		15	105	23,600	
		15	110	21,400	
8	15	15	110	24,800	56,600
		15	105	23,600	
		5	110	8,200	
9	16	15	110	26,400	39,000
		7.5	105	12,600	
10	11	7.5	110	9,100	9,100

Figure 10

Example of use of tabular form for computing weights of slices

Slice No.	W kip/ft	l ft	α deg	C kip/ft <sup>2</sup>	φ deg	u		W cos α kip/ft	Σ W cos α - ul	Σ W cos α - ul tan φ	c l	Σ W sin α
						Zero for Total Stress Analysis	ul					
1	8.2	17.5	-32	0.75	5			6.9		0.61	13.12	-4.35
2	21.4	16.2	-22	0.75	5			19.8		1.73	12.15	-8.22
3	35.2	15.5	-13	0.75	5			34.3		3.00	11.62	-7.92
4	50.3	15.1	-4	0.75	5			50.2		4.39	11.32	-3.51
5	62.4	15.1	4	0.75	5			62.2		5.44	11.32	4.35
6	71.5	15.5	13	0.75	5			69.7		6.10	11.62	16.08
7	69.8	16.2	22	0.75	5			64.7		5.66	12.15	26.15
8	56.6	17.5	32	0.75	5			48.0		4.20	13.12	30.00
9	39.0	22.0	43	0.10	30			28.5		16.45	2.20	26.60
10	9.1	18.5	55	0.06	35			5.2		3.64	1.11	7.45
									Σ	51.2	99.7	86.8



c = cohesion intercept  
 φ = friction angle  
 u = pore pressure

} at base of slice

$$F = \frac{\Sigma(W \cos \alpha - ul) \tan \phi + \Sigma cl}{\Sigma W \sin \alpha}$$

$$F = \frac{150.1}{86.8} = 1.74$$

Figure 11

Example of use of tabular form for calculating factor of safety  
by ordinary method of slices

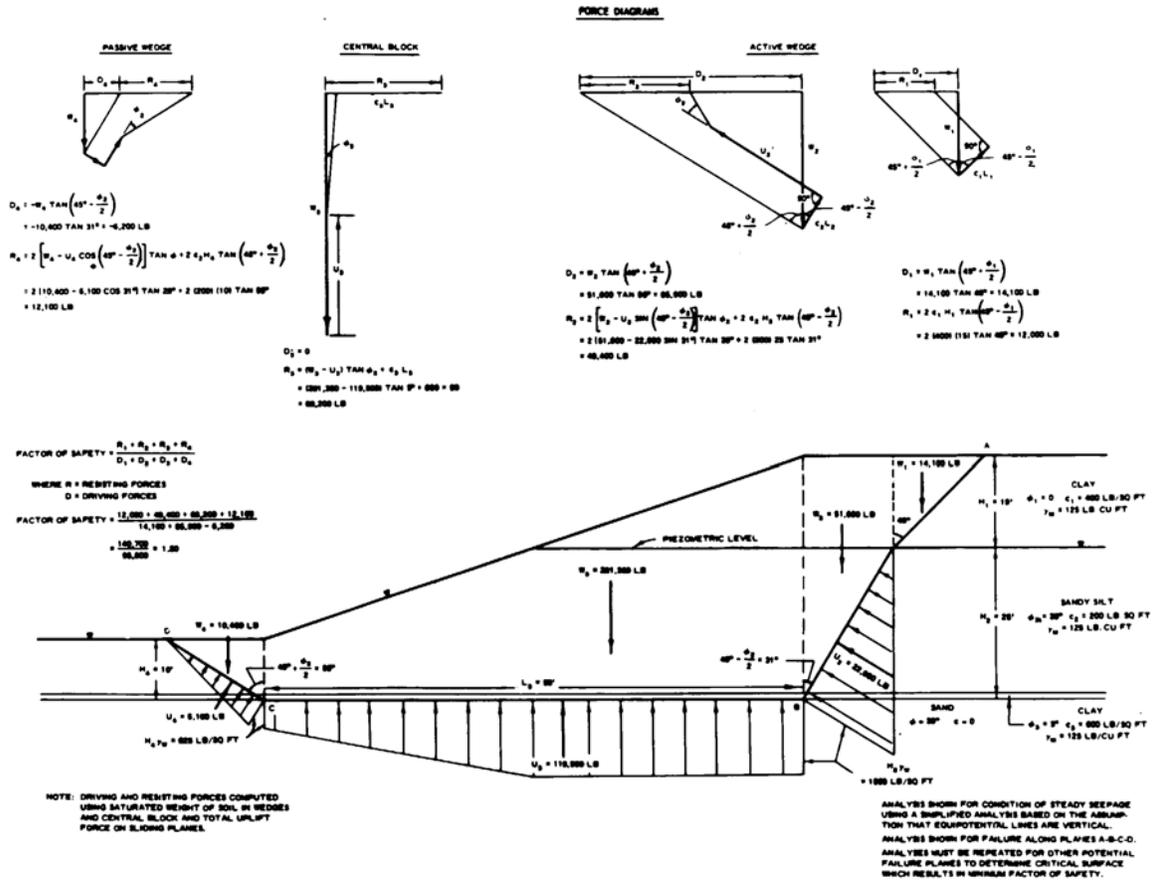


Figure 12

Example of simplified wedge analysis