CHAPTER 8

SLOPE STABILITY ANALYSIS

8-1. General. This chapter 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 chapter is on simple, routine procedures. It does not deal with specialized problems, such as the stability of excavated slopes during earthquakes.

8-2. 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.

   a. Cohesionless slopes resting on firm soil or rock. The stability of slopes consisting of cohesionless soils depends on the angle of internal friction ϕ', 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 ϕ' for stability analyses are determined from laboratory tests or estimated from correlations (para 3-6). 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, as discussed in Chapter 17.

   b. 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:

   (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 unconfined 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) *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.

   (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.

   c. 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 methods described in chapter 4.

8-3. Slopes In soils presenting special problems.

   a. 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.

b. 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.

c. 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.

d. 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.

e. Hydraulic fills. See Chapter 15.

8-4. Slope stability charts.

a. Uniform soil, constant shear strength, $\phi = 0$, rotational failure.

(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 8-1 to compute the safety factor. If the center and depth of the critical circle are desired, obtain them from the lower diagrams of figure 8-1.

(2) *Partial slope submergence, seepage*

surcharge loading, tension cracks. The effect of partial submergence of a slope is given by a factor μ_w in figure 8-2; seepage is given by a factor μ_w' in figure 8-2; surcharge loading is given by a factor μ_q in figure 8-2; and tension cracks is given by a factor μ_t in figure 8-3. Compute safety factor from the following:

$$F = \frac{\mu_w \mu_w' \mu_q \mu_t N_0 C}{\gamma H + q - \gamma_w H_w'} \qquad (8\text{-}1)$$

where

γ = total unit weight of soil
q = surcharge loading
N_0 = stability number from figure 8-1

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.

b. Stratified soil layers, $\phi = O$, rotational failure.

(1) Where the slope and foundation consist of a number of strata, each having a constant shear strength, the charts given in figures 8-1 through 8-3 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 8-1. 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.

(2) It may be necessary to calculate the safety factor for failure surfaces at more than one depth, as illustrated in figure 8-4.

c. Charts for slopes in uniform soils with $\phi > 0$.

(1) A stability chart for slopes in soils with $\phi > 0$ is shown in figure 8-5. Correction factors for surcharge loading at the top of the slope, submergence, and seepage are given in figure 8-2; and for tension cracks, in figure 8-3.

(2) The location of the critical circle can be obtained, if desired, from the plot on the right side of figure 8-5. 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 8-5 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.

d. Infinite slopes. Conditions that can be analyzed accurately using charts for infinite slope analyses shown in figure 8-6 are-

(1) Slopes in cohesionless materials where the critical failure mechanism is shallow sliding or surface raveling.

(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.

e. Shear strength increasing with depth and ϕ *= 0.* A chart for slopes in soils with shear strength increasing with depth and + = 0 is shown in figure 8-7.

8-5. 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.

a. The method of moments for $\phi = 0$. This method is simple but useful for the analysis of circular slip surfaces in $\phi = 0$ soils, as shown in figure 8-8.

b. The ordinary method of slices. This simple and conservative procedure for circular slip surfaces can be used in soils with $\phi \geq 0$. For flat slopes with high pore pressures and $\phi > 0$, 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 8-9 through 8-11. Various trial circles must be assumed to find the critical one. If ϕ 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.

c. The simplified wedge method. This method is a simple and conservative procedure for analyzing noncircular surfaces. An example is shown in 8-12. 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.

8-6. 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. Methods for stabilizing slopes and landslides are summarized in table 8-1. Often one or more of these schemes may be used together. Schemes I through V are listed approximately in order of increasing cost.
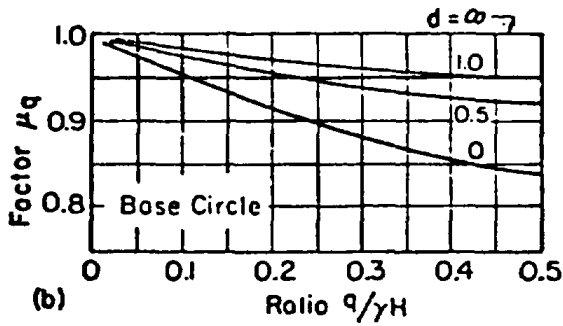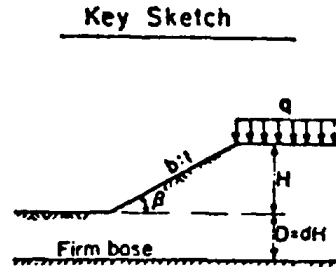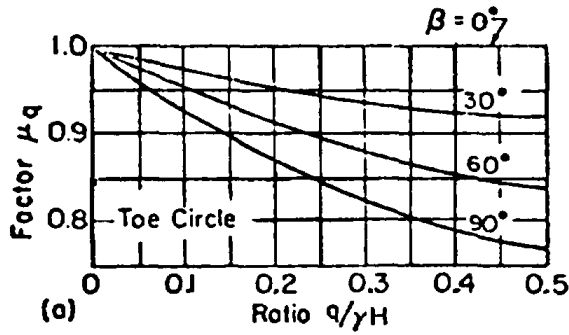
8-3

STABILITY NUMBER
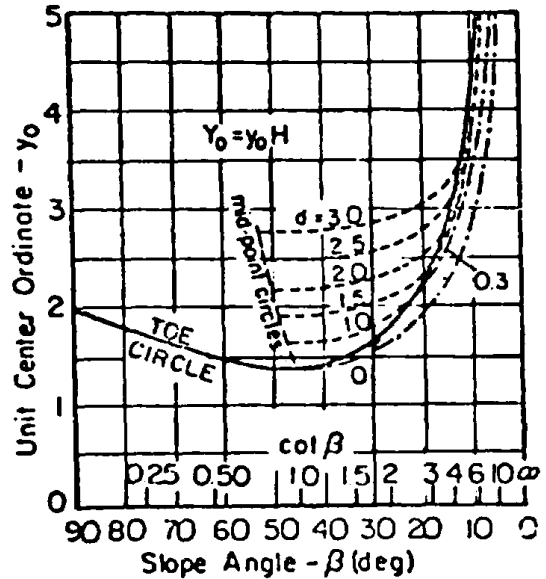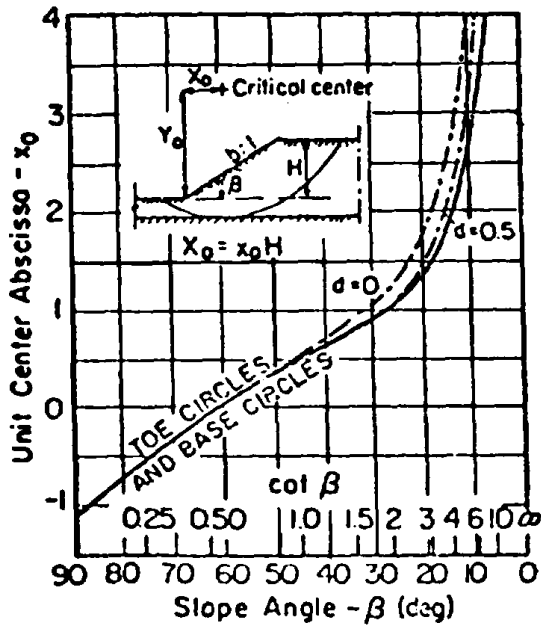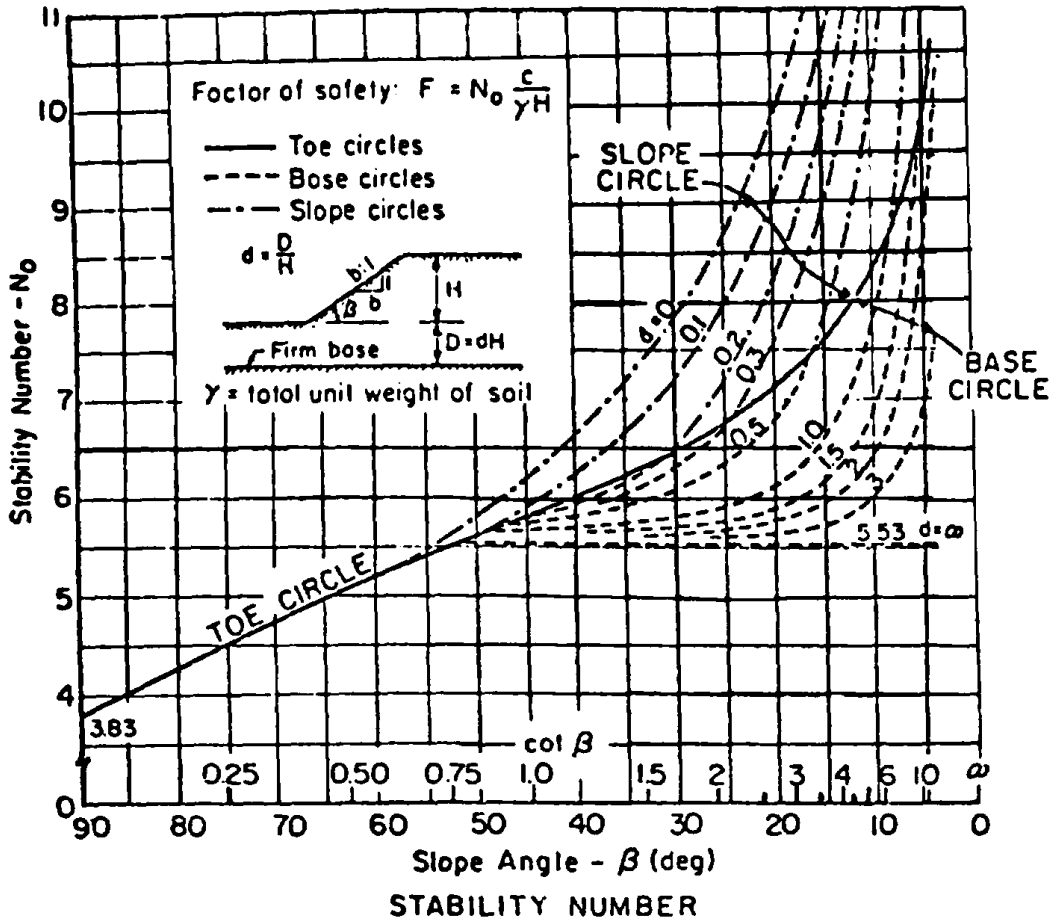
CENTER COORDINATES FOR CRITICAL CIRCLE

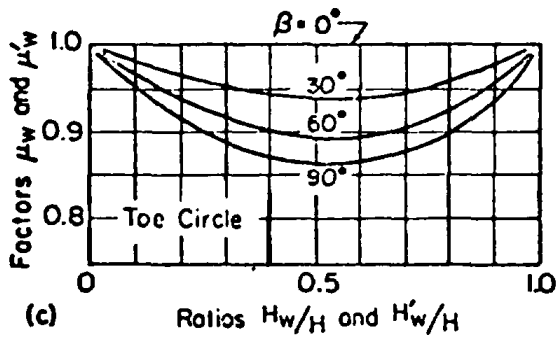U. S. Army Corps of Engineers
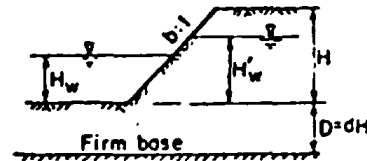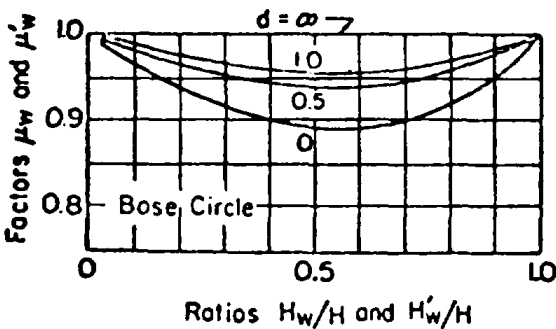
Figure 8-1. Slope stability charts for φ = 0 soils.

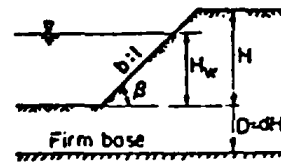## REDUCTION FACTORS FOR SURCHARGE LOADING (μ_q)

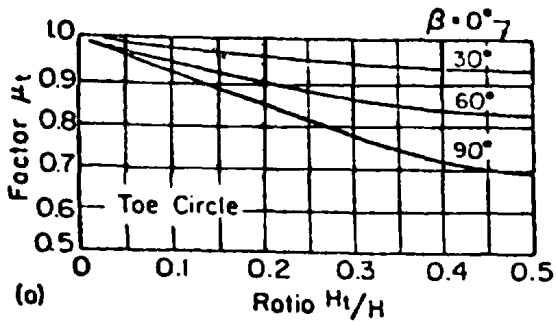## REDUCTION FACTORS FOR SUBMERGENCE (μ_w) AND SEEPAGE (μ_w')
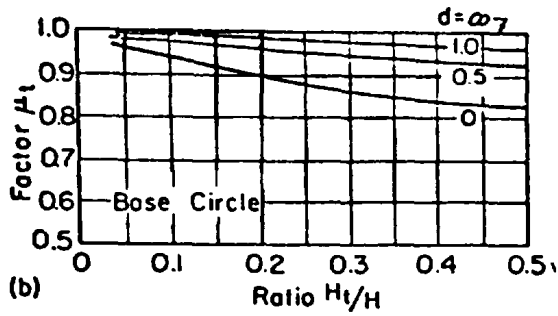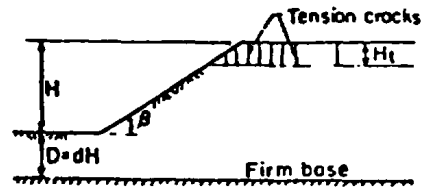

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Figure 8-2. Reduction factors (μ_q, μ_w, μ_w') for slope stability charts for ϕ =0 and ϕ >0 soils.
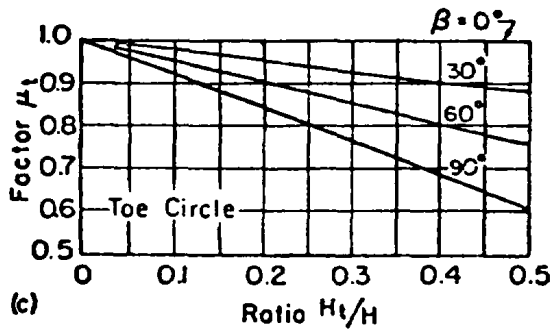
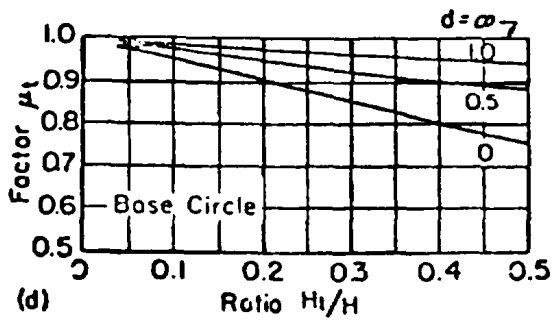REDUCTION FACTOR FOR TENSION CRACK
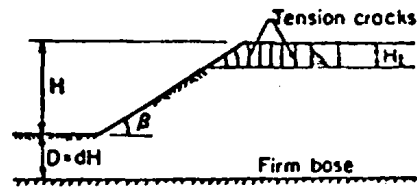No Hydrostatic Pressure in Crack

Key Sketch

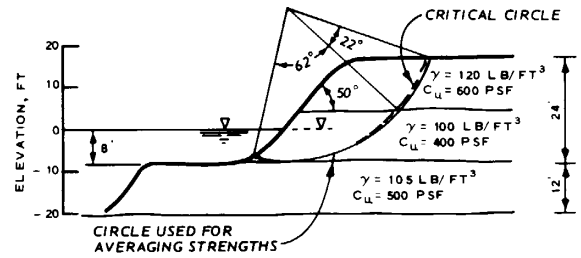REDUCTION FACTOR FOR TENSION CRACK
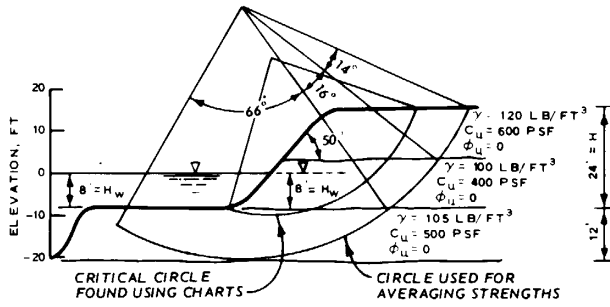Full Hydrostatic Pressure in Crack

Key Sketch

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Figure 8-3. Reduction factors (tension cracks, μ_t) for slope stability charts for $\phi = 0$ and $\phi > O$ soils.

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

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

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

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 $\beta = 50°$

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

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

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

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

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

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

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

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

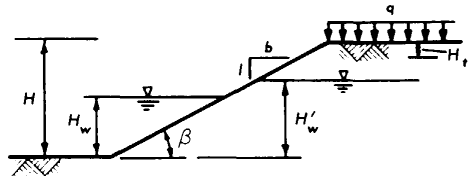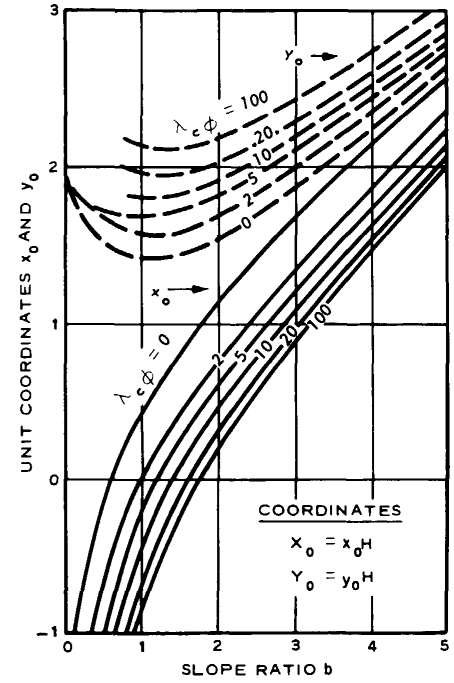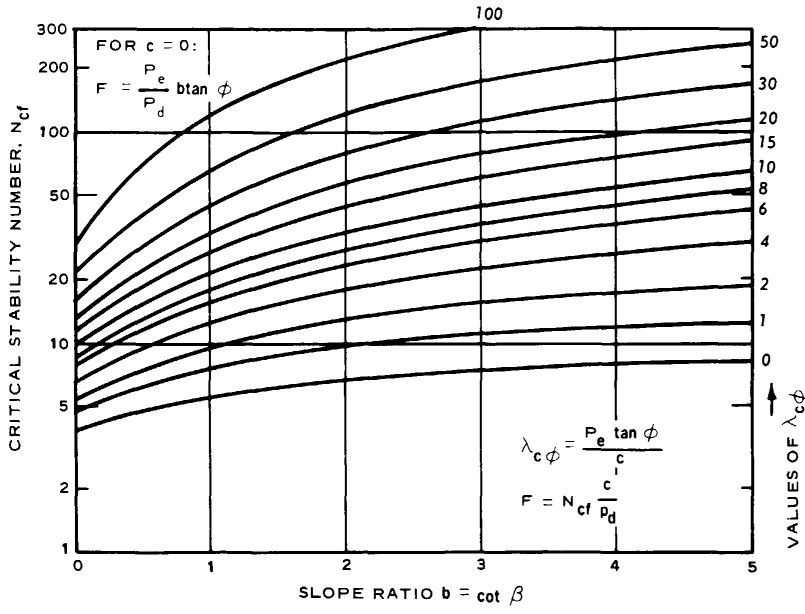$$F = \frac{(0.98)(5.8)(452)}{(110)(24) - (62.4)(8)} = 1.14$$

(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

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

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Figure 8-4. Example of use of charts forslopes in soils with uniform strength and $\phi = 0$.

$$F = \frac{P_e}{P_d} b \tan \phi \quad \text{FOR } c = 0$$

$$\lambda_{c\phi} = \frac{P_e \tan \phi}{c'}$$

$$F = N_{cf} \frac{c'}{P_d}$$

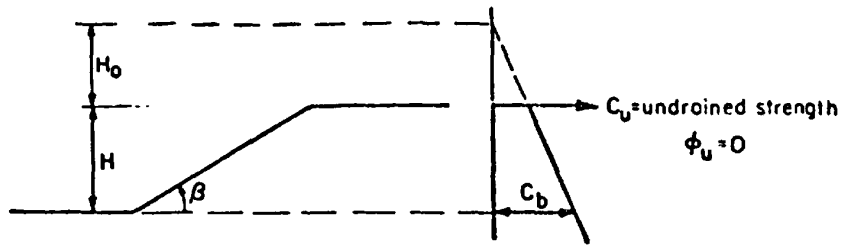$$P_d = \frac{\gamma H + q - \gamma_w H_w}{\mu_q \mu_w \mu_t}$$

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

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

CENTER COORDINATES FOR CRITICAL CIRCLE

COORDINATES
$$X_0 = x_0 H$$
$$Y_0 = y_0 H$$

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*Figure 8-5. Slope stabdilty charts for $\phi > 0$ soils.*

8-8

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 $F = N \dfrac{C_b}{\gamma(H+H_0)}$

Use $\gamma = \gamma_{buoyant}$ for submerged slope
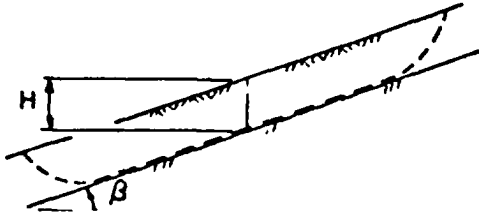
Use $\gamma = \gamma_m$ for no water outside slope

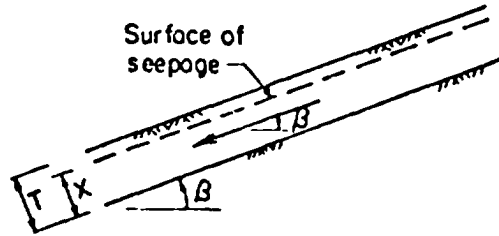Use average γ for partly submerged slope
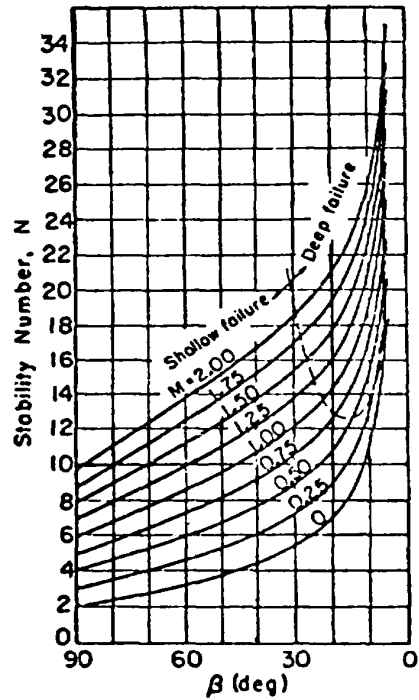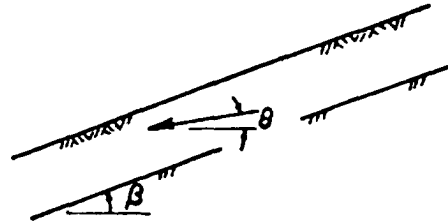

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Figure 8-6. Stability charts for infinite slopes.

8-9

γ = total unit weight of soil

γ_w = unit weight of water

c' = cohesion intercept
ϕ' = friction angle } Effective Stress

r_u = pore pressure ratio = $\dfrac{u}{\gamma H}$

u = pore pressure at depth H

Steps:

① Determine r_u from measured pore pressures or formulas at right

② Determine A and B from charts below

③ Calculate $F = A\dfrac{\tan\phi'}{\tan\beta} + B\dfrac{c'}{\gamma H}$

Seepage parallel to slope

$$r_u = \frac{X}{T}\frac{\gamma_w}{\gamma}\cos^2\beta$$

Seepage emerging from slope at angle θ

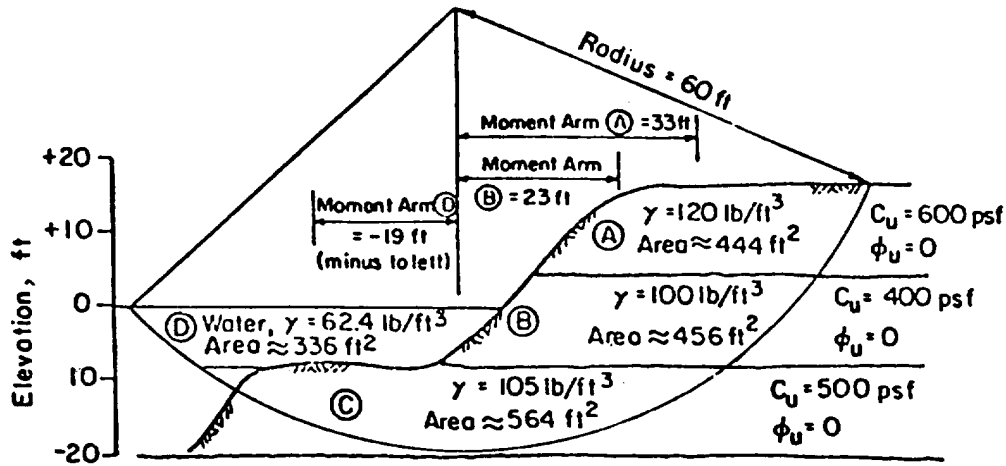$$r_u = \frac{\gamma_w}{\gamma}\frac{1}{1+\tan\beta\,\tan\theta}$$
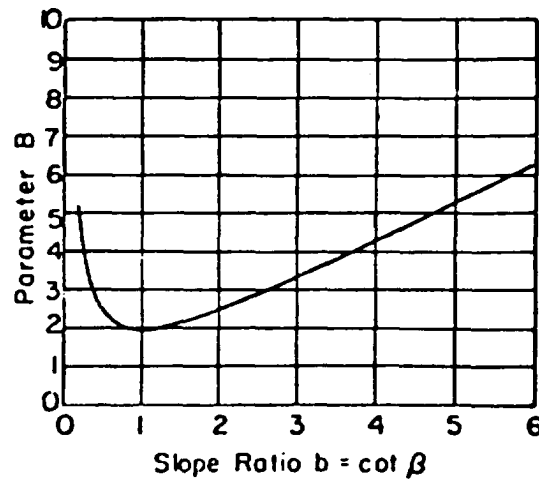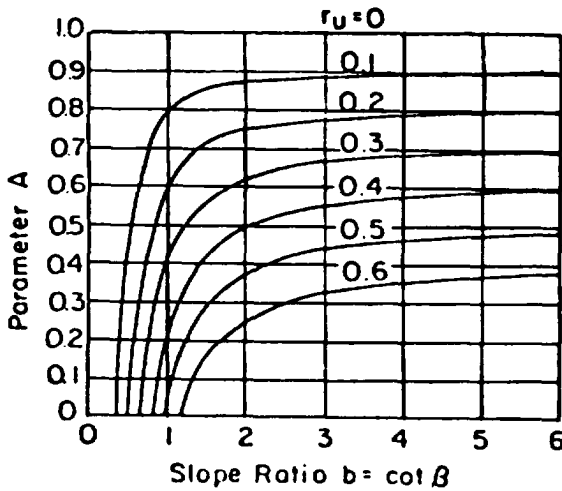
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Figure 8- 7. Slope stability charts for ϕ = 0 and strength increasing with depth.

| Section	Area (ft^2)	γ (lb/ft^3)	Weight (lb/ft)	Moment Arm (ft)	Moment (ft-lb/ft)
Ⓐ	444	120	53,280	+ 33	+ 1.76×10^6
Ⓑ	456	100	45,600	+ 23	+ 1.05×10^6
Ⓒ	564	105	59,220	0	0.0
Ⓓ	336	62.4	20,970	– 19	– 0.40×10^6

Total Overturning Moment = + 2.41×10^6

Section	Ave. Length (ft)	C_u(psf)	Force (lb/ft)	Moment Arm = Radius (ft)	Moment (ft-lb/ft)
Ⓐ	14	600	8,400	60	0.50×10^6
Ⓑ	16.5	400	6,600	60	0.40×10^6
Ⓒ	69	500	34,500	60	2.07×10^6
Ⓓ	18	0	0	60	0.00

Total Resisting Moment = 2.97×10^6

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

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Figure 8-8. Method of moments for ϕ = 0.

Layer	γ (lb/ft^3)	c (lb/ft^2)	ϕ (degrees)
Ⓐ	110	60	35
Ⓑ	105	100	30
Ⓒ	110	750	5
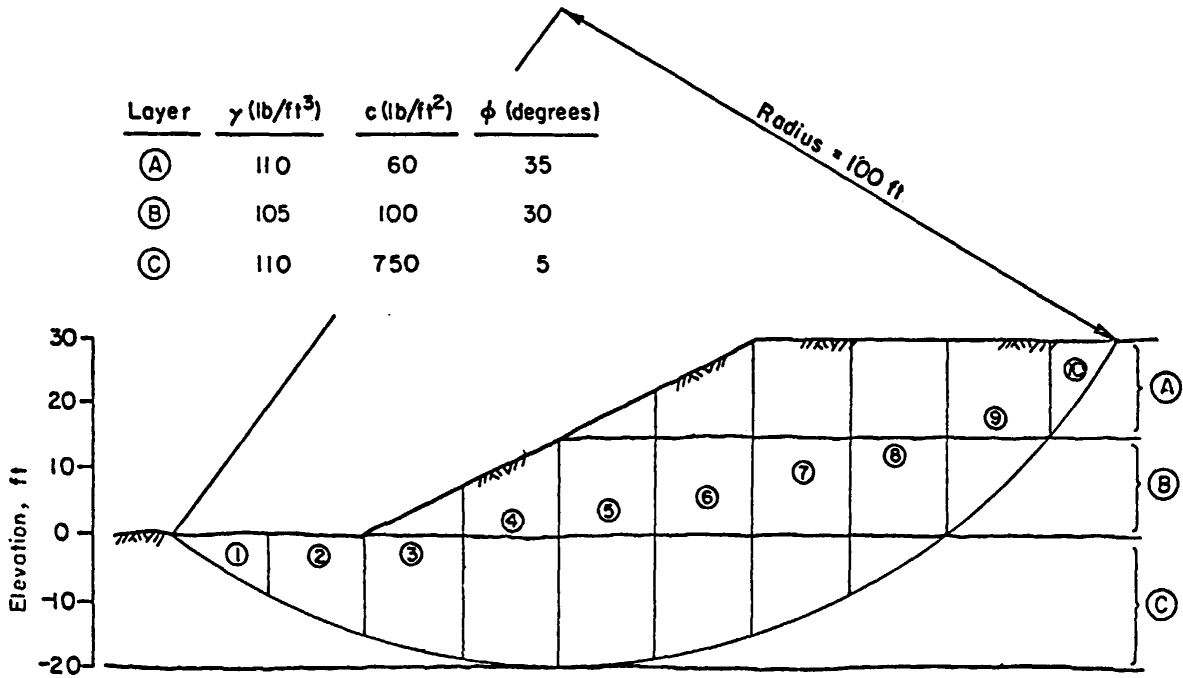
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Figure 8-9. Example problem for ordinary method of slices.

8-12
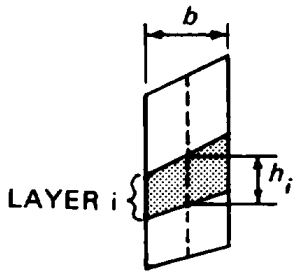
γ_i = unit weight of layer i

h_i = height of layer at center of slice

W_i = partial weight = $bh_i\gamma_i$

ΣW_i = total weight of slice

| Slice No.	b ft	h_i ft	γ_i lb/ft^3	W_i lb/ft	ΣW_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	
		17.5	110	28,900	35,200
4	15	11.5	105	18,100	
		19.5	110	32,200	50,300
5	15	4	110	6,600	
		15	105	23,600	
		19.5	110	32,200	62,400
6	15	11.5	110	19,000	
		15	105	23,600	
		17.5	110	28,900	71,500
7	15	15	110	24,800	
		15	105	23,600	
		15	110	21,400	69,800
8	15	15	110	24,800	
		15	105	23,600	
		5	110	8,200	56,600
9	16	15	110	26,400	
		7.5	105	12,600	39,000
10	11	7.5	110	9,100	9,100

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Figure 8-10. Example of use of tabular form for computing weights of slices.

Slice No.	W kip/ft	ℓ ft	α deg	C kip/ft²	φ deg	u Zero for Total Stress Analysis	uℓ	W cos α kip/ft	$\frac{W \cos \alpha - u\ell}{W}$	$\frac{(W \cos \alpha - u\ell)\tan\phi}{W}$	cℓ	W sin α
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
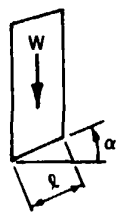
c = cohesion intercept
φ = friction angle } at base of slice
u = pore pressure

$$F = \frac{\Sigma(W \cos \alpha - u\ell)\tan\phi + \Sigma c\ell}{\Sigma W \sin \alpha}$$

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

Figure 8-11. Example of use of tabular form for calculating factor of safety by ordinary method of slices.
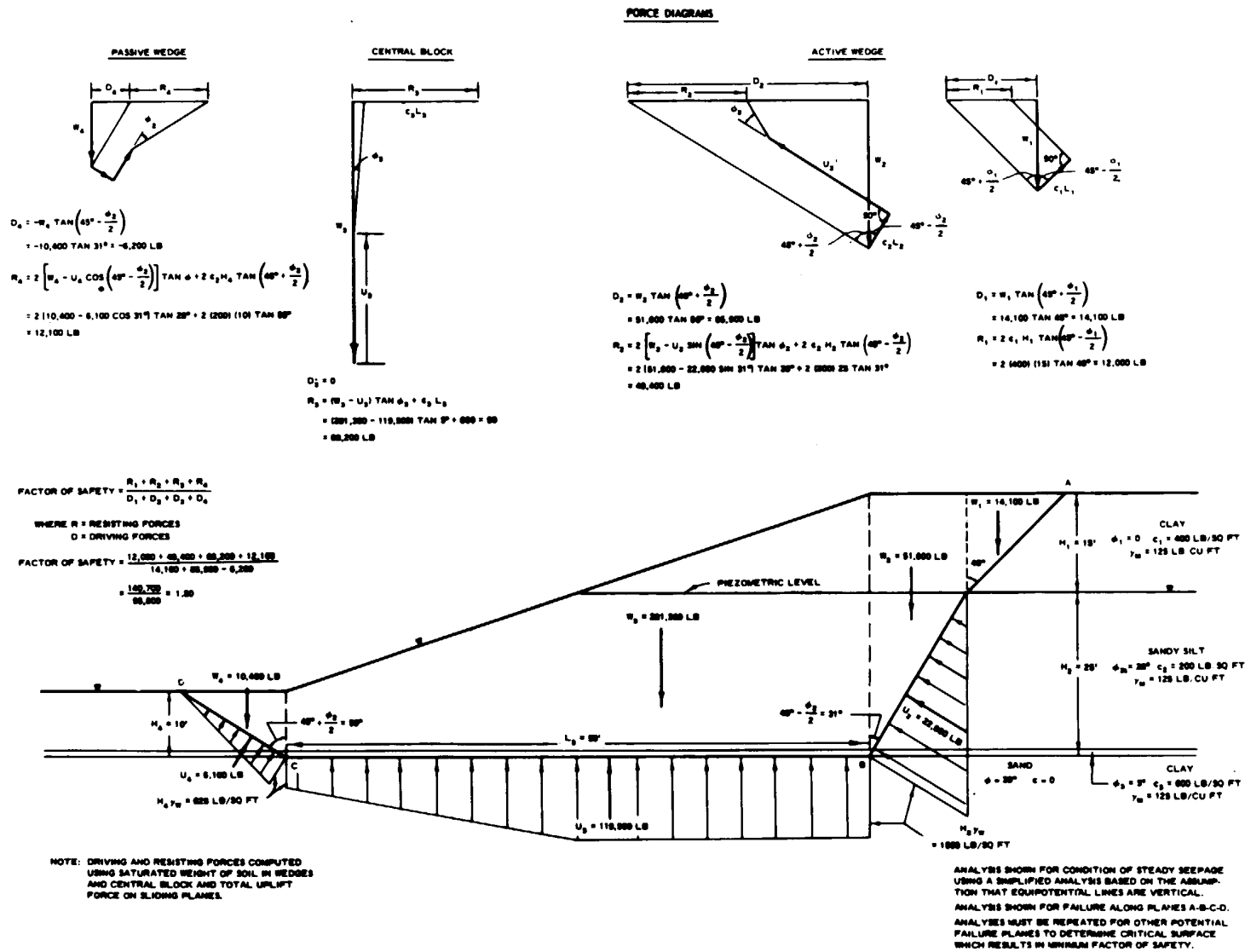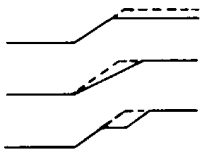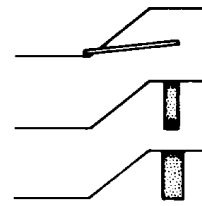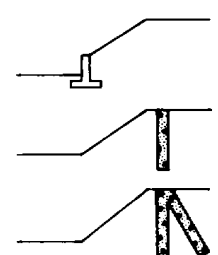
Figure 8-12. Example of simplified wedge analysis.

Scheme	Applicable Methods	Comments
I. Excavation	1. Reduce slope height by excavation at top of slope.	
	2. Flatten the slope angle.	Area has to be accessible to construction equipment. Disposal site needed for excavated soil. Drainage sometimes incorporated in this method.
	3. Excavate a bench in upper part of slope.	
	4. Excavate the entire slide mass.	
II. Drainage	1. Small diameter, horizontal drains (hydraugers).	1. Most effective if can tap natural aquifer. Drains are usually free-flowing.
	2. Continuous deep subdrain trench. Generally 3 to 15 ft deep.	2. Trench bottom should be sloped to drain and be tapped with an outlet pipe. Perforated pipe should be placed on trench bottom. Top of trench should be capped with impervious material.
	3. Drilled vertical wells - generally 18- to 36-in. diameter.	3. Can be pumped or tapped with a gravity outlet. Several wells in a row, joined at bottom can form a drainage gallery. Top of each well should be capped with impervious material.
	4. Improve surface drainage along top of slope with open ditch or paved gutter. Install deep-rooted, erosion-resistant plants on slope face.	4. Good practice for most slopes. Direct the discharge away from slide mass.
III. Earth or rock buttress (or berm fill)	1. Excavate slide mass and replace with compacted earth or rock buttress fill. Toe of buttress must be keyed into firm soil or rock below slide plane. Drain blanket with gravity flow outlet is provided in back slope of buttress fill.	1. Access for construction equipment and temporary stockpile area required. Excavated soil can usually be used in fill. Underpinning of existing structures may be required. Might have to be done in shore sections if stability during construction is critical.
	2. Compacted earth or rock berm placed at and beyond the toe. Drainage may be provided behind berm.	2. Sufficient width and thickness of berm required so failure will not occur below or through berm.
IV. Retaining structures	1. Retaining wall - crib or cantilever type.	1. Usually expensive. Cantilever walls might have to be tied back.
	2. Drilled, cast-in-place vertical piles, bottomed well below bottom of slide plane. Generally 18 to 36 in. in diameter and 4- to 8-ft spacing.	2. Spacing should be such that soil can arch between piles. Grade beam can be used to tie piles together. Very large diameter (6 ft ±) piles have been used for deep slides.
	3. Drilled, cast-in-place vertical piles tied back with battered piles or a deadman. Piles bottomed well below slide plane. Generally 12 to 30 in. in diameter and at 4- to 8-ft spacing.	3. Space close enough so soil will arch between piles. Piles can be tied together with grade beam.
	4. Earth anchors and rock bolts.	4. Can be used for high slopes, and in very limited stress. Conservative design should be used, especially for permanent support.
	5. Reinforced earth.	5. Usually expensive.
V. Special techniques	1. Grouting	1. and 2. Used successfully in a number of cases. Used at other times with little success. At the present, theory is not completely understood.
	2. Chemical injection	
	3. Electroomosis (in fine-grained soils).	3. Generally expensive.
	4. Freezing	4. and 5. Special methods which must be specifically evaluated at each site. Can be expensive.
	5. Heating	
		All of these techniques should be carefully evaluated in advance to determine the probable cost and effectiveness.

Table 8-1. Methods of Stabilizing Slopes and Landslides