21

Stability of Slopes

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21.1 Introduction

Slope stability analysis is performed to assess the potential for failure of the slope by rupture. Slopes that are typically assessed fall into a number of categories, as illustrated in Fig. 21.1. The primary objective of a stability analysis is to determine the factor of safety (FS) of a particular slope, to predict when failure is imminent, and to assess remedial treatments when necessary. In many practical situations, an analytical assessment of stability can be made. Other situations do not lend themselves to convenient analytical solutions. Analytical techniques can be applied to slope failures where peak strength occurs essentially simultaneously at every point along single or multiple failure surfaces and the mass moves as a unit or group of units. The sliding surface may be circular, planar, or irregular.

Slope failure forms that cannot be analyzed in the present state of the art include avalanches, flows, failure by lateral spreading, and progressive failure. All of these forms can be initiated by a slide failure at the toe of the mass. The only defense against these failure forms is recognition that their potential exists [Hunt, 1984].

Deformations can be a concern in slopes and embankments due to the effects on surface or buried structures, and because they often precede failure. The finite element method (FEM) has been used to approximate deformations in earth-dam embankments and rock slopes, but only infrequently in natural and cut slopes in soils [Vulliet and Hutter, 1988]. In any case, it is necessary to closely define material properties, slope geometry, and the initial state of stress, which often are difficult to assess accurately. The most common analytical approaches currently used by practitioners to assess slope stability are based on the limiting equilibrium method. It is an approximate solution that considers a state of equilibrium between the forces acting to cause failure (driving forces) and the forces resisting failure (mobilized shear stresses). Two general cases are illustrated in Fig. 21.2.

Limit equilibrium analyses assume the following:

1. The failure surface is of simple geometric shape (planar, circular, log-spiral).
2. The distribution of stresses acting along the failure surface causing failure are determinate.
3. The same percentage of mobilized shear strength acts simultaneously along the entire failure surface.

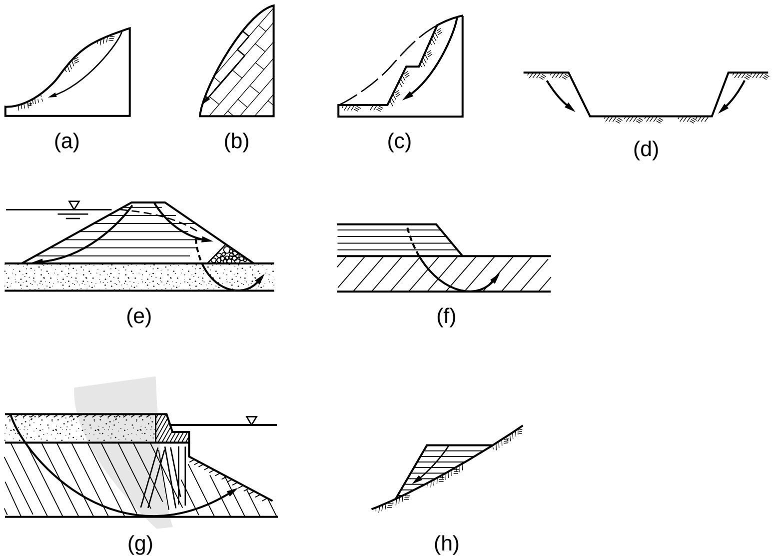


FIGURE 21.1 Categories of slope stability problems: (a) natural soil slope; (b) natural rock slope; (c) cut slope; (d) open excavation; (e) earth dam embankment; (f) embankment over soft soils; (g) waterfront structure; (h) sidehill fill. (*Source:* Hunt, R. E. 1986. *Geotechnical Engineering Techniques and Practices.* McGraw-Hill, New York. Reprinted with permission of McGraw-Hill Book Co.)

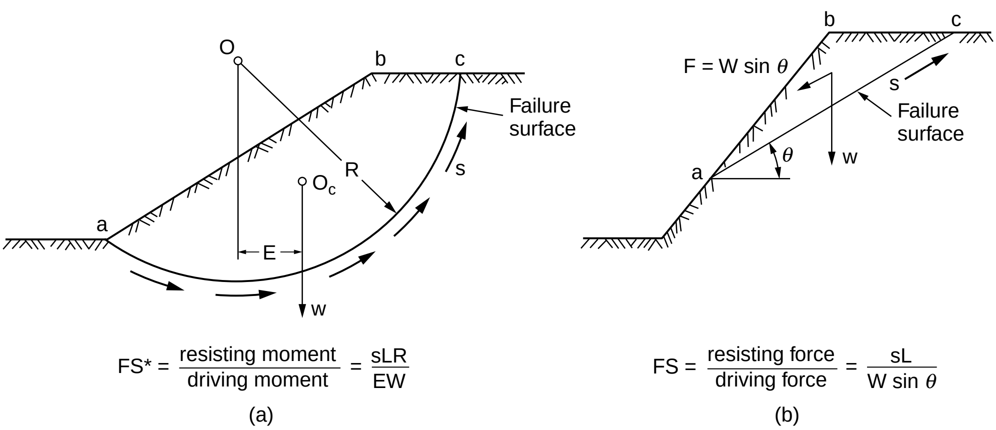


FIGURE 21.2 Forces acting on cylindrical failure surfaces. (a) Rotational cylindrical failure surface with length *L*. Safety factor against sliding FS. (b) Simple wedge failure on planar surface with length *L*. *Note:* The expression for FS is generally considered unsatisfactory (see text).

Limit equilibrium methods of analysis typically use the Mohr–Coulomb failure criterion, in which (see Table 21.1)

$$s = c + \sigma_n \tan \phi$$

where s is the shearing resistance, c is the cohesion, σ_n is the normal stress, and ϕ is the angle of internal friction. Appropriate total stress or effective stress strength parameters for specific loading conditions are provided in Table 21.2.

The factor of safety has been expressed in a number of forms, examples of which are given in Fig. 21.2 (see Table 21.3). In slope stability analysis, FS can be considered as the ratio of the total shearing strength

TABLE 21.1 Field Conditions and Strength Parameters Acting at Failure

Material	Field Conditions	Strength Parameter[a]
Cohesionless sands	Dry	ϕ ($i = \phi$)
	Submerged slope	ϕ ($i = \phi'$)
	Slope seepage with top flow line coincident with and parallel to slow surface	ϕ ($i = \phi'/2$)
Clays (except stiff fissured clays and clay shales)	Undrained conditions	S_u ($\phi = 0$)
	Drained conditions	c'^{b}
Stiff fissured clays and clay shales and existing failure surfaces	Without slope seepage	$\phi'_r(i \le \phi'_r)^b$
	With slope seepage	$\phi'_r(i \le \phi'_r/2^b)$
Cohesive mixtures	Undrained conditions	c_u, ϕ_u
	Drained conditions	$c'\phi^b$
Rock joints	Clean surfaces	ϕ or $\phi + j^c$
	With fillings	$c'\phi$, or ϕ'
	Clean but irregular surfaces after failure	$\phi_r + j^c$

Source: Hunt, R. E. 1986. *Geotechnical Engineering Techniques and Practices*. McGraw-Hill, New York. Reprinted with permission of McGraw-Hill Book Co.

[a] i = stable slope angle.

[b] Pore-water pressures: reduce frictional resistance in accordance with $(N - U)\tan\phi$.

[c] j = angle of asperities.

TABLE 21.2 Total versus Effective Stress Analysis

Condition	Preferred Method	Comment
Stability at intermediate times	$\bar{c}, \bar{\phi}$ analysis with estimated pore pressures	Actual pore pressures must be field-checked.
End of construction; partially saturated soil; construction period short compared to soil consolidation time	Either method: $\bar{c}_u, \bar{\phi}_u$ from CU tests, or $\bar{c}, \bar{\phi}$ plus estimated pore pressures	$\bar{c}, \bar{\phi}$ analysis permits check during construction using actual pore pressures
End of construction; saturated soil; construction period short compared to consolidation time	Total stress or s_u analysis with $\phi = 0$ and $c = s_u$	$\bar{c}, \bar{\phi}$ analysis permits check during construction using actual pore pressures
Long-term stability	$\bar{c}, \bar{\phi}$ analysis with pore pressures given by equilibrium ground-water conditions	Stability depends on amount of water-table rise and pore-pressure increase

Source: After Lambe, T. W., and Whitman, R. V. 1969. *Soil Mechanics*. John Wiley & Sons, New York.

available along the sliding surface to the total shearing stresses required to maintain limiting equilibrium, given as

$$\mathrm{FS} = \frac{cL + N\tan\phi}{c_m L + N\tan\phi_m}$$

where L is the length of the failure surface, N is the normal force on the assumed failure surface, c_m is the mobilized cohesion at equilibrium, ϕ_m is the mobilized friction angle at equilibrium, and $c/c_m = \phi/\phi_m$.

21.2 Factors to Consider

Selection of the proper method to be applied to the analysis of a slope problem requires consideration of a number of factors. Specific details can be found in the referenced works.

- Type of slope to be analyzed, such as natural or cut slope in soil [Bjerrum, 1973; Patton and Hendron, 1974; Brand, 1982; Leonards, 1979, 1982] or rock [Deere, 1976], earth-dam embankments [Lowe, 1967], embankments over soft ground [Chirapuntu and Duncan, 1976; Ladd, 1991], or sidehill fills.

TABLE 21.3 Minimum Values for FS for Earth and Rock-fill Dams[a,b]

Case	Design Conditions	FS_{min}	Shear Strength[c]	Remarks
I	End of construction	1.3[d]	*Q* or *S*[e]	Upstream and downstream slopes
II	Sudden drawdown from maximum pool	1.0[f]	*R*, *S*	Upstream slope only, use composite envelope
III	Sudden drawdown from spillway crest	1.2[f]	R, S	Upstream slope only, use composite envelope
IV	Partial pool with steady seepage	1.5	$(R + S)/2$ for $R < S$	Upstream slope only, use intermediate envelope
V	Steady seepage with maximum pool	1.5	$(R + S)/2$ for $R < S$	Downstream slope only, use intermediate envelope
VI	Earthquake (Cases I, IV, and V with seismic loading)	1.0	—[g]	Upstream and downstream slopes

[a] *Source:* From Wilson, S. D., and Marsal, R. J. 1979. *Current Trends in Design and Construction of Embankment Dams*. ASCE, New York.
[b] Not applicable to embankments on clay shale foundations; higher FS values should be used for these conditions.
[c] Q = quick (unconsolidated-undrained test), S = slow (consolidated-drained test), and R = intermediate (consolidated-undrained test).
[d] For embankments more than 50 ft high over relatively weak foundation use $FS_{min} = 1.4$.
[e] In zones where no excess pore-water pressures are anticipated use S strength.
[f] FS should not be less than 1.5 when drawdown rate and pore-water pressures developed from flow nets are used in stability analysis
[g] Use shear strength for case analyzed without earthquake. (Values for FS are based on pseudostatic approach.)

- Location, orientation, and shape of a potential or existing failure surface which is controlled by material type and structural features. The shape of the rupture zone can have single or multiple surfaces, and can be composed of single or multiple wedges. Failure surfaces can be planar, cylindrical or log-spiral, or irregular.
- Material distribution (stratigraphy) within and beneath the slope, divided generally into homogeneous zones wherein the properties are more or less similar in all directions, and nonhomogeneous zones wherein soils are stratified and rock masses contain major discontinuities.
- Material types and representative shear strength parameters; angle of internal friction ϕ, cohesion c, residual strength ϕ_r, and undrained strength s_u (see Table 21.1).
- Drainage conditions; appropriateness for either drained or undrained analysis, which depends on the relative rates of construction and pore pressure dissipation. Often considered as short-term (during construction) or long-term (postconstruction or natural slope) conditions; that is, total versus effective stress analysis (see Table 21.2).
- Distribution of piezometric levels (pore- or cleft-water pressures) along the potential failure surface and an estimate of the maximum value that may prevail.
- Potential earthquake loadings, which are a transient factor.

Hunt [1984, 1986] provides a detailed discussion of these necessary considerations for the various slope types.

21.3 Analytical Approaches

Most slope stability analyses are computationally intensive. Many computer programs have been developed to perform stability analyses on personal computers. Commonly used packages include PCSTABL (Purdue University) and UTEXAS3 (University of Texas). Data is input for the problem geometry, material stratigraphy, and the phreatic surface based on a coordinate system, and material properties are then entered. The programs search for the potential failure surface that produces the lowest value for FS. These programs are used routinely in virtually all geotechnical consulting offices. The following discussion is intended to present the salient features of the assumptions and analytical approach used to perform the stability analysis.

General

Failure, sudden or gradual, results when the mobilized stresses in a slope or its foundation equal the available strength. Limit equilibrium analysis is the basis for most methods available for slope stability evaluations. Consideration is given to a free body of the soil mass bounded by the slope and an assumed "slip" or failure surface. The known or assumed forces acting on the body and the shearing resistance required for stability are estimated. Most practical problems are statically indeterminate and require simplifying assumptions regarding the position and direction of forces to render the problem determinate.

The primary assumption of the limit equilibrium method is that the assumed strength can be mobilized throughout the length of the failure surface simultaneously. Strain compatibility is not considered. This assumption is applicable for stress–strain conditions that can be modeled as perfectly plastic at the failure strength. When soils have some post-peak strength reduction (as with most natural soil), engineering judgment is required to select appropriate strength parameters and safety factors.

Two common free-body assumptions are illustrated in Fig. 21.2. In the cylindrical form shown in Fig. 21.2(a), the mass weight W acting through lever arm E produces a driving moment. This moment is resisted by strength S mobilized along the failure surface of length L that acts through a lever arm R. In the simple wedge [Fig. 21.2(b)], strength S acting along the planar surface of length L resists the driving forces resulting from gravity acting on weight W. A plane strain condition is assumed in most analytical methods currently in use so the potential failure mass is analyzed per unit width. Resistance that would be generated at the lateral extremities of the failure zone are considered insignificant compared to the area of the potential failure surface. When three-dimensional analyses are performed, FS(3-D) > FS(2-D) [Duncan, 1992] for most cases; therefore, two-dimensional analysis normally is conservative.

Common methods of analysis consider a system of forces. The soil mass is divided into a system of "slices," "blocks," or "wedges," and force and/or moment equilibrium conditions are evaluated for the individual components of the soil mass. The soil strength is uniformly adjusted by a scaling factor until the system is in a state of equilibrium. The actual soil strength divided by the strength required to satisfy equilibrium is defined as the factor of safety.

Slices as Free Bodies

In modern force systems the sliding mass is divided into "slices" as shown in Fig. 21.3(a), which illustrates a "finite" slope with a circular failure surface. The forces that act on a slice [Fig. 21.3(b)] include the material weight W, normal force N, and shear force T distributed along an assumed failure surface; water pressure force U; water pressure V acting in a tension crack; and forces E_r, X_r, E_L, and X_L acting along

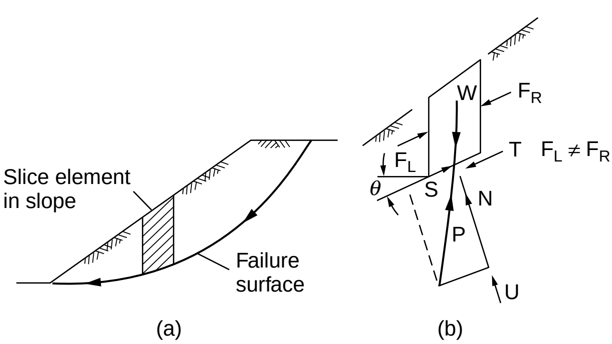


FIGURE 21.3 Finite slope with cylindrical failure surface (a) and forces on an element (b).

the sides of the slice. A discussion of the treatment of tension cracks can be found in Tschebotarioff [1973] and Spencer [1967]. Earthquakes impose dynamic forces in the form of the acceleration of gravity acting on the mass component.

Planar Surfaces, Blocks, and Wedges

Simple planar failures involve a single surface which can define a wedge in soil or rock [Fig. 21.2(b)], a sliding block, or a wedge in rock with a tension crack. The force system is similar to that given in Fig. 21.3, except that the side forces are neglected. Complex planar failures involve a number of planes dividing the mass into two or more blocks, and in addition to the force system for the single block, these solutions provide for interblock forces.

Infinite Slope

The infinite slope pertains where the depth to a planar failure surface is small compared to its length, which is considered as unlimited. Such conditions are found in slopes composed of the following:

- Cohesionless materials, such as clean sands
- Cohesive soils, such as residuum or colluvium, over a sloping rock surface at shallow depth
- OC fissured clays or clay shales with a uniformly deep, weathered zone
- Large slabs of sloping rock layers underlain by a weakness plane

Circular Failure Surfaces

General

In rotational slide failures, methods are available to analyze a circular or log-spiral failure surface, or a surface of any general shape. The location of the critical failure surface is found by determining the lowest value of safety factor obtained from a large number of assumed failure surface positions.

Slice Methods

Common to all slice methods is the assumption that the assumed soil mass and failure surface can be divided into a finite number of slices. Equilibrium conditions are considered for all slices. The problem is strongly indeterminate, requiring several basic assumptions regarding the location of application or resultant directions of applied forces.

The slice methods can be divided into two groups: nonrigorous and rigorous. Nonrigorous methods satisfy either force or moment equilibrium, whereas rigorous methods satisfy both force and moment equilibrium. The factor of safety estimated from rigorous methods is relatively insensitive to the assumptions made to obtain determinacy [Duncan, 1992; Espinoza et al., 1992, 1994]. However, nonrigorous solutions can produce significantly different estimates of safety depending on the assumptions made. In general, a nonrigorous solution satisfying only moment equilibrium is superior to one satisfying only force equilibrium and will provide solutions close to a rigorous method.

Ordinary Method of Slices

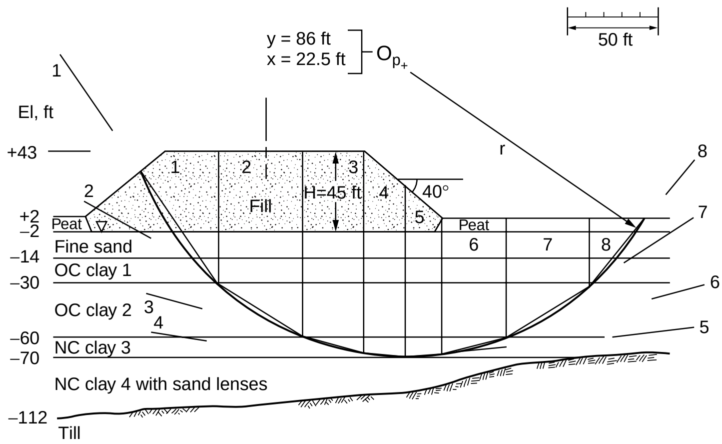
The ordinary method of slices, also known as the Swedish method, was developed by Fellenius [1936] to analyze failures in homogeneous clays occurring along Swedish railways in the 1920s. The solution is a trial-and-error technique that locates the critical failure surface, or that circle with the lowest value for FS.

The ordinary method is not a rigorous solution because the shear and normal stresses and pore-water pressures acting on the sides of the slice are not considered. In general the results are conservative. In slopes with low ϕ angles and moderate inclinations, FS may be 10 to 15% below the range of the more exact solutions; with high ϕ and slope inclination, FS can be underestimated by as much as 60%.

For the $\phi = 0$ case, normal stresses do not influence strength, and the ordinary method provides results similar to rigorous methods [Johnson, 1974]. An example analysis using the ordinary method for the $\phi = 0$ case as applied to an embankment over soft ground is given in Fig. 21.4.

Note: For s_u (fill)
$s = \bar{\sigma} \tan \phi = 22.5 \times 0.119 \times \tan 25 = 1.2$ ksf

Soil properties		
Soil	γ, kips/ft^3	Su, kips/ft^2
Fill	0.119	1.2
Fine sand	0.119	1.2
Clay 1	0.119	1.2
Clay 2	0.117	0.8
Clay 3	0.117	0.6
Clay 4	0.123	0.8



Slice	s_u, ksf	ΔL, ft	$S_u \Delta L$	θ, degrees	W, kips*	$W \sin\theta$
1	1.2	76	91.2	56	203.39	168.62
2	0.8	58	46.4	32	359.73	190.63
3	0.6	34	20.4	17	399.74	116.87
4	0.6	24	14.4	5	187.15	16.31
5	0.6	21	12.6	−5	117.60	−10.25
6	0.6	39	23.4	−14	128.46	−31.08
7	0.8	58	46.4	−32	113.40	−60.09
8	1.2	42	50.4	−51	30.24	−23.50
L = 352	Σ = 305.2	Σ = 367.5				

$$FS = \frac{\Sigma S_u \Delta L}{\Sigma W \sin\theta}$$

$$FS = \frac{305.2}{367.5} = 0.83$$

FIGURE 21.4 Embankment over soft clay: stability analysis for $\phi = 0$ case using ordinary method of slices. (*Source:* Hunt, R. E. 1986. *Geotechnical Engineering Techniques and Practices*. McGraw-Hill, New York. Reprinted with permission of McGraw-Hill Book Co.)

A counterberm added to increase stability (Fig. 21.5) must be of adequate width to cause the critical circle to pass beyond the toe with an acceptable value for FS.

Bishop Slice Methods

The rigorous Bishop method [Bishop, 1955] considers the complete system of forces acting on a slice, as shown in Fig. 21.6. In addition to N_i¢ U_i, and T_i, included along each side of the slice are shear stresses (x_i with width b_i), effective normal stresses (E_i), and pore-water pressures (U_L and U_r). In analysis, distributions of ($x_i - x_{i+1}$) are found by successive approximation until a number of equilibrium conditions are satisfied. Computations are considerable even with a computer.

The modified (simplified) Bishop method [Bishop, 1955; Janbu et al., 1956] is a simplification of the rigorous method. In the modified method it is assumed that the total influence of the tangential forces on the slice sides is small enough to be neglected.

Other Slice Methods

A number of other methods have been developed that differ in the statics employed to determine FS and the assumptions used to render the problem determinate. Included are Spencer's method [Spencer, 1967], Janbu's rigorous and simplified methods [Janbu et al., 1956], and the Morganstern–Price method

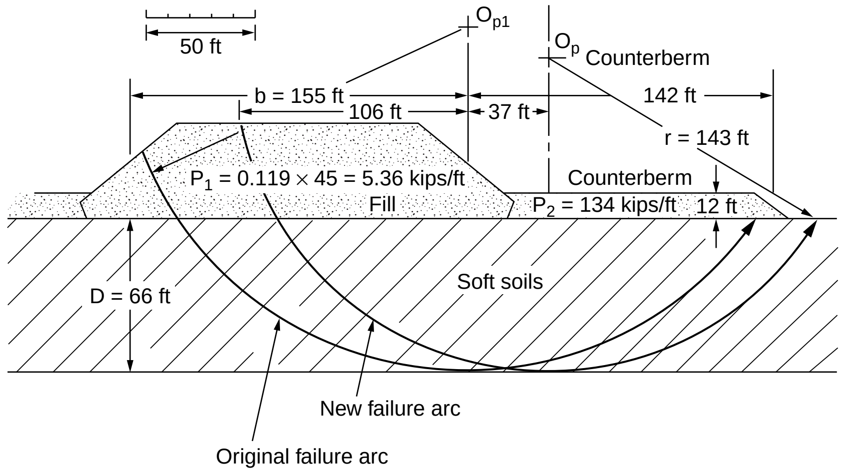


FIGURE 21.5 Counterberm to provide stability for embankment shown in Fig. 21.3. FS = 1.2. (*Source:* Hunt, R. E. 1986. *Geotechnical Engineering Techniques and Practices*. McGraw-Hill, New York. Reprinted with permission of McGraw-Hill Book Co.)

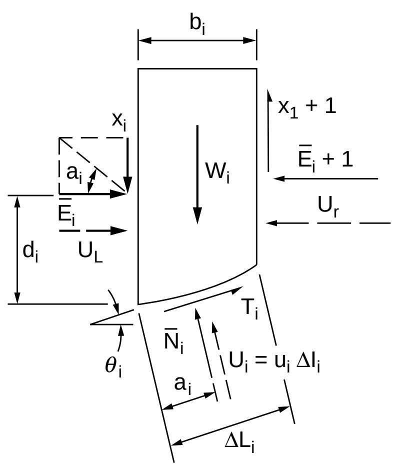


FIGURE 21.6 Complete force system acting on a slice. (*Source:* Lambe, T. W., and Whitman, R. V. 1969. *Soil Mechanics*. John Wiley & Sons, New York. Used with permission of John Wiley & Sons, Inc.)

[Morganstern and Price, 1965]. Espinoza et al. [1992, 1994] present a general framework to evaluate all limit equilibrium methods of stability analysis and illustrate the variability among methods for circular and irregular failure surfaces.

Chart Solutions for Homogeneous Slopes

Various chart solutions have been developed for simple homogeneous slopes, including Taylor [1937, 1948], Janbu [1968], Hunter and Schuster [1968], and Cousins [1978]. They are useful for preliminary analysis of the f = 0 case, and discussion and examples can be found in NAVFAC [1982] and Duncan et al. [1987].

Irregular and Planar Failure Surfaces

Geological conditions in many slopes are not amenable to circular failures, particularly when the potential failure surface is shallow relative to its length. Several methods are suitable for analyses of these conditions,

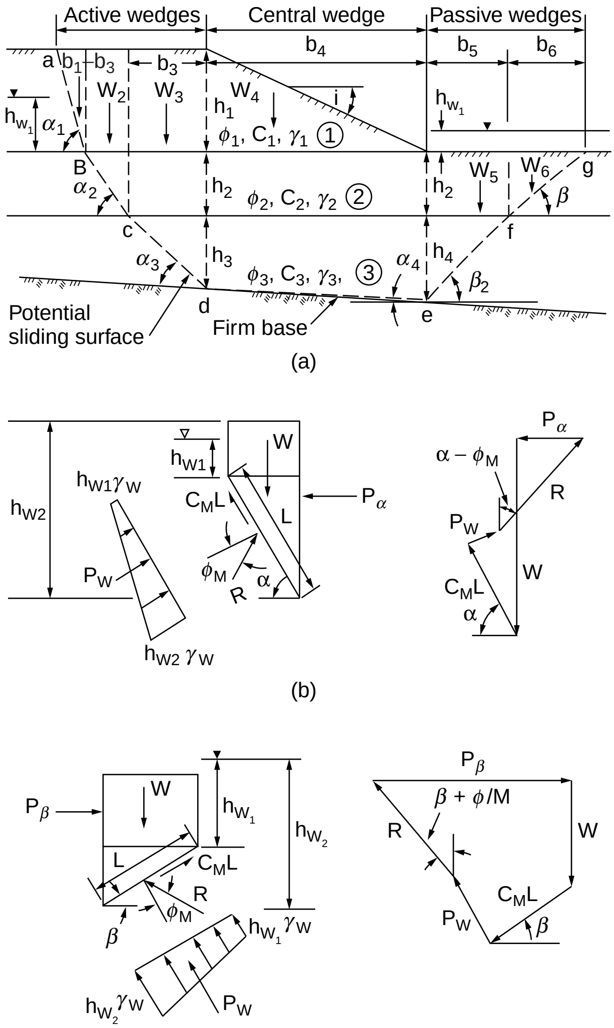


FIGURE 21.7 Stability analysis of translational failure (a). The resultant horizontal force for a wedge sliding along surface *abcde* is P_a (b), and for sliding along surface *efg* is P_b (c). (*Source:* NAVFAC. 1982. *Design Manual, Soil Mechanics, Foundations and Earth Structures, DM-7.* Naval Facilities Engineering Command, Alexandria, VA.)

including Spencer's method [Spencer, 1967], the Morganstern–Price method [Morganstern and Price, 1965], and Janbu's method [Janbu et al., 1956; Janbu, 1973].

In many natural situations the failure surface is planar and can be approximated by one or more straight lines which divide the mass into wedges or blocks. Solutions for one-, two-, and three-block problems are available in many sources, including Hunt [1986], Huang [1983], and NAVFAC [1982]. A detailed discussion of the analysis of blocks as applied to rock slopes is given in Hoek and Bray [1977].

The translation failure method [NAVFAC, 1982] is based on earth pressures and is suitable for multiple blocks, although interwedge forces are ignored. It is useful where soil conditions consist of several masses with different parameters such as in the case illustrated in Fig. 21.7.

Three-dimensional or tetrahedral wedge failures are common in open-pit mines on heights of one or two benches (60 to 100 ft) but become progressively less prevalent as slope height increases. Failure seems to be associated with weakness planes that are of the same order of size as the slope height involved, with

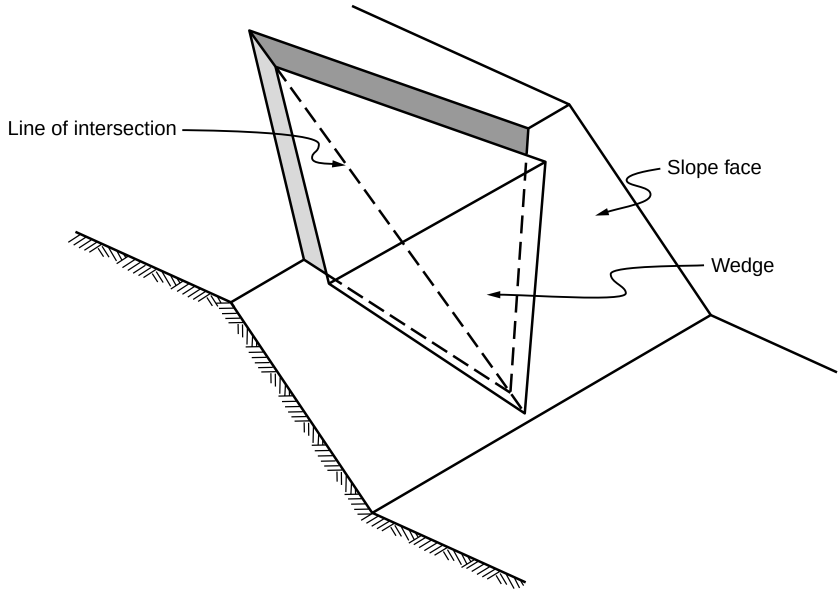


FIGURE 21.8 Geometry of a triangular wedge failure. (*Source:* Hoek, E., and Bray, J. W. 1977. *Rock Slope Engineering,* 2nd ed. The Institute of Mining and Metallurgy, London.)

the planes representing three or more intersecting joint sets. The “free blocks” approach tetrahedrons in shape, as shown in Fig. 21.8. A discussion of the analysis of the tetrahedral wedge can be found in Hoek and Bray [1977].

Earthquake Forces

Earthquake forces include cyclic loads which decrease the stability of a slope by increasing shear stresses, pore air and water pressures, and decreasing soil strength. In the extreme case, increases in pore pressure can lead to liquefaction. Sensitive clays and loose fine-grained granular soils above or below groundwater level (GWL), and metastable soils such as loess even when dry, are very susceptible to failure during cyclic loading. The presence of even a thin layer of saturated fine-grained soil, such as silt or clayey silt, can lead quickly to instability in any slope. Embankments over fine-grained saturated soils are particularly susceptible to failure, especially in areas where lateral restraint is limited.

Natural slopes composed of low-sensitivity clay, dense granular soils above or below GWL, or loose coarse-grained soils below GWL generally are stable even during strong ground shaking. Earth-dam embankments can withstand moderate to strong shaking when well-built to modern standards. The greatest risk of damage or failure lies with dams constructed of saturated fine-grained cohesionless materials. The general effect of ground shaking on embankments is slope bulging and crest settlement.

Pseudostatic Analysis

In the conventional approach, stability is determined as for static loading conditions and the effects of an earthquake are accounted for by including an equivalent horizontal force acting on the mass. The horizontal force, as shown in Fig. 21.9, is expressed as the product of the weight and a seismic coefficient k, which is related to induced accelerations. The effects on pore pressure are not considered, and a decrease in soil strength is accounted for only indirectly. Hall and Newmark [1977] developed design accelerations for horizontal ground motion for slope stability studies as related to magnitude, as given in Table 21.3. Augello et al. [1994] provide additional guidance for selecting appropriate design accelerations.

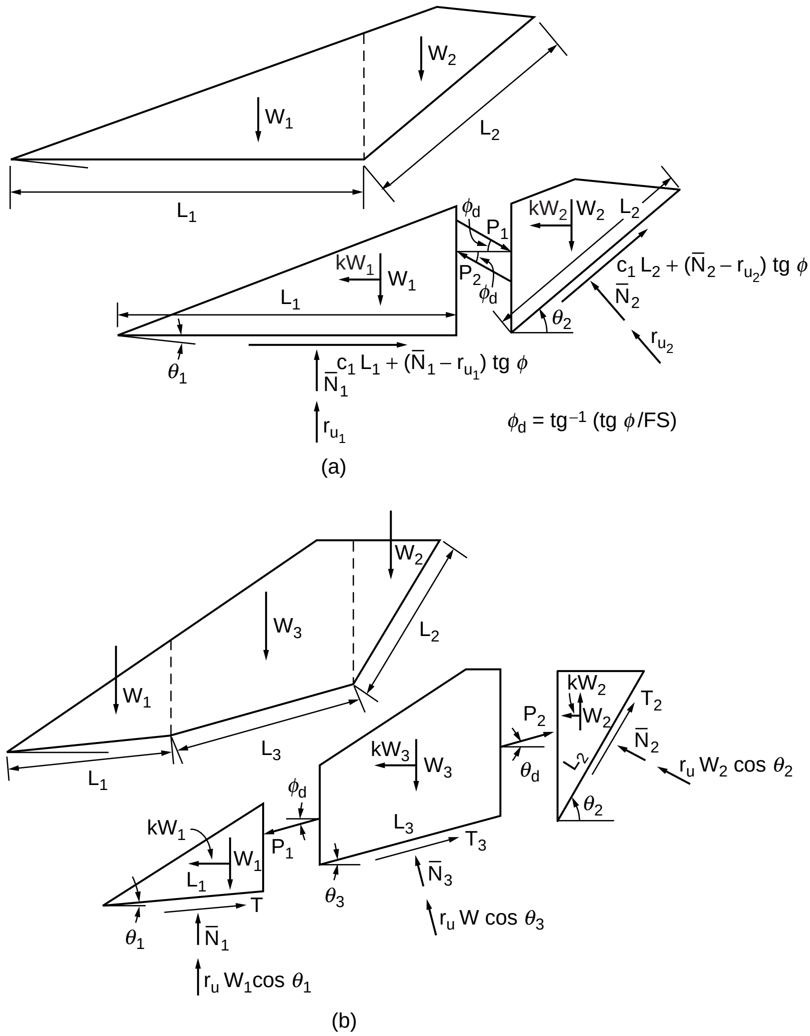


FIGURE 21.9 Complex wedge systems: (a) free-body diagrams for two blocks and (b) free-body diagrams for three blocks. (*Source:* After Huang, Y. H. 1983. *Stability Analysis of Earth Slopes.* Van Nostrand-Reinhold, New York.)

Time–History Dependence

Pseudostatic analysis is based on peak accelerations producing dynamic inertial forces. They may be sufficiently large to drop FS below unity for brief intervals during which displacements occur, but movement ceases when accelerations drop. The time element of the dynamic loadings is extremely significant and is not typically considered in analysis. Augello et al. [1995] and Bray et al. [1995] evaluate the time–history-dependent movements associated with stability of landfill slopes.

If dynamic loads are applied in cycles with small periods, but relatively long duration, soil will not have time to drain between loadings, pore pressures will continue to increase, and failure by liquefaction may occur. The Alaskan event of 1964 lasted almost three minutes, causing disastrous slides at Turnagain Heights where previous earthquakes of equal magnitude but shorter duration had caused little damage.

Dynamic Analysis

Dynamic analysis of embankments is performed employing finite element methods [Newmark, 1965; Seed, 1966; Seed et al., 1975; Byrne, 1991; Finn, 1993]. Embankment response to base excitation is evaluated and the dynamic stresses included in representative elements of the embankment are computed incorporating nonlinear dynamic material properties by using strain-dependent shear modulus and damping values. Recent work includes the generation of excess pore-water pressure during dynamic loading and the onset of liquefaction. Both the overall deformations and the stability of the embankment section are evaluated.

21.4 Treatments to Improve Stability

General Concepts

Selection Basis

Slope treatments can be placed in one of two broad categories:

- Preventive treatments applied to stable, but potentially unstable natural slopes, slopes to be cut, sidehill fills to be placed, or embankments to be constructed
- Remedial or corrective treatments applied to existing unstable, moving slopes, or to failed slopes

The slope treatment selected is a function of the degree of the hazard and the risk to the public. In natural slopes these factors are very much related to the form of slope failure (fall, slump, avalanche, or flow), the identification of which requires evaluation and prediction by an experienced engineering geologist.

Rating the Hazard and the Risk

Hazard degree relates to the potential failure itself in terms of its possible magnitude and probability of occurrence. Magnitudes can range from a small displacement and material volume, as is common in slump slides, to a large displacement and material volume, such as in a massive debris avalanche. Probability can range from certain to remote. Risk degree relates to the consequences of failure, such as a small volume of material covering portions of a roadway but not endangering lives, to the high risk from the failure of an earth dam resulting in the loss of many lives and much property damage. Safe but economical construction is always the desired result, but the acceptable degree of safety varies with the degree of hazard and risk. An example of a rating system for hazard and risk is given in Hunt [1984].

Treatment Options

Avoid the High-Risk Hazard

There are natural conditions where slope failure is essentially unpredictable and not preventable by reasonable means and the consequences are potentially disastrous. It is best to avoid construction in mountainous terrain subject to massive planar slides or avalanches, slopes in tropical climates subject to debris avalanches, or slopes subject to liquefaction and flows.

Accept the Failure Hazard

In some cases, low to moderate hazards may be acceptable because postfailure cleanup is less costly than some stabilization treatment. Examples are partial temporary closure of a roadway, which is often the approach in underdeveloped countries, or a slide in an open-pit mine where failure is predictable but prevention is considered uneconomical.

In many cases, failures are self-correcting, and eventually a slope may reach a stable condition or work back to where failures do not affect construction. An innovation being used in southern California to protect homes against debris slides, avalanches, and flows is the "A" wall [Hollingsworth and Kovacs, 1981] illustrated in Fig. 21.10. The purpose of the wall is to deflect moving earth masses away from the building.

Eliminate or Reduce the Hazard

Where failure is essentially predictable and preventable, or is occurring or has occurred and is suitable for treatment, slope stabilization methods are applied. For low-to-moderate-risk conditions, the approach can be either to eliminate or to reduce the hazard, depending on comparative economics. For high-risk

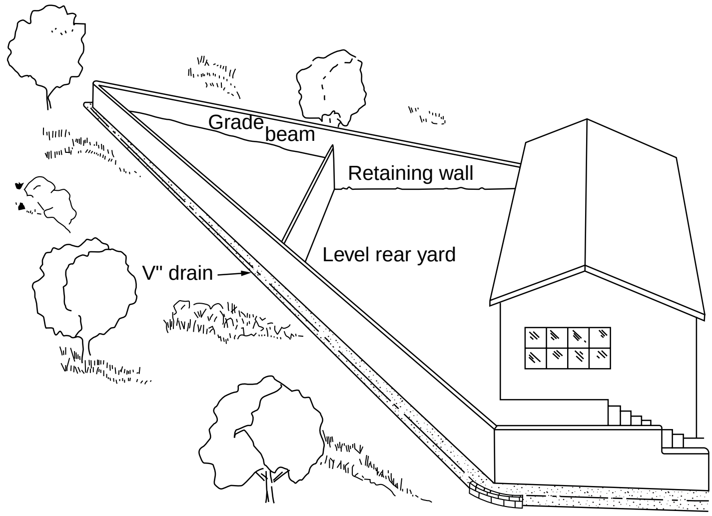


FIGURE 21.10 Typical A-wall layout to deflect debris slides, avalanches, and flows. (*Source:* Hollingsworth, R., and Kovacs, G. S. 1981. Soil slumps and debris flows: Prediction and protection. *Bul. Assoc. Eng. Geol.* 18(1):17–28.)

conditions the hazard should be eliminated. The generally acceptable safety factor determined by stability analysis can be taken as follows: low risk, FS = 1.3; significant risk, FS = 1.4; high risk, FS = 1.5.

Slope Stabilization

Slope stabilization methods may be placed in five general categories, as follows (Fig. 21.11):

1. Change slope geometry to decrease the driving forces or increase the resisting forces.
2. Control surface water to prevent erosion, and infiltration to reduce seepage forces.
3. Control internal seepage to reduce the driving forces.
4. Increase material strengths to increase resisting forces.
5. Provide retention to increase the resisting forces.

Where slopes are in the process of failing, the time factor must be considered. Time may not be available for carrying out measures that will eliminate the hazard; therefore, the hazard should be reduced and perhaps eliminated at a later date. The objective is to arrest the immediate movement. Where possible, treatments should be performed during the dry season, when movements will not affect remediation such as breaking horizontal drains.

Changing Slope Geometry

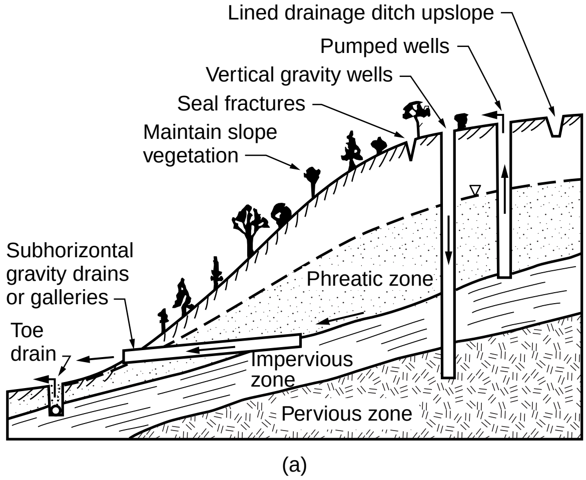
Natural Slope Inclinations

In many cases the natural slope represents the maximum long-term inclination, but there are cases where the slope is not stable and is moving. The inclination of existing slopes should be noted during field reconnaissance, since an increase in inclination by cutting may result in failure.

Cut Slopes in Rock

The objective of any cut slope is to form a stable inclination without retention. Controlled blasting procedures are required in rock masses to avoid excessive rock breakage resulting in extensive fracturing. Line drilling and presplitting during blasting operations minimize disturbance of the rock face.

Hard masses of igneous or metamorphic rocks, widely jointed, are commonly cut to 1H:4V (76°) [Terzaghi, 1962] [Fig. 21.12(a)]. Hard rock masses with joints, shears, or bedding representing major discontinuities dipping downslope are excavated along the dip of the discontinuity, as shown in



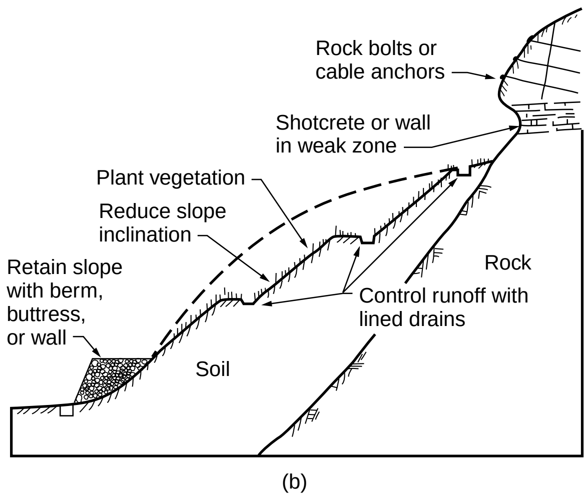


FIGURE 21.11 The general methods of slope stabilization: (a) control of seepage forces, (b) reducing the driving forces and increasing the resisting forces. (*Source:* Hunt, R. E. 1984. *Geotechnical Engineering Investigation Manual*. McGraw-Hill, New York.)

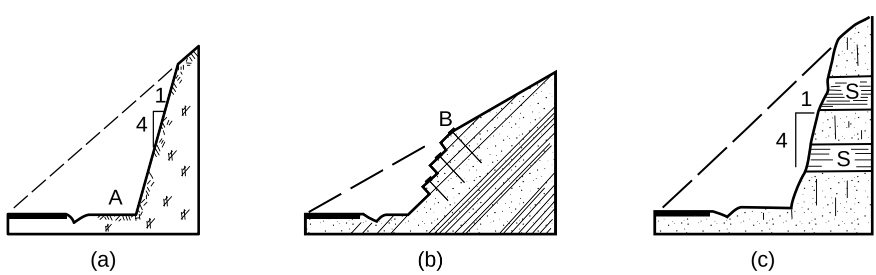


FIGURE 21.12 Typical cut slopes in hard rock for a roadway. Leave space at *A* for storage of block falls and topples and scale slope of loose blocks. (a) Closely jointed, competent rock. (b) Dipping beds; follow the dip unless excessively flat, in which case retain with bolts *B*. (c) Horizontally bedded sandstones and shales; apply gunite to the shales *S* if subjected to differential weathering. (*Source:* Hunt, R. E. 1986. *Geotechnical Engineering Techniques and Practices*. McGraw-Hill, New York. Reprinted with permission of McGraw-Hill Book Co.)

Fig. 21.12(b). All material should be removed until the original slope is intercepted. If the dip is too shallow for economical excavation, slabs can be retained with rock bolts (see "Retention," later in this chapter).

Hard sedimentary rocks with bedding dipping vertically or into the face, or horizontally interbedded hard sandstones and shales [Fig. 21.12(c)], are often cut to 1H:4V, but in the latter case, the shales should be protected from weathering with shotcrete or gunite to retard differential weathering. Weathered or closely jointed masses (except clay shales and dipping major discontinuities) require a reduction in inclination to between 1H:2V to 1H:1V (63° to 45°) depending on conditions, or require some form of retention. Clay shales, unless interbedded with sandstones, are often excavated to 6H:1V (9.5°).

Benching has been common practice in high rock cuts but there is disagreement among practitioners as to its value. Some consider benches to be undesirable because they provide takeoff points for falling blocks [Chassie and Goughnor, 1976]. To provide for storage they must be of adequate width. Block storage space should always be provided at the slope toe with adequate shoulder width to protect the roadway from falls and topples.

Cut Slopes in Soils

Most soil formations are commonly cut to an average inclination of 2H:1V (26°) but consideration must be given to seepage forces and other physical and environmental factors to determine if retention is required. Soil cuts are normally designed with benches, especially for cuts over 25 to 30 ft high. Because the slope angle between benches may be increased, benches reduce the amount of excavation necessary to achieve overall lower inclinations. Drains are installed as standard practice along the slopes and the benches to control runoff, as illustrated in Fig. 21.11(b) and Fig. 21.13.

In soil–rock transition (strong residual soils to weathered rock) such as in Fig. 21.13, cuts are often excavated to between 1H:2V to 1H:1V (63° to 45°) although potential failure along relict discontinuities must be considered. Where there is thin soil cover over rock the soil should be removed or retained as the condition normally will be unstable in cut. Figure 21.13 illustrates an ideal case, often misinterpreted from test boring data, and not present in the slope. In mountainous terrain all of the formations may be dipping, as shown on Fig. 21.14, a potentially very unstable condition. In such conditions, downslope seismic refraction surveys are valuable to define stratigraphy.

Failing Slopes

If a slope is failing and undergoing substantial movement, the removal of material from the head to reduce the driving forces can be the quickest method of arresting movement, and benching may be effective in the early stages. Placing material at the toe to form a counterberm increases the resisting forces. An alternative is to remove debris from the toe and permit failure to occur. Eventually the mass may naturally attain a stable inclination.

Changing slope geometry to achieve stability once failure has begun usually requires either the removal of very large volumes or the implementation of other methods. Space is seldom available in critical situations to permit placement of material at the toe, since very large volumes normally are required. As will be discussed, subhorizontal drains are often a very effective intermediate solution.

Surface Water Control

Purpose

Surface water is controlled to eliminate or reduce infiltration and to provide erosion protection. External measures are generally effective, however, only if the slope is stable and there is no internal source of water to cause excessive seepage forces.

Infiltration and Erosion Protection

Planting the slope with thick, fast-growing native vegetation strengthens the shallow soils with root systems and discourages desiccation, which causes fissuring. Not all vegetation works equally well, and

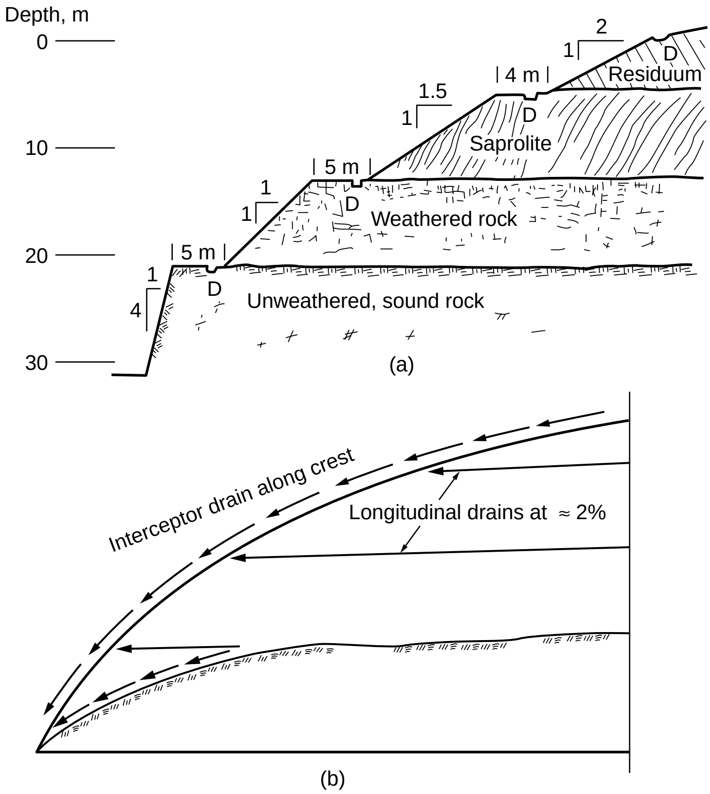


FIGURE 21.13 Benching a cut slope. (a) Typical section with drains located at *D*. Slope in weathered rock varies with rock quality; in saprolite, varies with orientation of foliations. Attempt is made to place benches on contact between material types. (b) General scheme to control runoff and drainage shown in face view. (*Source:* Hunt, R. E. 1986. *Geotechnical Engineering Techniques and Practices*. McGraw-Hill, New York. Reprinted with permission of McGraw-Hill Book Co.)

selection requires experience. In the Los Angeles area of California, for example, Algerian ivy has been found to be quite effective in stabilizing steep slopes [Sunset, 1978]. Newly cut slopes should be immediately planted and seeded. Burlap bags or sprayed mulch helps increase growth rate and provide protection against erosion during early growth stages. In addition to plantings, erosion protection along the slope can be achieved with wattling bundles, as illustrated in Fig. 21.15.

Sealing cracks and fissures with asphalt or soil cement will reduce infiltration but will not stabilize a moving slope since the cracks will continue to open. Grading a moving area results in filling cracks with soil, which helps reduce infiltration.

Surface Drainage Systems

Cut slopes should be protected with interceptor drains installed along the crest of the cut, along benches, and along the toe (Figs. 21.11 and 21.13). On long cuts the interceptors are connected to downslope collectors [Fig. 21.13(b)]. All drains should be lined with nonerodible materials, free of cracks or other openings, and designed to direct all concentrated runoff to discharge offslope.

With failing slopes, installation of an interceptor along the crest beyond the head of the slide area will reduce runoff into the slide. But the interceptor is a temporary expedient, since in time it may break up and cease to function as the slide disturbance progresses upslope.

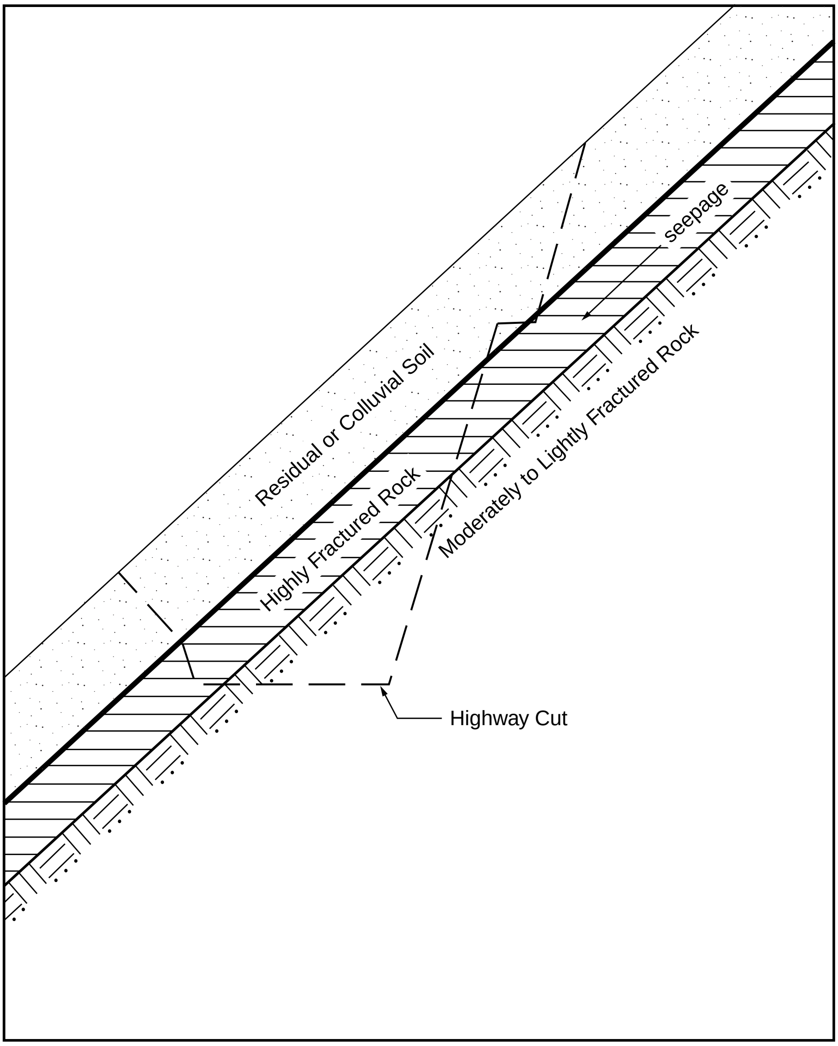


FIGURE 21.14 Slope conditions common along steep slopes in mountainous terrain.

Internal Seepage Control

General

Internal drainage systems are installed to lower the piezometric level below the potential or existing sliding surface. Selection of the drainage method is based on consideration of the geologic materials, structure, and groundwater conditions (static, perched, or artesian), and the location of the phreatic surface.

As the drains are installed, the piezometric head is monitored by piezometers and the efficiency of the drains is evaluated. The season of the year and the potential for increased flow during wet seasons must be considered, and if piezometric levels are observed to rise to dangerous values (as determined by stability analysis, or from monitoring slope movements), the installation of additional drains is required.

Cut Slopes

Systems to relieve seepage forces in cut slopes are seldom installed in practice, but they should be considered more frequently, since there are many conditions where they would aid significantly in maintaining stability.

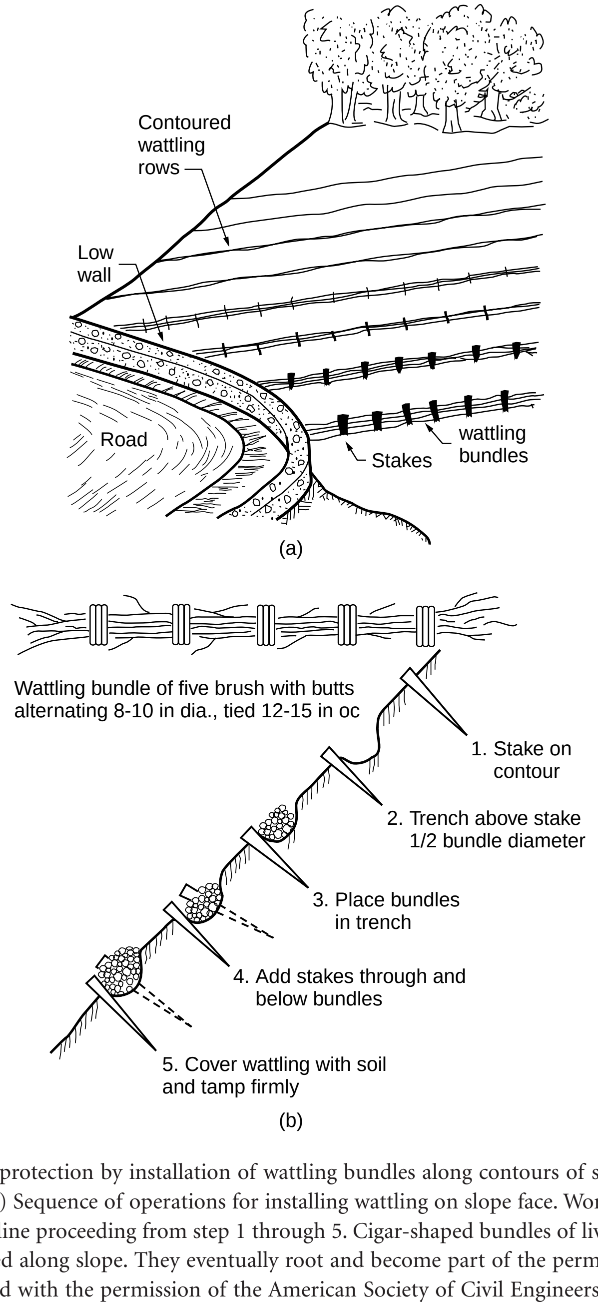


FIGURE 21.15 Erosion protection by installation of wattling bundles along contours of slope face. (a) Contoured wattling on slope face. (b) Sequence of operations for installing wattling on slope face. Work starts at bottom of cut or fill with each contour line proceeding from step 1 through 5. Cigar-shaped bundles of live brush of species which root are buried and staked along slope. They eventually root and become part of the permanent slope cover. (After Gray *et. al.*, 1980. Adapted with the permission of the American Society of Civil Engineers.)

Failing Slopes

The relief of seepage pressures is often the most expedient means of stabilizing a moving mass. The primary problem is that, as mass movement continues, the drains will be cut off and cease to function; therefore, it is often necessary to install the drains in stages over a period of time. Installation must be planned and performed with care, since the use of water during drilling could possibly trigger a total failure.

Methods

Deep wells have been used to stabilize many deep-seated slide masses, but they are costly since continuous or frequent pumping is required. Check valves normally are installed so that when the water level rises, pumping begins. Deep wells are most effective if installed in relatively free-draining material below the failing mass.

Vertical gravity drains are useful in perched water-table conditions where an impervious stratum overlies an open, free-draining stratum with a lower piezometric level. The drains permit seepage by gravity through the confining stratum and thus relieve hydrostatic pressures. Clay strata over granular soils, or clays or shales over open-jointed rock, offer favorable conditions for gravity drains where a perched water table exists.

Subhorizontal drains are one of the most effective methods to improve stability of a cut slope, or to stabilize a failing slope. Installed at a slight angle upslope to penetrate the phreatic zone and permit gravity flow, they usually consist of perforated pipe, 2 in. diameter or larger, forced into a predrilled hole of slightly larger diameter than the pipe. Horizontal drains have been installed to lengths of more than 300 ft. Spacing depends on the type of material being drained; fine-grained soils may require spacing as close as 10 to 30 ft, whereas for more permeable materials, 30 to 50 ft may suffice.

Drainage galleries are very effective for draining large moving masses but their installation is difficult and costly. They are used mostly in rock masses where roof support is less of a problem than in soils. Installed below the failure zone to be effective, they are often backfilled with stone. Vertical holes drilled into the galleries from above provide for drainage from the failure zone into the galleries.

Interceptor trench drains can be installed upslope to intercept groundwater flowing into a cut or sliding mass, but they must be sufficiently deep. Perforated pipe is laid in the trench bottom, embedded in sand, and covered with free-draining material, then sealed at the surface. Interceptor trench drains are generally not practical on steep, heavily vegetated slopes because installation of the drains and access roads requires stripping the vegetation, which will tend to decrease stability.

Relief trenches relieve pore pressures at the slope toe. They are relatively simple to install. Excavation should be made in sections and quickly backfilled with stone so as not to reduce the slope stability and possibly cause a total failure. Generally, relief trenches are most effective for small slump slides where high seepage forces in the toe area are the major cause of instability.

Electroosmosis has been used occasionally to stabilize silts and clayey silts, but the method is relatively costly, and not a permanent solution unless operation is maintained.

Increased Strength

Chemicals have sometimes been injected to increase soil strength. In a number of instances the injection of a quicklime slurry into predrilled holes has arrested slope movements as a result of the strength increase from chemical reaction with clays [Handy and Williams, 1967; Broms and Boman, 1979]. Strength increase in saltwater clays, however, was found to be low.

Resistance along an existing or potential failure surface can be increased with drilled piers [Oakland and Chameau, 1989; Lippomann, 1989], shear pins (reinforced concrete dowels), or stone columns. In the latter case the increased resistance is obtained from a significantly higher friction angle obtained in the stone along its width intercepting the failure surface.

Sidehill Fills

Failures

Construction of a sidehill embankment using slow-draining materials can be expected to block natural drainage and evaporation. As seepage pressures increase, particularly at the toe (as shown in Fig. 21.16), the embankment strains and concentric tension cracks form. The movements develop finally into a rotational failure. Fills placed on moderately steep to steep slopes of residual or colluvial soils, in particular,

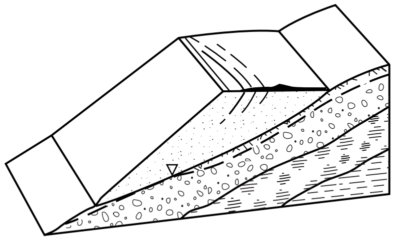


FIGURE 21.16 Early failure stage in sidehill fill as concentric cracks form in the pavement.

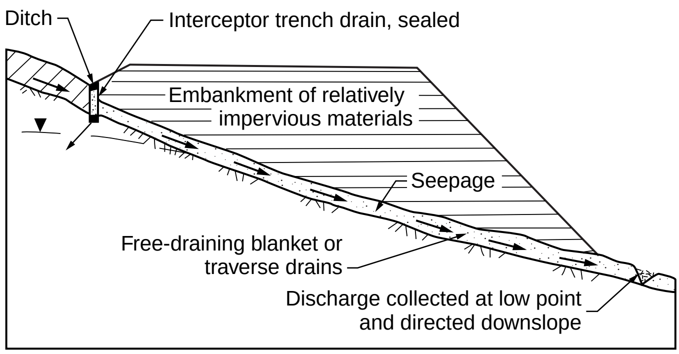


FIGURE 21.17 Proper drainage provisions for a sidehill fill *A* constructed of relatively impervious materials over relatively impervious natural soils *S* subject to surface creep. Upper soils are stripped and replaced with free-draining blanket *B*. Interceptor trench drain *C* installed and sealed with lined ditch *D*. Groundwater discharge collected at low point *E* and directed downslope. (*Source:* Hunt, R. E. 1986. *Geotechnical Engineering Techniques and Practices*. McGraw-Hill, New York. Reprinted with permission of McGraw-Hill Book Co.)

are prone to be unstable unless seepage is properly controlled, or the embankment is supported by a retaining structure.

Preventive Treatments

Interceptor trench drains should be installed along the upslope side of all sidehill fills as standard practice to intercept flow, as shown in Fig. 21.17. Perforated pipe is laid in the trench bottom, embedded in sand, covered by free-draining materials, and then sealed at the surface. Surface flow is collected in open drains and all discharge, including that from the trench drains, is directed away from the fill area.

Where anticipated flows are low to moderate, transverse drains extending downslope and connecting with the interceptor ditches upslope, parallel to the roadway, may provide adequate subfill drainage. Wherever either the fill or the natural soils are slow-draining, however, a free-draining blanket should be installed over the entire area between the fill and the natural slope materials to relieve seepage pressures from shallow groundwater conditions (Fig. 21.17). It is prudent to strip potentially unstable upper soils, which are often creeping on moderately steep to steep slopes, to a depth where stronger soils are encountered. Stepped excavations improve stability. Discharge should be collected at the low point of the fill and drained downslope in a manner that will provide erosion protection.

Retaining structures may be economical on steep slopes that continue for some distance beyond the fill, if stability is uncertain.

Corrective Treatments

If movement downslope has begun in a slow initial failure stage, subhorizontal drains may be adequate to stabilize the embankment if closely spaced. They should be installed during the dry season, if practical, since the use of water to drill holes during the wet season may accelerate total failure. An alternative is

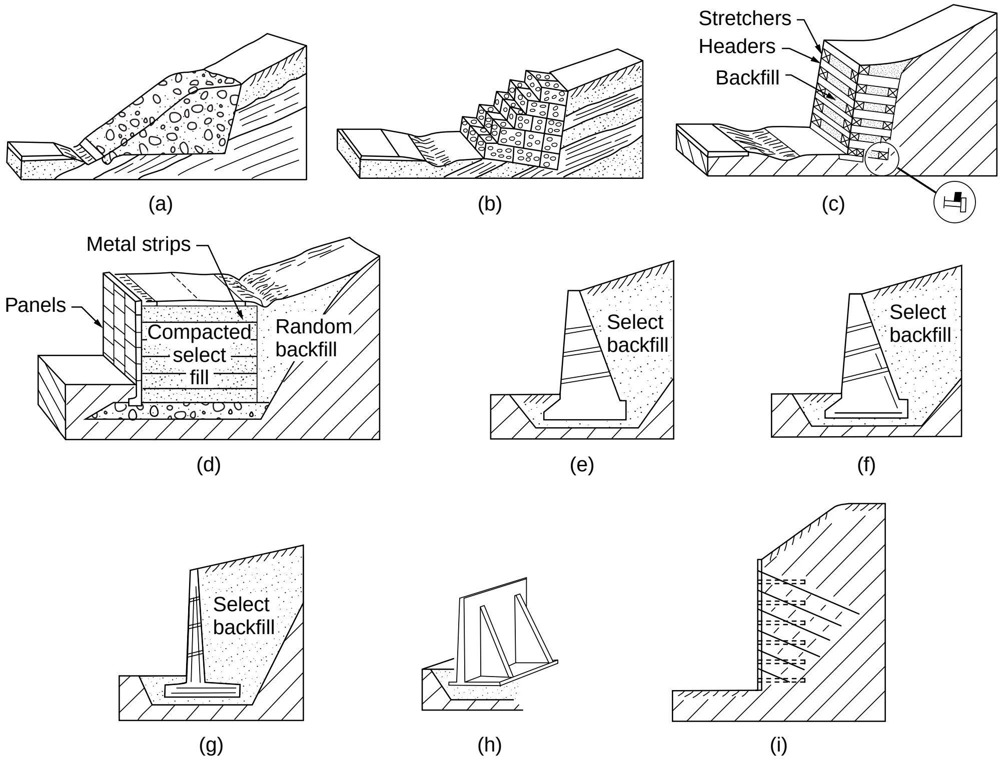


FIGURE 21.18 Various types of retaining walls: (a) rock-filled buttress; (b) gabion wall; (c) crib wall; (d) reinforced earth wall; (e) concrete gravity wall; (f) concrete-reinforced semigravity wall; (g) cantilever wall; (h) counterfort wall; (i) anchored curtain wall. (*Source:* Hunt, R. E. 1986. *Geotechnical Engineering Techniques and Practices*. McGraw-Hill, New York. Reprinted with permission of McGraw-Hill Book Co.)

to retain the fill with an anchored curtain wall (Fig. 21.18). After total failure, the most practical solutions are either reconstruction of the embankment with proper drainage, or retention with a wall.

Retention

Rock Slopes

Various methods of retaining hard rock slopes are illustrated in Fig. 21.19. They can be described briefly as follows.

- Concrete pedestals are used to support overhangs, where their removal is not practical because of danger to existing construction downslope [Fig. 21.19(a)].
- Rock bolts are used to reinforce jointed rock masses or slabs on a sloping surface [Fig. 21.19(b)]. Ordinary or temporary rock bolts, and fully grouted or permanent rock bolts are described by Lang [1972].
- Concrete straps and rock bolts are used to support loose or soft rock zones or to reduce the number of bolts [Fig. 21.19(c)].
- Cable anchors are used to reinforce thick rock masses [Fig. 21.19(d)]. The reinforcement of a single block by bolts or cables is shown in Fig. 21.20.

Shotcrete, when applied to rock slopes, usually consists of a wet-mix mortar with aggregate as large as 2 cm (3/4 in.) which is projected by air jet directly onto the slope face. The force of the jet compacts the mortar in place, bonding it to the rock, which first must be cleaned of loose particles and loose blocks.

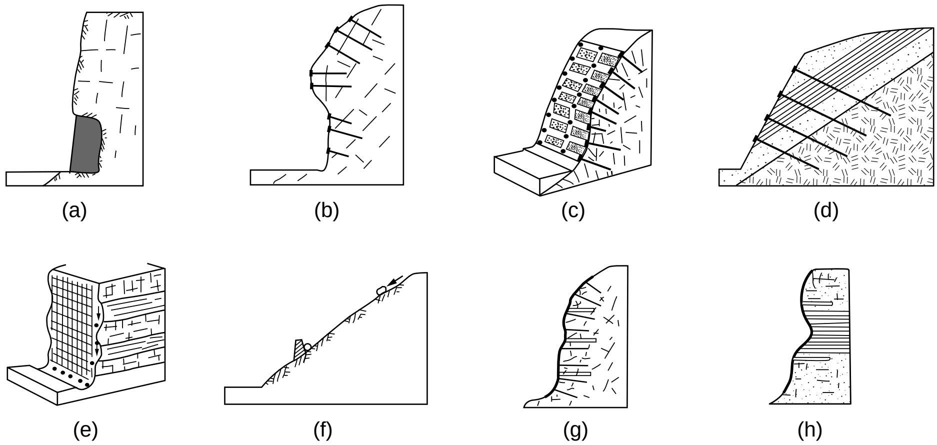


FIGURE 21.19 Various methods of retaining hard rock slopes: (a) concrete pedestals for overhangs; (b) rock bolts for jointed masses; (c) bolts and concrete straps for intensely jointed masses; (d) cable anchors to increase support depth; (e) wire mesh to constrain falls; (f) impact walls to deflect or contain rolling blocks; (g) shotcrete to reinforce loose rock, with bolts and drains; (h) shotcrete to retard weathering and slaking of shales. (*Source:* Hunt, R.E. 1984. *Geotechnical Engineering Investigation Manual.* McGraw-Hill, New York.)

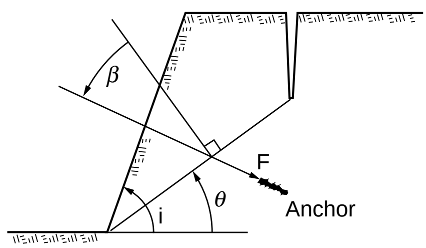


FIGURE 21.20 Definition of angle b for reinforcement of a single rock block with rock bolts. (*Source:* Hunt, R. E. 1986. *Geotechnical Engineering Techniques and Practices.* McGraw-Hill, New York. Reprinted with permission of McGraw-Hill Book Co.)

Application is in 3- to 4-in. layers, each of which is permitted to set before application of subsequent layers. Weep holes are installed to relieve seepage pressures behind the face. Since shotcrete acts as reinforcing and not as support, it is used often in conjunction with rock bolts. The tensile strength can be increased significantly by adding 25-mm-long wire fibers to the concrete mix.

Soil Slopes

Walls are used to retain earth slopes where space is not available for a flat enough slope or excessive volumes of excavation are required, or to obtain more positive stability under certain conditions. Except for anchored concrete curtain walls, wall types which require cutting into the slope for construction are seldom suitable for retention of a failing slope.

Various types of walls are illustrated in Fig. 21.18. They may be divided into four general classes, with some wall types included in more than one class: gravity walls, nongravity walls, rigid walls, and flexible walls.

Gravity walls provide slope retention by either their weight alone, or their weight combined with the weight of a soil mass acting on a portion of their base or by the weight of a composite system. They are free to move at the top, thereby mobilizing active earth pressure. Included are concrete gravity walls, cantilever walls, counterfort walls, rock-filled buttresses, gabion walls, crib walls, and reinforced earth walls. With the advent of geosynthetics there are many variations of “reinforced earth” [Koerner, 1993], and other innovations such as “soil nailing” [FHWA, 1993].

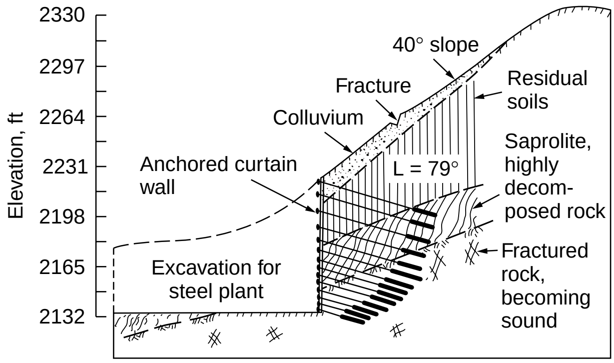


FIGURE 21.21 Section through a 25.5 m (85 ft) high anchored curtain wall constructed to retain a steep slope with a history of landslides. Joao Monlavade, M. G. Brazil. (*Source:* Hunt, R. E., and da Costa Nunes, A. J. 1978. Retaining walls: Taking it from the top. *Civ. Eng., ASCE,* May, 73–75.)

Nongravity walls are restrained at the top and not free to move. They include basement walls, some bridge abutments, and anchored concrete curtain walls. Anchored concrete curtain walls, such as the one illustrated in Fig. 21.21, can be constructed to substantial heights and have a very high retention capacity. As illustrated in Fig. 21.22, they are constructed from the top down by excavation of a series of benches into the slope and formation of a section of wall, retained by anchors, in each bench along the slope. Since the slope is thus retained completely during the wall construction, the system is particularly suited to potentially unstable or unstable slopes. A variation of the anchored curtain wall consists of anchored premolded concrete panels. The advantage of the system is that the wall easily conforms to the slope configuration.

Rigid walls include concrete walls, gravity and semigravity walls, cantilever walls, and counterfort walls. Anchored concrete curtain walls are considered as semirigid. Flexible walls include rock-filled buttresses, gabion walls, crib walls, reinforced earth walls, and anchored sheet-pile walls.

Embankments

Earth Dams

During design and construction of earth dams, stability is provided by controlled compaction of the embankment materials, adequate support by founding materials, and control of seepage through and beneath the embankment. Stable slope inclinations are related to the materials used to construct the embankment, and to the foundation materials. Relatively weak foundation materials either require removal by excavation or the flattening of embankment slopes.

Control of seepage through, beneath, and around the embankment is a critical aspect of design and construction. In addition to water loss in a reservoir, uncontrolled seepage can result in internal erosion of the embankment or high uplift pressures in the foundation, either of which can lead to complete failure.

Embankments over Soft Ground

Failures usually occur during or shortly after construction when embankments are raised over soft foundation soils. The major postconstruction concern is long-term settlements. In most situations it is desirable to avoid failure because the remolding of the soils that occurs significantly decreases their strength, worsening the situation. Therefore, embankments are constructed in stages. During each stage consolidation and strength increase occurs in the foundation soils, enabling the construction of subsequent stages [Ladd, 1991]. Vertical drains, such as wick drains, significantly increase the rate of consolidation and shorten the time interval between stages.

Geosynthetics are used to improve embankment strength. They may be placed on the soft ground prior to placing the first lift, and then subsequently placed within the embankment. Hird [1986] and Low et al. [1990] provide methods for assessing stability of reinforced embankments on soft ground. Construction of counterberms (Fig. 21.5) also improves stability.

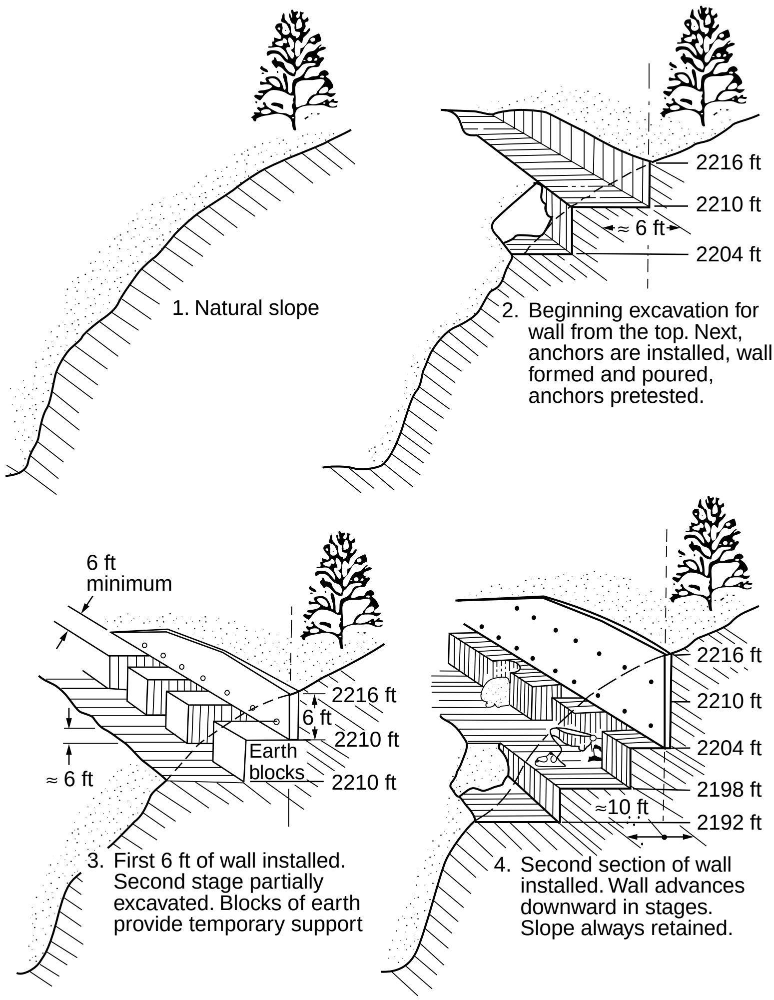


FIGURE 21.22 The "Brazilian method" of construction of an anchored curtain wall from the top down. (*Source:* Hunt, R. E., and da Costa Nunes, A. J. 1978. Retaining walls: Taking it from the top. *Civ. Eng., ASCE,* May, 73–75.)

21.5 Investigation and Monitoring

Exploration

Preliminary Phases

The objectives of the preliminary phases of investigation are to identify potential forms, magnitudes, and incidences of slope failures, and to plan an exploration program. The study scope includes collection of existing data, generation of new data through terrain analysis, and field reconnaissance. Data to be collected include site geology, topographic maps, and remotely sensed imagery, and historical information on regional and local slope failures, climatic conditions of precipitation and temperature, and seismicity.

Using the topographic maps and remotely sensed imagery (air-photo interpretation techniques) terrain analysis is performed to identify unstable and potentially unstable areas, and to establish preliminary conclusions regarding geologic conditions. For a large study area, a preliminary map is prepared on which is shown topography, drainage, active and ancient failures, and geology. The preliminary map is developed into a hazard map after field reconnaissance. At the project location, more detailed maps are prepared illustrating the foregoing items. The methodology identifies the significant features to be examined during field reconnaissance.

The site location is visited and notations are made regarding seepage points, vegetation, creep indications (tilted and bent tree trunks), tension cracks, failure scars, hummocky ground, natural slope inclinations, exposed geology, and prevailing and recent weather conditions.

From the data collected, preliminary evaluations are made regarding slope conditions and an exploration program is planned. The entire slope should be explored, not only the specific area of failure or area to be cut.

Explorations

Seismic refraction profiling is useful to determine the depth to sound rock and the probable groundwater table, and is most useful in differentiating between colluvial or residual soils and the fractured-rock zone. Surveys are made both longitudinal and transverse to the slope. They are particularly valuable on steep slopes with a deep weathering profile where test borings are time-consuming and costly.

Resistivity profiling may be performed to determine the depth to groundwater and to rock. Profiling is generally only applicable to depths of about 15 to 30 ft, but very useful in areas of difficult access. In the soft, sensitive clays of Sweden, the failure surface or potential failure surface is often located by resistivity measurements since the salt content, and therefore the resistivity, often changes suddenly at the slip surface [Broms, 1975].

Test borings are made to confirm the stratigraphy determined by the geophysical explorations, to recover samples of the various materials, and to provide holes for the installation of instrumentation. Borings should extend to adequate penetration below the depth of a cut, and below the depth of any potential failure surface. Sampling should be continuous through a potential or existing rupture zone, and in residual soils and rock masses care should be taken to identify slickensided surfaces. Groundwater conditions must be defined carefully and the static water table, perched, and artesian conditions noted. It is important to remember that the conditions existing at the time of investigation are not likely to be those during failure.

Evaluations

Evaluations are made of the safety factor against total failure on the basis of existing topographic conditions, then under conditions of the imposed cut or fill. For preliminary studies, shear strengths may be estimated from published data, or measured by laboratory or *in situ* testing. In the selection of the strength parameters, consideration is given to field conditions as well as to changes that may occur with time (reduction from weathering, leaching, solution). Other transient conditions such as weather and earthquakes also require consideration, especially if the safety factor for the entire slope is low and could go below unity with some environmental change.

Instrumentation and Monitoring

Purpose

Where movement is occurring, where safety factors against sliding are low, or where a major work would become endangered by a slope failure, instrumentation is required to monitor changing conditions and provide early warning of impending failure.

Slope–stability analysis is far from an exact science, regardless of the adequacy of the data available, and sometimes the provision for an absolutely safe slope is prohibitively costly.

In cut slopes, instrumentation monitors movements and changing stress conditions to provide early warning and permit invoking contingency plans for remedial measures when low safety factors are accepted in design. In unstable or moving slopes, instrumentation is installed to locate the failure surface and determine pore-water pressures for analysis, and to measure surface and subsurface movements, velocities, and accelerations which provide indications of impending total failure.

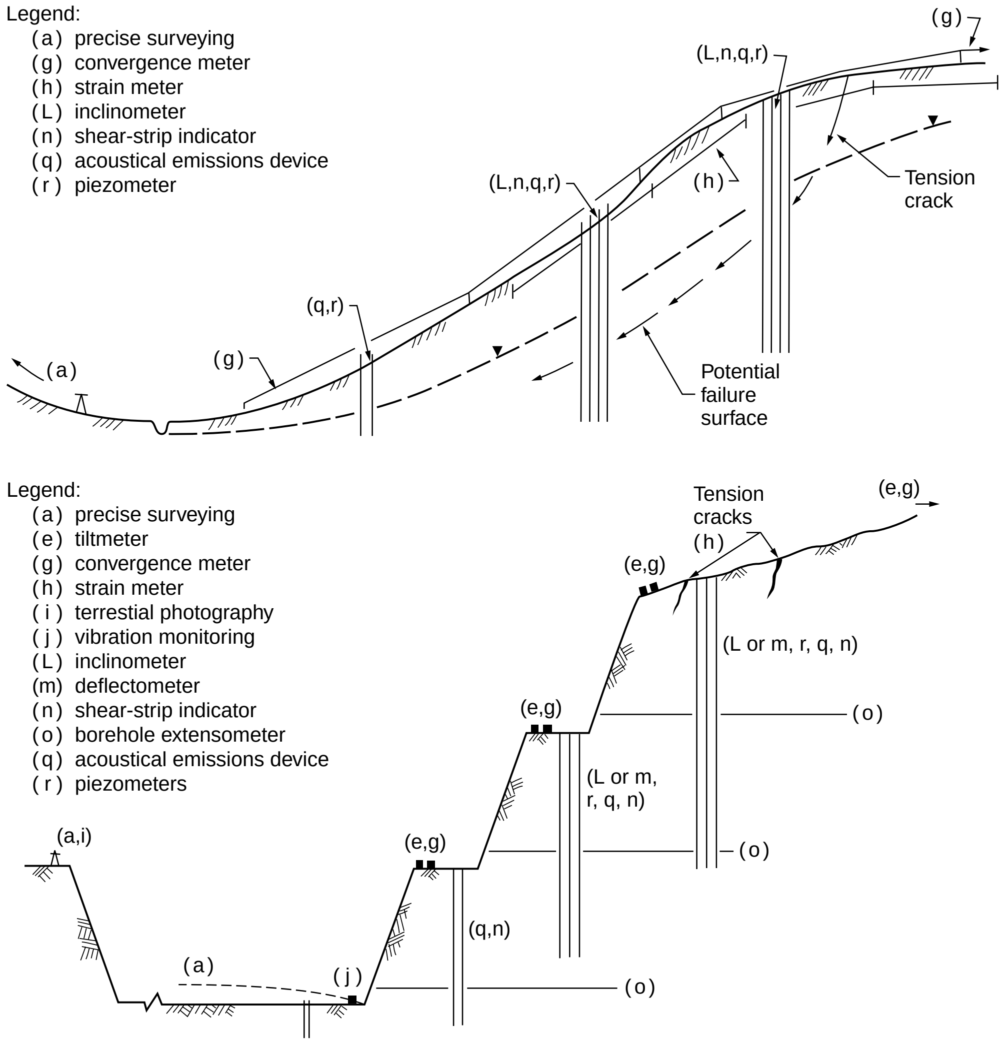


FIGURE 21.23 Slope instrumentation and monitoring: (a) potentially unstable soil slope and (b) rock cut. (*Source:* Hunt, R. E. 1984. *Geotechnical Engineering Investigation Manual.* McGraw-Hill, New York.)

Methods Summarized

Instrumentation is discussed in detail in Hunt [1984] and Dunnicliff [1988], and for slopes is illustrated in Fig. 21.23. Surface movements are monitored by survey nets, tiltmeters (on benches), convergence meters, surface extensometers, and terrestrial photography. Subsurface deformations are monitored with inclinometers, deflectometers, shear-strip indicators, steel wire and weights in boreholes, and the acoustical emissions device. Pore-water pressures are monitored with piezometers.

All instruments should be monitored periodically and the data plotted as it is obtained to show changing conditions. Movement accelerations are most significant.

References

Augello, A. J., Bray, J. D., Leonards, G. A., Repetto, P. C., and Byrne, R. J. 1995. Response of landfills to seismic loading. Proceedings, Geoenvironment 2000, *ASCE* (in press).

Bishop, A. W. 1955. The use of the slip circle in the stability analysis of earth slopes. *Geotechnique.* 5(1):7–17.

Bjerrum, L. 1973. Problems of soil mechanics and construction on soft clays and structurally unstable soils. In *Proc. 8th Int. Conf. Soil Mech. Found. Eng.* 3:111–159.

Brand, E. W. 1982. Analysis and design in residual soils. In *Proc. Conf. Eng. Constr. Tropical Soils. ASCE,* Honolulu, pp. 89–143.

Bray, J. D., Augello, A. J., Repetto, P. C., Leonards, G. A., and Byrne, R. J. 1995. Seismic analytical procedures for solid waste landfills. *J. Geotech. Eng., ASCE* (in press).

Broms, B. B. 1975. *Foundation Engineering Handbook,* H. F. Winterkorn and H. Y. Fang, editors, Van Nostrand Reinhold, New York, pp. 373–401.

Broms, B. B., and Boman, P. 1979. Lime columns — A new foundation method. *J. Geotech. Eng., ASCE.* 105(GT4):539–556.

Byrne, P. M. 1991. A cyclic shear-volume coupling and pore pressure model for sand. In *Proc. 2nd Int. Conf. Recent Adv. Geotech. Earth. Eng. Soil Dynamics*. Rolla, I:47–56.

Chassie, R. G., and Goughnor, R. D. 1976. States intensifying efforts to reduce highway landslides. *Civ. Eng.,* Apr., p. 65.

Chirapuntu, S., and Duncan, J. M. 1976. The role of fill strength in the stability of embankments on soft clay. University of California, Berkeley, NITS AD-A027-087.

Cousins, B. F. 1978. Stability charts for simple earth slopes. *J. Geotech. Eng. ASCE*. 104(GT5):267–279.

Deere, D. U. 1976. Dams on rock foundations — Some design questions. In *Proc. Rock Eng. Found. Slopes. ASCE,* New York, pp. 55–85.

Duncan, J. M. 1992. State-of-the-art: Static stability and deformation analysis. *Proc. ASCE, Stability and Performance of Slopes and Embankments — II*. Geotech. Spec. Pub. No. 1(31):222–266.

Duncan, J. M., Buchignani, A. L., and De Wet, M. 1987. *An Engineering Manual for Slope Stability Studies*. Virginia Tech Report.

Dunnicliff, J. 1988. *Geotechnical Instrumentation for Monitoring Field Performance*. John Wiley & Sons, New York.

Espinoza, R. D., Bourdeau, P. L., and Muhunthan, B. 1994. Unified formulation for analysis of slopes with general slip surfaces. *J. Geotech. Eng. ASCE*. 120(7):1185–1204.

Espinoza, R. D., Repetto, P. C., and Muhunthan, B. 1992. General framework for stability analysis of slopes. *Geotechnique*. 42(4):603–616.

Fellenius, W. 1936. Calculation of the stability of earth dams. *Trans. 2nd Cong. Large Dams,* Vol. 4, Washington, D.C.

FHWA. 1993. *FHWA Tour for Geotechnology — Soil Nailing*. U.S. Department of Transportation, Federal Highway Administration, Washington, D.C.

Finn, W. D. L. 1993. Practical studies of the seismic response of a rockfill dam and a tailings impoundment. In *Proc. 3rd Int. Conf. Case Histories Geotech. Eng*. St. Louis.

Gray, D. H., Leiser, A. T., and White, C. A. 1980. Combined vegetative-structural slope stabilization. *Civ. Eng., ASCE*. October, pp. 73–77.

Hall, W. J., and Newmark, N. M. 1977. Seismic design criteria for pipelines and facilities. In *The Current State of Knowledge of Lifeline Earthquake Engineering, Proc. ASCE,* New York, pp. 18–34.

Handy, R. L., and Williams, W. W. 1967. Chemical stabilization of an active landslide. *Civ. Eng.,* August, pp. 62–65.

Hird, C. C. 1986. Stability charts for reinforced embankments on soft ground. *Geotext. Geomembr*. 4(2):107–127.

Hoek, E., and Bray, J. W. 1977. *Rock Slope Engineering,* 2nd ed. The Institute of Mining and Metallurgy, London.

Hollingsworth, R., and Kovacs, G. S. 1981. Soil slumps and debris flows: Prediction and protection. *Bul. Assoc. Eng. Geol*. 18(1):17–28.

Huang, Y. H. 1983. *Stability Analysis of Earth Slopes*. Van Nostrand-Reinhold, New York.

Hunt, R. E. 1984. *Geotechnical Engineering Investigation Manual*. McGraw-Hill, New York.

Hunt, R. E. 1986. *Geotechnical Engineering Techniques and Practices*. McGraw-Hill, New York.

Hunt, R. E., and da Costa Nunes, A. J. 1978. Retaining walls: Taking it from the top. *Civ. Eng.,* May, pp. 73–75.

Hunter, J. H., and Schuster, R. L. 1968. Stability of simple cuttings in normally consolidated clays. *Geotechnique*. 13(3):372–378.

Janbu, N. 1968. *Slope Stability Computations*. Soil Mechanics and Foundation Engineering Report, The Technical University of Norway.

Janbu, N. 1973. Slope Stability Computations. In *Embankment Dam Engineering,* eds. R. C. Hirschfield and S. J. Poulos, pp. 47–86. John Wiley & Sons, New York.

Janbu, N., Bjerrum, L., and Kjaernsli, B. 1956. *Soil Mechanics Applied to Some Engineering Problems,* pp. 5–26. Norwegian Geotechnical Institute, Oslo, Publ. 16.

Johnson, S. J. 1974. Analysis and design relating to embankments. In *Proc. ASCE Conf. Anal. Design Geotech. Eng*. Austin, Texas, Vol. II, pp. 1–48.

Koerner, R. M. 1993. *Designing with Geosynthetics,* 3rd ed. Prentice Hall, Englewood Cliffs, NJ.

Ladd, C. C. 1991. Stability evaluation during staged construction. *J. Geotech. Eng., ASCE*. 117(4):540–615.

Lamb, T. W., and Whitman, R. V. 1969. *Soil Mechanics*. John Wiley & Sons, New York.

Lang, T. A. 1972. Rock reinforcement. *Bull. Assoc. Eng. Geol*. 9(3):215–239.

Leonards, G. A. 1979. Stability of slopes in soft clays. In *6th Panamerican Conf. Soil Mech. Found. Eng*. 1:225–274.

Leonards, G. A. 1982. Investigation of failures. *J. Geotech. Eng., ASCE*. 108(GT2):187–246.

Lippomann, R. 1989. Dowelled clay slopes: Recent example. In *Proc. 12th Int. Conf. Soil Mech. Found. Eng*. Rio de Janeiro, 2:1269–1271.

Low, B. K., Wong, K. S., Lim, C., and Broms, B. B. 1990. Slip circle analysis of reinforced embankments on soft ground. *Geotext. Geomembr*. 9(2):165–181.

Lowe, J., III. 1967. Stability analysis of embankments. *J. Soil Mech. Found., ASCE*. 93(SM4):1–34.

Morganstern, N. R., and Price, V. E. 1965. The analysis of the stability of general slip surfaces. *Geotechnique*. 15(1):79–93.

NAVFAC. 1982. *Design Manual, Soil Mechanics, Foundations and Earth Structures, DM-7*. Naval Facilities Engineering Command, Alexandria, VA.

Newmark, N. M. 1965. Effects of earthquakes on dams and embankments. *Geotechnique*. 15(2):139–160.

Oakland, M. W., and Chameau, J. L. 1989. Analysis of drilled piers used for slope stabilization. *Trans. Res. Rec*. 1219:21–32.

Patton, F. D., and Hendron, A. J., Jr. 1974. General report on mass movements. In *Proc. 2nd Int. Cong. Int. Assoc. Eng. Geol.* São Paulo, p. V-GR 1.

Seed, H. B. 1966. A method for earthquake resistant design of earth dams. *J. Soil Mech. Found., ASCE*. 92(SM1):13–41.

Seed, H. B., Idriss, I. M., Lee, K. L., and Makdisi, F. I. 1975. Dynamic analysis of the slide of the lower San Fernando Dam during the earthquake of February 9, 1971. *J. Geotech. Eng., ASCE*. 101(GT9):889–911.

Spencer, E. 1967. A method of analysis of the stability of embankments assuming parallel inter-slice forces. *Geotechnique*. 17(1):11–26.

Sunset. 1978. If hillside slides threaten in southern California, November is planting action month. *Sunset Magazine,* November, pp. 122–126.

Taylor, D. W. 1937. Stability analysis of earth slopes. *Journal of the Boston Society of Civil Engineers,* 24(3). Reprinted in *Contributions to Soil Mechanics 1925–1940,* BSCE, 1940, pp. 337–386.

Taylor, D. W. 1948. *Fundamentals of Soil Mechanics*. John Wiley & Sons, New York.

Terzaghi, K. 1962. Stability of steep slopes on hard, unweathered rock. *Geotechnique*. 12(4):251–270.

Tschebotarioff, G. P. 1973. *Foundations, Retaining and Earth Structures*. McGraw-Hill, New York.

Vulliet, L., and Hutter, K. 1988. Viscous-type sliding laws for landslides. *Can. Geotech. J*. 25(3):467–477.

Wilson, S. D., and Marsal, R. J. 1979. *Current Trends in Design and Construction of Embankment Dams. ASCE,* New York.