1

Landslides and Other Slope Failures

1.1 Introduction

1.1.1 General

Origins and Consequences of Slope Failures

Gravitational forces are always acting on a mass of soil or rock beneath a slope. As long as the strength of the mass is equal to or greater than the gravitational forces, the forces are in balance, the mass is in equilibrium, and movement does not occur. An imbalance of forces results in slope failure and movement in the forms of creep, falls, slides, avalanches, or flows.

Slope failures can range from being a temporary nuisance by partially closing a roadway, to destroying structures, to being catastrophic and even burying cities.

Failure Oddities

- *Prediction:* Some failures can be predicted, others cannot, although most hazardous conditions are recognizable.
- *Occurrence*: Some forms occur without warning; many other forms give warning, most commonly in the form of early surface cracks.
- *Movement velocities*: Some move slowly, others progressively or retrogressively, others at great velocities.
- *Movement distances*: Some move short distances; others can move for many miles.
- *Movement volume*: Some involve small blocks; others involve tremendous volumes.
- *Failure forms*: Some geologic formations have characteristic failure forms; others can fail in a variety of forms, often complex.
- *Mathematical analysis*: Some conditions can be analyzed mathematically, many cannot.
- *Treatments*: Some conditions cannot be treated to make them stable; they should be avoided.

Objectives

The objectives of this chapter are to provide the basis for:

- Prediction of slope failures through the recognition of the geologic and other factors that govern failure.
- Treatment of slopes that are potentially unstable and pose a danger to some existing development.
- Design and construction of stable cut slopes and side-hill fills.
- Stabilization of failed slopes.

1.1.2 Hazard Recognition

General

Slope failures occur in many forms. There is a wide range in their predictability, rapidity of occurrence and movement, and ground area affected, all of which relate directly to the consequences of failure. Recognition permits the selection of some slope treatment which will either *avoid, eliminate, or reduce the hazard.*

Hazard recognition and successful treatment require thorough understanding of a number of factors including:

- Types and forms of slope failures (classification)
- Relationship between geologic conditions and the potential failure form
- Significance of slope activity, or amount and rate of movement
- Elements of slope stability
- Characteristics of slope failure forms (see Section 1.2)
- Applicability of mathematical analysis (see Section 1.3)

Classification of Slope Failures

A classification of slope failures is given in Table 1.1. The most important classes are *falls, slides, avalanches*, and *flows*.

TABLE 1.1

A Classification of Slope Failures

Type	Form	Definition
Falls	Free fall	Sudden dislodgment of single or multiple blocks of soil or rock which fall in free descent.
	Topple	Overturning of a rock block about a pivot point located below its center of gravity.
Slides	Rotational or slump	Relatively slow movement of an essentially coherent block (or blocks) of soil, rock, or soil–rock mixtures along some well-defined arc-shaped failure surface.
	Planar or translational	Slow to rapid movement of an essentially coherent block (or blocks) of soil or rock along some well-defined planar failure surface.
	Subclasses	
	Block glide	A single block moving along a planar surface.
	Wedges	Block or blocks moving along intersecting planar surfaces.
	Lateral spreading	A number of intact blocks moving as separate units with differing displacements.
	Debris slide	Soil–rock mixtures moving along a planar rock surface.
Avalanches	Rock or debris	Rapid to very rapid movement of an incoherent mass of rock or soil–rock debris wherein the original structure of the formation is no longer discernible, occurring along an ill-defined surface.
Flows	Debris Sand Silt Mud Soil	Soil or soil–rock debris moving as a viscous fluid or slurry, usually terminating at distances far beyond the failure zone; resulting from excessive pore pressures (subclassed according to material type).
Creep		Slow, imperceptible downslope movement of soil or soil–rock mixtures.
Solifluction		Shallow portions of the regolith moving downslope at moderate to slow rates in Arctic to sub-Arctic climates during periods of thaw over a surface usually consisting of frozen ground.
Complex		Involves combinations of the above, usually occurring as a change from one form to another during failure with one form predominant.

Major factors of classification include:

- *Movement form*: Fall, slide, slide flow (avalanche), flow
- *Failure surface form*: Arc-shaped, planar, irregular, ill-defined
- *Mass coherency*: Coherent, with the original structure essentially intact although dislocated, or incoherent, with the original structure totally destroyed
- *Constitution*: Single or multiple blocks, or a heterogeneous mass without blocks, or a slurry
- *Failure cause*: Tensile strength or shear strength exceeded along a failure surface, or hydraulic excavation, or excessive seepage forces

Other factors to consider include:

- *Mass displacement*: Amount of displacement from the failure zone, which can vary from slight to small, to very large. Blocks can move together with similar displacements, or separately with varying displacements.
- *Material type*: Rock blocks or slabs, soil–rock mixtures (debris), sands, silts, blocks of overconsolidated clays, or mud (weak cohesive soils).
- *Rate of movement during failure*: Varies from extremely slow and barely perceptible to extremely rapid as given in Table 1.2.

TABLE 1.2
Velocity of Movement for Slope Failure Forms[a]

Velocity (m/s)	Movement	Rate		Classification
10^2, 10	Extremely rapid		Final stage (Planar slide-rock mass)	Avalanches and flows
		3 m/sec		
1, 10^{-1}, 10^{-2}	Very rapid			
		0.3 m/min		
10^{-3}, 10^{-4}	Rapid		Final stage (Rotational slide)	Slides
		1.5 m/day		
10^{-5}	Moderate			
10^{-6}		1.5 m/month		
	Slow		Inter-mediate	
10^{-7}		1.5 m/year		
10^{-8}	Very slow		Early stage	Creep
		0.3 m/5 year		
10^{-9}	Extremely slow			

[a] After Varnes, D. J., *Landslides and Engineering Practice*, Eckel, E. B., Ed., Highway Research Board Special Report No. 29, Washington, DC, 1958. Reprinted with permission of the Transportation Research Board.

Slope Failure Forms Related to Geologic Conditions

Anticipation of the form of slope failure often can be based on geologic conditions as summarized in Table 1.3. Detailed descriptions of the various forms are given in Section 1.2. Some forms of falls and slides in rock masses are illustrated in Figure 1.1. and Figure 1.2, slides in soil formations in Figure 1.3, and avalanches and flows in rock, soil, and mixtures in Figure 1.4.

Slope Activity

Slope activity relates to the amount and rate of slope movement that occur. Some failure forms occur suddenly on stable slopes without warning, although many forms occur slowly through a number of stages. Failure implies only that movement has occurred, but not necessarily that it has terminated; therefore, it is necessary to establish descriptive criteria for failure, or stability, in terms of stages. The amount and rate of movement vary with the failure stage for some failure forms.

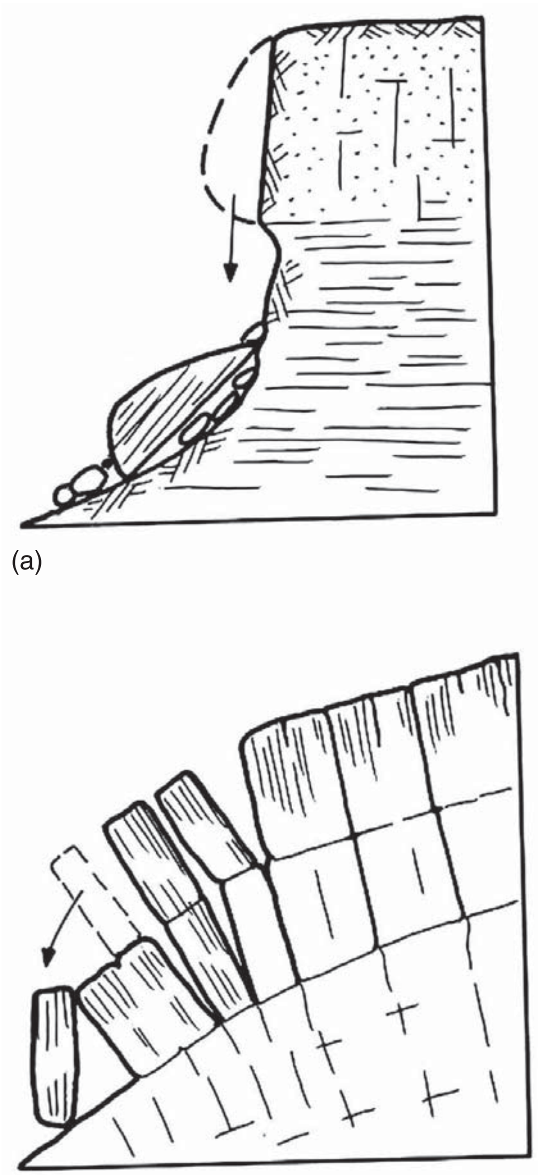
Slide forms of failure may be classified by five stages of activity:

1. *Stable slope*: No movement has occurred in the past, or is occurring now.
2. *Early failure stage*: Creep occurs, with or without the development of tension cracks on the surface (see Figure 1.22). Slump form movement velocities are generally of the order of a few inches per year.

TABLE 1.3

Geologic Conditions and Typical Forms of Slope Failures

Geologic Condition	Typical Movement Forms
Rock masses: general	Falls and topples from support loss
	Wedge failure along joints, or joints, shears, and bedding
	Block glides along joints and shears
	Planar slide along joints and shears
	Multiplanar failure along joint sets
	Dry rock flow
Metamorphic rocks	Slides along foliations
Sedimentary rocks	Weathering degree has strong effect
Horizontal beds	Rotational, or a general wedge through joints and along bedding planes
Dipping beds	Planar along bedding contacts; block glides on beds from joint separation
Marine shales, clay shales	Rotational, general wedge, or progressive through joints and along mylonite seams
Residual and colluvial soils	Depends on stratum thickness
Thick deposit	Rotational, often progressive
Thin deposit over rock	Debris slide, planar; debris avalanche or flow
Alluvial soils	Depends on soil type and structure
Cohesionless	Runs and flows
Cohesive	Rotational or planar wedge
Stratified	Rotational or wedges, becoming lateral spreading in fine-grained soils
Aeolian deposits	Variable
Sand dunes or sheets	Runs and flows
Loess	Block glides: flows during earthquakes
Glacial deposits	Variable
Till	Rotational
Stratified drift	Rotational
Lacustrine	Rotational becoming progressive
Marine	Rotational to progressive: rotational becoming lateral spreading: flows



(a) (b)

FIGURE 1.1
Forms of falls in rock masses: (a) free fall; (b) toppling by overturning.

3. *Intermediate failure stage*: Progressive slumps and scarps begin to form during rotational slides, and blocks begin to separate during planar slides, as tension cracks grow in width and depth. Movement velocities may range up to about 2 in./day (5 cm/day), accelerating during rainy seasons and storms and diminishing during dry periods. Movement is affected also by flooding, high tides, and earthquake forces. The slope is essentially intact and may remain in this condition for many years (see Figure 1.89).
4. *Partial total failure*: A major block or portion of the unstable mass has moved to a temporary location leaving a large scarp on the slope (Figure 1.25a).
5. *Complete failure*: The entire unstable mass has displaced to its final location (see Figure 1.93), moving rapidly at rates of about 3 ft/min (1m/min) for the case of

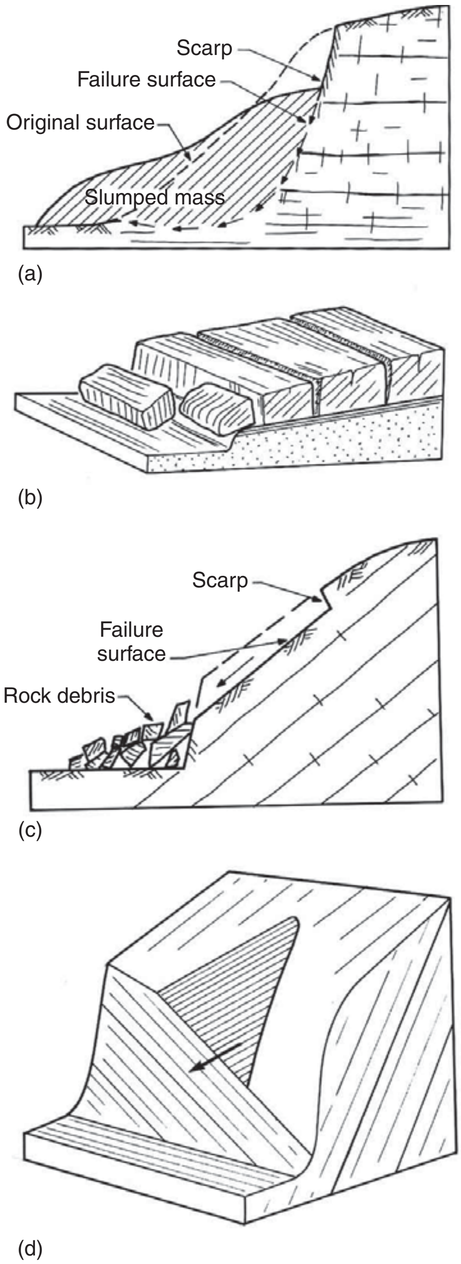


FIGURE 1.2
Slide forms in rock masses. (a) Rotational slide failed through joints and weak basal horizontal bed. (b) Translational sliding of blocks along a weak planar surface such as shale. (c) Planar slide failed along steeply dipping beds after cutting along lower slope. (d) Wedge failure scar. Failure occurred along intersecting joints and bedding planes when cut was made in obliquely dipping beds (see Sections 1.2.3 and 1.2.4).

rotational slides (Table 1.2). Planar slides in rock masses commonly reach velocities of 10 to 50 m/h (Banks and Strohn, 1974). Large planar slides in rock masses can achieve tremendous velocities, at times of the order of 200 m/h, as has been computed for the Vaiont slide (see Section 1.2.3). Habib (1975) considers these high velocities to be the result of movement of the rock mass over a cushion of water that negates all frictional resistance. The cushion is caused by heat, generated by shearing forces, which vaporizes the pore water. Such velocities are the major reason for the often disastrous effects of planar rock slides. Slide failures are usually progressive, and can develop into failure by lateral spreading, as well as into avalanches and flows.

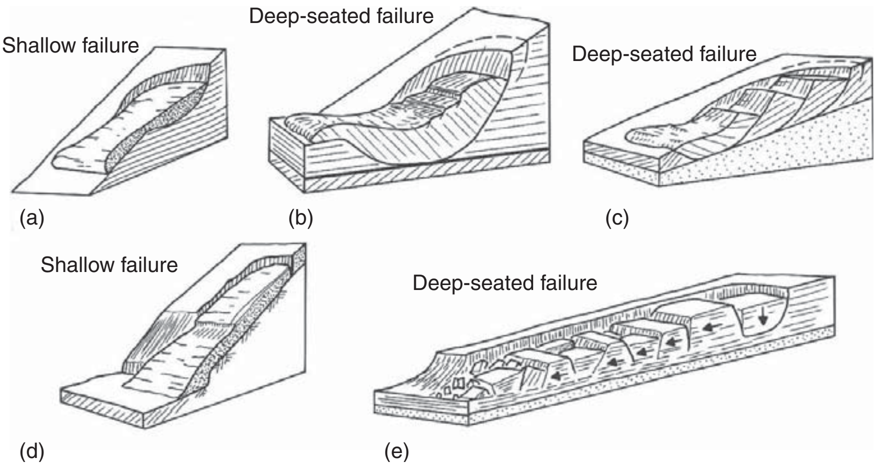


FIGURE 1.3
Slide forms in soil formations. (a) Single block failed along slope as a result of high groundwater level or strength increase with depth in cohesive soils. (b) Single block in homogeneous cohesive soils failed below toe of slope because of either a stronger or a weaker soil boundary at base. (c) Failure of multiple blocks along the contact with strong material. (d) Planar slide or slump in thin soil layer over rock. Often called debris slides. Common in colluvium and develop readily into flows. (e) Failure by lateral spreading. Occurs in glaciomarine or glaciolacustrine soils (parts a–c are rotational forms; parts d and e are planar or translational forms).

Avalanches and flows may develop from slide forms as mentioned above, or may undergo an early stage, but total final failure often occurs suddenly without warning on a previously stable slope as the result of some major event such as a very large rainfall or an earthquake. Velocities are usually very rapid to extremely rapid as given in Table 1.2.

Falls may occur suddenly, but often go through an early stage evidenced by the opening of tension cracks.

Deposition

Talus is rock debris at slope toes resulting from falling blocks. *Colluvium* is the residue of soil materials composing the soil mass, generally resulting from complete failure.

1.1.3 Rating the Hazard and the Risk

Significance

An existing or potential slope failure must be evaluated in terms of the degree of the hazard and the risk when plans for the treatment are formulated (see Section 1.4). Some conditions cannot be improved and should be avoided; in most, however, the hazard can be eliminated or reduced.

Hazard refers to the slope failure itself in terms of its potential magnitude and probability of occurrence.

Risk refers to the consequences of failure on human activities.

Hazard Degree

The rating basis for hazard is the potential magnitude and probability of failure.

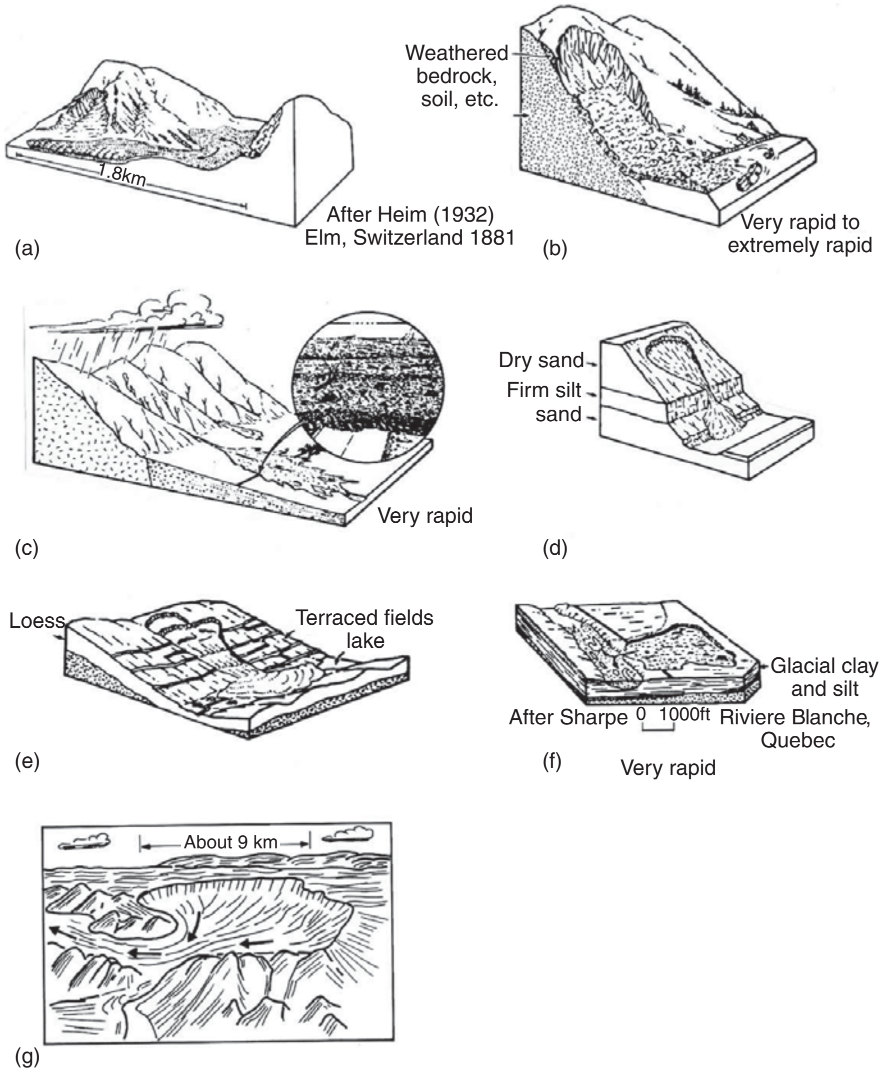


FIGURE 1.4
Avalanches and flows in rock, debris, and soil. (a) Rock fragment flow or rockfall avalanche. (This type of movement occurs only when large rockfalls and rockslides attain unusual velocity. Extremely rapid [more than 130 ft/sec] at Elm, Switzerland.) (b) Debris avalanche. (c) Debris flow. (d) Sand run: rapid to very rapid. (e) Dry loess flow caused by earthquake (Kansu Province, China, 1920). Extremely rapid movement. (f) Soil or mud flow. (g) Achacolla mud flow (La Paz, Bolivia). Huge mass of lacustrine soils slipped off the altiplano and flowed downstream for 25 km (see Figure 1.57). (Parts a–f from Varnes, D. J., *Landslides and Engineering Practice*, Eckel, E. B., Ed., Highway Research Board, Washington, DC, 1958. Reprinted with permission of the Transportation Research Board.)

- *Magnitude* refers to the volume of material which may fail, the velocity of movement during failure, and the land area which may be affected. It depends very much on the form of failure as related to geology, topography, and weather conditions.
- *Probability* is related in a general manner to weather, seismic activity, changes in slope inclinations, and other transient factors.

No hazard: A slope is not likely to undergo failure under any foreseeable circumstances.

Low hazard: A slope may undergo total failure (as compared with partial failure) under extremely adverse conditions which have a low probability of occurrence (for example, a 500 year storm, or a high-magnitude earthquake in an area of low seismicity), or the potential failure volume and area affected are small even though the probability of occurrence is high.

Moderate hazard: A slope probably will fail under severe conditions that can be expected to occur at some future time, and a relatively large volume of material is likely to be involved. Movement will be relatively slow and the area affected will include the failure zone and a limited zone downslope (moderate displacement).

High hazard: A slope is almost certain to undergo total failure in the near future under normal adverse conditions and will involve a large to very large volume of materials; or, a slope may fail under severe conditions (moderate probability), but the potential volume and area affected are enormous, and the velocity of movement very high.

Risk Degree

The rating basis for risk is the type of project and the consequences of failure.

No risk: The slope failure will not affect human activities.

Low risk: An inconvenience easily corrected, not directly endangering lives or property, such as a single block of rock of small size causing blockage of a small portion of roadway and easily avoided and removed.

Moderate risk: A more severe inconvenience, corrected with some effort, but not usually directly endangering lives or structures when it occurs, such as a debris slide entering one lane of a roadway and causing partial closure for a brief period until it is removed. Figure 1.5 illustrates a debris avalanche that closed a roadway for some days.

High risk: Complete or partial loss of a roadway or important structure, or complete closure of a roadway for some period of time, but lives are not necessarily endangered during the failure. Figure 1.6 illustrates a partial loss of a roadway. If failure continues it will result in total loss of the roadway and will become a *very high risk* for traffic.

Very high risk: Lives are endangered at the time of failure by, for example, the destruction of inhabited structures or a railroad when there is no time for a warning. The scars on the steep slopes in Figure 1.7 are the result of debris slides and avalanches resulting from roadway cuts upslope. The town shown on the lower right of the photo is located on the banks of a river. Concerns were from debris avalanches (1) filling and damming the river resulting in flooding of the town, and (2) falling on the town. Studies showed that the width, depth, and flow velocity of the river would remove any foreseeable volume of debris, and damming would not be expected. As long as the vegetation upslope of the town remained, the slope would be stable. Treatments were recommended to stabilize the areas upslope where failures had occurred. Therefore, the possible very high risk was reduced to low.

1.1.4 Elements of Slope Stability

General

Dependent Variables

Stated simply, slope failures are the result of gravitational forces acting on a mass which can creep slowly, fall freely, slide along some failure surface, or flow as a slurry. Stability can depend on a number of complex variables, which can be placed into four general categories as follows:

1. Topography — in terms of slope inclination and height
2. Geology — in terms of material structure and strength
3. Weather — in terms of seepage forces and run-off quantity and velocity
4. Seismic activity — as it affects inertial and seepage forces

FIGURE 1.5
Debris avalanche closes a roadway in Ecuador for some days. A temporary bypass was constructed and used until the debris was removed.

It is important to note that, although topography and geology are usually constant factors, there are situations where they are transient.

Mechanics of Sliding Masses

Masses that fail by sliding along some well-defined surface, moving as a single unit (as opposed to progressive failure or failure by lateral spreading), are the only slope failure form that can be analyzed mathematically in the present state of the art (see Section 1.3).

FIGURE 1.6
Partial loss of mountain roadway in Ecuador. Failure resulted from discharge from roadway drains causing downslope erosion. Additional failure will result in total loss of roadway and closure probably for months.

The diagrams given in Figure 1.8 illustrate the concept of failure that occurs when driving forces exceed resisting forces.

In the figure, the weight of mass *W* bounded by slice abc (in [a] acted on by the lever arm E; in [b] a function of the inclination of the failure surface) causing the driving force, is resisted by the shear strength *s* mobilized along the failure surface of length *L* (in case [a] acted on at "a" by lever arm R). The expression for factor of safety (FS) given in the figure is commonly encountered but is generally considered unsatisfactory because the resisting moment and the driving moment in (a) are ambiguous. For example, the portion of the rotating mass to the left of the center of rotation could be considered as part of the resisting moment. For this reason, FS is usually defined as

$$\mathrm{FS} = \frac{\text{shearing strength available along sliding surface}}{\text{shearing stresses tending to produce failure along surface}}$$

The four major factors influencing slope stability are illustrated in Figure 1.9 and described in the following sections.

Slope Geometry (Figure 1.9a)

Significance

Driving forces and *runoff* are increased as slope inclination and height increase. Runoff quantity and velocity are related directly to amount of erosion, and under severe conditions cause "hydraulic excavation," resulting in avalanches and flows (see discussion of runoff below).

Inclination

Geologic formations often have characteristic inclinations at which they are barely stable in the natural state, for examples, residual soils at 30 to 40°, colluvium at 10 to 20°, clay shales at 8 to 15°, and loess, which often stands vertical to substantial heights.

FIGURE 1.7
Debris avalanches resulted from roadway construction on a steep slope in the Bolivian Andes. The small town of Pacallo in the photo lower right is located adjacent to a fast-flowing mountain stream. The major concern was with future avalanches damming the stream with debris resulting in flooding of the town.

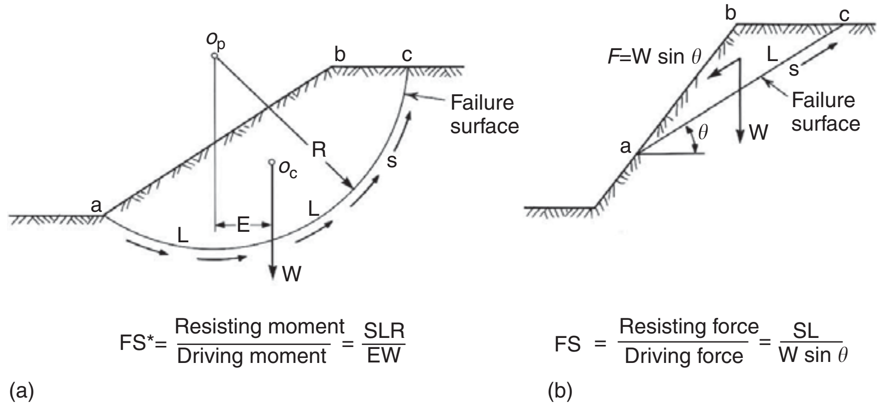


FIGURE 1.8
Forces acting on cylindrical and planar failure surfaces. (a) Rotational cylindrical failure surface with length *L*. Safety factor against sliding, FS. (b) Simple wedge failure on planar surface with length *L*. (*Note that the expression for FS is generally considered unsatisfactory; see text.)

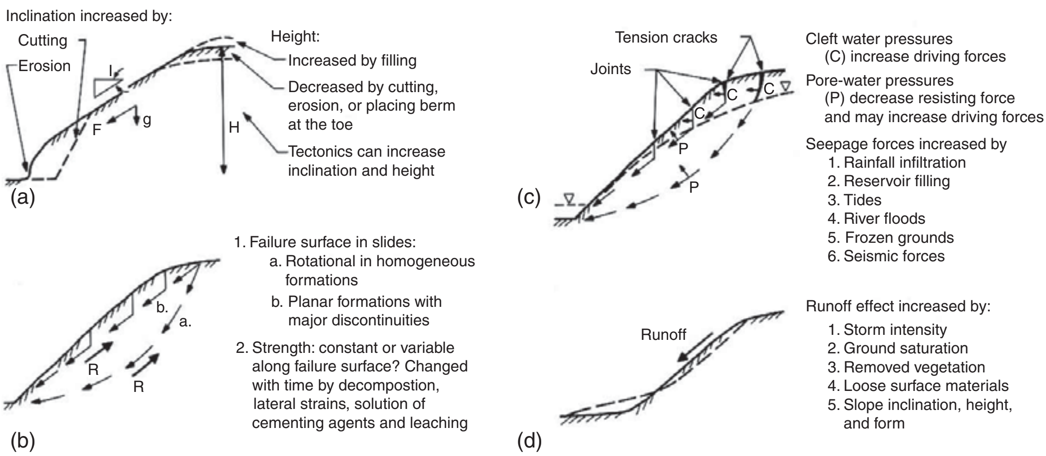


FIGURE 1.9
The major factors influencing slope stability: (a) increasing slope inclination and height increases the driving forces *F*; (b) geologic structure influences form and location of failure surface, material strength provides the resisting force *R*; (c) seepage forces reduce resisting forces along failure surface and increase driving forces in joints and tension cracks; (d) runoff quantity and velocity are major factors in erosion, avalanches, and flows.

Inclination is increased by:

- Cutting during construction, which should be controlled by analysis and judgment.
- Erosion, as a result of undercutting at the slope toe by wave or stream activity, of seepage exiting from the slope face, or of removal of materials by downslope runoff. All these are significant natural events.
- Tectonic movements in mountainous terrain, a very subtle and long-term activity which provides a possible explanation for the very large failures that occur from time to time and for which no other single explanation appears reasonable. An example is the disastrous rock slide at Goldau, Switzerland (see Section 1.2.3).

Slope Height

Slope height is increased by filling at the top, erosion below the toe, or tectonic activity. It is decreased by excavation and erosion at the top, or by placing a berm at the bottom. The driving forces are affected in failure forms where the limited slope condition applies (see Figure 1.8).

Material Structure (Figure 1.9b)

Significance

Material structure influences the failure form and the location and shape of the potential failure surface, and can be considered in two broad categories: uniform and nonuniform.

Uniform Materials

Uniform materials consist of a single type of soil or rock, essentially intact and free of discontinuities. From the aspect of slope stability, they are restricted to certain soil formations. Rotational failure is normal; the depth of the failure surface depends on the location of the phreatic surface and on the variation of strength with depth. Progressive failures are common, and falls and flows possible; flows are common in fine-grained granular soils.

Nonuniform Materials

Formations containing strata of various materials, and discontinuities represented by bedding, joints, shears, faults, foliations, and slickensides are considered nonuniform. The controlling factor for stability is the orientation and strength of the discontinuities, which represent surfaces of weakness in the slope.

Planar slides occur along the contacts of dipping beds of sedimentary rock and along joints, fault and other shear zones, slickensides, and foliations. Where a relatively thin deposit of soil overlies a sloping rock surface, progressive failure is likely and may develop into a debris avalanche. Along relatively flat-lying strata of weak material, failure can develop progressively in the form of lateral spreading, and can develop into a flow.

Rotational slides occur in horizontally bedded soil formations, and in certain rock formations such as clay shales and horizontally bedded sedimentary rocks.

Falls occur from lack of tensile strength across joints in overhanging or vertical rock masses. Changes in the orientation of the discontinuities with respect to the slope face occur normally as a result of excavation, but can also be caused by tectonic activity. Joint intensity can be affected by construction blasting.

Material Strength (Figure 1.9b)

Significance

Material strength provides the resisting forces along a surface of sliding. It is often neither the value determined by testing, nor the constant value assumed in analysis.

Variations along the Failure Surface

Slopes normally fail at a range of strengths, varying from peak to residual, distributed along the failure surface as a function of the strains. Slopes that have undergone failure in the past will have strengths at or near residual, depending upon the time for restitution available since failure.

Changes with Time

Chemical weathering is significant in residual soils and along discontinuities in rock masses in humid climates, and provides another possible explanation for the sudden failure of rock-mass slopes that have remained stable for a very long period of time under a variety of weather and seismic conditions.

Lateral strains in a slope tend to reduce the peak strength toward the residual, a significant factor in the failure of slopes in clay shales and some overconsolidated clays containing recoverable strain energy (Bjerrum, 1966), as well as in materials where slope movements have occurred.

Solution of Cementing Agents Reduces Strength.

Leaching of salts from marine clays increases their sensitivity and, therefore, their susceptibility to liquefaction and flow (Bjerrum et al., 1969).

Seepage Forces (Figure 1.9c)

Significance

Seepage forces may reduce the resisting forces along the failure surface or increase the driving forces.

Factors Causing Increased Seepage Forces

In general, seepage forces are increased by rainfall infiltration or reservoir filling, which raises the water table or some other phreatic surface (perched water level); sudden drawdown of a flooded stream or an exceptionally high tide; melting of a frozen slope that had blocked seepage flow; and earthquake forces.

Rising groundwater level is a common cause. Variables affecting such a rise include rainfall accumulation and increase in ground saturation for a given period, the intensity of a particular storm, the type and density of ground vegetation, drainage characteristics of the geologic materials, and the slope inclination and other features of topographic expression. Vegetation, geology, and topography influence the amount of infiltration that can occur, and careful evaluation of these factors often can provide the reasons for failure to occur at a particular location along a slope rather than at some other position during a given storm or weather occurrence.

Earthquake forces can cause an increase in pore-air pressures, as well as porewater pressures. Such an increase is believed to be the cause of the devastating extent of the massive landslides in loess during the 1920 earthquake in Kansu, China, which left 200,000 or more dead.

Runoff (Figure 1.9d)

Significance

The quantity and velocity of runoff are major factors in erosion, and are a cause of avalanches and flows. Storm intensity, ground saturation, vegetation, frozen ground, the nature of the surficial geologic materials, and slope inclination and other topographic features affect runoff.

Hydraulic Excavation

Many avalanches and flows are caused by hydraulic excavation during intense storms, a common event in tropical and semiarid climates. Water moving downslope picks up soils loosened by seepage forces, and as the volume and velocity increase, the capacity to remove more soil and even boulders increases, eventually resulting in a heavy slurry which removes everything loose in its path as it flows violently downslope. The scar of a debris avalanche is illustrated in Figure 1.10. Failure could hardly have been foreseen at that particular location along the slope, since conditions were relatively uniform.

FIGURE 1.10
Exposed rock surface remaining after runoff from torrential rains removed all vegetation, soil, and loose rock, depositing the debris mass at the toe of the slope (BR 116, km 56, Teresopolis, R. J., Brazil).

1.2 Slope Failure Form Characteristics

1.2.1 Creep

General

Creep is the slow, imperceptible deformation of slope materials under low stress levels, which normally affects only the shallow portion of the slope, but can be deep-seated where a weak zone exists. It results from gravitational and seepage forces, and is indicative of conditions favorable for sliding.

Recognition

Creep is characteristic of cohesive materials and soft rock masses on moderately steep to steep slopes. Its major surface features are parallel transverse slope ridges (“cow paths”) as illustrated in Figure 1.11, and tilted fence posts, poles, and tree trunks. Straight tilted tree trunks indicate recent movement (Figure 1.12), whereas bent tree trunks indicate old continuing movement (Figure 1.13) (see Section 1.5.2, Dating Relict Slide Movements).

1.2.2 Falls

General

Falls are the sudden failures of vertical or near-vertical slopes involving single or multiple blocks wherein the material descends essentially in free fall. Toppling, or overturning of rock blocks, often results in a fall.

In soils, falls are caused by the undercutting of slopes due to stream or wave erosion, usually assisted by seepage forces. In rock masses, falls result from undercutting by erosion or human excavation; increased pressures in joints from frost, water, or expanding materials; weathering along joints combined with seepage forces; and differential weathering wherein less-resistant beds remove support from stronger beds.

Their engineering significance lies normally in the occurrence of a single or a few blocks falling on a roadway, or occasionally encountering structures on slopes. At times, however, they can be massive and very destructive as shown in Figure 1.14.

Recognition

Falls are characteristic of vertical to near-vertical slopes in weak to moderately strong soils and jointed rock masses. Before total failure some displacement often occurs, as indicated by tension cracks; after total failure, a fresh rock surface remains and talus debris accumulates at the toe.

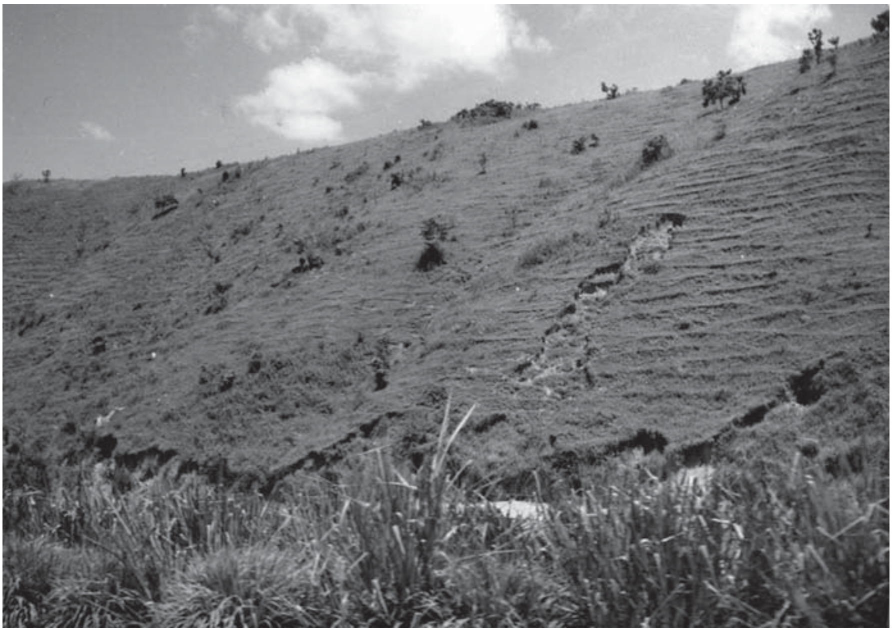


FIGURE 1.11
Creep ridges and erosion in residual soils after removal of vegetation (state of Rio de Janeiro, Brazil).

FIGURE 1.12
Trees bent in the lower portions and then growing straight up indicate long-term slope movements. The scarp in the photo is the head of a progressive failure in marine shales extending downslope for over a kilometer near Bandung, Java.

1.2.3 Planar Slides in Rock Masses

General

Forms of planar slides in rock masses include:

- Block glide involving a single unit of relatively small size (photo, Figure 1.15).
- Slab glide involving a single unit of relatively small to large size (photo, Figure 1.16).
- Wedge failures along intersecting planes involving single to multiple units, small to very large in size (Figure 1.2d). A small wedge failure is illustrated in Figure 1.81.
- Translational slide: Sliding as a unit, or multiple units, downslope along one or more planar surfaces (Figure 1.2b). Failure often is progressive (Section 1.2.6).
- Massive rock slide involving multiple units, small to very large in size, often with very high velocities (Figure 1.17).

Block and slab slides can be destructive, but massive rock slides are often disastrous in mountainous regions and in many cases cannot be prevented, only avoided.

FIGURE 1.13
Tilting tree trunks on a creeping hillside of varved clays indicate relatively recent movement (Tompkins Cove, New York).

FIGURE 1.14
Rockfall destroyed a powerhouse (Niagara Falls, New York). Failure may actually be in the form of a huge topple. (Photo by B. Benedict, 1956.)

FIGURE 1.15
Small granite block glide (Rio de Janeiro, Brazil).

FIGURE 1.16
Exfoliation loosening granite slabs. Impact wall on right was constructed to deflect falling and sliding blocks from buildings on lower slopes. Damage from falls and slab slides is a serious problem in Rio de Janeiro.

FIGURE 1.17
The scar of the Gros Ventre slide as seen from the Gros Ventre River, Wyoming, in August 1977.

Recognition

Planar slides are characteristic of:

- Bedded formations of sedimentary rocks dipping downslope at an inclination similar to, or less than, the slope face. They result in block glides or massive rock slides (see Examples below).
- Faults, foliations, shears or joints forming long, continuous planes of weakness that intersect the face of the slope.
- Intersecting joints result in wedge failures, which can be very large in open-pit mines.
- Jointed hard rock results in block glides.
- Exfoliation in granite masses results in slab glides.

Surface features:

- Before total failure, tension cracks often form during slight initial displacement.
- After total failure, blocks and slabs leave fresh scarps. Massive rock slides leave a long fresh surface denuded of vegetation, varying in width from narrow to wide and with a large debris mass at the toe of the slope and beyond. Since they can achieve very high velocities, they can terminate far beyond the toe.

Examples of Major Failures

Goldau, Switzerland

In September 1806, a massive slab 1600 m long, 330 m wide, and 30 m thick broke loose and slid downslope during a heavy rainstorm, destroying a village and killing 457 persons. The slab consisted of Tertiary conglomerate with a calcareous binder resting on a 30° slope. At its interface with the underlying rock was a porous layer of weathered rock.

Three possible causes were offered by Terzaghi (1950):

1. The slope inclination gradually increased from tectonic movements.
2. The shearing resistance at the slab interface gradually decreased because of progressive weathering or from removal of cementing material.

3. The piezometric head reached an unprecedented value during the rainstorm. Terzaghi was hesitant to accept this as the only cause, since he considered it unlikely that in the entire geologic history of the region, there had not been a more severe storm. Therefore, he concluded that the slide resulted from two or more changing conditions.

Gros Ventre, Wyoming

On June 23, 1925, following heavy rains and melting snow, approximately 50 million yd^3 slid in a few minutes down the mountainside along the Gros Ventre River near Grand Teton National Park in Wyoming. The debris formed a natural dam as high as 250 ft which blocked the river, and resulted in a lake 3 mi long. Almost 2 years later, in May 1927, water from heavy rains and melting snow filled the reservoir, over-topped the natural dam, eroded a large channel, and released flood waters which resulted in a number of deaths.

The slide scar which is still evident in 1977, 52 years later, is illustrated in Figure 1.17. A geologic section is given in Figure 1.18. Failure occurred along clay layers in the carbonaceous Amsden formation, dipping downslope. It appears that water entered the joints and pores of the Tensleep sandstone saturated the clay seams, and reduced or eliminated the normal stresses.

Vaiont, Italy

On October 9, 1963, the worst dam disaster in history occurred when more than 300 million m^3 of rock slid into the reservoir formed by the world's highest thin-arch concrete dam causing a tremendous flood which overtopped the dam and flowed into the Piave River valley, taking some 2600 lives. The slide involved an area on the south side of the valley roughly 2.3 km in width and 1.3 km in length, as shown in Figure 1.19. The natural slope was of the order of 20 to 30°.

A geologic section is given in Figure 1.20. The valley had formed in the trough of a syncline, and the beds forming the limbs dipped downslope at inclinations a few degrees steeper than the slope. The south slope consisted of Jurassic sedimentary rocks, primarily limestones and marls occasionally interbedded with clay seams (bentonite clay at residual strength; Patton, F. D. and Hendron, A. J., unpublished). Tectonic activity had caused regional folding, faulting, and fracturing of strata, and some of the tectonic stresses

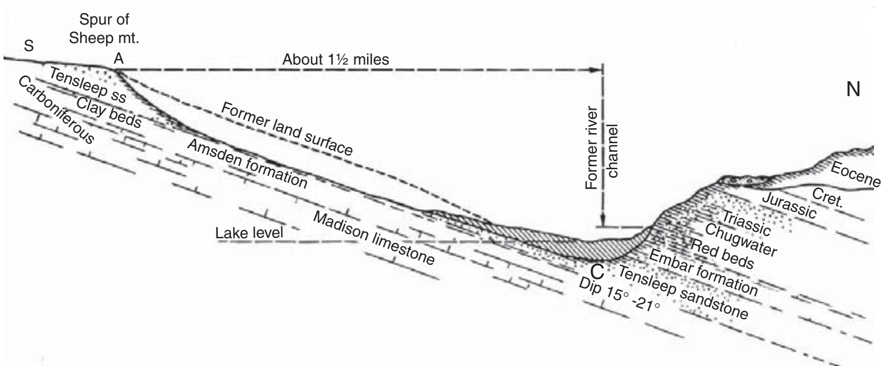


FIGURE 1.18
Section showing geologic conditions after the Gros Ventre landslide. The landslide dammed the Gros Ventre River. (From Alden, W. C., in *Focus on Environmental Geology*, Tank, R., Ed., Oxford University Press, New York [1973], 1928, pp. 146–153. With permission.)

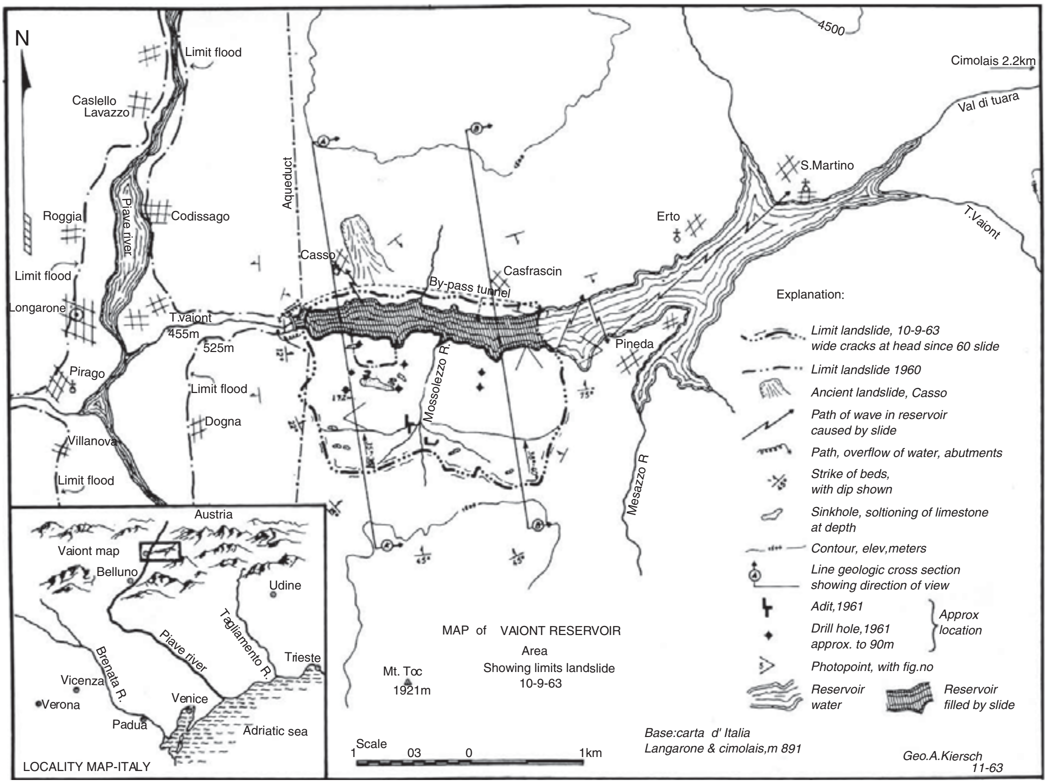


FIGURE 1.19
Map of Vaiont Reservoir slide. (From Kiersch, G. A., in *Focus on Environmental Geology*, Tank, R., Ed., Oxford University Press, New York [1973], Section 17, 1965, pp. 153–164. With permission.)

probably remained as residual stresses in the mass. Erosion of the valley caused some stress relief of the valley walls, resulting in numerous rebound joints that produced blocky masses. In addition, groundwater had attacked the limestone, leaving cavities and contributing to the generally unstable conditions (Kiersch, 1965).

The slide history is given by Kiersch (1965). Large-scale slides had been common on the Vaiont valley slopes, and evidence of creep had been observed near the dam as early as 1960, when the dam was completed at its final height of 267 m. During the spring and summer of 1963, the slide area was creeping at the rate of 1 cm/week. Heavy rains occurred during August and September and movement accelerated to 1 cm/day. In mid-September, movement accelerated to 20 to 30 cm/day, and on the day of failure, 3 weeks later, it was 80 cm/day. Since completion of the dam, the pool had been filled gradually and the elevation maintained at about 50 m below the crest or lower. During September, the pool rose at least 20 m higher, submerged the toe of the sliding mass, and caused the groundwater level to rise in the sliding mass. Collapse was sudden and the entire mass to a depth of 200 m broke loose and slid to the valley floor in 30 to 60 sec, displacing the reservoir and causing a wave that rose as much as 140 m above reservoir level. The dam itself was only slightly damaged by the wall of water but was rendered useless.

Sliding was apparently occurring along the clay seams, but the actual collapse is believed to have been triggered by artesian pressures and the rising groundwater levels that decreased the effective weight of the sliding mass and, thereby, the resisting force at the toe.

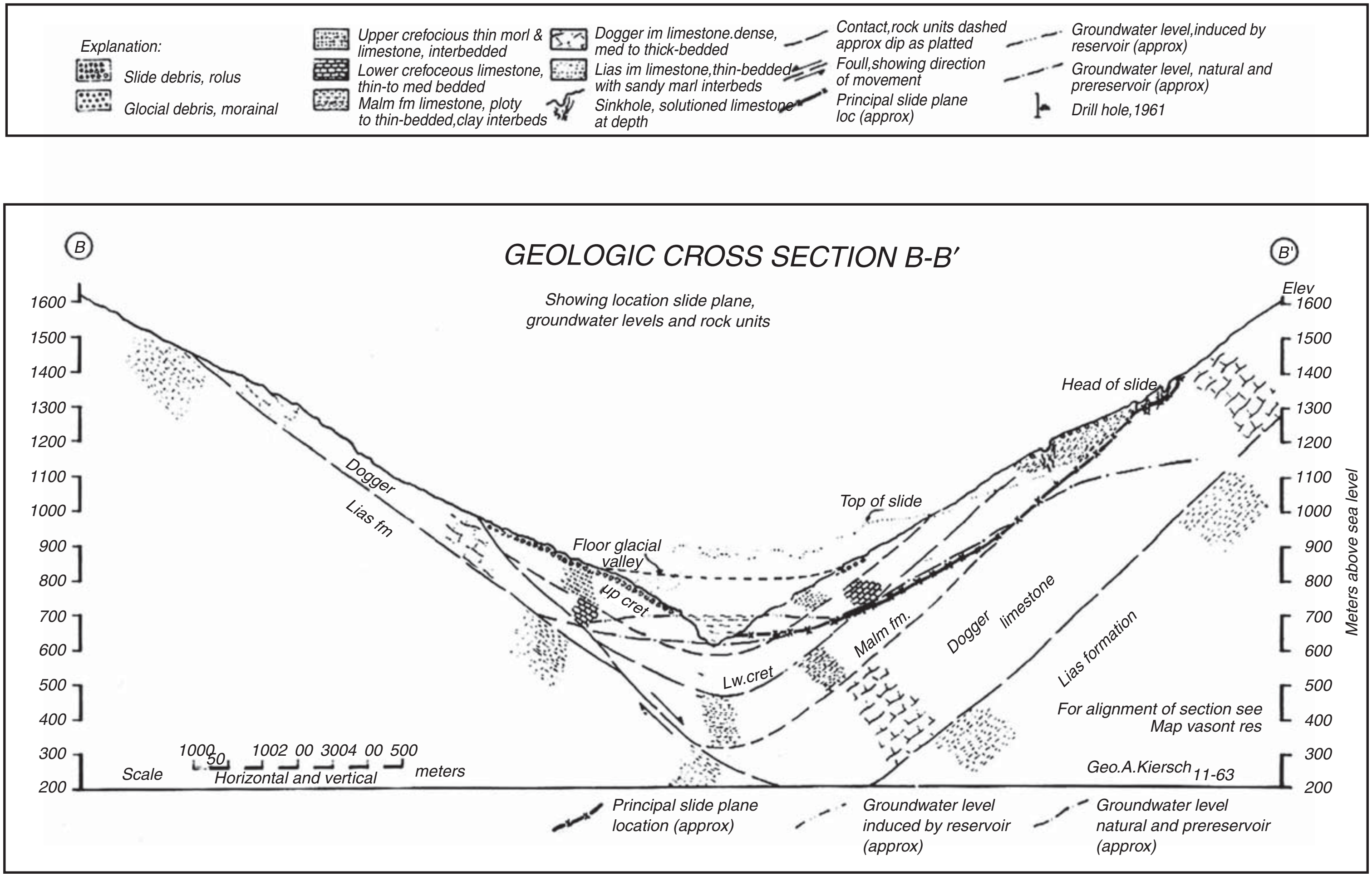


FIGURE 1.20
Geologic section through the Vaiont Reservoir slide. (From Kiersch, G. A., *Focus on Environmental Geology*, Tank, R., Ed., Oxford University Press, New York [1973], Section 17, 1965, pp. 153–164. With permission.)

1.2.4 Rotational Slides in Rock

General

In the rotational slide form, a spoon-shaped mass begins failure by rotation along a cylindrical rupture surface; cracks appear at the head of the unstable area, and bulging appears at the toe as the mass slumps (Figure 1.2a). At final failure, the mass has displaced substantially, and a scarp remains at the head (see Section 1.2.5 for nomenclature). The major causes are an increase in slope inclination, weathering, and seepage forces.

Recognition

Rotational slides are essentially unknown in hard-rock formations, but are common in marine shales and other soft rocks, and in heavily jointed stratified sedimentary rocks with weak beds.

Marine shales, with their characteristic expansive properties and highly fractured structure, are very susceptible to slump failures, and their wide geographic distribution makes such failures common. Natural slope angles are low, about 8 to 15°, and stabilization is often difficult. Failure is often progressive and can develop into large moving masses (see Section 1.2.6).

Stratified sedimentary rocks can on occasion result in large slides, and in humid climates slope failures can be common (Hamel, 1980) (see Example below).

Surface features before total failure are tension cracks; after total failure, a head scarp remains along with spoon-shaped slump topography (see Section 1.2.5).

Example of Major Failure

Event

At the Brilliant cut, Pittsburgh, Pennsylvania, on March 20, 1941, a rotational slide involving 120,000 yd^3 of material displaced three sets of railroad tracks and caused a train to be derailed (Hamel, 1972). A plan of the slide area is given in Figure 1.21b.

Geological conditions are illustrated on the section given in Figure 1.21a. The basal stratum, Zone 1, is described as "soft clay shale and indurated clay (a massive slickensided claystone)." The Birmingham shale of Zone 4 is heavily jointed vertically.

Slide History

In the 1930s, a large tension crack opened at the top of the slope. Sealing with concrete to prevent infiltration was unsuccessful in stopping movement and the crack continued to open over a period of several years. The rainfall that entered the slope through the vertical fractures normally drained from the slope along pervious horizontal beds. On the day of failure, which followed a week of rainfall, the horizontal passages were blocked with ice. Hamel (1972) concluded that final failure was caused by water pressure in the mass, and the failure surface was largely defined by the existing crack at the top of the slope and the weak basal stratum.

1.2.5 Rotational Slides in Soils

General

A common form of sliding in soil formations is the rotation about some axis of one or more blocks bounded by a more or less cylindrical failure surface (Figures 1.3a–c). The major causes are seepage forces and increased slope inclination, and relict structures in residual soils.

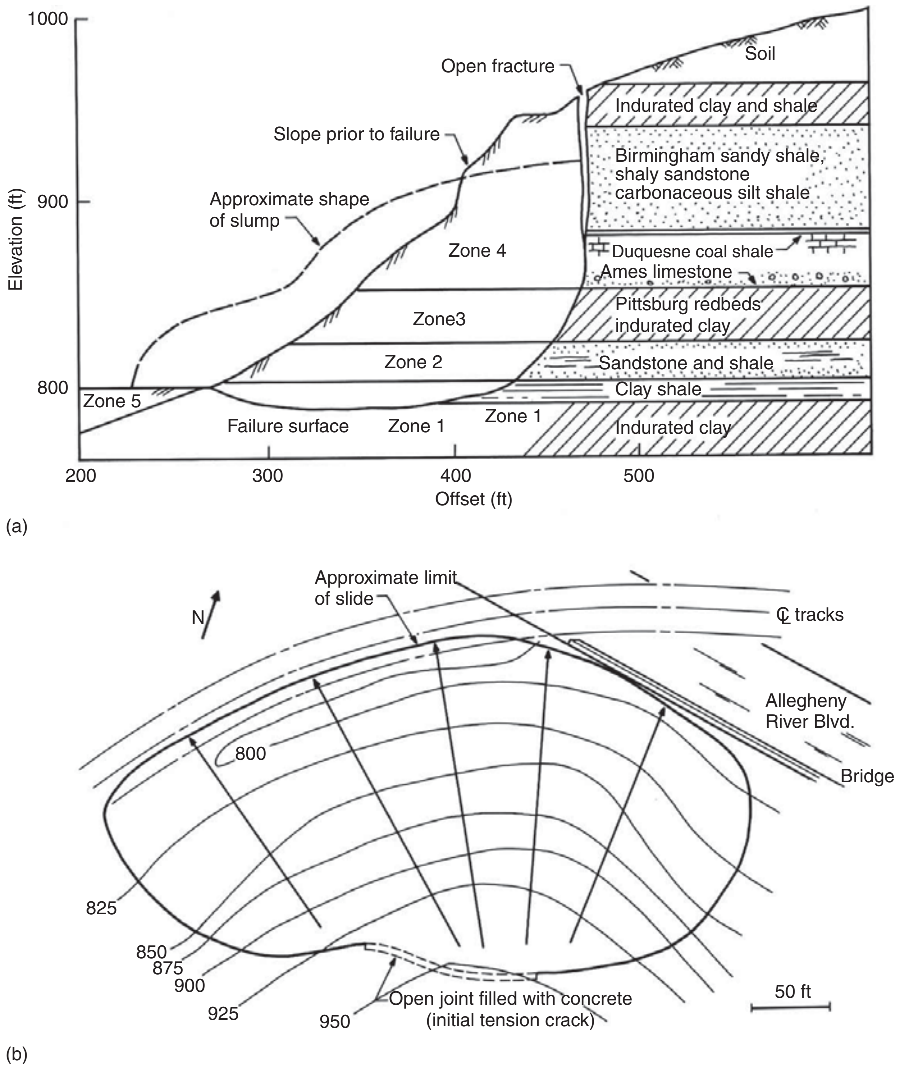


FIGURE 1.21
Rotational slide in rock at Brilliant Cut (Pittsburgh, Pennsylvania, March 28, 1941). (a) Generalized section through Brilliant Cut; (b) plan of slide area. (From Hamel, J. V., *Proceedings of the ASCE, 13th Symposium on Rock Mechanics*, Urbana, Illinois [1971], 1972, pp. 487–572. With permission.)

Usually, neither the volume of mass involved nor the distance moved is great; therefore, the consequences are seldom catastrophic although slump slides cause substantial damage to structures. If their warning signs are recognized they usually can be stabilized or corrected.

Recognition

Occurrence

Slump or rotation slides are characteristic of relatively thick deposits of cohesive soils without a major weakness plane to cause a planar failure. The depth of the failure surface varies with geology.

Deep-seated failure surfaces are common in soft to firm clays and glaciolacustrine, and glaciomarine soils. Deep to shallow failure surfaces are common in residual soils, depending on the strength increase with depth and relict rock defects. Relatively shallow failure surfaces are characteristic of colluvial soils.

Surface Features

During *early failure stages* tension cracks begin to form as shown in Figure 1.22 and Figure 1.23. After *partial failure*, in a progressive mode, the slope exists as a series of small slumps and scarps with a toe bulge as shown in Figure 1.24, or it may rest with a single large scarp and a toe bulge as illustrated in Figure 1.25(a). After *total failure*, surface features include a large head scarp and a mass of incoherent material at the toe as shown in Figure 1.25(b) and Figure 1.93.

Slump landforms remaining after total failures provide forewarning of generally unstable slope conditions. They include spoon-shaped irregular landforms, as seen from the air (Figure 1.26 and Figure 1.27), cylindrical scarps along terraces and water courses, and hummocky and irregular surfaces, as seen from the ground (Figure 1.28 and Figure 1.29). In the stereo-pair of aerial photos shown in Figure 1.26, the slump failure mass has stabilized temporarily but probably will reactivate when higher than normal seasonal rainfall arrives. A small recent failure scar exists along the road in the center of the slide mass. The rounded features of the mass, resulting from weathering, and vegetation growth indicate that the slide is probably 10 to 15 years old, or more. In the photo, it can be seen that the steep highway cut on the opposite side of the valley appears stable, indicative of different geologic conditions. In general, the geology consists of residual soils derived from metamorphic rocks in a subtropical climate. In Figure 1.27, an old slump scar in residual soils, weathering has strongly modified the features. In the photo, the tongue lobe at the intersection of the trails and the creep ridges are to be noted. The location is near the slide of Figure 1.26.

1.2.6 Lateral Spreading and Progressive Failure

General

Failure by lateral spreading is a form of planar failure which occurs in both soil and rock masses. In general, the mass strains along a planar surface, such as shown in Figure 1.3e, represent a weak zone. Eventually, blocks progressively break free as movement retrogresses toward the head. The major causes are seepage forces, increased slope inclination and height, and erosion at the toe.

Failure in this mode is essentially unpredictable by mathematical analysis, since one cannot know at what point the first tension crack will appear, forming the first block. Nevertheless, the conditions for potential instability are recognizable, since they are characteristic of certain soil and rock formations. Failure usually develops gradually, involving large volumes, but can be sudden and disastrous. Under certain conditions, it is unavoidable and uncontrollable from the practical viewpoint, and under other conditions control is difficult at best.

Recognition

The failure mode is common in river valleys, particularly where erosion removes material from the river banks. Characteristically, occurrence is in stiff fissured clays, in clay shales, and in horizontal or slightly dipping strata with a continuous weak zone such as those that occur in glaciolacustrine and glaciomarine soils. Colluvium over gently sloping residual soils or rock also fails progressively in a form of lateral spreading.

Surface features are characterized during the early stages by tension cracks, although failure can be sudden under certain conditions such as earthquake loadings. During the

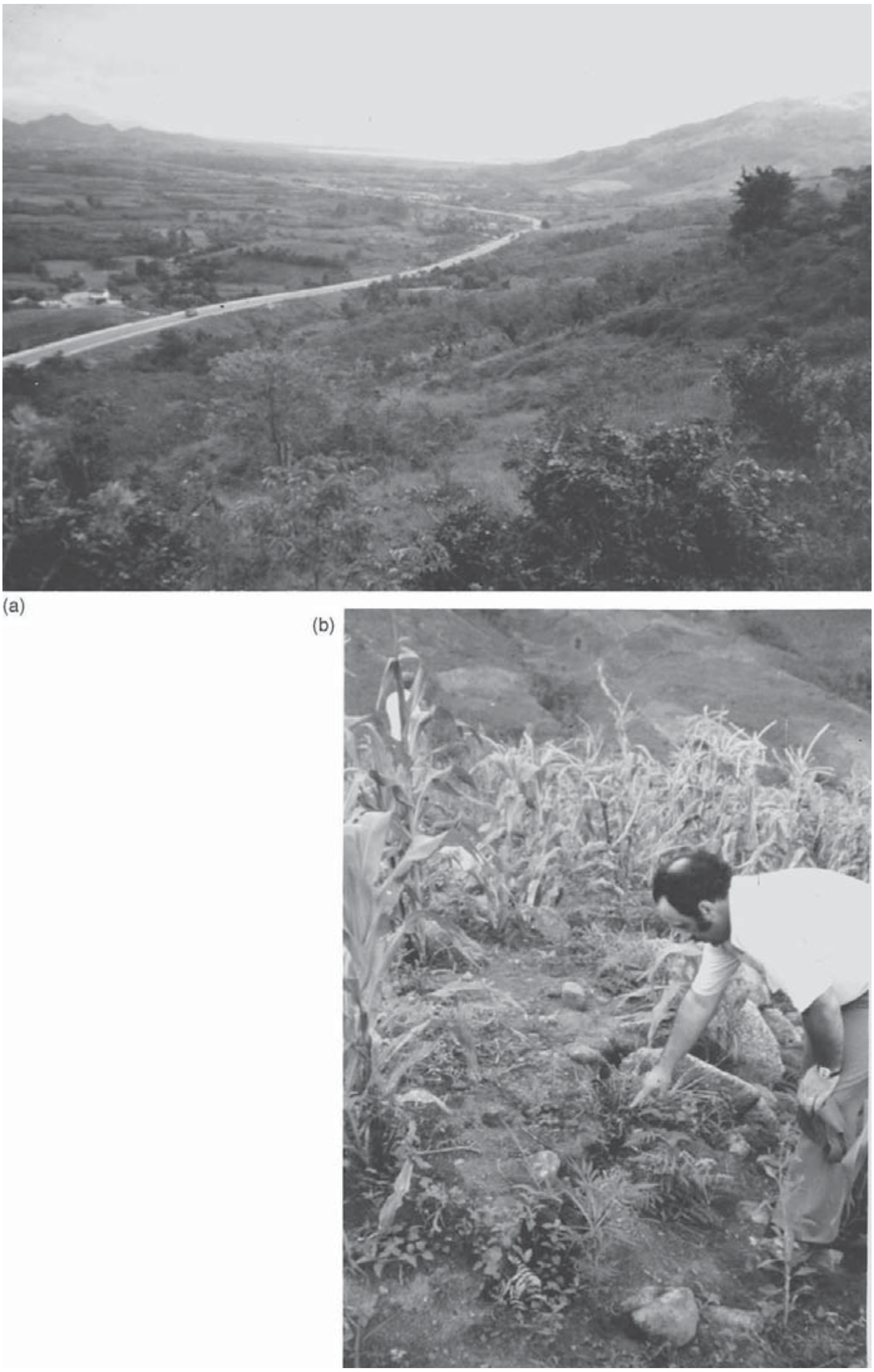


FIGURE 1.22
(a) Small scarp along tension crack appears in photo (middle right). Small highway cut is far below to the left. Scarp appeared after soil was removed from small slump failure at the toe (BR 101, Santa Catarina, Brazil). Movement is in residual soil. If uncorrected, a very large failure will develop. (b) Tension crack in the same slope found in another location.

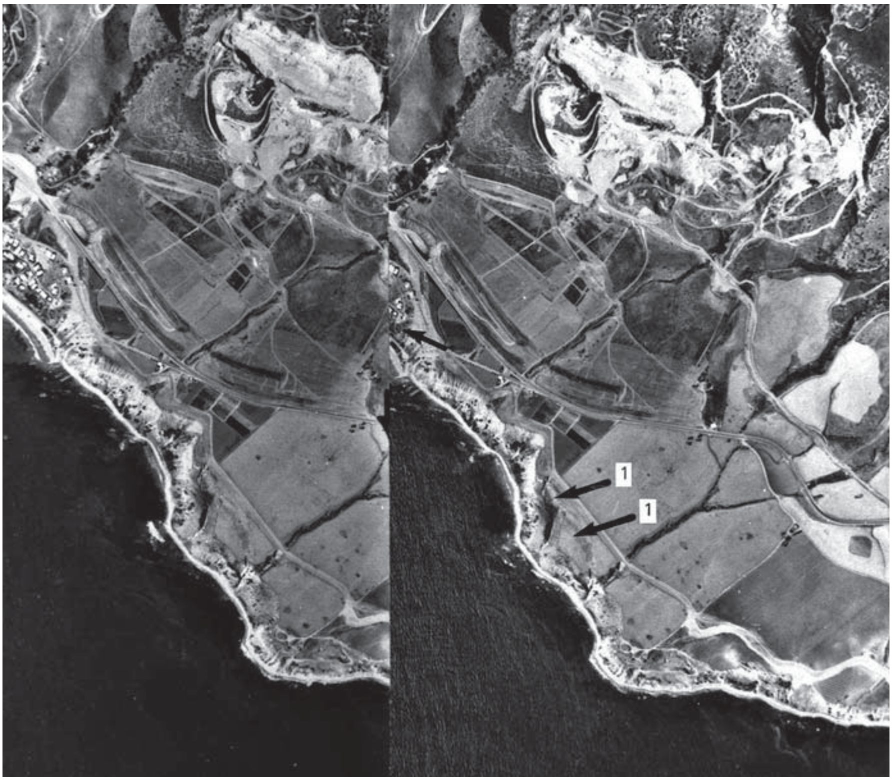


FIGURE 1.23
Stereo-pair of aerial photos showing tension cracks of incipient slides, such as at (1) along the California coast, a short distance from Portuguese Bend.

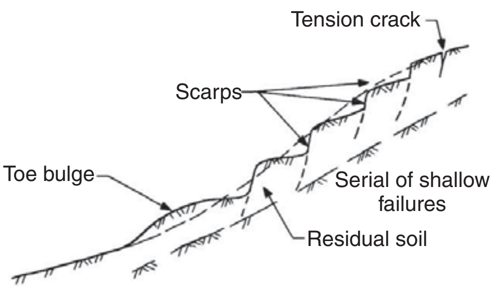


FIGURE 1.24
During the intermediate stage (during partial failure), residual soils often fail progressively, forming a series of slumps in tropical climates. Blocks move downhill during rainy periods and stabilize during dry periods.

progressive failure tension cracks open and scarps form, separating large blocks. The cracks can extend far beyond the slope face when a large mass goes into tension, even affecting surface structures as shown in Figure 1.30. Final failure may not develop for many years, and when it occurs it may be in a form resembling a large slump slide, or it may develop into a flow with individual blocks floating in a highly disturbed mass, depending upon natural conditions as described in the examples below.

Failure Examples

Marine Shales: General

Clay shales, particularly those of marine origin, are susceptible to several modes of slope failures as shown in Figure 1.31, of which progressive failure involves the largest volumes

(a)

(b)

FIGURE 1.25
Slump failure occurred after cutting in fine-grained glacial till (Mountainside, New Jersey). (a) Head scarp, toe bulge, and seepage at the toe. (b) Total failure some weeks later. Slope stabilized by benching, installation of trench drain, and counterberm along the toe.

and can be the most serious from the engineering viewpoint. Their most significant characteristics are their content of montmorillonite and their high degree of overstress. Excavation, either natural or human, results in lateral strains causing the strength along

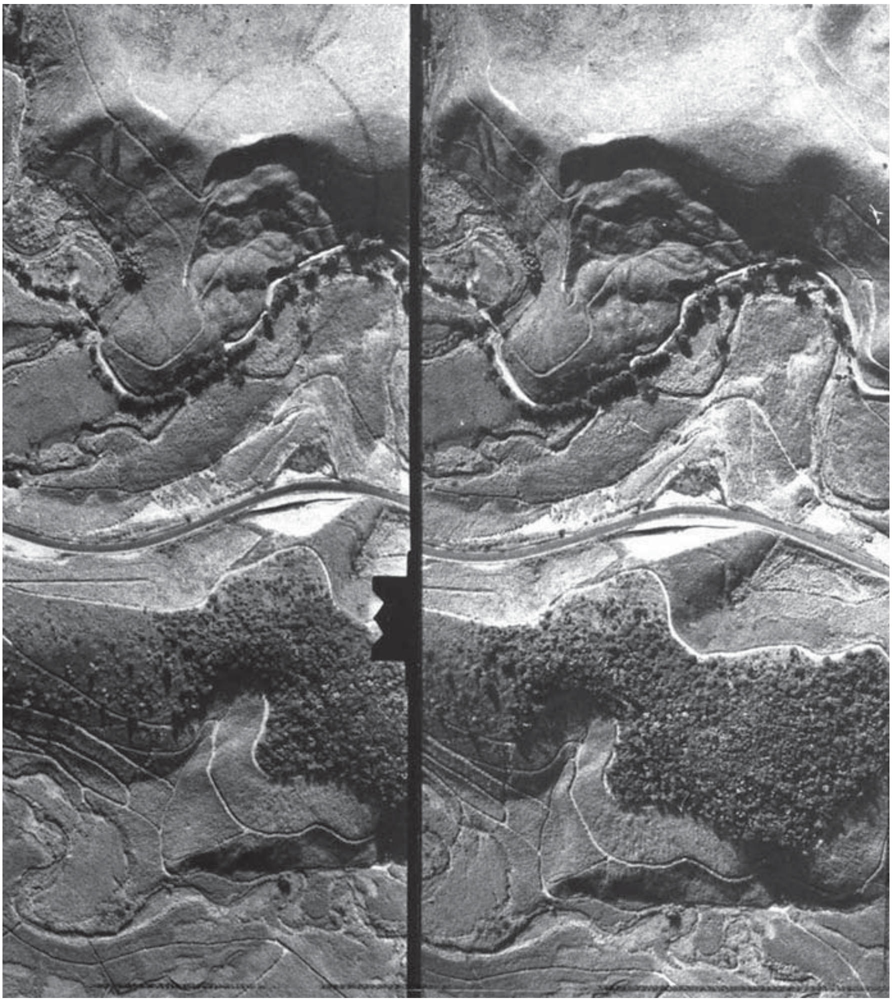


FIGURE 1.26
Stereo-pair of aerial photos showing *slump failure landform* (scale 1:8000). (From Hunt, R. E. and Santiago, W. B., *Proceedings of the 1st Congress Brasileiro de Geologia de Engenharia*, Rio de Janeiro, August, Vol. 1, 1976, pp. 79–98. With permission.)

certain planes to be reduced to residual values. Water entering the mass through open tension cracks and fractures assists in the development of failure conditions.

Marine Shale: Forest City Landslide

The Forest City Landslide, located on the banks of the Oahe Reservoir in South Dakota, includes an area of about 700 acres (Hunt et al., 1993). The hummocky landform, typical of marine shales and a large head scarp, is shown in the aerial oblique of Figure 1.32. Movements toward the reservoir, of the order of several inches or more per year, were causing distress in a large bridge structure. Investigations, including test borings and inclinometer data, identified the main failure surface at depths of the order of 100 ft, extending upslope to the head scarp, a distance of 2200 ft. The approach roadway embankment was moving laterally on shallower failure surfaces. A geologic section is given in Figure 1.33.

FIGURE 1.27
Old slump slide in residual soils located near slide in Figure 1.26.

FIGURE 1.28
Slump landform in glaciolacustrine soils showing shallow slopes, creep ridges, and seepage (Barton River, Vermont). Trees in upper left are growing on slide area. Slope failures are common in this region in the spring when the ground thaws and rains arrive.

FIGURE 1.29
Slump-slide landform (valley of the Rio Choqueyapa, La Paz, Bolivia). High center scarp in strong sands and gravels remains after failure of underlying lacustrine soils. Slopes were extremely unstable prior to channelization of the river, because of river erosion and flood stages. Grading of old slide in upper left is not arresting slope movements as evidenced by cracks in new highway retaining wall (not apparent in photo). Slope failures continue to occur from time to time throughout the valley (photo taken in 1973).

FIGURE 1.30
One-year-old church being split into half from slope movements although located over 1 km from the slope shown in Figure 1.29 (La Paz, Bolivia, 1972).

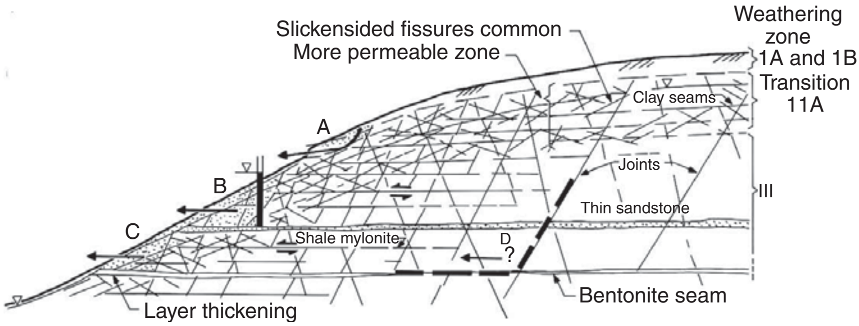


FIGURE 1.31
Failure forms in weathered clay shales: (A) surface slump in shallow weathered zone; (B) wedge failure along joints and sandstone seam; (C) wedge failure along thin bentonite seam may develop into large progressive failure to (D) or beyond. (From Deere, D. U., and Patton, F. D., *Proceedings of ASCE, 4th Pan American Conference on Soil Mechanics and Foundation Engineering*, San Juan, P. R., 1971, pp. 87–170. With permission.)

FIGURE 1.32
Aerial oblique, Forest City Landslide, South Dakota. Note the head scarp and hummocky landform. (Photo by Vermon Bump, SDDOT.)

Slope failures probably began in early postglacial times when the Missouri River incised its channel. Modern reactivation was caused by the filling of the valley with the reservoir, and subsequent relatively rapid changes in reservoir water levels. The failing mass consisted of a number of blocks, evidenced by surface tension cracks.

Stabilization of the overall sliding mass was essentially achieved by excavating a large cut at the escarpment at the head and relocating the approach roadway into the cut. The approach embankment, failing separately, was remediated by the installation of reinforced concrete "dowels."

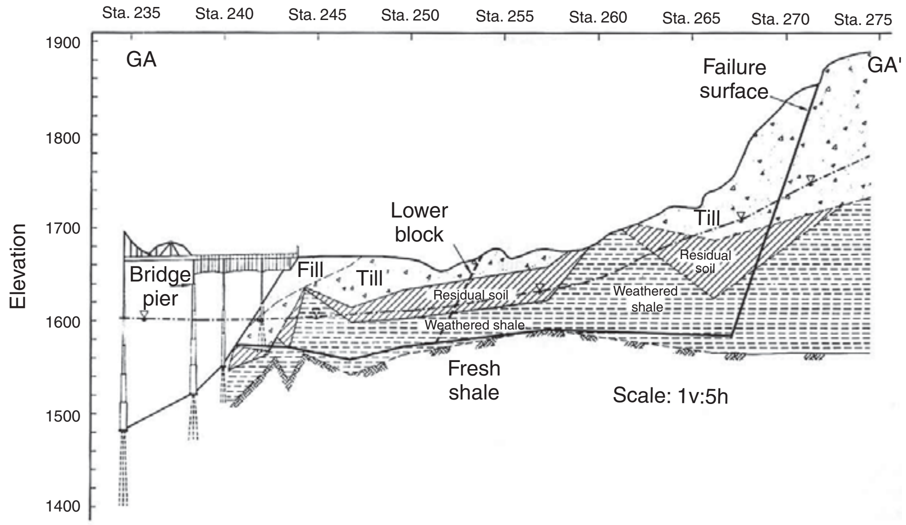


FIGURE 1.33
Geologic section, Forest City Landslide, South Dakota (From Hunt, R. E. et al., *3rd International Conference, Case Histories in Geotechnical Engineering*, St. Louis, MD, 1993. With permission.)

Marine Shale: Panama Canal Slides

Event: Massive slides occurred during 1907 and 1915 in the excavation for the Panama Canal in the Culebra Cut (Binger, 1948; Banks, 1972).

Geology: On the plan view of the slide areas (Figure 1.34), the irregular to gentle topography of the Cucaracha formation (Tertiary) is apparent. The Cucaracha is a montmorillonitic shale with minor interbedding of sandstone and siltstone more or less horizontally bedded but occasionally dipping and emerging from natural slopes. It is heavily jointed and slickensided, and some fractures show secondary mineral fillings. Natural slopes in the valley were relatively gentle, as shown on the geologic section given in Figure 1.35, generally about 20° or less. Laboratory consolidation tests gave values for preconsolidation pressure as high as 200 tsf.

Slide history: Excavations of the order of 300 ft in depth were required in the Cucaracha formation. Some minor sliding occurred as the initial excavations were made on slopes of 1:1 through the upper weathered zones to depths of about 50 ft. The famous slides began to occur when excavations reached about 100 ft. They were characterized by a buckling and heaving of the excavation floor, at times as great as 50 ft; a lowering of the adjacent ground surface upslope; and substantial slope movements. Continued excavation resulted in progressive sliding on a failure surface extending back from the cut as far as 1000 ft. The causes of the sliding are believed to be stress relief in the horizontal direction, followed by the expansion of the shale, and finally rupture along a shallow arc surface (Binger, 1948).

Analysis: Banks (1972) found that at initial failure conditions, the effective strength envelope yielded $\phi = 19°$ and $c' \approx 0$. For the case of an infinite slope (see Section 1.3.2) without slope seepage these values would produce a stable slope angle of 19°, or for the case of seepage parallel and coincident with the slope face, $1/2\phi$, or 1.5°. Since movements had occurred, the value 9.5° is considered to be the residual strength.

Solution: The slides were finally arrested by massive excavation and cutting the slopes back to 9.5° ($1/2\phi_r$), which is flatter than the natural slopes. Banks reported that measurements with slope inclinometers indicated that movement was still occurring in 1969, and that the depth of sliding was at an elevation near the canal bottom.

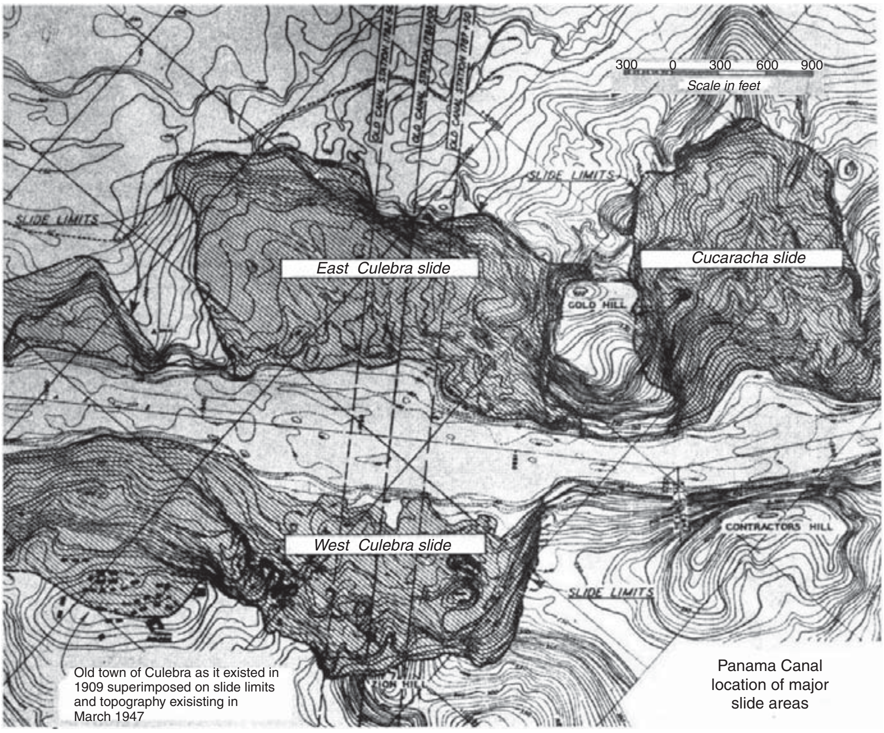


FIGURE 1.34
Plan view of slides and topography, Culebra Cut, Panama Canal. (From Binger, W. V., *Proceedings of the 2nd International Conference on Soil Mechanics and Foundation Engineering*, Rotterdam, Vol. 2, 1948, pp. 54–60. With permission.)

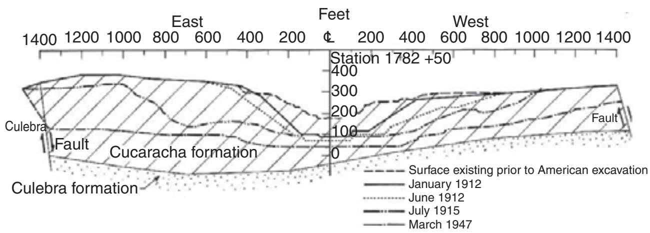


FIGURE 1.35
Sketch of east and west Culebra slides (Panama Canal) showing progress of slide movement: Cucaracha tuffaceous shale and Culebra tuffaceous shale, siltstone, and sandstone. (From Binger, W. V., *Proceedings of the 2nd International Conference on Soil Mechanics and Foundation Engineering*, Rotterdom, Vol. 2, 1948, pp. 54–60. With permission.)

Coastal Plain Sediments: Portuguese Bend Slide

Event: At Portuguese Bend, Palos Verdes Hills, California (see Figure 2.3 for location), a slide complex with a maximum width of roughly 4000 ft and a head-to-toe length of about 4600 ft began moving significantly in 1956 and, as of 1984, was still moving. Coastal plain sediments are involved, primarily marine shales. This slide may be classified as progressive block glides or failure by lateral spreading. It is one of the most studied active slides in the United States (Jahns and Vonder Linden, 1973).

Physiography: The limit of the slide area is shown in Figure 1.36, and the irregular hummocky topography is shown on the stereo-pair of aerial photos in Figure 1.37. In the slide

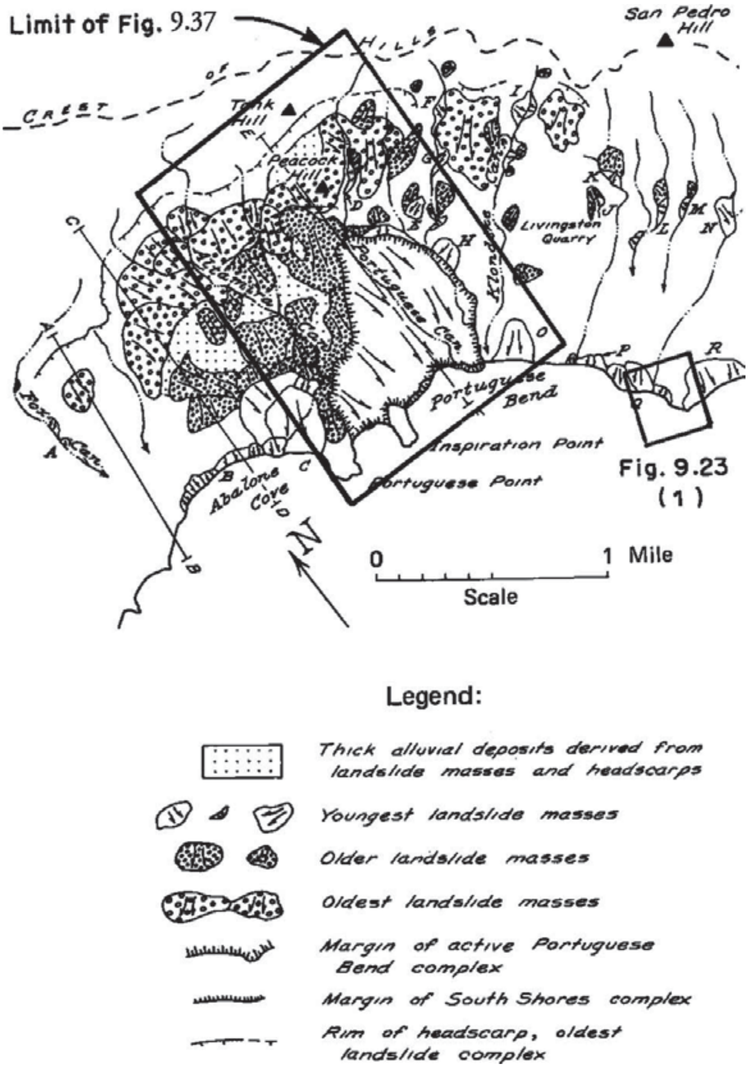


FIGURE 1.36
Distribution of principal landslides and landslide complexes in Palos Verdes Hills, California. (From Jahns, R. H. and Vonder Linden, C., *Geology, Seismicity and Environmental Impact*, Special Publication Association Engineering Geology, Los Angeles, 1973, pp. 123–138. With permission.)

area, the land rises from the sea in a series of gently rolling hills and terraces to more than 600 ft above sea level. The hills beyond the slide area rise to elevations above 1200 ft and the cliffs along the oceanfront are roughly 150 ft above the sea. A panoramic view of the slide is given in Figure 1.38.

Geology: The slide zone occurs in Miocene sediments of heavily tuffaceous and sandy clays interbedded with relatively thin strata of bentonitic clays. When undisturbed, the beds dip seaward at about 10 to 20°, which more or less conforms with the land surface as illustrated on the section (Figure 1.39). A badly crushed zone of indurated clayey silt forming a soil "breccia" (Figure 1.40) is found in the lower portions of the slide area. Present movement of the slide appears to be seated at a depth of about 100 ft below the surface in the "Portuguese tuff," originally deposited as a marine ash flow.

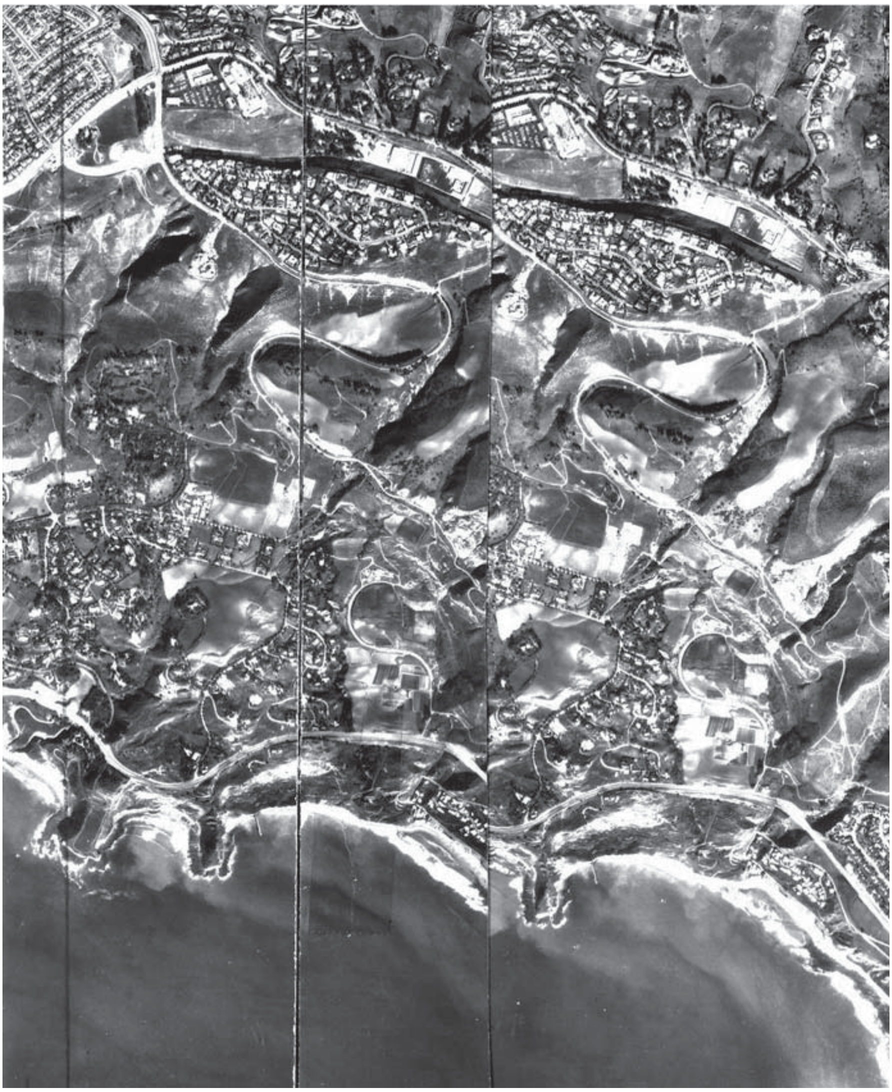


FIGURE 1.37
Stereo-pair of aerial photos of the Portuguese Bend area of the Palos Verdes Hills (January 14, 1973, scale 1:24,000).

Slide history: The area has been identified as one with ancient slide activity (see Figure 1.36). Using radiometric techniques, colluvium older than 250,000 years has been dated, and intermediate activity dated at 95,000 years ago. In recent times some block movement was noted in 1929, but during the 1950s, when housing development began on the present slide surface, the slide was considered as inactive. Significant modern movement began in 1956, apparently triggered by loading the headward area of the slide with construction fill for a roadway.

Slide movements: The mass began moving initially during 1956 and 1957, at rates of 2 to 5 in./year, continuing at rates varying from 6 to 24 in./year during 1958, then 3 to 10 ft/year during 1961 to 1968. After 1968, a significant increase in movement occurred. Eventually, 120 houses were destroyed over a 300 acre area. Studies have correlated acceleration in rate of advance with earthquake activity, abnormally high tides, and rainfall (Easton, 1973). The average movement in 1973 ranged from about 3 in./day during the dry season, to 4 in./day

FIGURE 1.38
Panorama of the Portuguese Bend landslide looking south. The highway on the left is continually moving, and the old abandoned road appears in the photo center. The broken ground on the right is the head scarp of rotational slides in the frontal lobe of the unstable mass (photo taken in 1973).

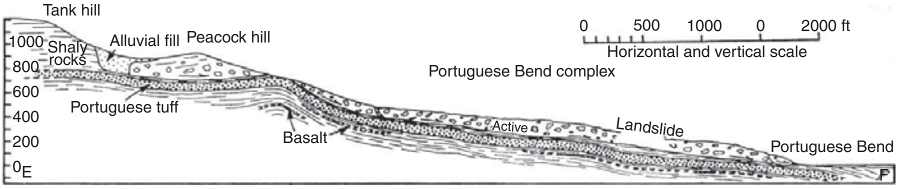


FIGURE 1.39
Geologic section through the Portuguese Bend landslide. For location see Figure 1.36, section along lines e–f. (From Jahns, R. H. and Vonder Linden, C., *Geology, Seismicity and Environmental Impact*, Special Publication Association Engineering Geology, Los Angeles, 1973, pp. 123–138. With permission.)

during the rainy season, to peaks of 6 in./day during heavy rains. Rainfall penetrating deeply into the mass through the many large tension cracks builds up considerable hydrostatic head to act as a driving force on the unstable blocks supported by material undoubtedly at residual strength. The maximum horizontal displacement between 1968 and 1970 was about 130 ft and the maximum vertical displacement about 40 ft. An interesting feature of the slide is the gradual and continuous movement without the event of total collapse.

Stabilization: Because of the large area involved and the geologic and other natural conditions, there appears to be no practical method of arresting slide movements. The cracks on the surface are too extensive to consider sealing to prevent rainwater infiltration, and the strength of the tuff layer is now inadequate to restrain gravity movement even during the dry season. A possible solution to provide stability might be to increase the shearing resistance of the tuff by chemical injection. Since this would be extremely costly, it appears prudent to leave the unstable area as open space although continuous maintenance of the roadway in Figure 1.38 has been necessary.

Glaciolacustrine Soils

Glaciolacustrine soils composing slopes above river valleys normally are heavily overconsolidated. Shear strengths, as measured in the laboratory, are often high, with cohesion ranging from 1 to 4 tsf. Therefore, these soils would not usually be expected to be slide-prone on moderately shallow slopes, and normal stability analysis would yield an adequate factor of safety against sliding (Bjerrum, 1966). Sliding is common, however, and often large in scale, even on shallow slopes.

FIGURE 1.40
Soil "breccia" of fragments of indurated clayey silt in a crushed uplifted zone at Portuguese Bend.

In the Seattle Freeway slides (Figure 1.41), failure occurred along old bedding plane shears associated with lateral expansion of the mass toward the slopes when the glacial ice in the valley against the slopes disappeared (Palladino and Peck, 1972). Similar conditions probably existed at the site of the slide occurring at Kingston, New York, in the Hudson River Valley in August 1915 (Terzaghi, 1950). The Kingston slide was preceded by a period of unusually heavy rainfall. Factors contributing to failure, as postulated by Terzaghi, were the accumulation of stockpiles of crushed rock along the upper edge of the slope and perhaps the deforestation of outcrops of the aquifer underlying the varved clays which permitted an increase in pore-water pressures along the failure surface.

Glaciomarine Soils: South Nation River Slide

Event: The South Nation River slide in Casselman, Ontario, of May 16, 1971, is typical of many slides occurring in the sensitive Champlain clays of glaciomarine origin, in Quebec province, Canada (Eden et al., 1971). These clays are distributed in a broad belt along the St. Lawrence River and up the reaches of the Saguenay River. Most of the slides occur along riverbanks, commencing as either a slump or block glide and retrogressing through either slumping or lateral spreading. At times, the frontal lobes of the slides liquefy and become flows (Figure 1.4f) (see Section 1.2.11).

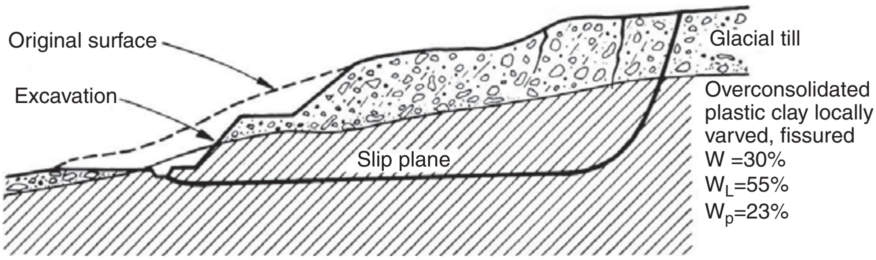


FIGURE 1.41
Failure surface in overconsolidated, fissured clays, undergoing progressive failure as determined by slope inclinometer measurements along the Seattle Freeway. (From Bjerrum, L., *Terzaghi Lectures 1963-1972, ASCE* [1974], 1966, pp. 139–189. With permission.)

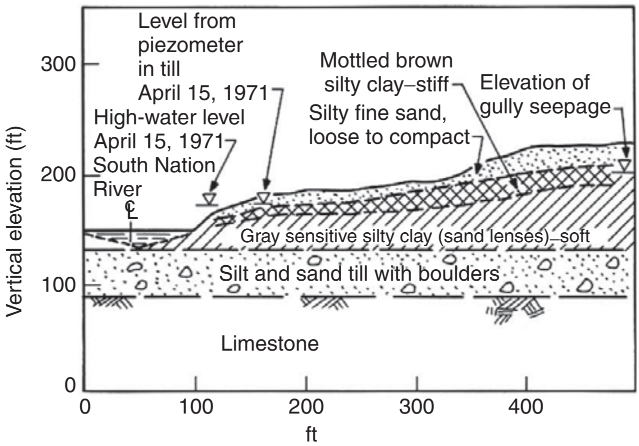


FIGURE 1.42
Stratigraphy prior to the South Nation River slide. An overconsolidated crust over soft, sensitive clays. (From Eden, W. J. et al., *Can. Geotech. J.*, 8, 1971. Reprinted with permission of the National Research Council of Canada.)

Geology: The stratigraphy at the South Nation River slide prior to failure consisted of 6 to 23 ft of stratified silty fine sands overlying the Champlain clay (Leda clay) as shown in Figure 1.42. The undrained strength of the clay was about 0.5 tsf, its sensitivity ranged from 10 to 100, the average plastic limit was 30%, and liquid limit 70%, and the natural water content was at the liquid limit.

Slide history: An all-time record snowfall of 170 in. (432 cm) occurred during the 1970 to 1971 winter and gradual melting resulted in saturation of the upper clays. The slide occurred at the end of the snow-melting season during a heavy rainstorm. A contributing factor was the river level at the slide toe area. It had risen as much as 30 ft during spring floods, remained at that level for a week, then dropped back rapidly to preflood levels. At the time of the slide, groundwater at the lower part of the slope was observed to be nearly coincident with the surface. From the appearance of the ground after failure, it appears that the slide retrogressed as a series of slumps as shown in Figure 1.43.

Glaciomarine Soils: Turnagain Heights Slide

Event: Much of the damage to the Anchorage, Alaska, area from the March 1964 earthquake was caused by landslides induced by seismic forces. The slides occurred in the city in the Ship Creek area and along the waterfront formed by the Knik Arm of Cook Inlet. The largest slide occurred at Turnagain Heights, a bluff some 60 ft high overlooking Knik Arm. Many homes were destroyed in the slide area of 125 acres as illustrated in Figure 1.44. The slide at Turnagain Heights is an example of sliding along horizontal strata. It was planar and

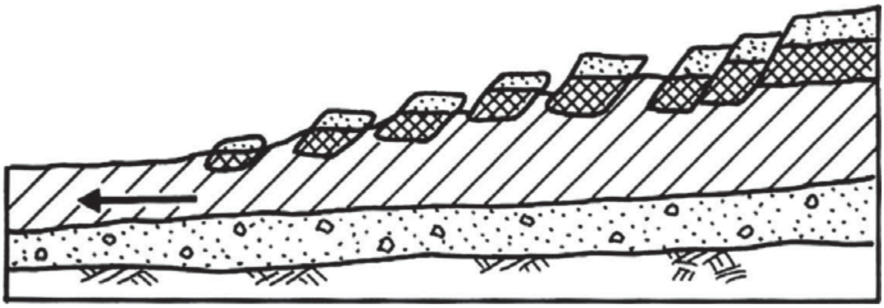


FIGURE 1.43
Stratigraphy after slide at South Nation River. Blocks broke loose and moved by lateral spreading. (After Mollard, J. D., *Reviews in Engineering Geology*, Vol. III, *Landslides*, Geological Society of America, 1977, pp. 29–56.)

FIGURE 1.44
Failure by block gliding and lateral spreading resulting from the 1964 earthquake, Turnagain Heights, Anchorage, Alaska. (Photo courtesy of U.S. Geological Survey, Anchorage.)

evolved by block gliding or slump failure at the bluff, followed by lateral spreading of the mass for a width of 8000 ft, and extending as much as 900 ft inland.

Geology: Anchorage and the surrounding area are underlain by the Bootlegger Cove clay of glaciomarine origin. Soil stratigraphy at the bluff consisted of a thin layer of sand and gravel overlying a clay stratum over 100 ft thick as shown in Figure 1.45a. The consistency of the upper portions of the clay was stiff to medium, becoming soft at a depth of about 50 ft. The soft zone extended to a depth of about 23 ft below sea level. Layers of silt and fine sand were present at depths of a several feet or so above sea level.

Slide history: Seed and Wilson (1967) postulated that cyclic loading induced by the earthquake caused liquefaction of the silt and fine sand lenses resulting in instability and block gliding along the bluff. Blocks continued to break loose and glide retrogressively, resulting

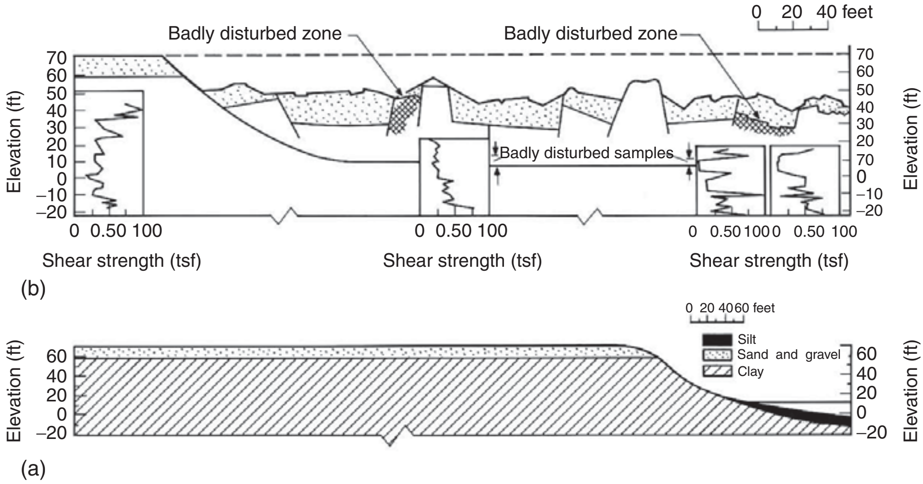


FIGURE 1.45
Soil profiles through east end of slide area at Turnagain Heights (a) before and (b) after failure. (From Seed, H.B. and Wilson, S.D., *Proceedings of ASCE, J. Soil Mechanics Foundation Engineering Division,* Vol. 93, 1967, pp. 325–353. With permission.)

in lateral spreading of the mass which came to rest with a profile more or less as illustrated in Figure 1.45b. The movement continued for the duration of the earthquake (more than 3 min), but essentially stopped once strong ground motion ceased.

Conclusions: The magnitude of the 1964 event was 8.5 (Richter) with an epicenter 80 mi east of Anchorage. Previous earthquakes of slightly lower magnitudes but closer epicenters had occurred, but the Turnagain Heights area had not been affected (see Section 3.3.4). Seed and Wilson (1967) concluded that, in light of previous earthquake history, the slide was the result of a continuous increase in pore pressures caused by the long duration of the 1964 event; and, that it is extremely unlikely that any analysis would have anticipated the extent of inland transgression of the failure. Considering the local stratigraphy and seismic activity, however, the area certainly should be considered as one with a high slope failure hazard.

1.2.7 Debris Slides

General

Debris slides involve a mass of soil, or soil and rock fragments, moving as a unit or a number of units along a steeply dipping planar surface. They often occur progressively and can develop into avalanches or flows. Major causes are increased seepage forces and slope inclination, and the incidence is increased substantially by stripping vegetation. Very large masses can be involved, with gradually developing progressive movements, but at times total failure of a single block can occur suddenly.

Recognition

Occurrence is common in colluvial or residual soils overlying a relatively shallow, dipping rock surface. During the initial stages of development, tension cracks are commonly formed. After partial failure, the tension cracks widen and the complete dislodgment of one or more blocks may occur, often leaving a clean rock surface and an elliptical failure scar as shown in Figure 1.46. Total failure can be said to have occurred when the failure

FIGURE 1.46
Small debris-slide scar along the Rio Santos Highway, Brazil. Note seepage along the rock surface, the sliding plane. Failure involved colluvium and the part of the underlying rock. The area subsequently was stabilized by a concrete wall.

surface reaches to the crest of the hill. If uncorrected, failure often progresses upslope as blocks break loose.

Examples of Major Failures

Pipe Organ Slide

Description: The Pipe Organ slide in Montana (Noble, 1973) triggered by a railroad cut made at the toe involved about 9 million yd^3 of earth. The failure surface developed at depths below the surface of 120 to 160 ft in a Tertiary colluvium of stiff clay containing rock fragments, which overlay a porous limestone formation.

Slide movements: During sliding, movement continued for a year at an average rate of 2 in./week, developing in a progressive mode. Total movement was about 12 ft and the length of the sliding mass was 2000 ft along the flatter portions of a mountain slope.

Stabilization: Movement was arrested by the installation of pumped wells, which were drilled into the porous limestone and later converted into gravity drains. The water perched on the sliding surface near the interface between the colluvium and the limestone drained readily into the limestone and the hydrostatic pressures were relieved.

Golden Slide

Description: A railroad cut into colluvium of about 50 ft thick caused a large slide near Golden, Colorado (Noble, 1973). The colluvium is an overconsolidated clay with fragments of clay shale and basalt overlying a very hard blue-gray clayey siltstone. The water table was midway between the ground surface and the failure surface, and there was evidence of artesian pressure at the head of the slide.

Slide movements: Movement apparently began with a heavy rainfall and was about 1 in./day. Tension cracks developed in the surface and progressed upslope with time. About 500,000 yd^3 of material were moving within a length of about 1000 ft.

Stabilization: Cutting material from the head of the slide and placing approximately 100,000 yd^3 against a retaining wall at the toe that penetrated into underlying sound rock was unsuccessful. Movement was finally arrested by the installation of horizontal drains as long as 400 ft, and vertical wells which were being pumped daily at the time that Noble (1973) prepared his paper.

Colluvium on Shale Slopes

The Pennington shale of the Cumberland plateau in Tennessee and the sedimentary strata in the Appalachian plateau of western Pennsylvania develop thick colluvial overburden, which is the source of many slide problems in cuts and side-hill fills. The geology, nature of slope problems, and solutions are described in detail by Royster (1973, 1979) and Hamel (1980).

1.2.8 Debris Avalanches

General

Debris avalanches are very rapid movements of soil and rock debris which may, or may not, begin with rupture along a failure surface. All vegetation and loose soil and rock material may be scoured from a rock surface as shown in Figure 1.7. Major causes are high seepage forces, heavy rains, snowmelts, snowslides, earthquakes, and the creep and gradual yielding of rock strata.

Failure is sudden and without warning, and essentially unpredictable except for the recognition that the hazard exists. Effects can be disastrous in built-up areas at the toes of high steep slopes under suitable geologic conditions (see Examples below).

Occurence

Debris avalanches are characteristic of mountainous terrain with steep slopes of residual soils where topography causes runoff concentration (see Figure 1.86), or badly fractured rock such as illustrated in Figure 1.47.

There is usually no initial stage, although occasionally tension cracks may be apparent under some conditions. Total failure occurs suddenly either by a rock mass breaking loose or by "hydraulic excavation" which erodes deep gullies in soil slopes during torrential rains as shown in Figure 1.48. All debris may be scoured from the rock surface and deposited as a terminal lobe at a substantial distance from the slope. As shown in Figure 1.49, the force is adequate to move large boulders, and erosion can cause the failure area to progress laterally to affect substantial areas.

In the Andes Mountains of South America, debris avalanches are the most common form of slope failures. They occur occasionally in natural slopes, not impacted on by construction, but normally are caused by roadway cuts. Typical geologic conditions are shown in Figure 1.50. Illustrated in Figure 1.51 are the steep slopes and irregular topography that result in the necessity for numerous cuts for roadway construction. The slope shown in the photo, Figure 1.52, taken in 2002, was free of slope failures in 1995. Investigation and treatments of slopes in the Andes is discussed in Section 1.5.

Examples of Major Failures

Rio de Janeiro, Brazil

Event: A debris avalanche occurred during torrential rains in 1967 in the Laranjeiras section of Rio de Janeiro, which destroyed houses and two apartment buildings, causing the death of more than 130 persons. The avalanche scar and a new retaining wall are shown in Figure 1.53.

FIGURE 1.47
Scarred surface remaining after a rock and debris avalanche in a limestone quarry. The rock is heavily jointed with sets oriented more or less parallel to the slope and across the bedding plane. Failure was induced by wedging from water and ice pressures, and occurred in the early spring.

Climatic conditions: Hundreds of avalanches and slides occurred in Rio de Janeiro and the nearby mountains during the unusually heavy rains of 1966 and 1967 when intensities as high as 200 mm/h (8 in./h) were recorded (Jones, 1973).

During a 3-day storm beginning on January 10, 1966, a gaging station at Alto da Boa Vista, in the mountains a few kilometers from the city, recorded 675 mm (26.2 in.) of rainfall. It was an unprecedented amount. Although heavy rains occur each year during the summer months of January and February, with rainfall averaging 171 mm (6.7 in.) during January, most of the rainfalls during intense storms. The potential for slope failures is very much dependent upon the accumulated rainfall and associated water-table conditions for a given rainy season (see Section 1.3.4).

Local geology: Typical profiles in the residual soils (see Figure 1.69) along the coastal mountains of Brazil show that these soils are most impervious near the surface and that permeability increases downward through the soil profile into the underlying decomposed and fractured crystalline rocks. Fissures in the outer portions of the residual and colluvial soils close during rainfall; they thereby block drainage and cause a rapid increase in pore pressures, resulting in sudden failure, which combined with high runoff develops into an avalanche or even a flow. To minimize the slope-failure hazard, the city of Rio has zoned some areas of high steep slopes to prohibit construction, and has undertaken the construction of numerous stabilization works throughout the city.

FIGURE 1.48
The force of hydraulic excavation is evident in this photo taken in a typical V-shaped scarred zone of a debris avalanche. Location is near the crest of the hill in Figure 1.49. The bedrock surface is exposed.

FIGURE 1.49
Debris avalanche that covered BR 101 near Tubarao, Santa Catalina, Brazil during torrential rains in 1974. Note the minibus for scale. The debris lobe crossed the highway and continued for a distance of about 200 m and carried boulders several meters in diameter. Debris has been removed from the roadway. It is unlikely that a failure of such a magnitude could have been foreseen for this particular location.

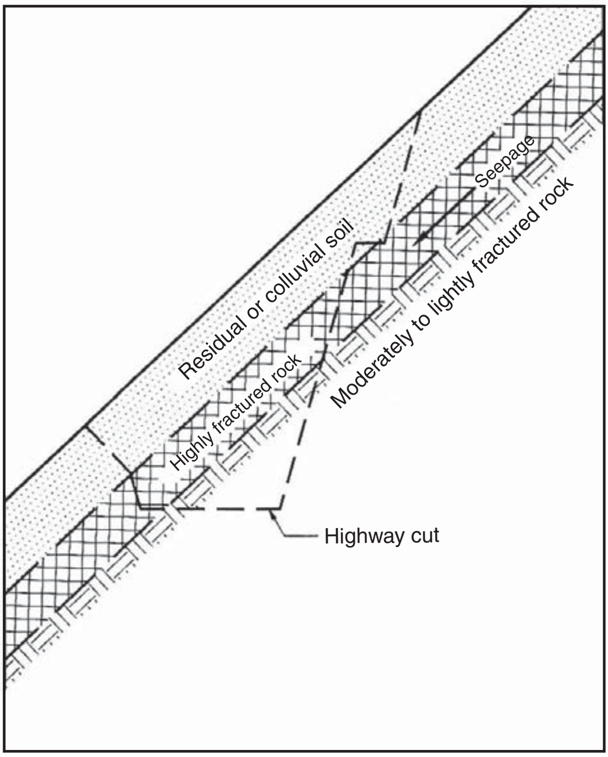


FIGURE 1.50
Typical geologic conditions in the Andes Mountains. Slopes often inclined at 35 to 40°.

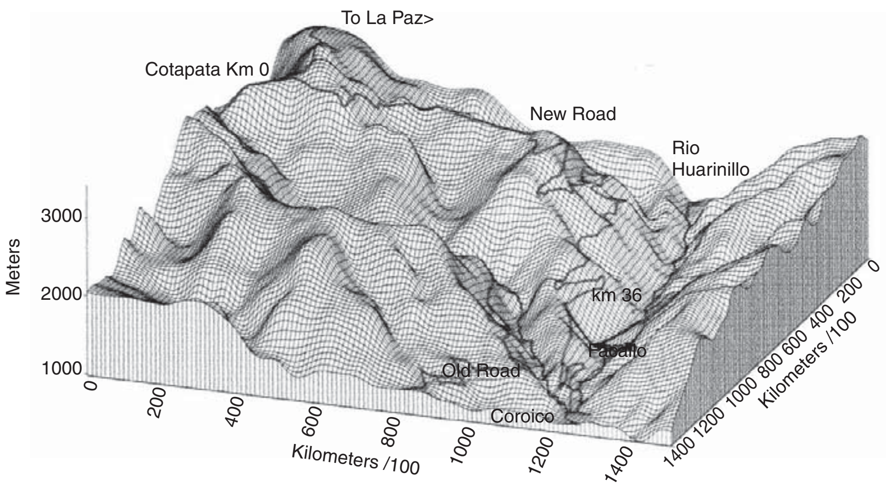


FIGURE 1.51
Three-dimensional diagram of a new roadway in the Andes of Bolivia. Steep slopes and irregular landform results in numerous cuts. See Figure 1.7 for debris avalanches above town of Pacallo, and at km 36, Figure 1.52.

Ranrahirca and Yungay, Peru

Event no. 1: One of the most disastrous debris avalanches in modern history occurred in the Andes Mountains of Peru on January 10, 1962. In a period of 7 min, 3500 lives were lost and seven towns, including Ranrahirca, were buried under a mass of ice, water, and debris (McDonald and Fletcher, 1962). The avalanche began with the collapse of Glacier 512 from

FIGURE 1.52
Debris avalanches at km 36 along the roadway shown in Figure 1.51. There were no slope failures at the beginning of construction in 1995. The photo, taken by the author, shows conditions during 2002, after several years of El Niño. At this time construction was not complete.

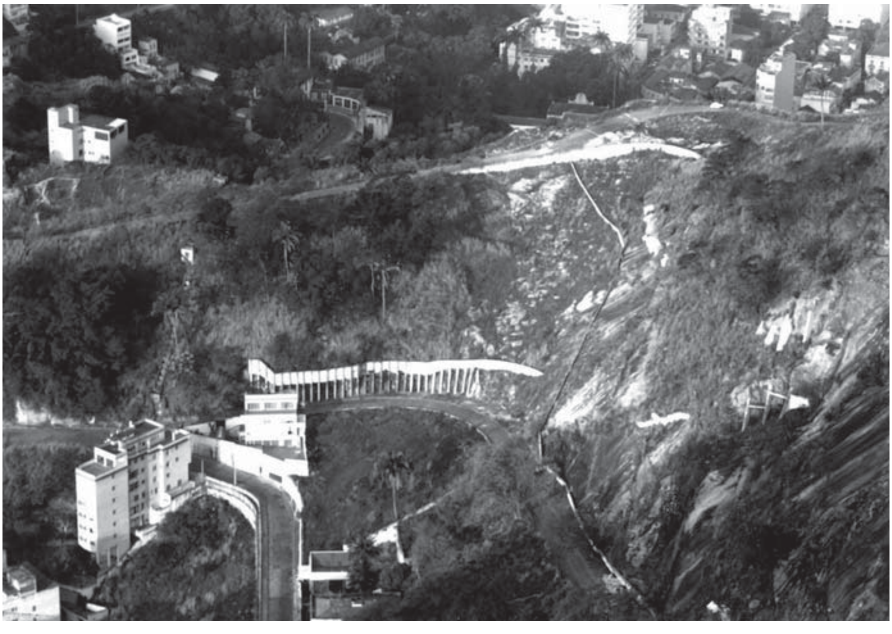


FIGURE 1.53
Scar of debris avalanche of February 18, 1967, which destroyed two apartment buildings and took 132 lives in the Larenjeiras section of Rio de Janeiro. The buttressed wall was constructed afterward. Debris avalanches need not be large to be destructive.

the 7300-m-high peak, Nevada Huascaran. Triggered by a thaw, 3 million tons of ice fell and flowed down a narrow canyon picking up debris and spilling out onto the fertile valley at an elevation 4000 m lower than the glacier and a distance of 15 km. The debris remaining in the towns ranged from 10 to 20 m thick.

Event No. 2: The catastrophe was almost duplicated on May 32, 1970, when the big Peruvian earthquake caused another avalanche from Nevada Huascaran that buried Yungay, adjacent to Ranrahirca, as well as Ranrahirca again, taking at least 18,000 lives (Youd, 1978). During the 1962 event, Yungay had been spared. The average velocity of the avalanche has been given as 320 km/h (200 mi/h) (Varnes, 1978), and the debris flowed upstream along the Rio Santa for a distance of approximately 2.5 km. As with the 1962 failure, the avalanche originated when a portion of a glacier on the mountain peak broke loose.

1.2.9 Debris Flows

General

Debris flows are similar to debris avalanches, except that the quantity of water in the debris-flow mass causes it to flow as a slurry; in fact, differentiation between the two forms can be difficult. The major causes are very heavy rains, high runoff, and loose surface materials.

Recognition

Occurrence is similar to debris avalanches, but debris flows are more common in steep gullies in arid climates during cloudbursts, and the failing mass can move far from its source (see Figure 1.4c).

1.2.10 Rock-Fragment Flows

General

A rock mass can suddenly break loose and move downslope at high velocities as a result of the sudden failure of a weak bed or zone on the lower slopes causing loss of support to the upper mass. Weakening can be from weathering, frost wedging, or excavation. Failure is sudden, unpredictable, and can be disastrous.

Recognition

High, steep slopes in jointed rock masses offer the most susceptible conditions. The avalanche illustrated in Figure 1.47 could also be classified as a dry rock flow because of its velocity and lack of water. In the initial stages tension cracks may develop; after the final stage a scarred surface remains over a large area, and a mass of failed debris may extend far from the toe of the slope.

Example of Major Failure

Event

The Turtle Mountain slide of the spring of 1903 destroyed part of the town of Frank, Alberta, Canada. More than 30 million m^3 of rock debris moved downslope and out onto the valley floor for a distance of over 1 km in less than 2 min.

Geology

The mountain is the limb of an anticline composed of limestone and shales as shown in the section in Figure 1.54a. Failure was sudden, apparently beginning in bedding planes in the lower shales (Krahn and Morgenstern, 1976) which are steeply inclined. The slide scar is shown in Figure 1.54b.

Cause

Terzaghi (1950) postulated that the flow was caused by joint weathering and creeping of the soft shales, accelerated by coal mining operations along the lower slopes.

1.2.11 Soil and Mud Flows

General

Soil and mud flows generally involve a saturated soil mass moving as a viscous fluid, but at times can consist of a dry mass. Major causes include earthquakes causing high pore-air pressures (loess) or high pore-water pressures; the leaching of salts from marine clays increasing their sensitivity, followed by severe weather conditions; lateral spreading followed by a sudden collapse of soil structure; and heavy rainfall on a thawing mass or the sudden drawdown of a flooded water course.

Flows occur suddenly, without warning, and can affect large areas with disastrous consequences.

Recognition

Occurrence is common in saturated or nearly saturated fine-grained soils, particularly sensitive clays, and occasional in dry loess or sands (sand runs).

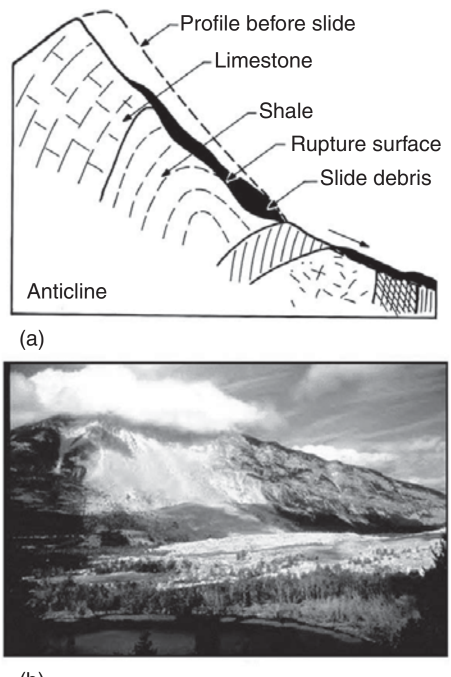


FIGURE 1.54
Turtle Mountain rock slide, Frank, Alberta in 1903. (a) Geologic section (From Krahn, J. and Morgenstern, N. R., *Proceedings of ASCE, Rock Engineering for Foundations and Slopes*, Vol. 1, 1976, pp. 309–332. With permission.) (b) Photo of scar (source unknown).

During initial stages flows may begin by slump failure followed by lateral spreading in the case of sensitive clays. During final failure, a tongue-shaped lobe of low profile extends back to a bottleneck-shaped source area with a small opening at the toe of the flow as shown in Figure 1.4f, and a distinct scarp remains at the head. Flowing masses can extend for great distances, at times measured in kilometers.

Examples of Major Failures

Achocallo Mudflow, La Paz, Bolivia

Believed triggered by an earthquake some thousands of years ago, an enormous portion of the rim of the Bolivian altiplano (elevation 4000 m), roughly 9 km across, slipped loose and flowed down the valley of the Rio Achocallo into the Rio La Paz at an elevation approximately 1500 m lower. The flow remnants extend downstream today for a distance of about 25 km, part of which are shown on the aerial oblique panorama given in Figure 1.55. The head scarp is shown on the photo (Figure 1.56).

The altiplano is the remains of an ancient lake bed, probably an extension of Lake Titicaca, underlain by a thick stratum of sand and gravel beneath which are at least several hundred meters of lacustrine clays and silts, interbedded with clays of volcanic origin. The photo (see Figure 2.36) of a large piping tunnel was taken in the bowl-shaped valley about 3 km downslope from the rim.

Province of Quebec, Canada

Event: In Saint Jean Vianney, on May 4, 1971, a mass of glaciomarine clays completely liquefied, destroying numerous homes and taking 31 lives (Tavenas et al., 1971).

Description: The flow began in the crater of a much larger 500-year-old failure (determined by carbon-14 dating). Soil stratigraphy consisted of about 100 ft of disturbed clays with sand pockets from the ancient slide debris, overlying a deep layer of undisturbed glaciomarine clay. Occurring just after the first heavy rains following the spring thaw, the flow apparently began as a series of slumps from the bank of a small creek, which formed a temporary dam. Pressure built up behind the dam, causing it to fail, and 9 million yd^3 of completely liquefied soils flowed downstream with a wavefront 60 ft in height and a velocity estimated at 16 m/h. The flow finally discharged into the valley of the Saguenay River, 2 mi from its source.

Causes: Slope failures in the marine clays of Quebec are concentrated in areas that seem to be associated with a groundwater flow regime resulting from the existence of valleys in the underlying rock surface (Tavenas et al., 1971). The valleys cause an upward flow gradient and an artesian pressure at the slope toes. The upper part of the soil profile is subjected to a downward percolation of surface water because of the existence of sand strata. The downward percolation and upward flow produce an intense leaching of the clay, resulting in a decrease of the undrained shear strength and an increase in sensitivity. The evidence tends to indicate that the leaching is a function of the gradient. Field studies have shown a close relationship between the configuration of the bedrock and the properties of the underlying clay deposit.

Norwegian "Quick" Clays

Regional geology: Approximately 40,000 km^2 of Norway has deposits of glaciomarine clays which overlie an irregular surface of granite gneiss, similar to conditions in Quebec. During postglacial times, the area has been uplifted to place the present surface about 180 m above sea level. Typical stratigraphy includes 5 to 7 m of a stiff, fissured clay overlying normally consolidated soft marine clay which extends to depths greater than 70 m in

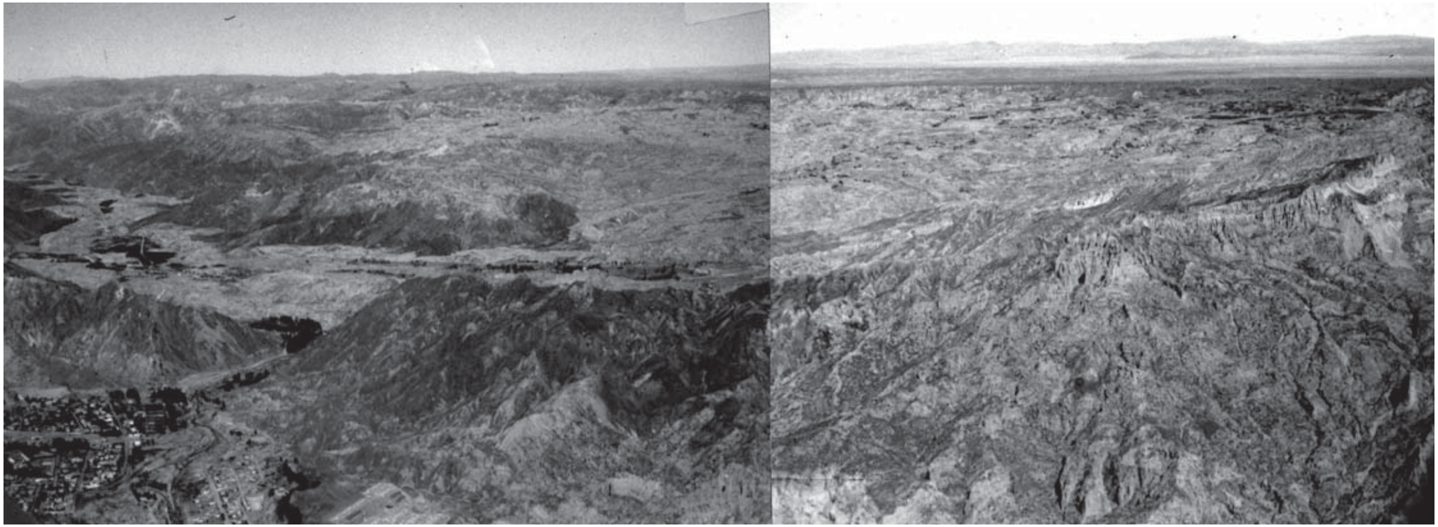


FIGURE 1.55
Believed triggered by an earthquake some thousands of years ago, an enormous portion of the rim of the Bolivian Altiplano, roughly 9 km across (photo right), slipped loose and flowed down the valley of the Rio Achocallo into the Rio La Paz, for a total distance of over 25 km (photo left). The flow is apparent in the photo as the light-colored area in the valley.

FIGURE 1.56
The head scarp of the Achocallo mud flow, near La Paz, Bolivia (see Figure 1.55).

some locations. Rock varies from outcropping at the surface in stream valleys to over 70 m in depth. The quick clays are formed by leaching of salts, but the leaching is believed to be caused by artesian pressure in the rock fractures from below, rather than downward percolation of water (Bjerrum et al., 1969). Sensitivity values are directly related to the amount of leaching and the salt content, and are greatest where rock is relatively shallow, about 15 to 35 m.

Slope failures are common events. The natural slopes are stable at about 20° where there is a stiff clay crust. Seepage parallel to the slopes occurs in fissures in the clay, and the stiff clay acts as a cohesionless material with slopes at $i = 1/2\phi'$, and $\phi = 38°$ (see the infinite slope problem in Section 1.3.2). Stream erosion causes small slides in the weathered stiff clay; the sliding mass moves into the soft clay, which upon deformation becomes quick and flows. In one case cited by Bjerrum et al. (1969), 200,000 m^3 flowed away from the source in a few minutes.

Solution: Since stream degrading appears to be a major cause of the flows, Bjerrum et al. (1969) proposed the construction of small weirs to impede erosion in streams where failures pose hazards.

1.2.12 Seafloor Instability

General

Various forms of slope failures have been recognized offshore, including deep rotational slides (Figure 1.57) and shallow slumps, flows, and collapsed depressions (Figure 1.58). Major causes are earthquakes, storm waves inducing bottom pressures, depositional loads accumulating rapidly and differentially over weak sediments, and biochemical degradation of organic materials forming large quantities of gases *in situ* which weaken the seafloor soils.

Offshore failures can occur suddenly and unpredictably, destroying oil production platforms, undersea cables, and pipelines. Large flows, termed "turbidity currents," can move tremendous distances.

Recognition

Occurrence is most common in areas subjected to earthquakes of significant magnitude and on gently sloping seafloors with loose or weak sediments, especially in rapidly accreting deltaic zones.

After failure the seafloor is distorted and scarred with cracks, scarps, and flow lobes similar to those features which appear on land as illustrated in Figure 1.58. Active areas are explored with side-scan sonar (Figure 1.59) and high-resolution geophysical surveys (see Figure 1.57).

Examples of Major Failures

Gulf of Alaska

The major slide illustrated on the high-resolution seismic profile given in Figure 1.57 apparently occurred during an earthquake, and covers an area about 15 km in length. Movement occurred on a 1° slope and is considered to be extremely young (Molnia et al., 1977). As shown in the figure, the slide has a well-defined head scarp, disrupted bedding, and a hummocky surface.

Gulf of Mexico

Movements are continually occurring offshore of the Mississippi River Delta. During hurricane Camille in August 1969, wave-induced bottom pressures caused massive seafloor movements that destroyed two offshore platforms and caused a third to be displaced over a meter on a bottom slope that was very flat, less than 0.5% (Focht and Kraft, 1977).

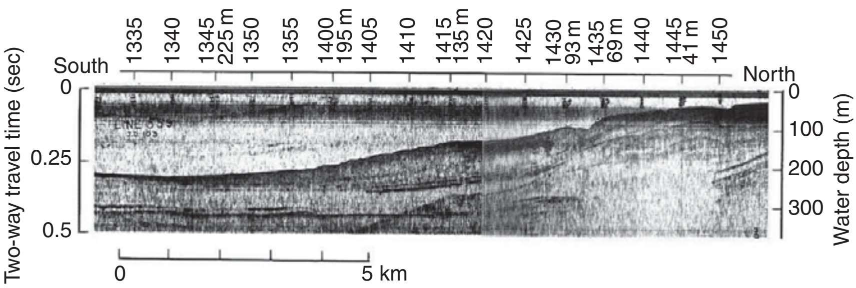


FIGURE 1.57
High-resolution seismic reflection profile showing a portion of the Kayak Trough slump slide in the Gulf of Alaska. (From Molnia, B. F. et al., in *Reviews in Engineering Geology*, Vol. VIII, *Landslides*, Coates, D.R., Ed., Geologic Society of America, 1977, pp. 137–148. With permission.)

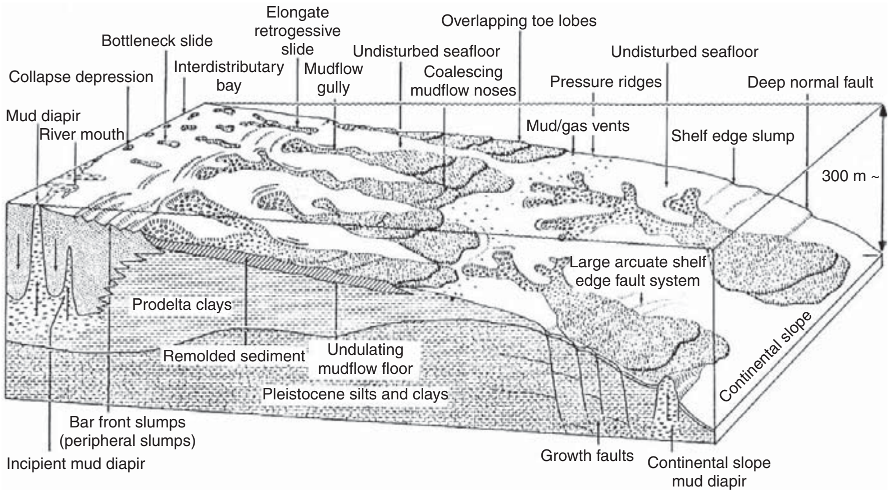


FIGURE 1.58
Schematic block diagram illustrating the various forms of slope failure in the offshore Mississippi River Delta. (From Coleman, J. M. et al., *Open File Report No. 80.01*, Bureau of Land Management, U.S. Department of Interior, 1980. With permission.)

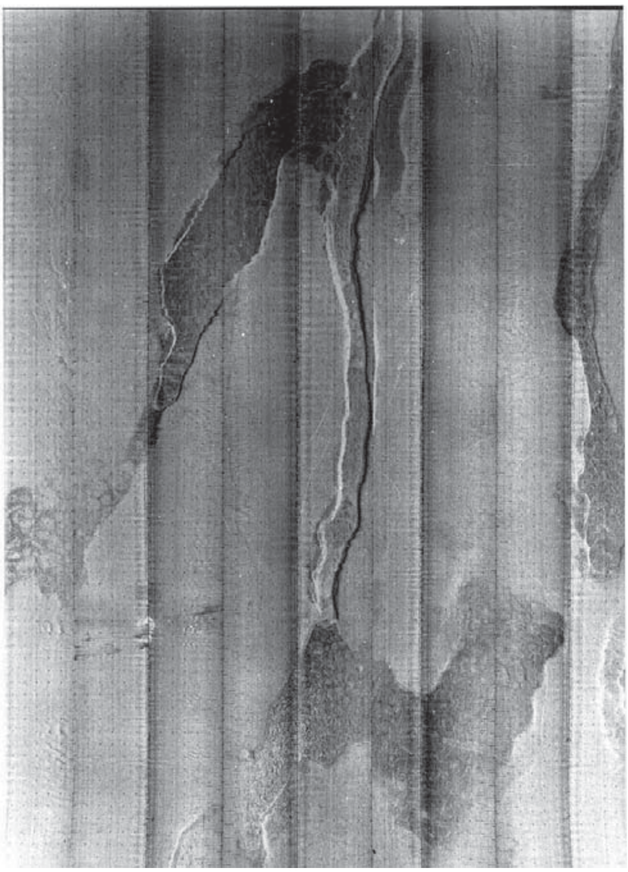


FIGURE 1.59
Side-scan sonar mosaic illustrating seafloor mudflows (offshore Gulf of Mexico). Grids are 25 m (82 ft) apart and the mosaic covers an area approximately 1.5 km in length. (Mosaic courtesy of Dr. J. M. Coleman, Coastal Studies Institute, Louisiana State University.)

Grand Banks, Newfoundland

The earthquake of November 1929 (see Section 3.3.4) caused a section of the continental shelf to break loose and subsequently mix with seawater. It moved offshore for a distance of about 925 km and broke a dozen submarine cables. Geologists called this flow a "turbidity current" (Richter, 1958).

1.3 Assessment of Slopes

1.3.1 General

Objectives

The assessment of an existing unstable or potentially unstable slope, or of a slope to be cut, provides the basis for the selection of slope treatments. Treatment selection requires forecasting the form of failure, the volume of material involved, and the degree of the hazard and risk. Assessment can be based on quantitative analysis in certain situations, but in many cases must be based on qualitative evaluation of the slope characteristics and environmental factors including weather and seismic activity.

Key Factors to Be Assessed

- History of local slope failure activity as the result of construction, weather conditions, seismic activity (see Section 3.3.4), or other factors, in terms of failure forms and magnitudes.
- Geologic conditions including related potential failure forms and their suitability for mathematical analysis, material shear strength factors (constant, variable, or subject to change or liquefaction, Section 1.3.2), and groundwater conditions.
- Slope geometry in terms of the influence of inclination, height, and shape on potential seepage forces, runoff, and failure volume.
- Surface indications of instability such as creep, scars, seepage points, and tension cracks.
- Degree of existing slope activity (see Section 1.1.2).
- Weather factors (rainfall and temperature) in terms of the relationship between recent weather history and long-term conditions (less severe, average, and more severe) in view of present slope activity, stability of existing cut slopes, groundwater levels, and slope seepage.

1.3.2 Stability Analysis: A Brief Review

General Principles

Basic Relationships

Stability analysis of slopes by mathematical procedures is applicable only to the evaluation of failure by sliding along some definable surface. Avalanches, flows, falls, and progressive failure cannot be assessed mathematically in the present state of the art.

Slide failure occurs when the shearing resistance available along some failure surface in a slope is exceeded by shearing stresses imposed on the failure surface. Static analysis of sliding requires knowledge of the location and shape of the potential failure surface, the shear strength along the failure surface, and the magnitude of the driving forces. Statically determinate failure forms may be classified as:

- Infinite slope — translation on a plane parallel to the ground surface whose length is large compared with its depth below the surface (end effects can be neglected) (Figure 1.63) (Morgenstern and Sangrey, 1978).
- Finite slope, planar surface — displacement of one or more blocks, or wedge-shaped bodies, along planar surfaces with finite lengths (Figure 1.66).
- Finite slope, curved surface — rotation along a curved surface approximated by a circular arc, log-spiral, or other definable cylindrical shape (Figure 1.76).

Failure origin: As stresses are usually highest at the toe of the slope, failure often begins there and progresses upslope, as illustrated in Figure 1.60, which shows the distribution of active and passive stresses in a slope where failure is just beginning. Failures can begin at any point along a failure surface, however, where the stresses exceed the peak strength. Because failure often is progressive, it usually occurs at some average shear strength that can be considerably less than the peak strength measured by testing techniques.

Limit Equilibrium Analysis

Most analytical methods applied to evaluate slope stability are based on limiting equilibrium, i.e., on equating the driving or shearing forces due to water and gravity to the resisting forces due to cohesion and friction.

Shearing forces result from gravity forces and internal pressures acting on a mass bounded by a failure surface. Gravity forces are a function of the weight of the materials, slope angle, depth to the failure surface, and in some cases, slope height. Pressures develop in joints in rock masses from water, freezing, swelling materials, or hydration of minerals and, in soils, from water in tension cracks and pores.

Resisting forces, provided by the shear strength along the failure surface, are decreased by an increase in pore pressures along the failure surface, by lateral strains in overconsolidated clays in clay shales, by dissolution of cementing agents and leaching, or by the development of tension cracks (which serve to reduce the length of the resisting surface).

Safety factor against rupture, given as

$$FS = \frac{\text{shearing strength available along the sliding surface}}{\text{shearing stresses tending to produce failure along the surface}} \tag{1.1}$$

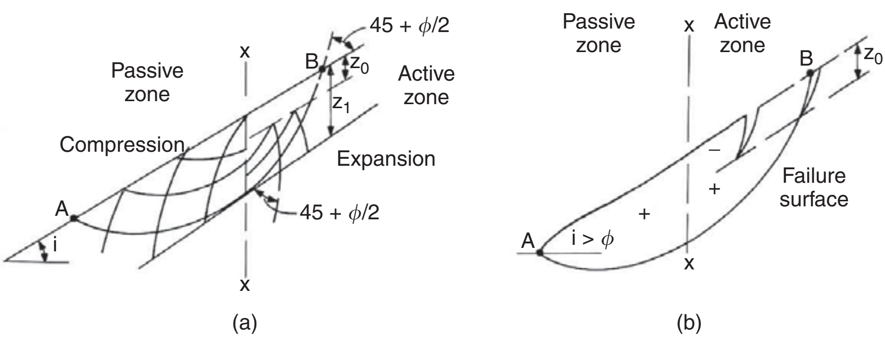


FIGURE 1.60
Active and passive stresses acting on a slope. The passive resistance at the lower portions is most significant. (a) Distribution of stresses on slope with $i > \phi$ (failure beginning). (b) A case of rupture where + indicates zones of passive stress and (−) indicates zones of active stress, subjected to tension cracks.

Shear Strength Factors

Strength Parameters

The basic strength parameters are the angle of internal friction ϕ and cohesion c. Frictional resistance based on ϕ is a function of the normal stress, and the maximum frictional shear strength is expressed as

$$S_{max}=N\tan\phi \tag{1.2}$$

Cohesion c is independent of the normal stress and acts over the area of the failure surface.

Total and Effective Stresses

In the total stress condition, the measured stress includes both pore-water pressures and stresses from grain-to-grain contact. In the effective stress condition, stresses from grain-to-grain contact are measured, which increase as pore pressures dissipate. Effective stress equals total stress minus pore pressure.

Pore-water pressures (U for total, u for unit pressures) are induced either by a load applied to a saturated specimen, or by the existence of a phreatic surface above the sliding surface. They directly reduce the normal force component N, and shearing resistance is then expressed as

$$S_{max}=(N-U)\tan\phi \tag{1.3}$$

In Figure 1.66, therefore, if pore pressures become equal to the normal component of the weight of the block, there will be no shearing resistance.

Failure Criteria

The Mohr–Coulomb criterion defines failure in terms of unit shear strength and total stresses as

$$s=c+\sigma_n\tan\phi \tag{1.4}$$

The Coulomb–Terzaghi criterion accounts for pore-water pressures by defining failure in terms of effective stresses as

$$s=c'+\sigma_n'\tan\phi' \tag{1.5}$$

where c' is the effective cohesion; σ_n' the effective normal stress ($= p - u$) with p total normal stress. In a slope, the total pressure p per unit of area at a point on the sliding surface equals $h_z\,\gamma_t/\cos^2\theta$, where h_z is the vertical distance from the point on surface of sliding to top of slope, γ_t the slope unit weight of soil plus water, and θ the inclination of surface of sliding at point with respect to horizontal; u the pore-water pressure, in a slope $u=h_w\gamma_w$, the piezometric head times the unit weight of water; and ϕ' the effective friction angle.

The strength parameters representing shearing resistance in the field are a function of the material type, slope history, drainage conditions, and time. Most soils (except purely granular materials and some normally consolidated clays) are represented by both parameters ϕ and c, but whether both will act during failure depends primarily on drainage conditions; and, the stress history of the slope.

Undrained vs. Drained Strength

Undrained conditions exist when a fully saturated slope is sheared to failure so rapidly that no drainage can occur, as when an embankment is placed rapidly over soft soils. Such conditions are rare except in relatively impervious soils such as clays. Soil behavior may then be regarded as purely cohesive and $\phi = 0$. Results are interpreted in terms of total stresses, and s_u, the undrained strength, applies. The case of sudden drawdown of an adjacent water body is an undrained condition, but analysis is based on the consolidated undrained (CU) strength of the soil before the drawdown. This strength is usually expressed in terms of the CU friction angle.

Drained or long-term conditions exist in most natural slopes, or some time after a cut is made and drainage permitted. Analysis is based on effective stresses, and the parameters ϕ' and c' will be applicable.

Peak and Residual Strength

The foregoing discussion, in general, pertains to peak strengths. When materials continue to strain beyond their peak strengths, however, resistance decreases until a minimum strength, referred to as the ultimate or residual strength, is attained. The residual strength, or some value between residual and peak strengths, normally applies to a portion of the failure surface for most soils; therefore, the peak strength is seldom developed over the entire failure surface.

Progressive failure, when anticipated, has been approximately evaluated by using the residual strength along the upper portion of the failure surface, and the peak strength at maximum normal stress along the lower zone (Conlon as reported in Peck, 1967; Barton, 1972).

Stiff fissured clays and *clay shales* seldom fail in natural slopes at peak strength, but rather at some intermediate level between peak and residual. Strength is controlled by their secondary structure. The magnitude of peak strength varies with the magnitude of normal stress, and the strain at which peak stress occurs also depends on the normal stress (Peck, 1967). Because the normal stress varies along a failure surface in the field, the peak strength cannot be mobilized simultaneously everywhere along the failure surface.

Residual strength applies in the field to the entire failure surface where movement has occurred or is occurring. Deere and Patton (1971) suggest using ϕ_r (the residual friction angle), where preexisting failure surfaces are present.

Other Strength Factors

Stress levels affect strength. Creep deformation occurs at stress levels somewhat lower than those required to produce failure by sudden rupture. A steady, constant force may cause plastic deformation of a stratum that can result in intense folding, as illustrated in Figure 1.61. Shear failure by rupture occurs at higher strain rates and stress levels and distinct failure surfaces are developed as shown in Figure 1.62. The materials are genetically the same, i.e., varved clays from the same general area.

The strength of partially saturated materials cannot be directly evaluated by effective stress analysis since both pore-air and pore-water pressures prevail. Residual soils, for example, are often partially saturated when sampled. In Brazil, effective stress analysis has sometimes been based on parameters measured from direct shear tests performed on saturated specimens to approximate the most unfavorable field conditions (Vargas and Pichler, 1957). Depending upon the degree of field saturation, the saturated strengths may be as little as 50% of the strength at field moisture. Apparent cohesion results from capillary forces in partially saturated fine-grained soils such as fine sands and silts; it constitutes a temporary strength that is lost upon saturation and, in many instances, on drying.

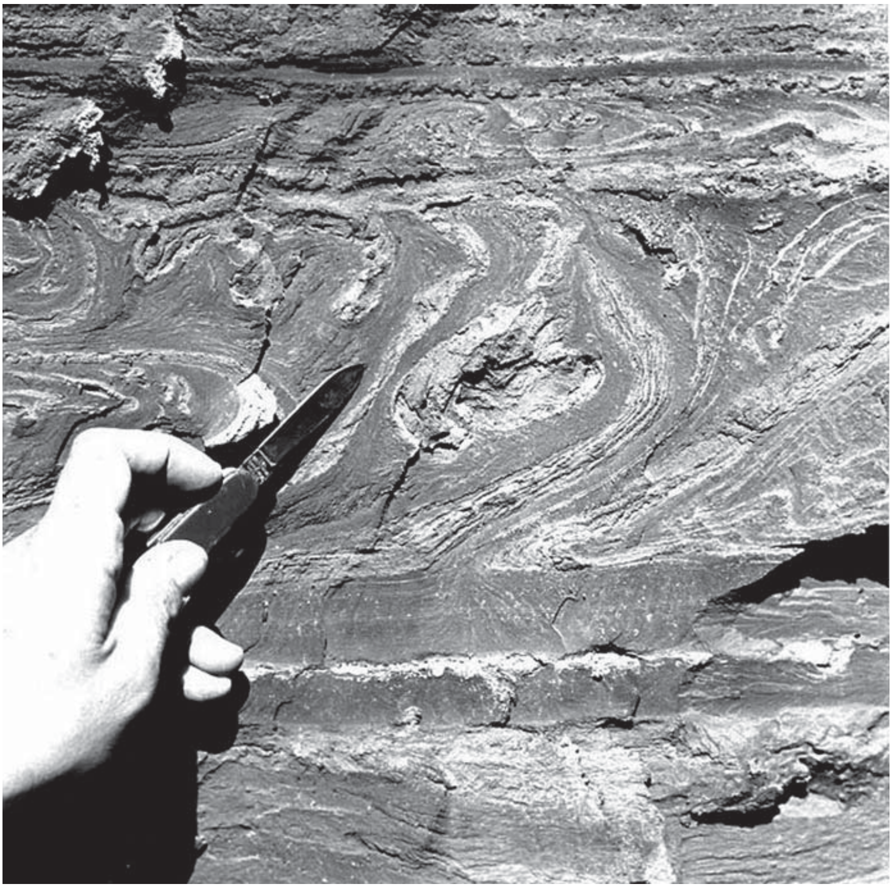


FIGURE 1.61
Creep deformation in varved clays (Roseton, New York).

Spontaneous liquefaction occurs and the mass becomes fluid in fine-grained, essentially cohesionless soils when the pore pressure is sufficiently high to cause a minimum of grain-to-grain contact. After failure, as the mass drains and pore pressures dissipate, the mass can achieve a strength higher than before failure. High pressures can develop in pore air or pore water.

Changes with time occur from chemical weathering, lateral strains, solution of cementing agents, or leaching of salts (see Section 1.1.4).

In Situ *Rock Strength*

Effective stress analysis normally is applicable because the permeability of the rock mass is usually high. In clay shales and slopes with preexisting failure surfaces, the residual friction angle ϕ_r is often applicable, with pore pressures corresponding to groundwater conditions.

Two aspects that require consideration regarding strength are that strength is either governed by (1) planes of weakness that divide the mass into blocks, or (2) the degree of weathering controls, and soil strength parameters apply.

Seepage or cleft-water pressures affect the frictional resistance of the rock mass in the same manner that pore pressures affect the strength of a soil mass.

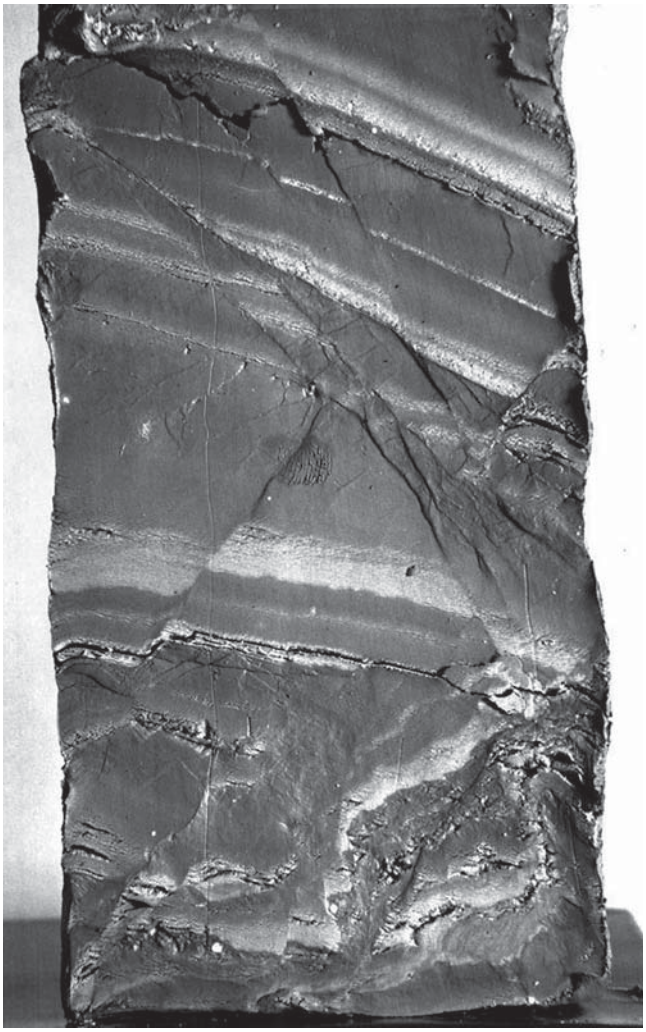


FIGURE 1.62
Section of 3-in.-diameter undisturbed specimen of varved clay taken from a depth of 11m in the failure zone showing the rupture surfaces after collapse of an excavation in Haverstraw, New York.

Failure Surface Modes and Stability Relationships

General: Two Broad Modes of Failure

Infinite slope mode involves translation on a planar surface whose length is large compared with its depth. This mode is generally applicable to cohesionless sands, some colluvial and residual soil slopes underlain by a shallow rock surface, and some cases of clay shale slopes.

Finite or limited slope mode involves movement along a surface limited in extent. The movement can be along a straight line, a circular arc, a log-spiral arc, or combinations of these. There are two general forms of finite slope failures: wedges and circular failures. Wedge analysis forms are generally applicable to jointed or layered rock, intact clays on steep slopes, stratified soil deposits containing interbedded strong and weak layers, and clay shale slopes. Cylindrical failure surfaces are typical of normally consolidated to slightly overconsolidated clays and common to other cohesive materials including residual, colluvial, and glacial soils where the deposit is homogeneous.

Infinite-Slope Analysis

The infinite slope and forces acting on an element in the slope are illustrated in Figure 1.63. In the infinite-slope problem, neither the slope height nor the length of the failure surface is considered when the material is cohesionless.

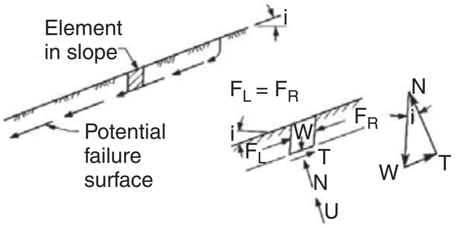


FIGURE 1.63
The infinite slope and forces on an element (total shear resistance $=T_{max}=N$ tan ϕ: $N=W$ cos i).

Relationships at equilibrium between friction ϕ and the slope angle i for various conditions in a cohesionless material are given in Figure 1.64, in which T is the total shearing resistance, summarized as follows:

- Dry slope: $i=\phi$ (angle of repose for sands), $T=N\tan\phi$.
- Submerged slope: $i=\phi$, $T=N'\tan\phi'$, and

$$\text{FS}=(W\cos \text{i})\tan\phi' / \ W\sin i \tag{1.6}$$

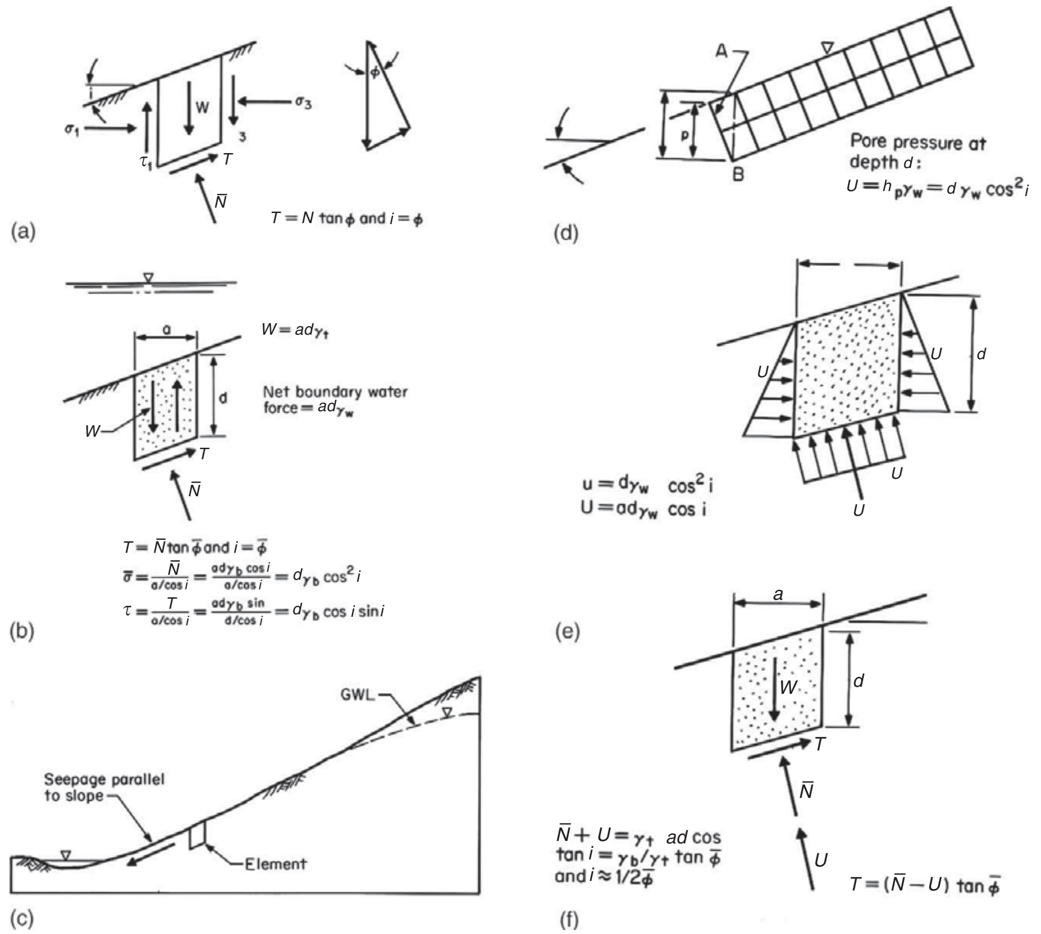


FIGURE 1.64
Equilibrium of an infinite slope in sand under dry, submerged, and slope seepage conditions: (a) slope in dry sand; (b) submerged slope in sand; (c) seepage in a natural slope; (d) flow net of seepage parallel to slope; (e) boundary pore pressures; (f) force equilibrium for element with seepage pressures. (After Lambe, T. W. and Whitman, R. V., *Soil Mechanics,* Wiley, New York, 1969. Adapted with permission of John Wiley & Sons, Inc.)

- Seepage parallel to slope with free water surface coincident with the ground surface (Figure 1.64c–f): $i = 1/2\phi'$, and $T = (N' - U)\tan\phi$.
- Infinite-slope conditions can exist in soils with cohesion which serves to increase the stable slope angle i. These conditions generally occur where the thickness of the stratum and, therefore, the position of the failure surface that can develop are limited by a lower boundary of stronger material. Many colluvial and clay shale slopes are found in nature at $i = 1/2\phi_r$, the case of seepage parallel to the slope with the free water surface coincident with the ground surface.

Finite Slope: Planar Failure Surface

Case 1: Single planar failure surface with location assumed, involving a single block and no water pressures (Figure 1.65). Driving force F (= $W\sin i$) is the block weight component. Resisting force $T = N\tan\phi = (W\cos i)\tan\phi$,

$$FS = (W\cos i)\tan\phi / W\sin i \tag{1.7}$$

where $i_{cr} = \phi$.

Case 2: Single block with cleft-water pressures and cohesion along the failure surface with location assumed (Figure 1.66):

$$FS = [cA + (W\cos i - U)\tan\phi] / W\sin i + V \tag{1.8}$$

where A is the block base area, V the total joint water pressure on upstream face of the block, U the total water pressure acting on the base area (boundary water pressures), c the cohesion, independent of normal stress, acting over the base area and W the total weight of block, based on γ_t.

Case 3: Simple wedge acting along one continuous failure surface with cohesion and water pressure; failure surface location known (Figure 1.67):

$$FS = [cL + (W\cos\theta - U)\tan\phi] / W\sin\theta \tag{1.9}$$

where L is the length of failure surface.

Case 4: Simple wedge with tension crack and cleft-water pressures V and U. Failure surface location known; tension crack beyond slope crest (Figure 1.68a); tension crack along

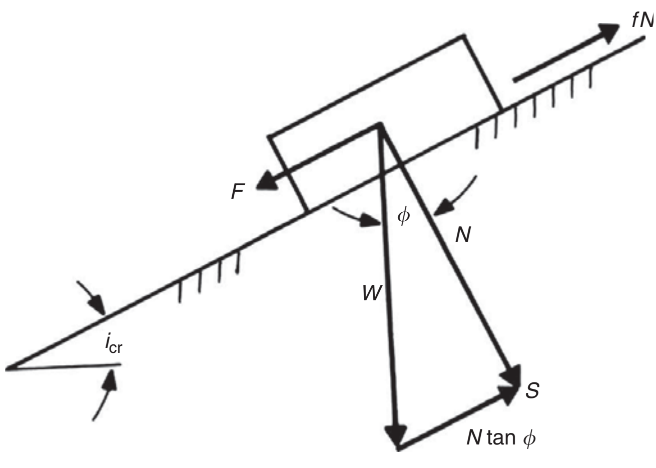


FIGURE 1.65
Simple sliding block.

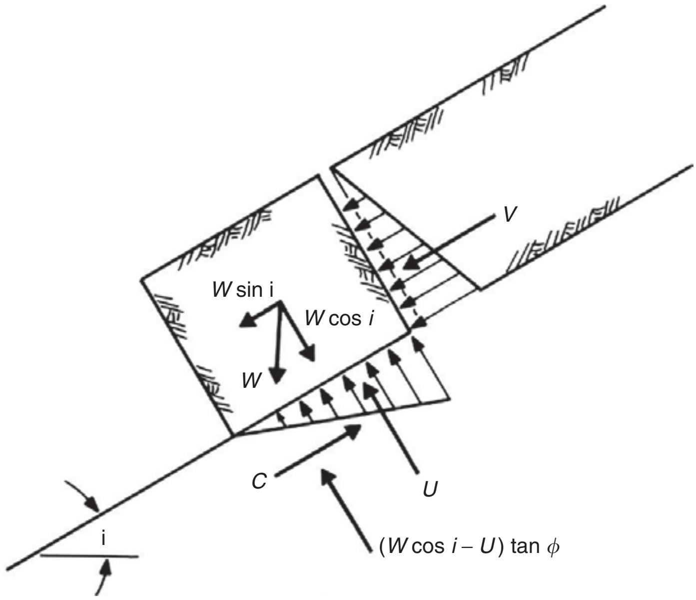


FIGURE 1.66
Block with cleft water pressures and cohesion.

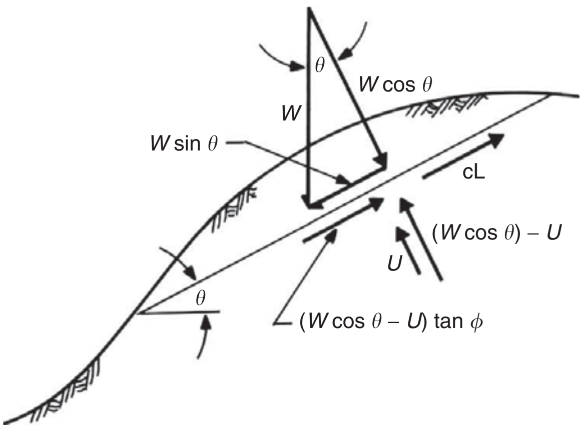


FIGURE 1.67
Simple wedge acting along continuous surface with cohesion.

slope (Figure 1.68b). Figure 1.69 gives an example of a simple wedge developing in residual soils. In Figure 1.68,

$$\mathrm{FS}=[cL+(W\cos\theta-U-V\sin\theta)\tan\phi]/\ W\sin\theta+V\cos\theta \qquad (1.10)$$

where

$L=(H-z)\ \mathrm{cosec}\ \theta,$

$U=1/2\gamma_{\mathrm{w}}\, z_{\mathrm{w}}\,(H-z)\ \mathrm{cosec}\ \theta,$

$V=1/2\gamma_{\mathrm{w}}\, z_{\mathrm{w}}^{\ 2},$

$W=1/2\gamma_{\mathrm{t}}\, H^2\ \{[1-(z/H)^2]\cot\theta-\cot i\}$ (Figure 1.68a)

or $W=1/2\gamma_{\mathrm{t}}\, H^2\ \{[1-z/H)^2\ \cot\theta\ (\cot\theta\tan i-1)]$ (Figure 1.68b)

Case 5: Single planar failure surface in clay: location unknown; Culmann's simple wedge (Figure 1.70). Assumptions are that the failure surface is planar and passes through the

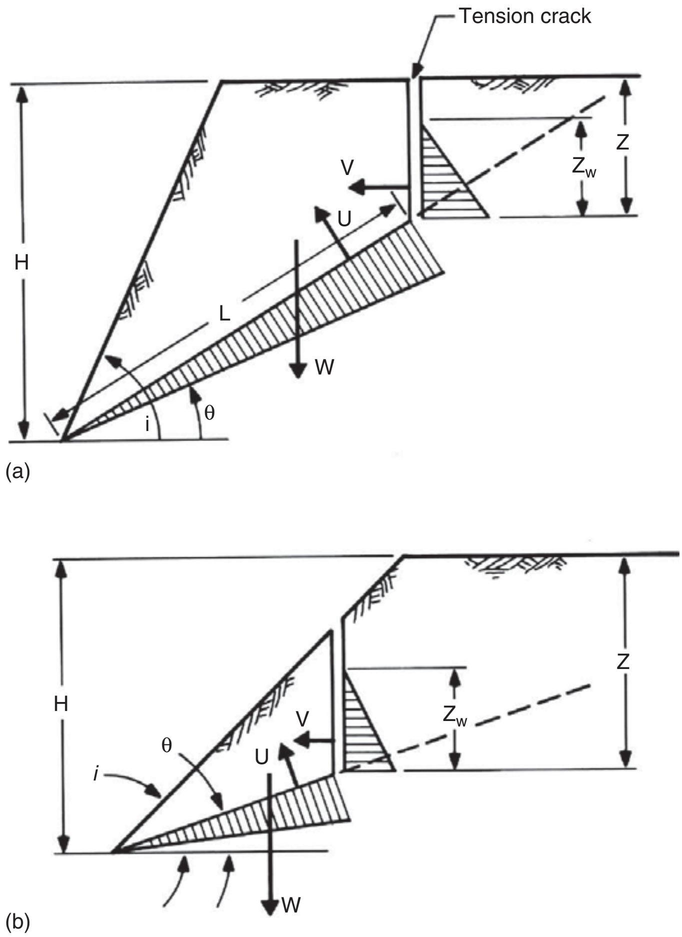


FIGURE 1.68
Plane failure analysis of a rock slope with a tension crack: (a) tension crack in upper slope surface; (b) tension crack in slope face. (From Hoek, E. and Bray, J. W., *Rock Slope Engineering*, 2nd ed., The Institute of Mining and Metallurgy, London, 1977. With permission.)

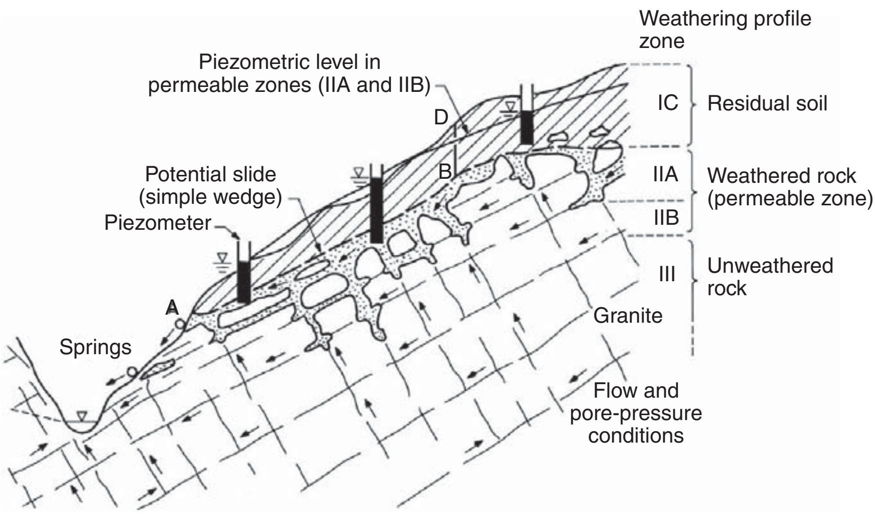


FIGURE 1.69
Development of simple wedge failure in residual soil over rock. (After Patton, F. D. and Hendron, A. J., Jr., *Proceedings of the 2nd International Congress, International Association of Engineering Geology*, São Paulo, 1974, p. V-GR 1.)

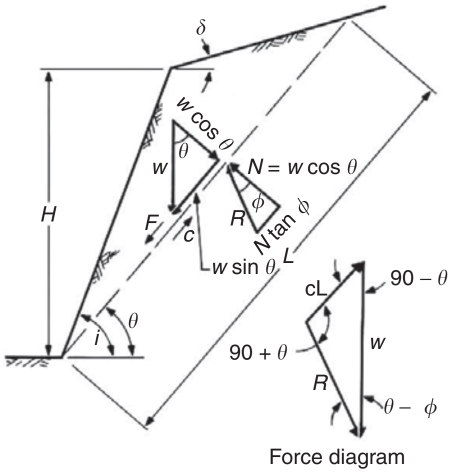


FIGURE 1.70
Culmann's simple wedge in clayey soil.

slope toe, shear strength is constant along the failure surface in a homogeneous section, and there are no seepage forces. In practice, seepage forces are applied as in Equation 1.9. The solution is generally considered to yield reasonable results in slopes that are vertical or nearly so, and is used commonly in Brazil to analyze forces to be resisted by anchored curtain walls (see Section 1.4.6). The solution requires finding the critical failure surface given by

$$\theta_{cr} = (i + \phi)/2 \tag{1.11}$$

$$FS = [cL + (W \cos \theta) \tan \phi] / W \sin \theta \tag{1.12}$$

where $W = 1/2\gamma_t LH$ cosec i sin $(i - \theta)$, with

$$H_{max=} 4c(\sin i)(\cos \phi)/\gamma_t [1\text{-} \cos (i - \phi)] \tag{1.13}$$

Case 6: Critical height and tension crack in clay (Figure 1.71). The critical height H_{cr} is defined as the maximum height at which a slope can stand before the state of tension, which develops as the slope yields, is relieved by tension cracks. Terzaghi (1943) gave the critical height in terms of total soil weight, having concluded that the tension crack would reach to one half the critical height, as

$$H'_{cr} = (2.67c/\gamma_t) \tan (45° + \phi/2) \tag{1.14}$$

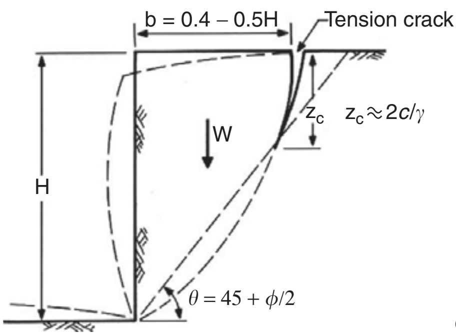


FIGURE 1.71
Critical height of a vertical slope in clay and the tension crack.

or for the case of $\phi = 0$,

$$H_{cr} = 2.67c/\gamma_t \tag{1.15}$$

Field observations indicate that the tension crack depth z_c ranges from $1/3H$ to $1/2H$. In practice, z_c is often taken as $1/2H_{cr}$ of an unsupported vertical cut or as

$$z_c = 2c/\gamma \tag{1.16}$$

which is considered conservative (Tschebotarioff, 1973).

Case 7: Multiple planar failure surfaces are illustrated as follows, relationships not included:

- Active and passive wedge force system applicable to rock or soil and rock slopes — Figure 1.72.
- General wedge or sliding block method applicable to soil formations and earth dams — Figure 1.73.
- Intersecting joints along a common vertical plane — Figure 1.74.
- Triangular wedge failure, applicable to rock slopes — Figure 1.75.

Finite Slope: Cylindrical Failure Surface

In rotational slide failures, methods are available to analyze a circular or log-spiral failure surface, or a surface of any general shape. In all cases, the location of the critical failure surface is found by trial and error, by determining the factor of safety (FS) for various trial positions of the failure surface until the lowest value of the FS is reached. The forces acting on a free body taken from a slope are given in Figure 1.76.

Most modern analytical methods are based on dividing the potential failure mass into slices, as shown in Figure 1.77. The various methods differ slightly based on the force system assumed about each slice (Figure 1.78). Iterations of the complex equations to find the "critical circle" has led to the development of many computer programs for use with the personal computer (PC). Equation 1.17 given under the Janbu method is similar to the Simplified Bishop. In most cases, for analysis, the parameters selected for input are far more important than the method employed. The most significant parameter affecting the FS is usually the value input for shear strength; assuming even a small amount of cohesion can result in FS = 1.2 rather than 1.02, if only internal friction is assumed.

Ordinary method of slices: In 1936, Fellenius published a method of slices based on cylindrical failure surfaces which was known as the Swedish Circle or Fellenius method. Modified for effective stress analysis, it is now known as the Ordinary Method of Slices.

As illustrated in Figure 1.77, the mass above a potential failure surface is drawn to scale and divided into a number of slices with each slice having a normal force resulting from its weight. A flow net is drawn on the slope section (Figure 1.77a), or more simply, a phreatic surface is drawn. Pore pressures are determined as shown in Figure 1.77b. The equilibrium of each slice is determined and FS found by summing the resisting forces and dividing by the driving forces as shown in Figure 1.77c. The operation is repeated for other circles until the lowest safety factor is found. The method does not consider all of the forces acting on a slice (Figure 1.78a), as it omits the shear and normal stresses and porewater pressures acting on the sides of the slice, but usually (although not always) it yields conservative results. However, the conservatism may be high.

Bishop's method of slices: This method considers the complete force system (Figure 1.78b), but is complex and requires a computer for solution. The results, however, are substantially

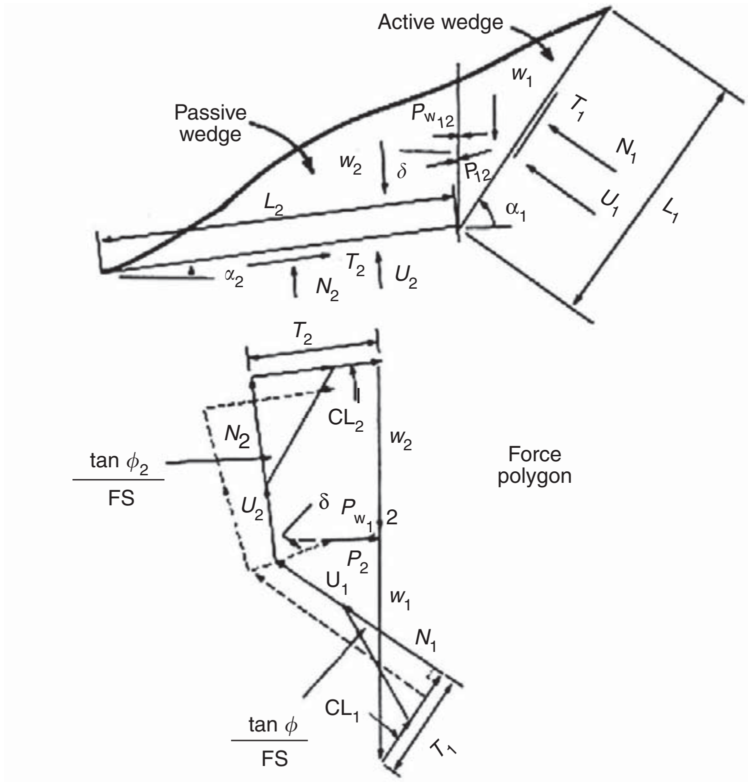


FIGURE 1.72
Forces acting on two wedges: one active, one passive. W_1, W_2 are the weights of the wedge, U_1, U_2 the resultant water pressure acting on the base of the wedge, N_1, N_2 the effective force normal to the base, T_1, T_2 the shear force acting along the base of the wedge, L_1, L_2 the length of the base, α_1, α_2 the inclination of the base to the horizontal, Pw_{12} the resultant water pressure at the interface, P_{12} the effective force at the interface, and δ the inclination of P_{12} to the horizontal. (From Morganstern, N. R. and Sangrey, D. A., *Landslides: Analysis and Control*, Schuster and Krizek, Eds., National Academy of Sciences, Washington, DC, 1978, pp. 255–272. Reprinted with permission of the National Academy of Sciences.)

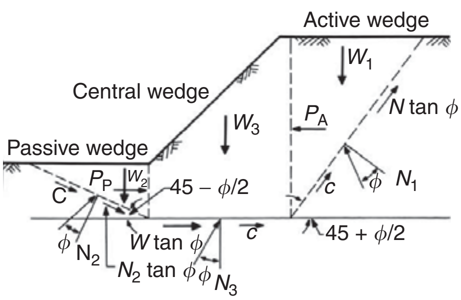


FIGURE 1.73
The general wedge or sliding block concept. (After NAVFAC, *Design Manual, Soil Mechanics, Foundation and Earth Structures,* DM-7.1, Naval Facilities Engineering Command, Alexandria, Virginia, 1982.)

more accurate than the ordinary method, and slightly more accurate than the modified Bishop's (1955) method.

Modified Bishop's method: This is a simplified Bishop's method (Janbu et al., 1956), widely used for hand calculations since it gives reasonably accurate solutions for circular failure surfaces. It is still widely used today on personal computers. The force system is given in Figure 1.78c.

Janbu's method: This is an approximate method applicable to circular as well as noncircular failure surfaces, as shown in Figure 1.79. It is sufficiently accurate for many practical

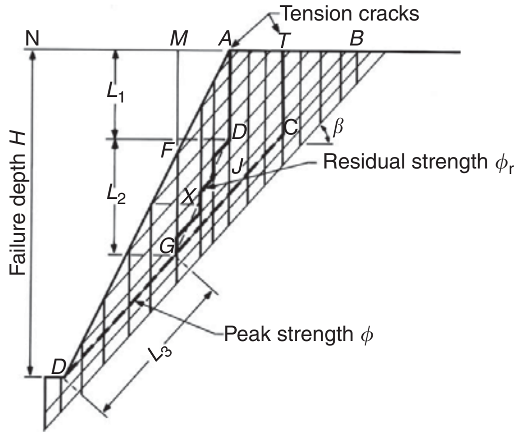


FIGURE 1.74
Geometry of two intersecting joint sets failing progressively (the multilinear failure surface). (From Barton, N., *Proceedings of ASCE, 13th Symposium on Rock Mechanics,* Urbana, Illinois [1971], 1972, pp. 139–170. With permission.)

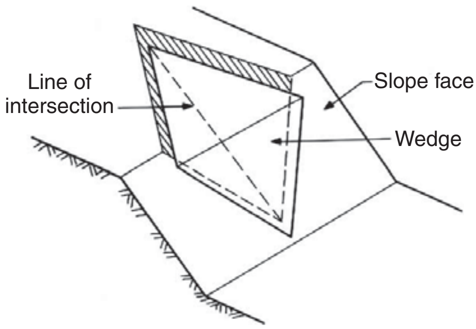


FIGURE 1.75
Geometry of a triangular wedge.

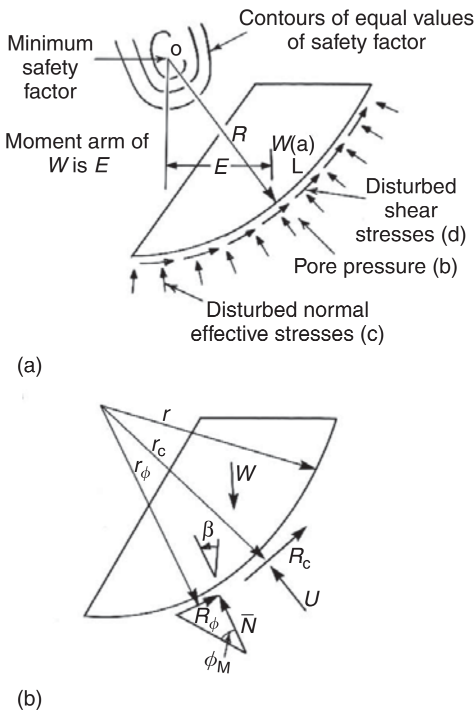


FIGURE 1.76
Forces acting on a free body with circular failure: (a) distributed stresses; (b) resultant forces.

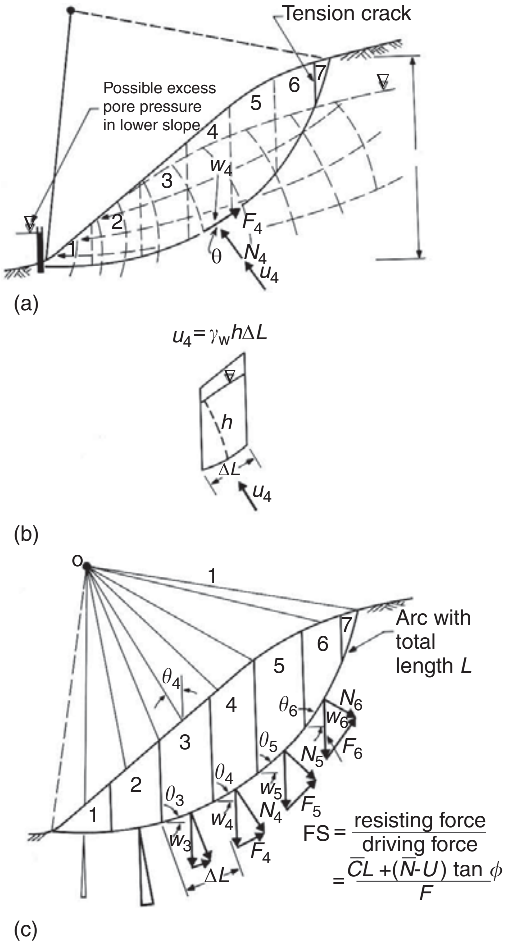


FIGURE 1.77
The ordinary method of slices. (a) Draw the slope and flow net to scale. Select failure surface and divide into equal slices of similar conditions. (b) Pore pressures for slice U_4. (c) Determine forces and safety factor:

$N_i = W_i\cos\theta - u_iL_i$ and $F = W_i\sin\theta$

$$FS = \frac{\sum \bar{c}\Delta L_i + \sum N_i \tan\phi}{\sum W_i \sin\theta_i}$$

Repeat for other r values to find FS_{min}.

cases (Janbu, 1973; Morganstern and Sangrey, 1978). Suitable for hand calculations it is particularly useful in slopes undergoing progressive failure on a long, noncylindrical failure surface where the location is known. It was used to analyze the failure illustrated in Figure 1.33. The equations are as follows:

$$FS = f_o\,(\Sigma\{[c'b + (W - ub)\tan\phi']\,[1/\cos\theta\, M_i(\theta)]\}/\Sigma W\tan\theta + V) \tag{1.17}$$

where

$$M_i(\theta) = \cos\theta_I\,(1 + \tan\theta_i\tan\phi'/FS) \tag{1.18}$$

$$f_o = 1 + 0.50\,[d/L\text{-}1.4(d/L^2]\quad \text{for } c > 0,\ \phi > 0 \tag{1.19}$$

$$f_o = 1 + 0.31\,[d/L\text{-}1.4(d/L^2]\quad \text{for } c = 0$$

$$f_o = 1 + 0.66\,[d/L\text{-}1.4(d/L^2]\quad \text{for } \phi = 0$$

Data are input for each slice. Substitution of Equation 1.18 into Equation 1.17 results in FS on both sides of the equation. For solution a value for FS_1 is assumed and FS_2 calculated,

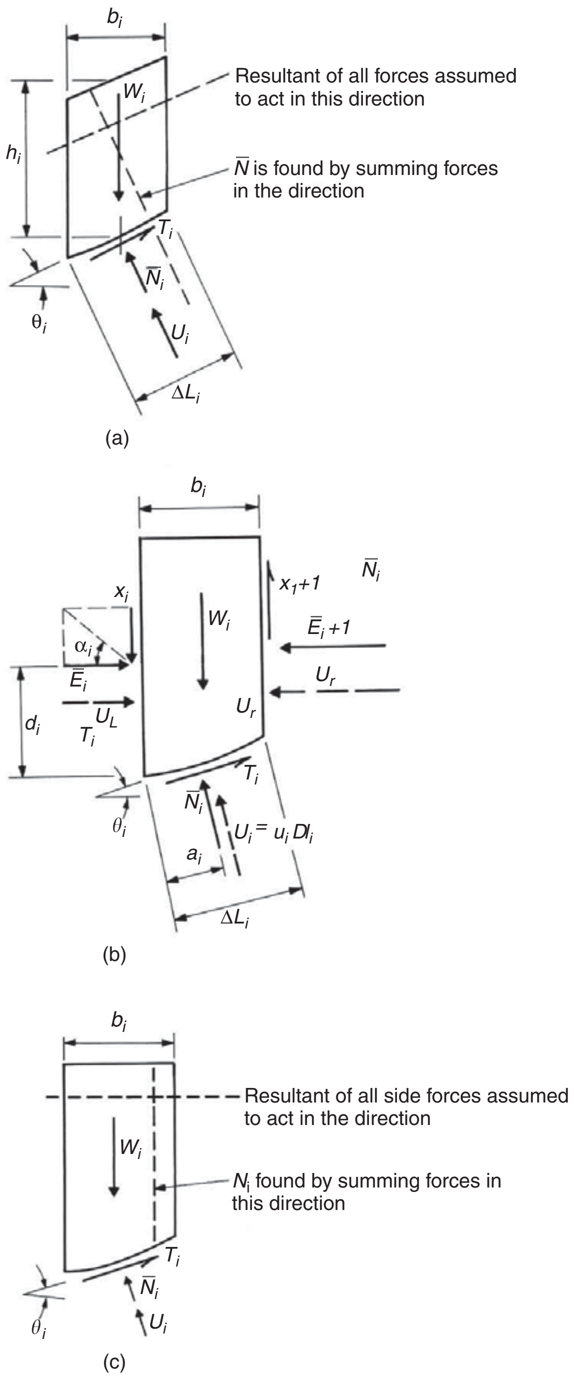


FIGURE 1.78
Force systems acting on a slice assumed by various analytical methods. (a) Ordinary method of slices. (b) Complete force system assumed by Bishop. (c) Force system for modified Bishop. (From Lambe, T. W. and Whitman, R. V., *Soil Mechanics,* Wiley, New York, 1969. Reprinted with permission of John Wiley & Sons, Inc.)

and the procedure is repeated until $FS_1 = FS_2$. Solutions converge rapidly. Parameters *d* and *L* are illustrated in Figure 1.79a. Janbu Equation 1.17 is similar to the modified Bishop, except for the parameter f_o and the denominator, which in the Bishop equation is $\sum W \sin\theta$.

The Morgenstern and Price method: This method (Morgenstern and Price, 1965) can be used to analyze any shape of failure surface and satisfies all equilibrium conditions. It is based on the Bishop method and requires a computer for solution. There are several

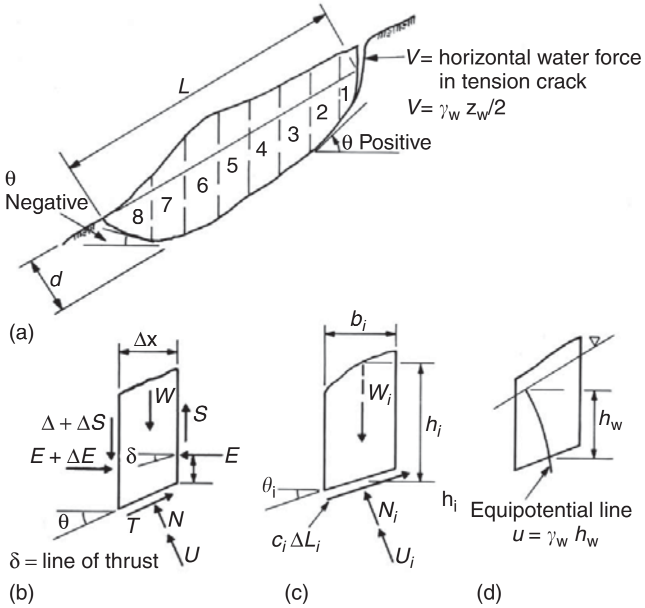


FIGURE 1.79
Nomenclature for Janbu's simplified method of analysis for noncircular failure surfaces: (a) failure mass geometry; (b) slice force system; (c) slice parameters for analysis; (d) determination of pore pressure u. Useful for cases where the failure surface is known or assumed.

theoretically possible positions for the line of action of the resultant forces between slices, and the line of action must be checked to determine if it is a possible one.

Spencer's method: This method (Spencer, 1967, 1973) is similar to the Morganstern and Price method.

Friction Circle method: See Taylor (1948).

Charts based on total stresses are used to find FS in terms of slope height and angle, and of soil parameters c, ϕ, and unit weight. The direction of the resultant normal stress for the entire free body is slightly in error because the resultant is not really tangent to the friction circle, but the analysis provides a lower bound for safety and is therefore conservative. The Taylor charts are strictly valid only for homogeneous slopes with no seepage. They consider that shear strength is mobilized simultaneously along the entire failure surface and that there is no tension crack. They are used for rough approximations and preliminary solutions of more complex cases. If the strength values vary along the failure surface they are averaged to obtain working values. This must be done with judgment and caution. For the foregoing conditions of validity, solutions using the charts are in close agreement with the method of slices described below.

Earthquake Forces

Pseudostatic methods have been the conventional approach in the past (Terzaghi, 1950). The stability of a potential sliding mass is determined for static loading conditions, and the effects of earthquake forces are accounted for by including equivalent horizontal forces acting on the mass. The horizontal force is expressed as the product of the weight of the sliding mass and a seismic coefficient that is expressed as a fraction of the acceleration due to gravity (see Section 3.3.4).

Dynamic analysis techniques provide for much more realistic results but also have limited validity. These techniques are described by Newmark (1965) and Seed (1968). The Newmark sliding block analyses are widely used for estimating the permanent displacements of slopes during earthquakes (Kramer and Smith, 1997; Wartman et al., 2003).

Summary

The applicability of mathematical analysis to various slope failure forms and the elements affecting slope failures are summarized in Table 1.4. General methods of stability analysis for sliding masses and the applicable geologic conditions are summarized in Table 1.5. Strength parameters acting at failure under various field conditions are summarized in Table 1.6.

1.3.3 Slope Characteristics

General

Qualitative assessment of slopes provides the basis for predicting the potential for failure and selecting practical methods for treatment, and for evaluating the applicability of mathematical solutions. The two major elements of qualitative assessment are slope characteristics (geology, geometry, surface conditions, and activity) and the environment (weather conditions of rainfall and temperature, and earthquake activity). The discussion in Section 1.2 presents relationships between the mode of slope failure and geologic conditions, as well as other slope characteristics, giving a basis for recognizing potential slope stability problems.

TABLE 1.4

Comparison of Elements and Classification of Geological and Engineering Failure Forms

	Elements of Slope Failures[a]						Engineering Failure Forms[b]				
Geologic Failure Forms	**Slope Inclination**	**Slope Height**	**Material Structure**	**Material Strength**	**Seepage Forces**	**Runoff**	**Infinite Slope**	**Single Planar Failure Surface (Simple Wedge and Sliding Block)**	**Multiple Planar Failure Surfaces (Oblique Surfaces Intersecting Parallel Surfaces)**	**Multiple Planar Failure Surfaces Intersecting Obliquely (Wedge)**	**Cylindrical Failure Surface**
Falls	P	N	P	P	P	N	N	N	N	P	N
Planar slides (translational block glides)	P	S	P	P	P	M	A	A	A	A	N
Rotational slides in rock	P	P	P	P	P	M	N	N	N	N	A
Rotational slides in soil	P	P	P	P	P	M	N	N	N	N	A
Lateral spreading and progressive failure	S	M	P	P	P	N	N	S	S	N	N
Debris slides	P	M	P	P	P	N	S	S	S	S	N
Debris avalanches	P	S	S	S	P	P	N	N	N	N	N
Debris flows	P	S	S	S	P	P	N	N	N	N	N
Rock fragment flow	P	S	P	P	P	N	N	N	N	N	N
Soil and mud flows	S	S	S	P	P	M	N	N	N	N	N
Submarine slides	S	S	P	P	P	N	N	N	N	N	S

[a] P — primary cause; S — secondary cause; M — minor effect; N — little or no effect.
[b] A — application; S — some application; P — poor application; N — no application.

TABLE 1.5

General Method of Stability Analysis and Applicable Geologic Condition for Slides

General Method of Analysis	Geologic Conditions
Infinite slope — (depth small compared with length of failure surface)	Cohesionless sands. Residual or colluvial soils over shallow rock. Stiff fissured clays and marine shales in the highly weathered zone
Limited slope	Sliding block
Simple wedge (single planar failure surface)	Interbedded dipping rock or soil Faulted or slickensided material Stiff to hard cohesive soil, intact, on steep slope
General wedge (multiple planar failure surfaces)	Sliding blocks in rock masses Closely jointed rock with several sets Weathered interbedded sedimentary rock Clay shales and stiff fissured clays Stratified soils Side-hill fills over colluvium
Cylindrical arc	Thick residual or colluvial soil Soft marine or clay shales Soft to firm cohesive soils

TABLE 1.6

Strength Parameters Acting at Failure under Various Field Conditions

Material	Field Conditions	Strength Parameters
(*a*) Cohesionless sands	Dry	ϕ
(*b*) Cohesionless sands	Submerged slope	$\bar{\phi}$
(*c*) Cohesionless sands	Slope seepage with top flow line coincident with and parallel to slope surface	$\bar{\phi}$
(*d*) Cohesive materials	Saturated slope, short-term or undrained conditions (ϕ=0)	s_u
(e) Cohesive materials (except for stiff fissured clays and clay shales)	Long-term stability	$\bar{\phi}$, $\bar{c}$
(*f*) Stiff fissured clays and clay shales	Part of failure surface	$\bar{\phi}_r$
(*g*) Soil or rock	Part of failure surface	$\bar{\phi}$, $\bar{c}$
	Existing failure surfaces	$\bar{\phi}_r$
(*h*) Clay shales or existing failure surfaces	Seepage parallel and top flow line coincident to slope surface	$\bar{\phi}_r$
(*i*) Pore-water pressures	Reduce ϕ in *e*, *f*, and *g* in accordance with seepage forces, γ applicable; or boundary water forces, γ_t applicable In (*c*) and (*h*), pore pressures reduce effectiveness by 50%	$(p–u)$ tan $\bar{\phi}$

Geologic Conditions

Significant Factors

- Materials forming the slope (for rock, the type and the degree of weathering; for soil, the type as classified by origin, mode of occurrence, and composition) as well as their engineering properties.
- Discontinuities in the formations, which for rock slopes include joints, shears, bedding, foliations, faults, slickensides, etc.; and, for soils include layering, slickensides, and the bedrock surface.
- Groundwater conditions: static, perched or artesian, and seepage forces.

Conditions with a High Failure Incidence

- Jointed rock masses on steep slopes can result in falls, slides, avalanches, and flows varying from a single block to many blocks.
- Weakness planes dipping down and out of the slope can result in planar failures with volumes ranging from very large to small.
- Clay shales and stiff fissured clays are frequently unstable in the natural state where they normally fail by shallow sloughing, but cuts can result in large rotational or planar slides.
- Residual soils on moderate to steep slopes in wet climates may fail progressively, generally involving small to moderate volumes, although heavy runoff can result in debris avalanches and flows, particularly where bedrock is shallow.
- Colluvium is generally unstable on any slope in wet climates and when cut can fail in large volumes, usually progressively.
- Glaciolacustrine soils normally fail as shallow sloughing during spring rains, although failures can be large and progressive.
- Glaciomarine and other fine-grained soils with significant granular components can involve large volumes in which failure may start by slumping, may spread laterally, and under certain conditions may become a flow.
- Any slope exposed to erosion at the toe, particularly by stream activity; cut too steeply; subject to unusually heavy rainfall; or experiencing deformation.

Some Examples

A general summary of typical forms of slope failures as related to geologic conditions is given in Table 1.3.

Dipping beds of sedimentary rocks in mountainous terrain are often the source of disastrous slides or avalanches (see Figure 1.18). Very large planar slides failing along a major discontinuity occur where the beds incline in the slope direction. On the opposite side of the failure in Figure 1.18 the slope is steeper and more stable because of the bedding orientation. Failures will generally be small, evolving under joint sets, although disastrous avalanches have occurred under these conditions, such as the one at Turtle Mountain, Alberta.

Orientation of joints with respect to the rock slope face controls stability and the form of failure. The near-vertical slope in the 40-year-old railroad cut illustrated in Figure 1.80 is stable in decomposed amphibolite gneiss because of the vertical jointing. The cut shown in Figure 1.81 is near that of Figure 1.80 but at a different station and on the opposite side of the tracks. Here, the slope is much flatter, approximately 1:1, but after 40 years is still experiencing failures such as that of the wedge shown in the photo that broke loose along the upper joints and slid along a slickensided surface. These examples illustrate how joint orientation controls slope stability, even in "soft" rock. The cuts were examined as part of a geologic study for 30 km of new railroad to be constructed in the same formation but some distance away.

Sea erosion undercutting jointed limestone illustrated in Figure 1.82 was causing concern over the possible loss of the roadway, which is the only link between the town of Tapaktuan, Sumatra, and its airport. A fault zone may be seen on the right-hand side of the photo. For the most part, the joints are vertical and perpendicular to the cliff face, shown as plane *a* in Figure 1.83, and the conditions are consequently stable. Where the joints are parallel to the face and inclined into it, as shown by plane *b* in the figure, a potentially unstable condition exists. This condition was judged to prevail along a short stretch of road beginning to the

FIGURE 1.80
Near-vertical slope in 40-year-old railroad cut standing stable in amphibolite gneiss because of vertical jointing. (Tres Ranchos, Goias, Brazil). Compare with Figure 1.81.

right of the photo, shown in Figure 1.84. The recommendation was to cut into the landward slope and relocate the roadway away from the sea cliff along this short stretch. The very costly alternative was to relocate the roadway inland around and over the coastal mountain.

The major cause of instability in colluvial soil slopes is illustrated in Figure 1.85. The slide debris impedes drainage at the toe and causes an increase in pore-water pressures in (b) over those in (a). The sketch also illustrates the importance of placing piezometers at different depths because of pressure variations. Conditions in (b) apply also to the case of a side-hill embankment for which a free-draining blanket beneath the fill would be necessary to provide stability.

Slope Geometry

General

The significant elements of slope geometry are inclination, height, and form. Aspects of inclination and height, as they relate to a particular point along a slope, are described in Section 1.1.4. This section is more concerned with the form and other characteristics of an entire slope as they affect seepage and runoff, which can be dispersed by the geometrical configuration of the slope or can be concentrated. The difference influences slope stability.

The examples given are intended to illustrate the *importance of considering the topography of an entire slope* during roadway planning and design, not only the immediate cut or fill area.

Topographic Expression

In both natural and cut slopes, the topographic expression has a strong influence on where failure may occur since landform provides the natural control over rainfall infiltration and runoff when geologic factors are constant. In Figure 1.86, runoff is directed away from the

FIGURE 1.81
Same cut as in Figure 1.80, but at a different station. Wedge failure along dipping joints.

convex nose form at (a) and a cut made there will be stable at a much steeper angle than at (b), where runoff is concentrated in the swale or concave form. Runoff and seepage at (c) are less severe than at (b) but still a problem to be considered. Natural slides, avalanches, and flows usually will not occur at (a), but rather at (b) and (c), with the highest incidence at (b) (see Figure 1.89 and Figure 1.90).

Location of Cut on Slope

Cuts in level ground or bisecting a ridge perpendicular to its strike will be stable at much steeper inclinations than cuts made along a slope, parallel to the strike (side-hill cuts). The side-hill cut in Figure 1.87 intercepts seepage and runoff from upslope and will be much less stable on its upslope side than on its downslope side where seepage is directed away from the cut. The treatment to provide stability, therefore, will be more extensive on the upslope side than on the opposite side.

The significance of cut locations along a steeply inclined slope in mountainous terrain in a tropical climate is illustrated in Figure 1.88. A cut made at location 1 will be much less stable than at location 3, and treatment will be far more costly because of differences in runoff and seepage quantities. River erosion protection or retention of the cut slope at 1 can be more costly than the roadway itself. Retention would not be required at 3 if a stable cut angle were selected, but might be required at 2 together with positive seepage control.

FIGURE 1.82
Sea erosion undercutting limestone and causing rockfalls. Slope failure could result in loss of roadway in photo middle (Tapaktuan, Sumatra) (see Figure 1.83 and Figure 1.84).

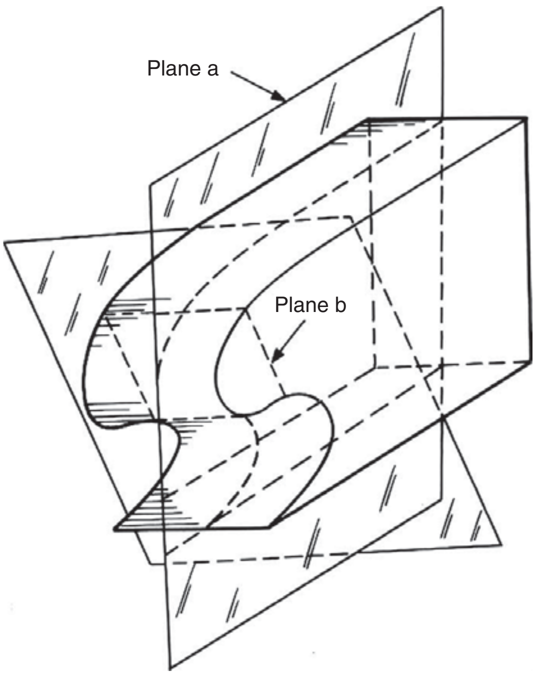


FIGURE 1.83
Orientation of fracture planes controls rock mass stability. Plane (a) represents stable conditions and plane (b) unstable.

Surface Conditions

Seepage Points

Observations of seepage points should be made in consideration of the weather conditions prevailing during the weeks preceding the visit, as well as the season of the year, and

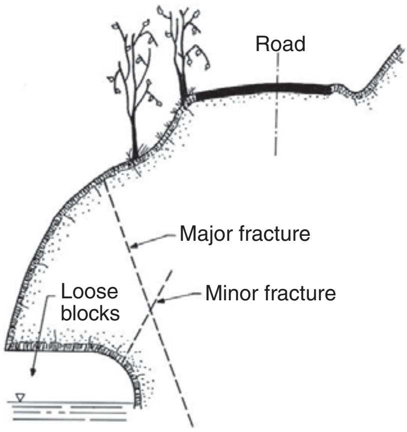


FIGURE 1.84
Possible orientation of fracture planes at km 10+750 which might lead to a very large failure (Tapaktuan, Sumatra).

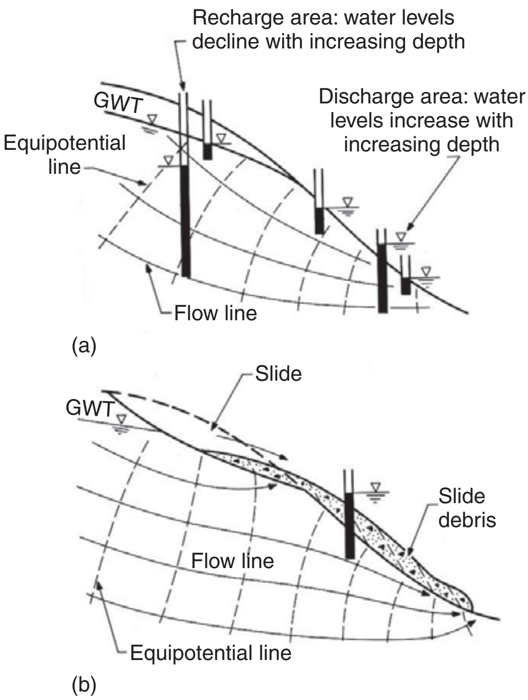


FIGURE 1.85
The effect of colluvium on groundwater flow in a slope: (a) flow in slope before slide; (b) flow in slope with mantle of slide debris. (From Patton, F. D. and Hendron, A. J., Jr., *Proceedings of the 2nd International Congress, International Association of Engineering Geology*, São Paulo, 1974, P.V–GR 1. With permission.)

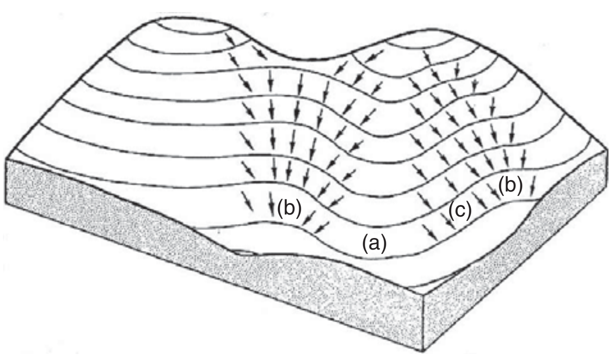


FIGURE 1.86
The influence of topography on runoff and seepage. Cuts and natural slopes at convex slopes at (a) are relatively stable compared with the concave slope at (b) or the slope at (c).

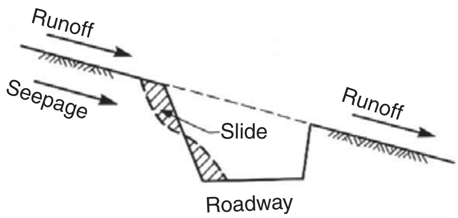


FIGURE 1.87
Upslope side of hillside cut tends to be much less stable than downslope side because of runoff and seepage.

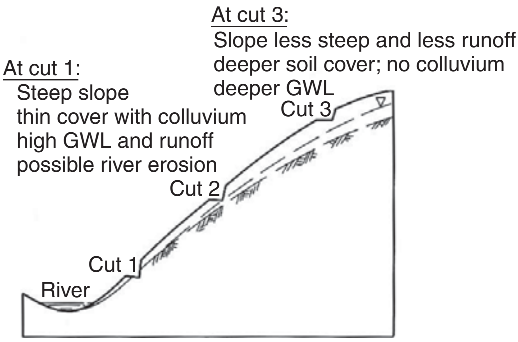


FIGURE 1.88
Stability problems may be very much related to the cut location along a steeply inclined slope in mountainous terrain in a tropical climate, and generally decrease in the upslope direction.

regional climatic history. No slope seepage during a rainy period may be considered as very favorable for stability, if there is no blockage from ice, colluvium, etc. On the other hand, seepage during a dry period signifies that a substantial increase in seepage will occur during wet periods. Toe seepage indicates a particularly dangerous condition, especially during dry periods.

Vegetation

Density of vegetation is an important factor in slope stability. Recently cleared upslope areas for logging, farming, or grazing are very likely to be locations where failures will occur in freshly made cuts, or in old cuts during severe weather conditions. Removal of vegetation permits an increase in erosion, a reduction in strength in the shallow portions of the slope from the loss of root structure, an increase in infiltration during rainy periods, and an increase in evaporation during dry spells resulting in surface desiccation and cracking.

Certain types of vegetation may be indicators of potential instability. For example, in tropical climates, such as in Brazil and Indonesia, banana plants seem to favor colluvial soil slopes, probably because colluvium has a higher moisture content than the residual soils in the same area.

Indications of Instability

Surface features indicating instability include tilted or bending tree trunks, tilted poles and fence posts, tension cracks along the slope and beyond the crest, and slump and hummocky topography, as described and illustrated in Sections 1.1.2 and 1.2.

Slope Activity

Degrees of Activity

Slopes reside at various degrees of activity, as discussed in Section 1.1.2, ranging through stable slopes with no movement, early failure stages with creep and tension cracks, intermediate failure stages with significant movement, partial total failures with substantial displacement, to complete failure with total displacement.

Evidence that a slope is unstable requires an assessment of the imminence of total collapse, and, if movement is occurring, how much time is available for treatment and stabilization. Tension cracks, in particular, serve as an early warning of impending failure and are commonly associated with the early stages of many failures. Their appearance, even together with scarps, does not necessarily mean, however, that failure is imminent. The slope shown in Figure 1.89 appears precipitous but the slide has moved very little over a 4-year period, an interval of lower than normal rainfall (see Section 1.3.4). Note the "nose" location; a massive slide has already occurred along the slope at the far left in the photo that closed the highway for a brief time. This "total failure," shown in Figure 1.90, occurred where the slope shape was concave, a few hundred meters from the slope in Figure 1.89.

Movement Velocity vs. Failure

The most significant factors indicating approaching total failure for many geologic conditions are velocities of movement and accelerations.

During surface monitoring, both vertical and horizontal movements should be measured, and evaluated in terms of velocity and acceleration. There is a lack in the literature of definitive observational data that relate velocity and acceleration to final or total failure in a manner suitable for formulating judgments as to when failure is imminent. Movement velocities before total failure and the stabilization treatment applied are summarized in Table 1.7 for a few cases from the literature. From the author's experience and literature review, it appears that, as a rule of thumb, if a slope of residual or colluvial soils is moving at a rate of the order of 2 to 5 cm/day (0.8 to 2.0 in./day) during a rainy season with the probability of storms, and if the velocity is increasing, final failure is imminent

FIGURE 1.89
Slump movement caused by cut made in residual soils for the Rio-Santos Highway (Itaorna, Brazil) has remained stable as shown for at least 4 years. (Photo taken in 1978.) The slope form is convex. The failure shown in Figure 1.90 was located around the roadway bend in the photo (left).

FIGURE 1.90
Debris avalanche scar along the Rio-Santos Highway at Praia Brava, Itaorna, Brazil. The slope form was concave; location is around the bend from Figure 1.89.

TABLE 1.7
Velocities of Slide Movements before Total Failure and Solutions[a]

Location	Material	Movement Velocity	Solution	Reference
Philippine Islands	Weathered rock (open-pit mine)	2 cm/day	Horizontal adits	Brawner (1975)
Santos, Brazil	Colluvium (cut)	2.5 cm/day	Trenches, galleries	Fox (1964)
Rio de Janeiro state, Brazil	Residuum (cut)	0.4–2.2 cm/day	None applied, no total failure in 20 years	Garga and DeCampos (1977)
Rio-Santos Highway, Brazil	Residuum (cut)	2–3.5 cm/day for first 2 weeks, 30 cm/day during 6th week, then failure	Removal of failure mass	Hunt (1978) (see Section 1.3.3)
Golden slide	Debris (cut)	2.5 cm/day	Horizontal drains, vertical wells	Noble (1973)
Pipe Organ slide	Debris (cut)	5 cm/week	Gravity drains	Noble (1973)
Vaiont, Italy	Rock-mass translation	1 cm/week, then 1 cm/day, then 20–30 cm/day and after 3 weeks, 80 cm/day and failure		Kiersch (1965)
Portuguese Bend	Lateral spreading	1956–1957, 5–12 cm/yr 1958, 15–60 cm/yr 1961–1968, 1–3m/yr 1968, dramatic increase, houses destroyed 1973, 8 cm/day during dry season 1973, 10 cm/day during wet season 1973, 15 cm/day during heavy rains No total mass failure	None	Easton (1973)

[a] From examples given in Chapter 1.

and may occur during the first heavy rain, or at some time during the rainy season. The following two case histories relate slope movements to total failure.

Case 1: A roadway cut made in residual soils in the coastal mountains of Brazil continued to show instability through creep, tension cracks, small slumps, and periodic encroachment on the roadway for a period of several years. An intermediate failure stage occurred on November 29, 1977, after a weekend of moderately heavy rain and a period during which the highway department had been removing material from the slope toe. Figure 1.91, a photo taken from a helicopter about 10 days after the failure, illustrates the general conditions. Tension cracks have opened at the base of the forward transmission tower, a large scarp has formed at midslope, and the small gabion wall at the toe has failed.

Figure 1.92 illustrates the tension crack and the distortion in the transmission tower shown in Figure 1.91. The maximum crack width was about 30 cm (12 in.) and the scarp was as high as 50 cm (20 in.). Slope movement measurements were begun immediately by optical survey and the transmission lines were quickly transferred to a newly constructed tower situated farther upslope.

For the first 2 weeks after the initial movement, a period of little rainfall, the vertical drop along the scarp was about 2 to 3.5 cm/day (0.8 to 1.4 in./day). In 5 weeks, with occasional rainy periods, the scarp had increased to 3 m (10 ft). Finally, after a weekend of heavy rains the slide failed totally in its final stage within a few hours, leaving a scarp about 30 m (100 ft) in height as shown in Figure 1.93, and partially blocking the roadway. Excavation removed the slide debris, and 2 years afterward the high scarp still remained. The tower in the photo was again relocated farther upslope. Although future failures will occur, they will be too far from the roadway to cover the pavement.

Remediating the slope by permitting failure is an example of one relatively low-cost treatment method. Cutting material from the head of the slide would be an alternate method, but there was concern that perhaps time for construction prior to total failure

FIGURE 1.91
Rotational slide at Muriqui, Rio de Janeiro, Brazil, initiated by the road cut, had been active for 4 years. Major movements began after a weekend of heavy rains, endangering the transmission tower. Tension crack and distorted transmission tower shown in the inset, Figure 1.92.

FIGURE 1.93
Total collapse of slope shown in Figure 1.91 left a high head scarp and temporarily closed the roadway. Failure occurred 6 weeks later during the rainy season after several days of heavy rains.

would be inadequate. Considering the slope activity, weather conditions, costs, and construction time, a retaining structure was considered as not practical.

Case 2: Another situation is similar to that described in Case 1 although the volume of the failing mass is greater. A 25° slope of residual soil has been moving for 20 years since it was activated by a road cut (Garga and DeCampos, 1977). Each year during the rainy season movement occurs at a velocity measured by slope inclinometer ranging from 0.4 to 2.2 cm/day (0.2 to 0.9 in./day). The movement causes slide debris to enter the roadway, from which it is removed. During the dry season movement ceases, and as of the time of the report (1977), total failure had not occurred.

1.3.4 Weather Factors

Correlations between Rainfall and Slope Failures

Significance

Ground saturation and rainfall are the major factors in slope failures and influence their incidence, form, and magnitude. Evaluating rainfall data are very important for anticipating and predicting slope failures. Three aspects are important:

1. Climatic cycles over a period of years, i.e., high annual precipitation vs. low annual precipitation.
2. Rainfall accumulation in a given year in relationship to normal accumulation.
3. Intensities of given storms.

Cumulative Precipitation vs. Mean Annual Precipitation

A study of the occurrence of landslides relative to the cumulative precipitation record up to the date of failure as a percentage of the mean annual precipitation (termed the cycle coefficient C_c) was made by Guidicini and Iwasa (1977). The study covered nine areas of the mountainous coastal region of Brazil, which has a tropical climate characterized by a wet season from January through March and a dry season, June through August.

Cumulative precipitation increases the ground saturation and a rise in the water table. A rainstorm occurring during the dry season or at the beginning of the wet season will have a lesser effect on slope stability than a storm of the same intensity occurring near the end of the wet season. A plot by month of the occurrence of failures as a function of the cycle coefficient is given in Figure 1.94. It is seen that the most catastrophic events occur toward the end of the rainy season, when the cumulative precipitation is higher than the mean annual.

Regarding rainfall intensity, Guidicini and Iwasa concluded that:

- Extremely intense rainfalls, about 12% greater than the mean annual rainfall (300 mm in 24 to 72 h) or more, can cause natural slope failures in their area, regardless of the previous rainfall history.
- Intense rainfalls, up to 12% of the mean annual, where the precipitation cycle is normal or higher, will cause failures, but if the preceding precipitation level is lower than the mean annual, failures are not likely even with intensities to 12%.
- Rainfalls of 8% or less of annual precipitation will generally not cause failures, regardless of the preceding precipitation, because the gradual increase in the saturation level never reaches a critical magnitude.
- A danger level chart (Figure 1.95) was prepared by Guidicini and Iwasa for each study area, intended to serve communities as a guide for assessing failure hazard in terms of the mean cumulative precipitation for a given year.

Such data can be used for temporarily closing mountain roadways, subject to slope failures, when large storms are expected during the rainy season. At the least warnings signs can be placed.

Evaluating Existing Cut Slope Stability

General

It is often necessary to evaluate a cut slope that appears stable to formulate judgments as to whether it will remain so. If a cut slope has been subjected during its lifetime to conditions

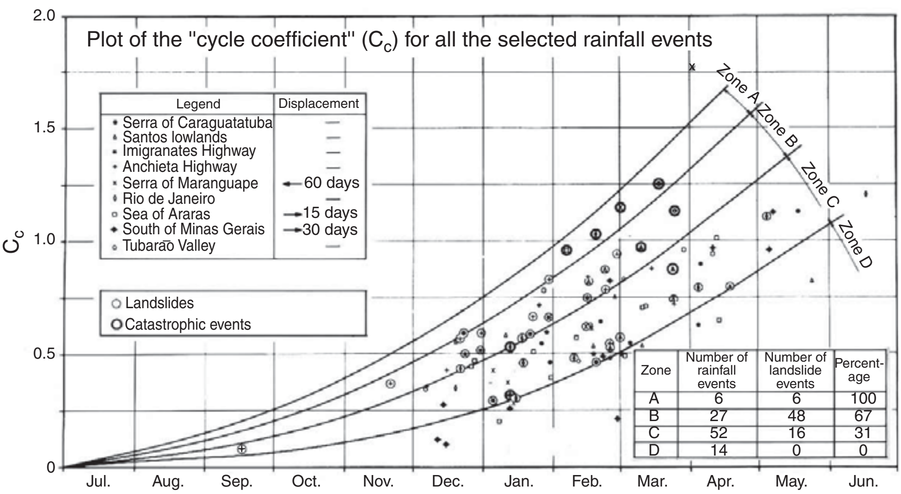


FIGURE 1.94
Comparison of landslide events and rainfall cycle coefficient C_c for coastal mountains of Brazil. (From Guidicini, G. and Iwasa, O. Y., *Bull. Int. Assoc. Eng. Geol.*, 16, 13–20, 1977. With permission.)

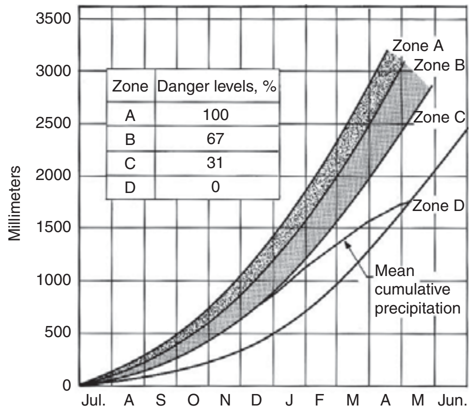


FIGURE 1.95
Correlation between rainfall and landslides (Serra de Caraguatatuba, São Paulo, Brazil). Chart based on rainfall record station No. E2-65-DAEE, installed in May 6, 1928, in the town of Caraguatatuba. Mean annual precipitation of 1905 mm based on 46 year record. Approximate station elevation of 10 m. (From Guidicini, G. and Iwasa, O. Y., *Bull. Int. Assoc. Eng. Geol.*, 16, 13–20, 1977. With permission.)

drier than normal and there have been no major storms, it can be stated that the slope has *not* been tested under severe weather conditions. It may be concluded that it is not necessarily a potentially stable slope. If a cut is failing under conditions of normal rainfall, it can be concluded that it will certainly undergo total failure at some future date during more severe conditions.

Case 1: In a study of slope failures on the island of Sumatra, examination was made of several high, steep slopes cut in colluvium which were subjected to debris and slump slides during construction. Failure occurred during a normal rainy season of 500 mm (20 in.) for the month of occurrence. The cuts were reshaped with some benching and flatter inclinations and have remained stable for a year of near-normal rainfall of about 2500 mm (98 in.) with monthly variations from 80 to 673 mm (3 to 26 in.).

Rainfall records were available during the study for only a 5-year period, but during the year before the cuts were made 1685 mm (66 in.) were reported for the month of December during monsoon storms. The cuts cannot be considered stable until subjected to a rainfall of this magnitude, unless there is an error in the data or the storm was a very unusual occurrence. Neither condition appears to be the case for the geographic location.

Case 2: A number of examples have been given of slope failures along the Rio Santos Highway that passes through the coastal mountains of Brazil. Numerous cuts were made in the years 1974 and 1975 without retention, and a large number of relatively small slides and other failures have occurred. The solution to the problem adapted by the highway department, in most cases, is to allow the failures and subsequently clean up the roadway. As of 1980, except for short periods, during the slide illustrated in Figure 1.93, the road has remained in service.

A review of the rainfall records for the region during the past 40 years (from 1984) revealed that the last decade had been a relatively dry period with rainfall averaging about 1500 to 2000 mm (59 to 79 in.). During the previous 30 years, however, the annual rainfall averaged 2500 to 3500 mm (98 to 138 in.). One storm in the period dropped 678 mm (27 in.) in 3 days (see Section 1.2.8). In view of the already unstable conditions along the roadway, if the weather cycle changes from the currently dry epoch to the wetter cycle of the previous epoch, a marked increase in incidence and magnitude of slope failures can be anticipated.

Temperatures

Freezing temperatures and the occurrence of frost in soil or rock slopes are highly significant. Ground frost can wedge loose rock blocks and cause falls, or in the spring months can block normal seepage, resulting in high water pressures which cause falls, debris avalanches, slides, and flows. A relationship among the number of rock falls, mean monthly temperature, and mean monthly precipitation is given in Figure 1.96. It is seen that the highest incidence for rock falls is from November through March in the Fraser Canyon of British Columbia.

1.3.5 Hazard Maps and Risk Assessment

Purpose

Degrees of slope-failure hazards along a proposed or existing roadway or other development can be illustrated on slope hazard maps. Such maps provide the basis not only for establishing the form of treatment required, but also for establishing the degree of urgency for such treatment in the case of existing works, or the programming of treatment for future works. They represent the product of a regional assessment.

Hazard Rating Systems

In recent years, various organizations have developed hazard rating systems. In the United States, the system apparently most commonly used is the "Rockfall Hazard Rating System" (RHRS) developed for the Federal Highway Administration by the Oregon State Highway Department (Pierson et al., 1990). Highway departments use the RHRS to inventory and classify rock slopes according to their potential hazard to motorists, and to identify those slopes that present the greatest degree of hazard and formulate cost estimates for treatments (McKown, 1999).

Some states, such as West Virginia (Lessing et al., 1994), have prepared landslide hazard maps. The Japanese have studied the relationships between earthquake magnitude and epicenter distance to slope failures in Japan and several other countries and have proposed procedures for zoning the hazard (Orense, 2003).

Example

The Problem

A 7-km stretch of existing mountain roadway with a 20-year history of slope failures, including rotational slides, debris slides, avalanches, and rock falls, was mapped in detail with respect to slope stability to provide the basis for the selection of treatments and the establishment of treatment priorities. A panoramic photo of the slopes in the higher elevations along the roadway is given in Figure 1.97.

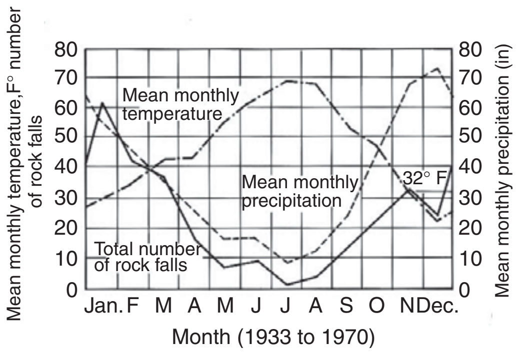


FIGURE 1.96
Number of rock falls, mean monthly temperature, and mean monthly precipitation in the Fraser Canyon of British Columbia for 1933–1970. (Peckover, 1975; from Piteau, D. R., *Reviews in Engineering Geology,* Vol. III, Coates, D. R., Ed., Geologic Society of America, Boulder, Colorado, 1977, pp. 85–112. With permission.)

FIGURE 1.97
(Upper left) km 52.5–53.5.

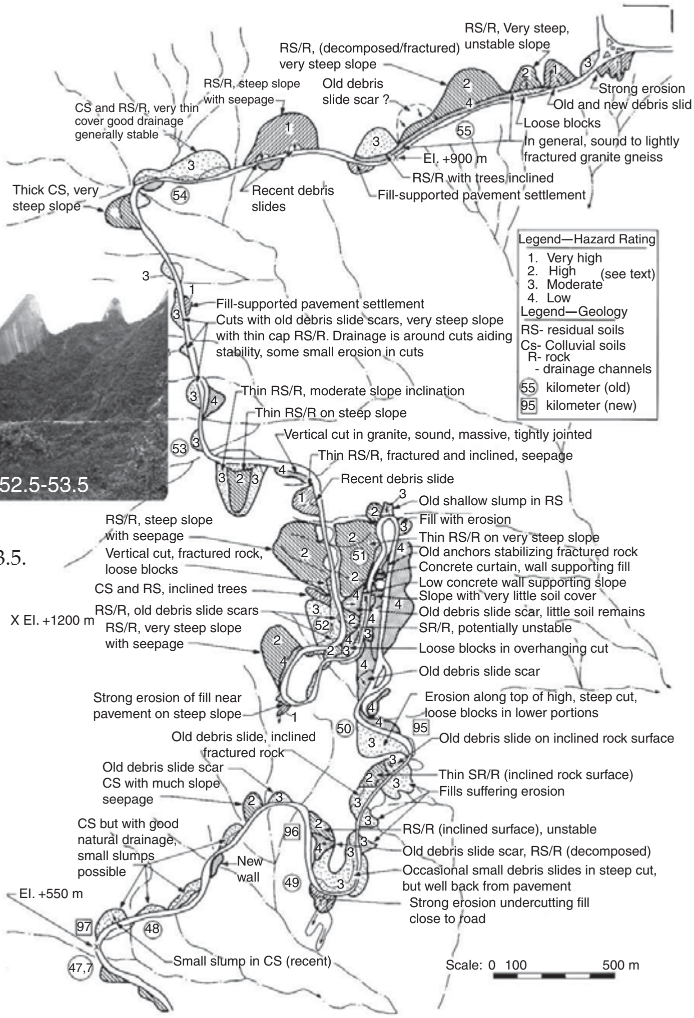


FIGURE 1.98
Slope failure hazard map for an existing roadway subject to numerous failures (BR-116, Rio de Janeiro, Brazil). A panorama of the road between km 52 and 53.5 is shown in the photo (Figure 1.97). A debris avalanche that occurred at km 56 is shown in Figure 1.10.

The Slope Failure Hazard Map

The map (Figure 1.98), prepared by the author during a 1979 study, illustrates the location of cuts and fills, drainage, and the degree of hazard. Maps accompanying the report gave geologic conditions and proposed solutions. The maps were prepared by enlarging relatively recent aerial photographs to a scale of 1:10,000 to serve as a base map for plotting, since more accurate maps illustrating the topography and locations of cuts and fills were not available.

Five degrees of hazard were used to describe slope conditions:

1. *Very high*: Relatively large failures will close the roadway. Slopes are very steep with a thin cover of residual or colluvial soils over rock, and substantial water penetrates the mass. Fills are unstable and have suffered failures.

2. *High*: Relatively large failures probably will close the road. Failures have occurred already in residual or colluvial soils and in soils over rock on moderately steep slopes. Fills are unstable.
3. *Moderate*: In general, failures will not close the road completely. Relatively small failures have occurred in residual soils on steep slopes, in colluvial soils on moderate slopes with seepage, in vertical slopes with loose rock blocks, and in slopes with severe erosion.
4. *Low*: Low cuts, cuts in strong soils or stable rock slopes, and fills. Some erosion is to be expected, but in general slopes are without serious problems.
5. *No Hazard*: Level ground, or sound rock, or low cuts in strong soils.

Note: Several years after the study, during heavy rains, a debris avalanche occurred at km 52.9 (a high-hazard location) carrying a minibus downslope over the side of the roadway resulting in a number of deaths

1.4 Treatment of Slopes

1.4.1 General Concepts

Selection Basis

Basic Factors

The first factor to consider in the selection of a slope treatment is its purpose, which can be placed in one of two broad categories:

- Preventive treatments which are applied to stable, but potentially unstable, natural slopes or to slopes to be cut or to side-hill fills to be placed
- Remedial or corrective treatments which are applied to existing unstable, moving slopes or to failed slopes

Assessment is then made of other factors, including the degree of the failure hazard and risk (see Section 1.1.3) and the slope condition, which can be considered in four general groupings as discussed below.

Slope Conditions

Potentially unstable natural slopes range from those subjects to falls or slides where development along the slope or at its base can be protected with reasonable treatments, to those where failures may be unpreventable and will have disastrous consequences. The potential for the latter failures may be recognizable, but since it cannot be known when the necessary conditions may occur, the failures are essentially unpredictable. Some examples of slope failures that could not have been prevented with any reasonable cost include those at Nevada Huascaran in the Andes that destroyed several towns (see Section 1.2.8), the Achocallo mudflow near La Paz (see Section 1.2.11), and the thousands of debris avalanches and flows that have occurred in a given area during heavy storms in mountainous regions in tropical climates (see Section 1.2.8).

Unstable natural slopes undergoing failure may or may not require treatment depending upon the degree of hazard and risk, and in some instances, such as Portuguese Bend (see Section 1.2.6), stabilization may not be practical because of costs.

Unstable cut slopes in the process of failure need treatment, but stabilization may not be economically practical.

New slopes formed by cutting or filling may require treatment by some form of stabilization.

Initial Assessment

An initial assessment is made of the slope conditions, the degree of the hazard, and the risk. There are then three possible options to consider for slope treatment:

- avoid the high-risk hazard, or
- accept the failure hazard, or
- stabilize the slope to eliminate or reduce the hazard.

Treatment Options

Avoid the High-Risk Hazard

Conditions: Where failure is essentially not predictable or preventable by reasonable means and the consequences are potentially disastrous, as in mountainous terrain subject to massive planar slides or avalanches, or slopes in tropical climates subject to debris avalanches, or slopes subject to liquefaction and flows, the hazard should be avoided.

Solutions: Avoid development along the slope or near its base and relocate roadways or railroads to areas of lower hazard where stabilization is feasible, or avoid the hazard by tunneling.

Accept the Failure Hazard

Conditions: Low to moderate hazards, such as partial temporary closure of a roadway, or a failure in an open-pit mine where failure is predictable but prevention is considered uneconomical, may be accepted.

Open-pit mines: Economics dictates excavating the steepest slope possible to minimize quantities to be removed, and most forms of treatment are not feasible; therefore, the hazard is accepted. Slope movements are monitored to provide for early warning and evacuation of personnel and equipment. In some instances, measures may be used to reduce the hazard where large masses are involved, but normally failures are simply removed with the equipment available.

Roadways: Three options exist besides avoiding the hazard, i.e., accept the hazard, reduce the hazard, or eliminate it. Acceptance is based on an evaluation of the degree of hazard and the economics of prevention. In many cases involving relatively small volumes failure is self-correcting and most, if not all, of the unstable material is removed from the slope by the failure; it only remains to clean up the roadway. These nuisance failures commonly occur during or shortly after construction when the first adverse weather arrives. The true economics of this approach, however, depends on a knowledgeable assessment of the form and magnitude of the potential failure, and assurance that the risk is low to moderate. Conditions may be such that small failures will evolve into very large ones, or that a continuous and costly maintenance program may be required. Public opinion regarding small but frequent failures of the nuisance type also must be considered.

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 conditions the hazard should be eliminated.

Slope Stabilization

Methods

Slope stabilization methods may be placed in five general categories:

1. Change slope geometry to decrease the driving forces or increase the resisting forces.
2. Control surface water infiltration to reduce seepage forces.
3. Control internal seepage to reduce the driving forces and increase material strengths.
4. Provide retention to increase the resisting forces.
5. Increase soil strength with injections. In a number of instances the injection of 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 Bowman, 1979). Strength increase in saltwater clays, however, was found to be low.

Stabilization methods are illustrated generally in Figure 1.99 and summarized in Table 1.8 with respect to conditions and general purpose. "General Purpose" indicates whether the aim is to prevent failure or to treat the slope by some remedial measure.

Selection

In the selection of the stabilization method or methods, consideration is given to a number of factors including:

- Material types composing the slope and intensity and orientation of the discontinuities.
- Slope activity.
- Proposed construction, whether cut or side-hill .
- Form and magnitude of potential or recurring failure (summary of preventive and remedial measures for the various failure forms is given in Table 1.9).

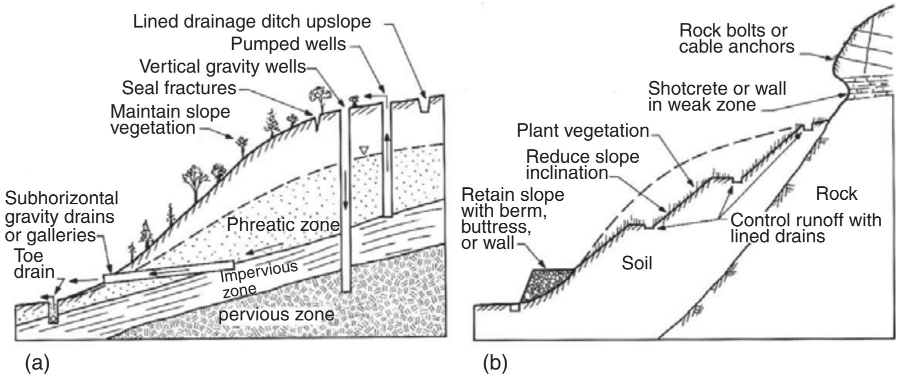


FIGURE 1.99
The general methods of slope stabilization: (a) control of seepage forces; (b) reducing the driving forces and increasing the resisting forces.

TABLE 1.8

Summary of Slope Treatment Methods for Stabilization

Treatment	Conditions	General Purpose (Preventive or Remedial)
	Change Slope Geometry	
Reduce height	Rotational slides	Prevent/treat during early stages
Reduce Inclination	All soil/rock	Prevent/treat during early stages
Add weight to toe	Soils	Treat during early stages
	Control Surface Water	
Vegetation	Soils	Prevent
Seal cracks	Soil/rock	Prevent/treat during early stages
Drainage system	Soil/decomposing rock	Prevent/treat during early stages
	Control Internal Seepage	
Deep wells	Rock masses	Temporary treatment
Vertical gravity drains	Soil/rock	Prevent/treat during early stages
Subhorizontal drains	Soil/rock	Prevent/treat early to Intermediate stages
Galleries	Rock/strong soils	Prevent/treat during early stages
Relief wells or toe trenches	Soils	Treat during early stages
Interceptor trench drains	Soils (cuts/fills)	Prevent/treat during early stages
Blanket drains	Soils (fills)	Prevent
Electroosmosis[a]	Soils (silts)	Prevent/treat during early stages: temporarily
Chemical[a]	Soils (clays)	Prevent/treat during early stages
	Retention	
Concrete pedestals	Rock overhang	Prevent
Rock bolts	Jointed or sheared rock	Prevent/treat sliding slabs
Concrete straps and bolts	Heavily jointed or soft rock	Prevent
Cable anchors	Dipping rock beds	Prevent/treat early stages
Wire meshes	Steep rock slopes	Contain falls
Concrete impact wells	Moderate slopes	Contain sliding or rolling blocks
Shotcrete	Soft or jointed rock	Prevent
Rock-filled buttress	Strong soils/soft rock	Prevent/treat during early stages
Gabion wall	Strong soils/soft rock	Prevent/treat during early stages
Crib wall	Moderately strong soils	Prevent
Reinforced earth wall	Soils/decomposing rock	Prevent
Concrete gravity walls	Soils to rock	Prevent
Anchored concrete curtain walls	Soils/decomposing rock	Prevent/treat — early to intermediate stages
Bored or root piles	Soils/decomposing rock	Prevent/treat — early stages

[a] Provides strength increase.

- Time available for remedial work on failed slopes, judged on the basis of slope activity, movement velocity and acceleration, and existing and near-future weather conditions.
- Degree of hazard and risk.
- Necessity to reduce or eliminate the hazard.

Hazard Elimination

The hazard can be eliminated by sufficient reduction of the slope height and inclination combined with an adequate surface drainage system or by retention.

Retention of rock slopes is accomplished with pedestals, rock bolts, bolts and straps, or cable anchors; retention of soil slopes is accomplished with the addition of adequate material at the toe of the slope or with properly designed and constructed walls.

TABLE 1.9

Failure Forms: Typical Preventive and Remedial Measures

Failure Form	Prevention during Construction	Remedial Measures
Rockfall	Base erosion protection Controlled blasting excavation Rock bolts and straps, or cables Concrete supports, large masses Remove loose blocks Shotcrete weak strata	Permit fall, clean roadway Rock bolts and straps Concrete supports Remove loose blocks Impact walls
Soil fall	Base erosion protection	Retention
Planar rock slide	Small volume: remove or bolt Moderate volume: provide stable inclination or bolt to retain Large volume: install internal drainage or relocate to avoid	Permit slide, clean roadway Remove to stable inclination or bolt Install internal drainage or relocate to avoid
Rotational rock slide	Provide stable inclination and surface drainage system Install internal drainage	Remove to stable inclination Provide surface drainage Install internal drains
Planar (debris) slides	Provide stable inclination and surface drainage control Retention for small to moderate volumes Large volumes: relocate	Allow failure and clean roadway Use preventive measures
Rotational soil slides	Provide stable inclination and surface drainage control, or retain	Permit failure, clean roadway Remove to stable inclination, provide surface drainage, or retain Subhorizontal drains for large volumes
Failure by lateral spreading	Small scale: retain Large scale: avoid and relocate, prevention difficult	Small scale: retain Large scale: avoid
Debris avalanche	Prediction and prevention difficult Treat as debris slide Avoid high-hazard areas	Permit failure, clean roadway: eventually self-correcting Otherwise relocate Small scale: retain or remove
Flows	Prediction and prevention difficult Avoid susceptible areas	Small scale: remove Large scale: relocate

Hazard Reduction

The hazard can be decreased by partially reducing the height and inclination or adding material at the toe; by planting vegetation, sealing cracks, installing surface drains, and shotcreting rock slopes; and by controlling internal seepage. In the last case, one can never be certain that drains will not clog, break off during movements, or be overwhelmed by extreme weather conditions.

Time Factor

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. To the extent possible, treatments should be performed during the dry season when movements will not close trenches, break off drains, or result in even larger failures when cuttings are made.

In general, the time required for various treatment measures are as follows:

- Sealing surface cracks and constructing interceptor ditches upslope are performed within several days, at most.
- Excavation at the head of a slide or the removal of loose blocks may require 1 to 2 weeks.
- Relief of internal water pressures may require 1 to 4 weeks for toe drains and trenches and 1 to 2 months for the installation of horizontal or vertical drains.
- Counterberms and buttresses at the toe require space, but can be constructed within 1 to 2 weeks.
- Retention with concrete walls can require 6 months or longer.

1.4.2 Changing Slope Geometry

Natural Slope Inclinations

Significance

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

Some examples of natural slope inclinations

- *Hard, massive rocks*: Maximum slope angle and height is controlled by the concentration and orientation of joints and by seepage. The critical angle for high slopes of hard, massive rock with random joint patterns and no seepage acting along the joints is about 70° (Terzaghi, 1962).
- *Interbedded sedimentary rocks*: Extremely variable, depending upon rock type, climate, and bedding thickness as well as joint orientations and seepage conditions. Along the river valleys, natural excavation may have reduced stresses sufficiently to permit lateral movement along bedding planes and produce bedding-plane mylonite shear zones. On major projects such shears should be assumed to exist until proven otherwise.
- *Clay shales*: 8 to 15°, but often unstable. When interbedded with sandstones, 20 to 45°.
- *Residual soils*: 30 to 40°, depending upon parent rock type and seepage.
- *Colluvium:* 10 to 20°, and often unstable.
- *Loess*: Often stands vertical to substantial heights.
- *Sands:* dry and "clean," are stable at the angle of repose ($i = \phi$).
- *Clays*: Depends upon consistency, whether intact or fissured, and the slope height.
- *Sand–clay mixtures*: Often stable at angles greater than repose as long as seepage forces are not excessive.

Cut Slopes in Rock

Excavation

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

Typical Cut Inclinations

Hard masses of igneous or metamorphic rocks, widely jointed, are commonly cut to 1H:4V (76°) as shown in Figure 1.100. Hard rock masses with joints, shears, or bedding representing major discontinuities and dipping downslope are excavated along the dip of the discontinuity as shown in Figure 1.101, although 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 Section 1.4.6).

Hard sedimentary rocks with bedding dipping vertically and perpendicularly to the face as in Figure 1.102 or dipping into the face; or horizontally interbedded hard sandstone

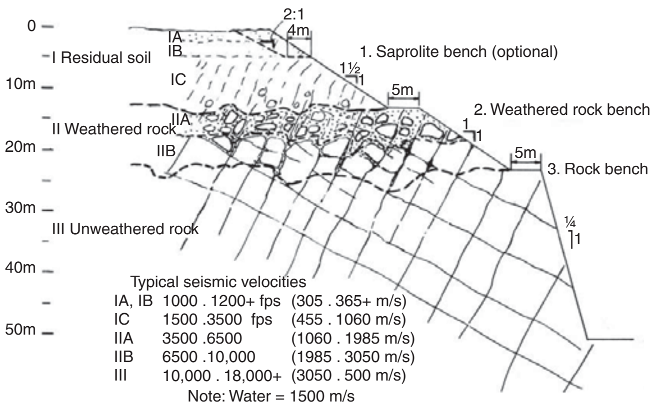


FIGURE 1.100
Typical cut slope angles for various rock and soil conditions. (From Deere, D. U. and Patton, F. D., *Proceedings of ASCE, 4th Pan American Conference on Soil Mechanics and Foundation Engineering,* San Juan, Puerto Rico, 1971, pp. 87–170. With permission.)

FIGURE 1.101
The face of the major joint surface in siltstone is stable but the overhanging blocks will fail along the same surface unless removed or retained (Sidikalang, Sumatra).

FIGURE 1.102
Folded, vertical beds in highly fractured, hard arenaceous shale are stable in near-vertical cuts (Sidikalang, Sumatra). Decreasing the slope inclination would increase susceptibility to erosion.

and shale are often cut to 1H:4V. The shale beds should be protected from weathering with shotcrete or gunite if they have expansive properties or are subjected to intense fracturing and erosion from weathering processes. Note that the steeper the cut slope the more resistant it is to erosion from rainfall.

Clay shale, unless interbedded with sandstone, is often excavated to 6H:1V (1.5°).

Weathered or closely jointed masses (except clay shale and dipping major discontinuities) require a reduction in inclination to between 1H:2V 1H:1V (63–45°) depending on conditions, or require some form of retention.

Benching

Benching is common practice in high cuts in rock slopes but there is disagreement among practitioners as to its value. Some consider benches as undesirable because they provide takeoff points for falling blocks (Chassie and Goughnor, 1975). To provide for storage they must be of adequate width. Block storage space should always be provided at the slope toe to protect the roadway from falls and topples.

Cut Slopes in Soils

Typical Inclinations

Thin soil cover over rock (Figure 1.103): The soil should be removed or retained as the condition is unstable.

Soil–rock transition (strong residual soils to weathered rock) such as in Figures 1.100 and 1.104 are often excavated to between 1H:1V to 1H:2V (45 to 63°), although potential failure along relict discontinuities must be considered. Saprolite is usually cut to 1.5H:1V (Figure 1.100).

Most soil formations are commonly cut to an average inclination of 2H:1V (26°) (Figure 1.100), but consideration must be given to seepage forces and other physical and environmental factors to determine if retention is required. Slopes between benches are usually steeper.

Benching and Surface Drainage

Soil cuts are normally designed with benches, especially for cuts over 25 to 30 ft (8 to 10 m) high. Benches reduce the amount of excavation necessary to achieve lower inclinations because the slope angle between benches may be increased.

FIGURE 1.103
Cuts in colluvium over inclined rock are potentially highly unstable and require either removal of soil or retention (Rio de Janeiro, Brazil).

FIGURE 1.104
Cut slopes at the beginning of a 300-m-deep excavation in highly decomposed igneous and metamorphic rocks for a uranium mine. Bench width is 20 m, height is 16 m, and inclination is 1H:1.5V (57°). Small wedge failure at right occurred along kaolinite-filled vertical relict joint. Part of similar failure shows in lower left.

Drains are installed as standard practice along the slopes and the benches to control runoff as illustrated in Figure 1.105 to Figure 1.107.

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 of relatively small failures. Placing material at the toe to form a counterberm increases the resisting forces. Benching may be effective in the early stages, but it did not fully stabilize the slope illustrated in Figure 1.29, even though a large amount of material was removed. An alternative is to permit movement to occur and remove debris from the toe; 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.

1.4.3 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 Protection

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

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 to 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 (Figure 1.105). On long cuts the interceptors are connected to downslope collectors (Figure 1.106). 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.

Roadway storm water drains should be located so as to not discharge on steep slopes immediate adjacent to the roadway. The failure shown in Figure 1.6 was caused by storm water discharge through a drainpipe connecting the catch basin on the upslope side of the roadway with a pipe beneath the roadway which exited on the slope.

1.4.4 Internal Seepage Control

General

Purpose

Internal drainage systems are installed to lower the piezometric level below the potential or existing sliding surface.

System Selection

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.

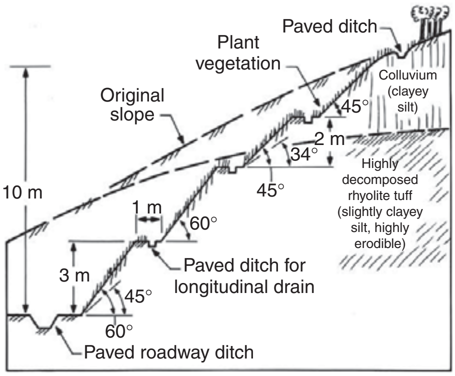


FIGURE 1.105
Benching scheme for cut in highly erodible soils in a tropical climate. Low benches permit maximum inclination to reduce the effect of runoff erosion.

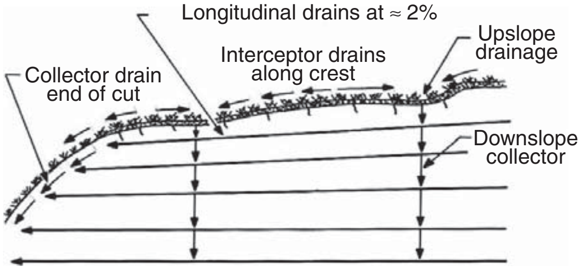


FIGURE 1.106
Sketch of slope face in Figure 1.105 showing system of longitudinal and downslope drains to control erosion.

Monitoring

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.

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 may 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 (see Figure 1.99a)

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, cylindrical 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 (see Section 1.2.7, discussion of the Pipe Organ Slide). 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 is 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, of 2in. diameter or larger, forced into a predrilled hole of slightly larger diameter than the pipe. Subhorizontal drains have been installed to lengths of more than 300 ft (100 m). Spacing depends on the type of material being drained; fine-grained soils may require spacing as close as 10 to 30 ft (3 to 8 m), whereas, for more permeable materials, 30 to 50 ft (8 to 15 m) may suffice.

Santi et al. (2003) report on recent installations of subhorizontal *wick drains* to stabilize slopes. Composed of geotextiles (polypropylene) they have the important advantages of stretching and not rupturing during deformation, and are resistant to clogging. Installation proceeds with a disposal plate attached to the end of a length of wick drain that is inserted into a drive pipe. The pipe, which can be a wire line drill rod, is pushed into the slope with a bulldozer or backhoe. Additional lengths of wicks and pipe are attached and driven into the slope. When the final length is installed, the drive pipe is extracted.

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 or slots are installed along a slope to intercept seepage in a cut or sliding mass, but they must be sufficiently deep. As shown in Figure 1.107, slotted pipe is laid in the trench bottom, embedded in sand, and covered with free-draining material, then sealed at the surface. The drain bottom should be sloped to provide for gravity drainage to a discharge point. 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 further decrease stability.

Relief trenches or slots 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 slump slides (Figure 1.25) where high toe seepage forces are the major cause of instability.

Electro-osmosis 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.

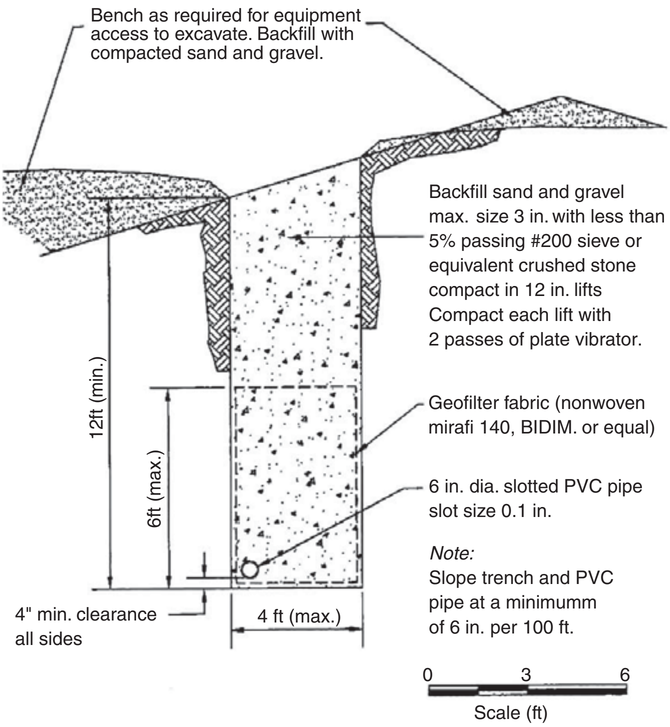


FIGURE 1.107
Typical slope trench drain.

Examples

Case 1: Open-Pit Mines (*Brawner, 1975*)

General: Problems encountered in open-pit mines in soft rock (coal, uranium, copper, and asbestos) during mining operations include both bottom heave of deep excavations (of the order of several hundred meters in depth) and slides, often involving millions of tons.

Solutions: Deep vertical wells that have relieved artesian pressures below mine floors where heave was occurring have arrested both the heave and the associated slope instability. Horizontal drainage in the form of galleries and boreholes as long as 150 m installed in the toe zone of slowly moving masses arrested movement even when large failures were occurring. In some cases, vacuum pumps were installed to place the galleries under negative pressures. Horizontal drains, consisting of slotted pipe installed in boreholes, relieve cleft-water pressures in jointed rock masses.

Case 2: Failure of a Cut Slope (*Fox, 1964*)

Geological conditions: The slope in Figure 1.108 consists of colluvial soils of boulders and clay overlying schist interbedded with gneiss. Between the colluvium and the relatively sound rock is a zone of highly decomposed rock.

Slide history: An excavation was made to a depth of 40 m into a slope with an inclination of about 28°. Upon its completion cracks opened, movement began, and springs appeared on the surface. The excavation was backfilled and the ground surface was graded to a uniform slope and covered with pitch. Monuments were installed to permit observations of movements. Even after the remedial measures were invoked, movement continued to endanger nearby structures. The greatest movement was about 2.5 cm/day. Failure had

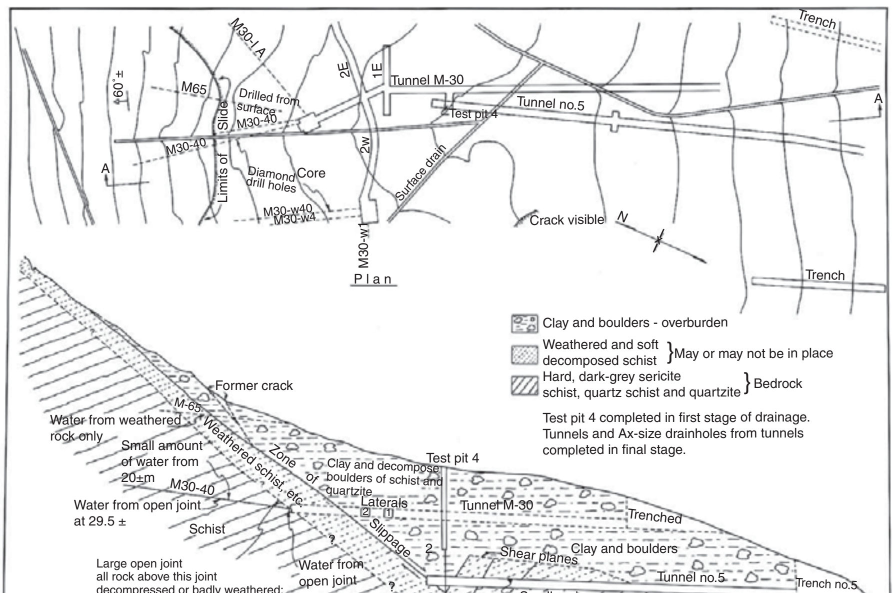


FIGURE 1.108
Stabilization of a failure in a colluvial soil slope using lateral drains and galleries. (From Fox, P.P., *Engineering Geology Case Histories Numbers 1-5,* The Geological Society of America, Engineering Geology Division, 1964, pp. 17–24. With permission.)

reduced the preexisting strengths to the extent that the original slope inclination was unstable. Piezometers installed as part of an investigation revealed that the highest pore pressures were in the fractured rock zone, under the colluvium.

Remedial measures: To correct the slide, Dr. Karl Terzaghi had a number of horizontal drill holes and galleries extended into the fractured rock as shown in Figure 1.108. The holes drained at rates of 10 to 100 L/min, and the water level in the piezometers continued to fall as work progressed. The slide was arrested and subsequent movements were reported to be minor.

Case 3: Construction of a Large Cut Slope (*D'Appolonia et al., 1967*)

Geologic conditions: As illustrated on the section, (Figure 1.109), conditions were characterized by colluvium, overlying sandstones and shales, and granular alluvium. Explorations were thorough and included test pits which revealed the overburden to be slickensided, indicating relict failure surfaces and a high potential for instability.

Treatment: Construction plans required a cut varying from 6 to 18 m in height in the colluvium along the slope toe. To prevent any movement, a system of trenches, drains, and galleries was installed. A cutoff trench, vertical drain, and gallery were constructed upslope, where the colluvium was relatively thin, to intercept surface water and water entering the colluvium from a pervious siltstone layer. A 2-m-diameter drainage gallery was excavated in the colluvium at about midslope to intercept flow from a pervious sandstone stratum and to drain the colluvium. Sand drains were installed downslope, near the proposed excavation, to enable the colluvium to drain by gravity into the underlying sand and gravel lying above the static water level, thereby reducing pore pressures in the colluvium. An anchored sheet-pile wall was constructed to retain the cut face; the other systems were installed to maintain the stability of the entire slope and reduce pressures on the wall.

1.4.5 Side-Hill Fills

Failures

Construction of a side-hill 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 Figure 1.110a, the embankment strains and concentric tension cracks form.

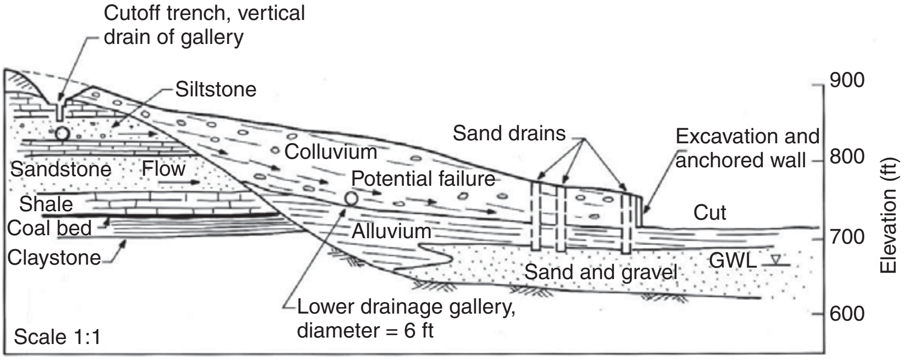
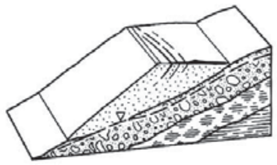
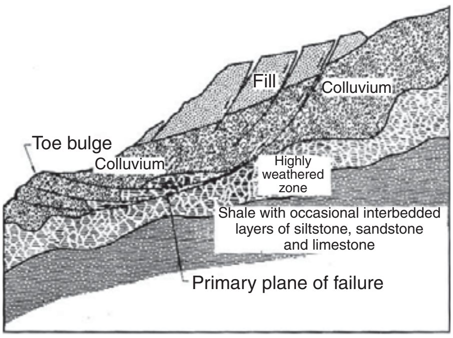


FIGURE 1.109
Stabilization of a colluvial soil slope (Weirton, West Virginia) with vertical drains and galleries. (From D'Appolonia, E. D. et al., 1967.)



(a)



(b)

FIGURE 1.110
Development of rotational failure in side-hill fill with inadequate subsurface drainage. (a) Early failure stage: concentric cracks show in pavement. (b) Rotational failure of side-hill fill over thick colluvium. (Part b from Royster, D. L., *Bull. Assoc. Eng. Geol.,* X, 1973. With permission.)

The movements develop finally into a rotational failure as shown in Figure 1.110b, a case of deep colluvium. Figure 1.111 illustrates a case of shallow residual soils.

Side-hill fills placed on moderately steep to steep slopes of residual or colluvial soils, in particular, are prone to be unstable unless seepage is properly controlled, or the embankment is supported by a retaining structure.

Stabilization

Preventive

Interceptor trench drains should be installed along the upslope side of all side-hill fills as standard practice to intercept flow as shown in Figure 1.112. 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.

A *free-draining blanket* should be installed between the fill and the natural slope materials to relieve seepage pressures from shallow groundwater conditions wherever either the fill or the natural soils are slow-draining, as shown in Figure 1.112. It is prudent to strip

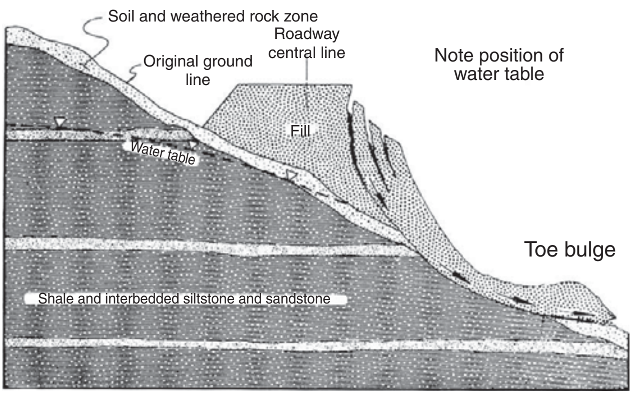


FIGURE 1.111
Development of rotational failure in side-hill fill underlain by thin formation of residual soils and inadequate subsurface drainage. (From Royster, D. L., *Bull. Assoc. Eng. Geol.,* X, 1973. With permission.)

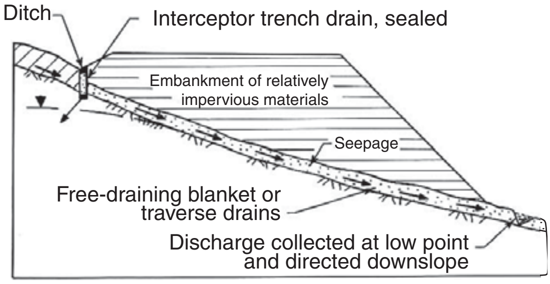


FIGURE 1.112
Proper drainage provisions for a side-hill fill.

potentially unstable upper soils, which are often creeping on moderately steep to steep slopes, to a depth where stronger soils are encountered, and to place the free-draining blanket over the entire area to be covered by the embankment. Discharge should be collected at the low point of the fill and drained downslope in a manner that will provide erosion protection.

Transverse drains extending downslope and connecting with the interceptor ditches upslope, parallel to the roadway, may provide adequate subfill drainage where anticipated flows are low to moderate.

Retaining structures may be economical on steep slopes that continue for some distance beyond the fill if stability is uncertain (see Figure 1.125).

Corrective

After the initial failure stage, subhorizontal drains may be adequate to stabilize the embankment if closely spaced, but they should be installed during the dry season since the use of water to drill holes during the wet season may accelerate total failure. An alternative is to retain the fill with an anchored curtain wall (see Figure 1.128).

After total failure, the most practical solutions are either reconstruction of the embankment with proper drainage, or retention with a wall.

1.4.6 Retention

Rock Slopes

Methods Summarized

The various methods of retaining hard rock slopes are illustrated in Figure 1.113 and described briefly below.

- Concrete pedestals are used to support overhangs, where their removal is not practical because of danger to existing construction downslope, as illustrated in Figure 1.114.
- Rock bolts are used to reinforce jointed rock masses or slabs on a sloping surface.
- Concrete straps and rock bolts are used to support loose or soft rock zones or to reduce the number of bolts as shown in Figure 1.115 and Figure 1.120.
- Cable anchors are used to reinforce thick rock masses.
- Wire meshes, hung on a slope, restrict falling blocks to movement along the face (Figure 1.116).
- Concrete impact walls are constructed along lower slopes to contain falling or sliding blocks or deflect them away from structures (see Figure 1.16).
- Shotcrete (Figure 1.117) is used to reinforce loose fractured rock, or to prevent weathering or slaking of shales or other soft rocks, especially where interbedded

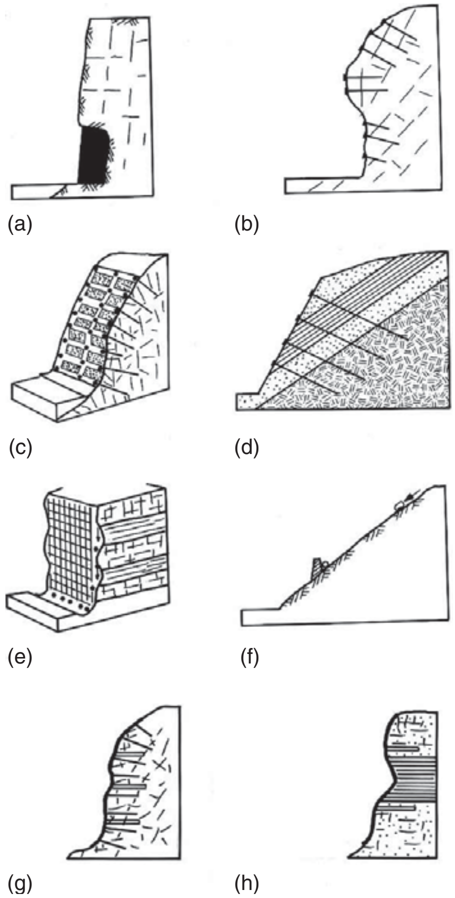


FIGURE 1.113
Various methods of retaining hard rock slopes: (a) concrete pedestals for overhangs; (b) rock bolts for jointed masses; (c) bolts and concrete straps for intensely jointed masses: (d) cable anchors to increase support depth; (e) wire mesh to constrain falls; (f) impact walls to deflect or contain rolling blocks; (g) shotcrete to reinforce loose rock, with bolts and drains; (h) shotcrete to retard weathering and slaking of shales.

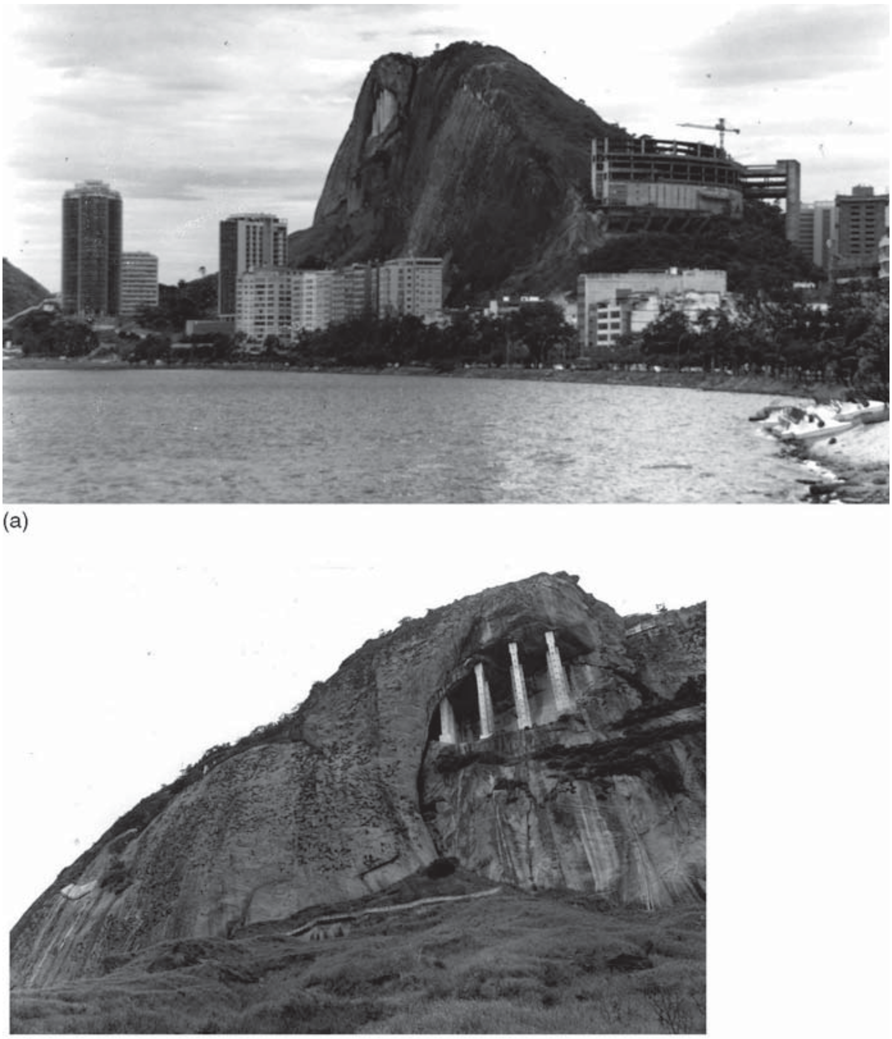
with more resistant rocks. Shotcrete is normally used with wire mesh and dowels, bolts or nails as discussed below.

- Gunite is similar to shotcrete except that the aggregate is smaller.

Reinforcing Rock Slopes

Rock anchors are tensile units, fixed at one end, used to place large blocks in compression, and should be installed as near to perpendicular to a joint as practical. The ordinary types consist of rods installed in drill holes either by driving and wedging, driving and expanding, or by grouting with mortar or resins as illustrated in Figure 1.118. Bolt heads are then attached to the rod and torqued against a metal plate to impose the compressive force on the mass. Weathering of rock around the bolt head may cause a loss in tension; therefore, heads are usually protected with concrete or other means, or used in conjunction with concrete straps in high-risk conditions.

Fully grouted rock bolts, illustrated in Figure 1.119, provide a more permanent anchor than those shown in Figure 1.118. The ordinary anchor is subject to loss in tension with time from several possible sources including corrosion from attack by aggressive water, anchorage slip or rock spalling around and under the bearing plate, and block movement along joints pinching the shaft. Care is required during grouting to minimize grout spread, which results in decreasing mass drainage, especially where bolts are closely spaced. Drain holes may be required.



(a)

(b)

FIGURE 1.114
Support of granite overhang with pedestals (Rio de Janeiro). Ancient slide mass of colluvium appears on lower slopes. (a) Side view and (b) face view.

A major installation of bolts and straps is illustrated in Figure 1.120, part of a 60-m-high rock slope (Figure 1.121), at the base of which is to be constructed a steel mill. The consultant selected the support system rather than shaving and blasting loose large blocks for fear of leaving a weaker slope in a high-risk situation.

Rock Dowels are fully grouted rock bolts, usually consisting of a ribbed reinforcing bar, installed in a drill hole and bonded to the rock over its full length (Franklin and Dusseult, 1989). Rock movement results in the dowels being self-tensioned. Grouting with resins is becoming more and more common because of easy installation and the rapid attainment of capacity within minutes of installation. Sausage-shaped resin packages are installed in the drill hole and a ribbed bar inserted and rotated to open the packages which contain resin and a catalyst (Figure 1.119c).

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

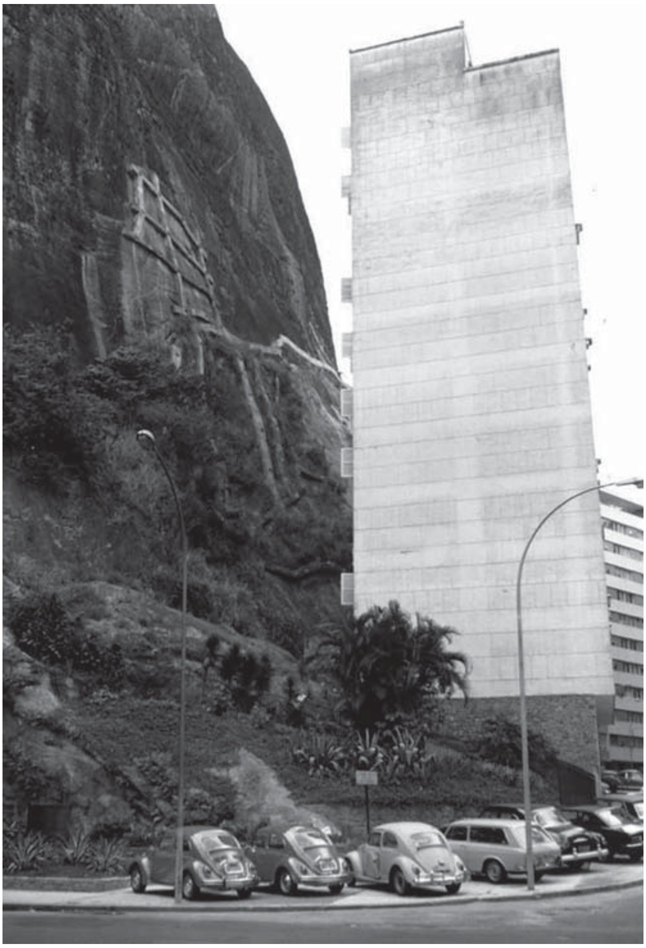


FIGURE 1.115
Stabilization of exfoliating granite with rock bolts and concrete straps (Rio de Janeiro).

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. Application is in 8 to 20 cm (3 to 4 in.) layers, each of which is permitted to set before application of subsequent layers. Originally, weep holes were installed to relieve seepage pressures behind the face, but modern installations include geocomposite drainage strips placed behind the shotcrete. 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. A typical installation is illustrated in Figure 1.124.

Soil Layer over Rock Slopes

As shown in Figure 1.50, cuts in mountainous terrain are inherently unstable where a relatively thin layer of soil overlies rock. The upper portion of the underlying rock normally is fractured and a conduit for seepage. Investigations made during the dry season may not encounter seepage in the rock, but flow during the wet season often is common and must be considered during evaluations.

Some typical solutions are given in Figure 1.122. In (a), design provides for inclining the cut in the rock and the soil; in (b), the soil is cut to a stable inclination and the rock cut made steeper by retention with shotcrete and rock nails; and in (c), the soil is retained with a top down wall (Figure 1.123) and the rock with shotcrete and nails (Figure 1.124).

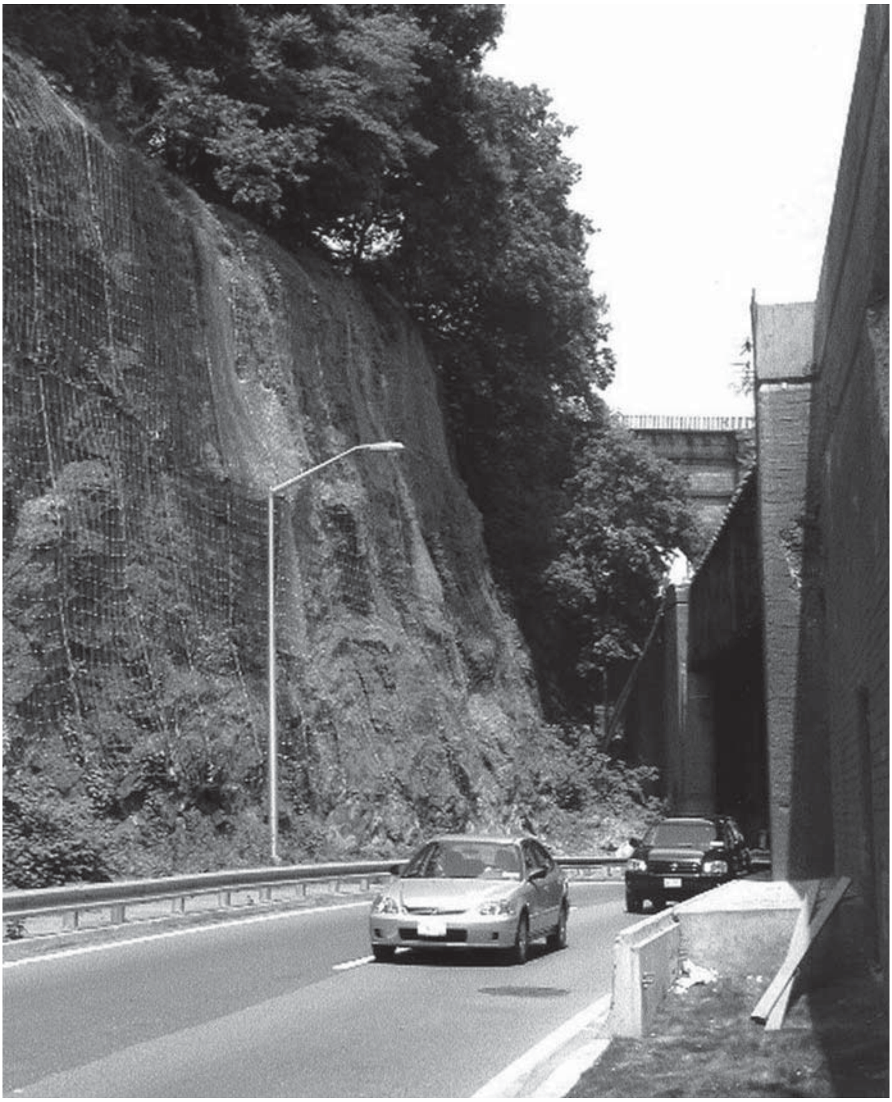


FIGURE 1.116
Wire mesh installed to prevent blocks of gneiss from falling on the Harlem River Drive, New York City.

Soil Slopes

Purpose

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, other types of walls that require cutting into the slope for construction are seldom suitable for retention of a failing slope.

Classes

The various types of walls are illustrated in Figure 1.125. 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.

FIGURE 1.117
Shotcrete applied to retain loose blocks of granite gneiss in cut. Untreated rock exposed in lower right (Rio de Janeiro).

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 rock-filled buttresses, gabion walls, crib walls, reinforced earth walls, concrete gravity walls, cantilever walls, and counterfort walls. A series of reinforced earth walls with a combined height of 100 ft is shown in Figure 1.126. Concrete walls are becoming relatively uncommon due to costs, construction time, and the fact that the slope is unsupported during construction.

Gebney and McKittrick (1975) report on a complex system of gravity walls installed to correct a debris slide along Highway 39, Los Angeles County, California. The reconstructed roadway was supported on a reinforced earth wall in turn supported by an embankment and a buttress at the toe of the 360-ft-high slope. Horizontal and longitudinal drains were installed to relieve hydrostatic pressures in that part of the slide debris which was not totally removed.

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 Figure 1.127, can be constructed to substantial heights and have a very high retention capacity. 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. An example of an anchored curtain wall retaining a side-hill fill is shown in Figure 1.128. A variation of the anchored curtain wall consists of anchored premolded concrete panels. The advantage of the system is that the wall conforms readily to the slope configuration, as shown in Figure 1.129.

An alternate "top-down" procedure, common to the United States, is to install anchored (tiebacks) soldier piles (Figure 1.123). As the excavation proceeds, breasting boards are installed in the soldier pile flanges to support the slope.

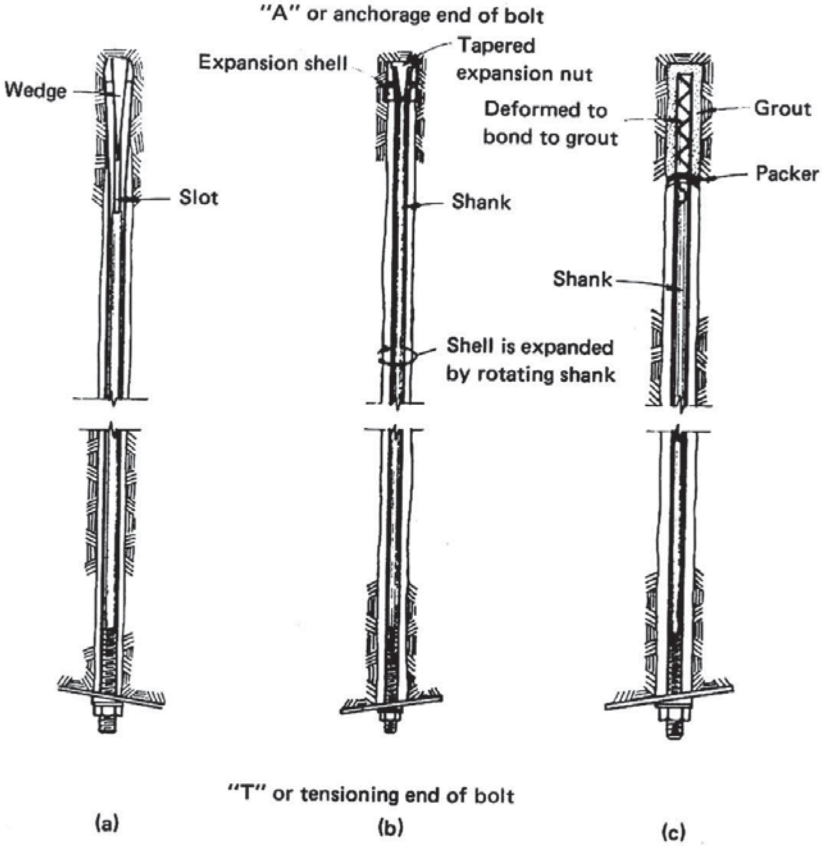


FIGURE 1.118
Types of ordinary rock bolts (anchors): (a) drive-set or slot and wedge bolt; (b) torque-set or expansion bolt; (c) grouted bolt. (From Lang, T. A., *Bull. Assoc. Eng. Geol.*, 9, 215–239, 1972. With permission.)

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.

Soil nailing is an *in situ* soil reinforcement technique that is finding increasing application. Long rods (nails) are installed to retain excavations or stabilize existing slopes. Nails are driven for temporary installations or drilled and grouted for permanent installations similar to the procedures described for shotcreting rock masses. Cohesive soils with LL $>$ 50 and PI $>$ 20 require careful assessment for creep susceptibility. Soil nailing is discussed in detail in Elias and Juran (1991).

Wall Characteristics

The general characteristics of retaining walls are summarized in Table 1.10. Also included are bored piles and root piles, not shown in Figure 1.125.

Wall Selection and Design Elements

The wall type is tentatively selected on the basis of an evaluation of the cut height, materials to be supported, wall purpose, and a preliminary economic study.

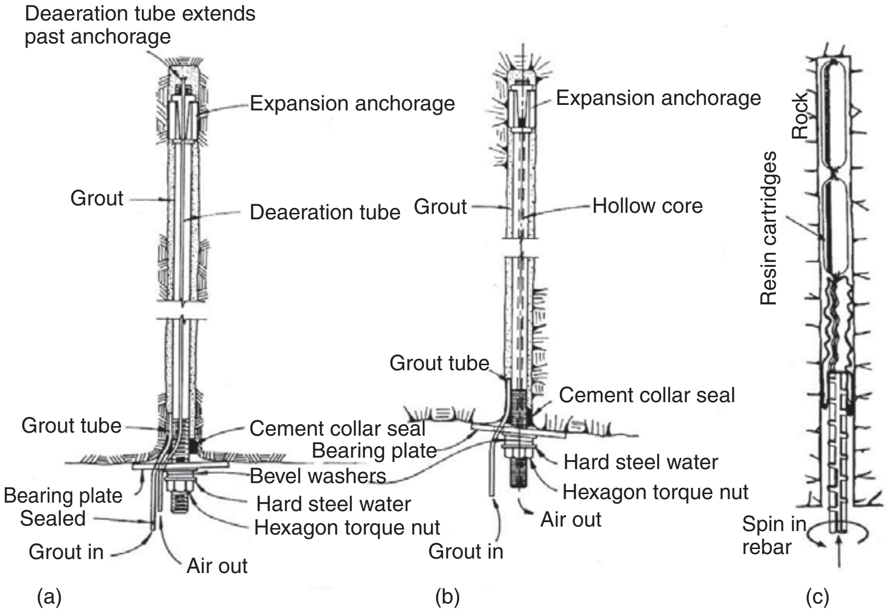


FIGURE 1.119
Fully grouted rock bolts or anchors: (a) grouted solid expansion anchorage bolt; (b) hollow-core grouted rock bolt. (a and b from Lang, T. A., *Bull. Assoc. Eng. Geol.*, 9, 215–239, 1972. With permission.) (c) Rebar and resin cartridges.

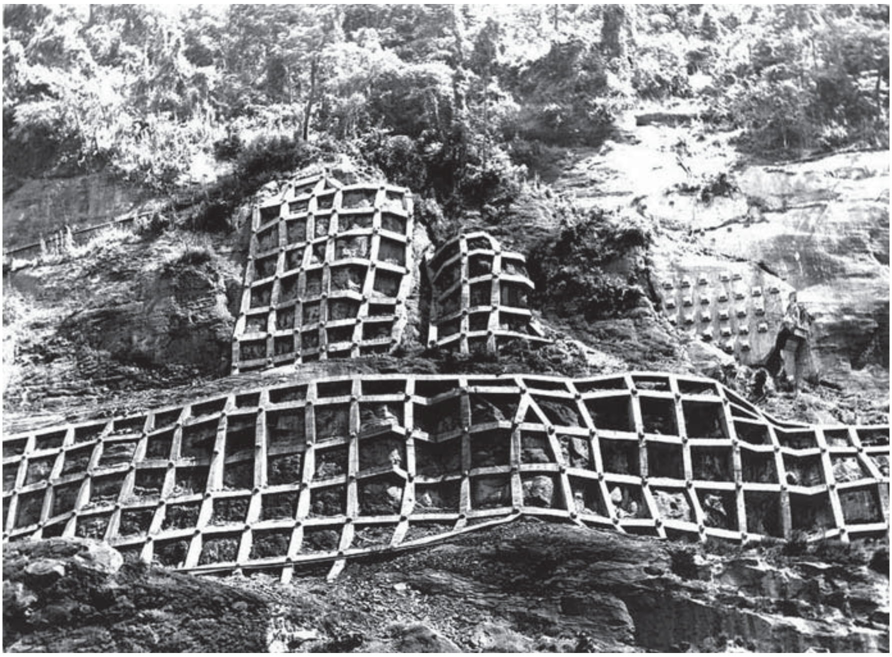


FIGURE 1.120
Loose granite gneiss blocks retained with rock bolts, and blocks of gneiss and a soft zone of schist retained with bolts and concrete straps on natural slope (Joao Monlevade, M. G., Brazil). (Courtesy of Tecnosolo. S. A.)

FIGURE 1.121
The 60-m-high slope of Figure 1.116 before treatment. The lower portions have been excavated into relatively sound gneiss. The workers (in circle) give the scale. Several large blocks weighing many tons broke loose during early phases when some scaling of loose blocks was undertaken.

Earth pressures are determined (magnitude, location, and direction), as influenced by the slope inclination and height; location and magnitude of surcharge loads; wall type, configuration and dimensions; depth of embedment; magnitude and direction of wall movement; soil parameters for natural materials and borrowed backfill; and seepage forces.

Stability of gravity walls is evaluated with respect to adequacy against overturning, sliding along the base, foundation bearing failure, and settlement. The slope must be evaluated with respect to formation of a possible failure surface beneath the wall.

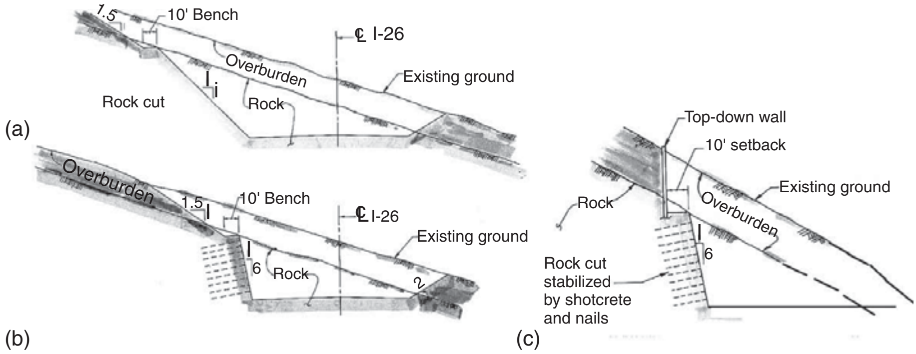


FIGURE 1.122
Various solutions for roadway cuts in soil over fractured rock. (a) Typical roadway rock cut; (b) roadway rock cut with nails; (c) roadway rock cut with top-down wall.

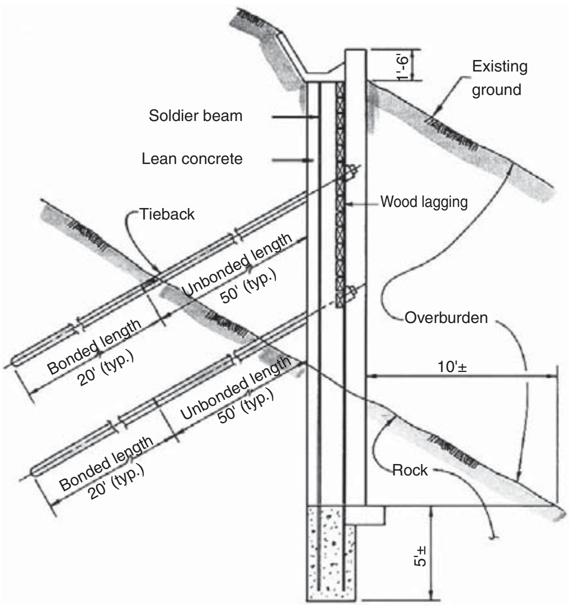


FIGURE 1.123
Typical top-down wall with anchored soldier beam and lagging.

Structural design proceeds when all of the forces acting on the wall have been determined. Beyond the foregoing discussion, the design of retaining walls is not within the scope of this volume.

1.5 Investigation: A Review

1.5.1 General

Study Scopes and Objectives

Regional Planning

Regional studies are performed to provide the basis for planning urban expansion, transportation networks, large area developments, etc. The objectives are to identify areas

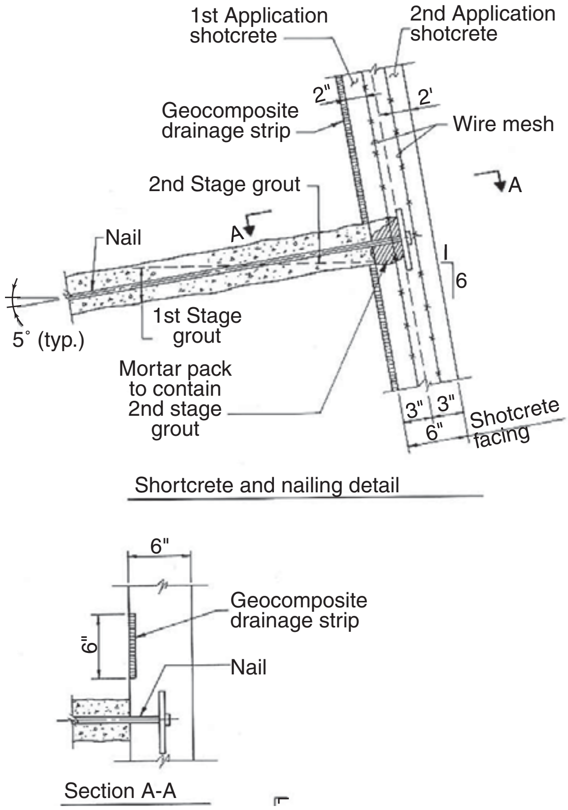


FIGURE 1.124
Retention with shotcrete and nails or bolts in soil or rock.

prone to slope failures, and the type, magnitude, and probability of occurrence. Hazard maps illustrate the findings.

Individual Slopes

Individual slopes are studied when signs of instability are noted and development is endangered, or when new cuts and fills are required for development. Studies should be performed in two phases: Phase 1, to establish the overall stability, is a study of the entire slope from toe to crest to identify potential or existing failure forms and their failure surfaces, and Phase 2 is a detailed study of the immediate area affected by the proposed cut or fill.

Considerations

Failure Forms and Hazard Degrees

Engineers and geologists must be aware of which natural slope conditions are hazardous, which can be analyzed mathematically with some degree of confidence, which are very sensitive to human activities on a potentially catastrophic scale, which can be feasibly controlled, and which are to be avoided. They should also be aware that in the present state of the art there are many limitations in our abilities to predict, analyze, prevent, and contain slope failures.

Rotational slides are the forms most commonly anticipated, whereas the occurrence of other forms is often neglected during slope studies. They are generally the least catastrophic of all forms, normally involve a relatively small area, give substantial warning in the form of surface cracking, and usually result in gradual downslope movement during the initial development stages. Several potential failure forms can exist in a given slope, however.

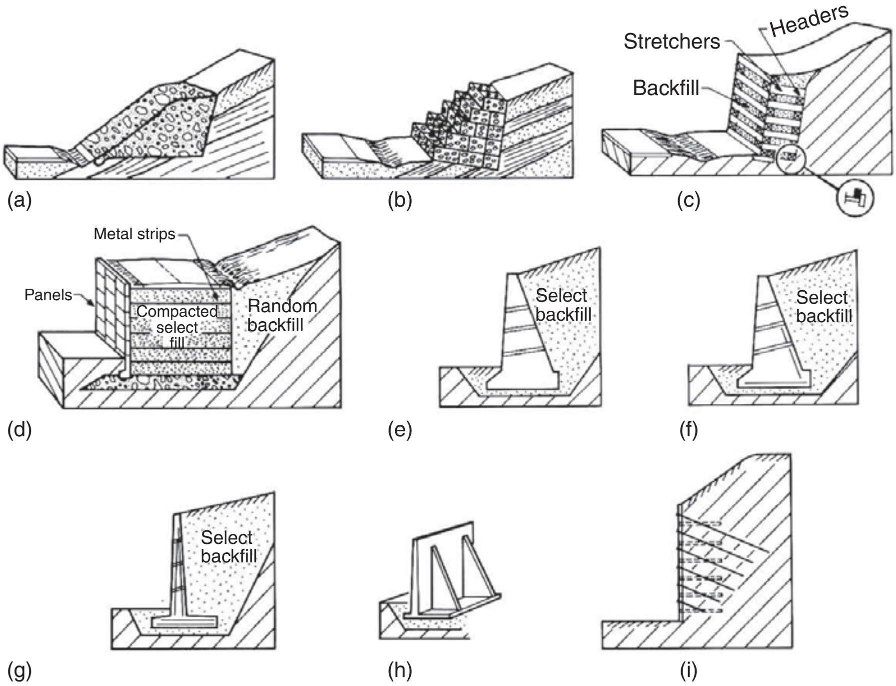


FIGURE 1.125
Various types of retaining walls: (a) rock-filled buttress; (b) gabion wall; (c) crib wall; (d) reinforced earth wall; (e) concrete gravity wall; (f) concrete-reinforced semigravity wall; (g) cantilever wall; (h) counterfort wall; (i) anchored curtain wall.

FIGURE 1.126
Reinforced earth walls over 100 ft in combined height support roadway fill, I–26, Sams Gap, Tennessee.

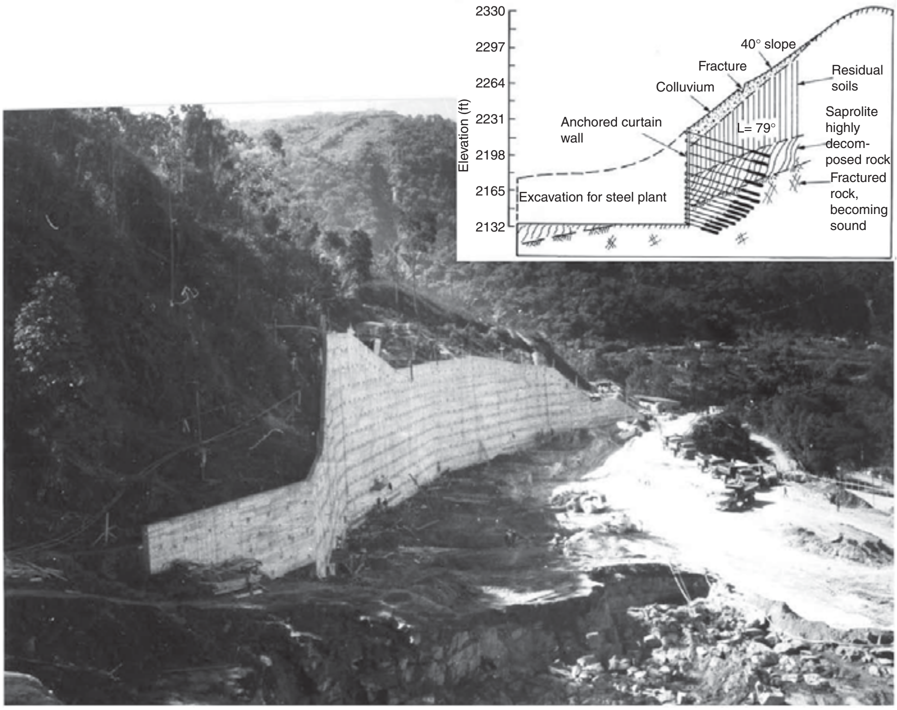


FIGURE 1.127
Anchored curtain wall being constructed to a height of 25 m and length of 150 m, completed to 15 m height (Joao Monlevade, M. G., Brazil). The wall, of maximum thickness of 50 cm, is constructed in sections 1.5 m high from the top down, with each section anchored to provide continuous support. Geologic Section is shown in the inset. (From Hunt, R. E. and Costa Nunes, A. J. da, *Civil Engineering, ASCE,* 1978, pp. 73–75. With permission.)

FIGURE 1.128
Anchored concete curtain wall supports fill placed over colluvium. Wall is pile-supported to rock. Slope movements are occurring downslope to the right but the wall is stable (Highway BR 277, Parana, Brazil).

FIGURE 1.129
Stabilization of a slump slide with anchored premolded concrete panels which conform readily to the shape of the slope (Itaorna, R. J., Brazil).

Planar slides in mountainous terrain, which usually give warning and develop slowly, can undergo sudden total failure, involving huge volumes and high velocities with disastrous consequences.

Falls, avalanches, and flows often occur suddenly without warning, move with great velocities, and can have disastrous consequences.

Stability Factors

Slope geometry and geology, weather conditions, and seismic activity are the factors influencing slope stability, but conditions are frequently transient. Erosion, increased seepage forces, strength deterioration, seismic forces, tectonic activity, as well as human activity, all undergo changes with time and work to decrease slope stability.

Selection of Slope Treatments

Slope treatments are selected primarily on the basis of judgment and experience, and normally a combination of methods is chosen.

For *active slides of large dimensions*, consideration should be given chiefly to external and internal drainage; retaining structures are seldom feasible.

Active slides of small dimensions can be stabilized by changing their geometry, improving drainage, and when a permanent solution is desired, containing them by walls. An alternative, which is often economically attractive, is to permit the slide to occur and to remove material continuously from the toe until a stable slope has been achieved naturally. The risk of total failure, however, must be recognized.

Cut slopes are first approached by determining the maximum stable slope angle; if too much excavation is required or if space does not permit a large cut, alternative methods employing retention are considered. It must be noted that side-hill cuts are potentially far

TABLE 1.10

Retaining Wall Characteristics

Wall Type	Description	Comments
Rock-filled buttress	Constructed of nondegradable, equidimensional rock fragments with at least 50% between 30 and 100 cm and not more than 10% passing 2-in. sleeve (Royster, (1979).	Gradation is important to maintain free-draining characteristics amid high friction angle, which combined with weight provides retention. Capacity limited by ϕ of approximately=40° and space available for construction.
Gabion wall	Wire baskets, about 50 cm each side, are filled with broken stone about 10 to 15 cm across. Baskets are then stacked in rows.	Free-draining. Retention is obtained from the stone weight and its interlocking and frictional strength. Typical wall heights are about 5 to 6 m, but capacity is limited by ϕ.
Crib wall	Constructed by forming interconnected boxes from timber, precast concrete, or metal members and then filling the boxes with crushed stone or other coarse granular material. Members are usually 2 m in length.	Free-draining. Height of single wall is limited to an amount twice the member length. Heights are increased by doubling box sections in depth. High walls are very sensitive to transverse differential settlements, and the weakness of cross members precludes support of high surcharge loads.
Reinforced earth walls	A compacted backfill of select fill is placed as metal strips, called ties, are embedded in the fill to resist tensile forces. The strips are attached to a thin outer skin of precast concrete panels to retain the face.	Free-draining and tolerant of different settlements, they can have high capacity and have been constructed to heights of at least 18 m. Relatively large space is required for the wall.
Concrete gravity wall	A mass of plain concrete.	Requires weep holes, free-draining backfill, large excavation. Can take no tensile stresses and is uneconomical for high walls.
Semigravity concrete wall	Small amount of reinforcing steel is used to reduce concrete volume and provide capacity for greater heights.	Requires weep holes, free-draining backfill, and large excavation. Has been constructed to heights of 32 m (Kulhawy, 1974).
Cantilever wall	Reinforced concrete with stem connected to the base. The weight of earth acting on the heel is added to the weight of the concrete to provide resistance.	Requires weep holes and free-draining backfill; smaller excavation than gravity walls but limited to heights of about 8 m because of inherent weakness of the stem-base connection.
Counterfort wall	A cantilever wall strengthened by the addition of counterforts.	Used for wall heights over 6 to 8 m.
Buttress wall	Similar to counterfort walls except that the vertical braces are placed on the face of the wall rather than on the backfill side.	As per cantilever and counterfort walls.
Anchored reinforced-concrete curtain wall	A thin wall of reinforced concrete is tied back with anchors to cause the slope and wall to act as a retaining system. A variation by Tecnosolo S. A. uses precast panels as shown in Figure 1.129.	Constructed in the slope from the top down in sections to provide continuous retention of the slope during construction. (All other walls require an excavation which remains open while the wall is erected.) Retention capacity is high and they have been used to support cuts in residual soils over 25 m in height. Drains are installed through the wall into the slope. *See also* Figure 1.127 and Figure 1.128.
Anchored steel sheet-pile wall	Sheet piles driven or placed in an excavated slope and tied back with anchors to form a flexible wall.	Seldom used to retain slopes because of its tendency to deflect and corrode and its costs, although it has been used successfully to retain a slope toe in conjunction with other stabilization methods (*see* Figure 1.109).
Bored piles	Bored piles have been used on occasion to stabilize failed slopes during initial stages and cut slopes.	Height is limited by pile capacity in bending. Site access required for large drill rig unless holes are hand-excavated.
Root piles	Three-dimensional lattice of small-diameter, cast-in-place, reinforced-concrete piles, closely spaced to reinforce the earth mass.	Trade name “Fondedile.” A retaining structure installed without excavation. Site access for large equipment required.

more dangerous than cuts made in level ground, even with the same cut inclination, depth, and geologic conditions. The significant difference is likely to be seepage conditions.

Side-hill fills must always be provided with proper drainage, and on steep slopes retention usually is prudent.

Slope Activity Monitoring

Where potentially dangerous conditions exist, monitoring of slope activity with instrumentation is necessary to provide early warning of impending failures.

Hazard Zoning

In cities and areas where potentially dangerous conditions exist and failures would result in disastrous consequences, such as on or near high, steep slopes or on sensitive soils near water bodies or courses, development should be prohibited by zoning regulations. Pertinent in this respect is a recent slide in Goteburg, Sweden (ENR, 1977), a country with a long history of slope failures in glaciolacustrine and glaciomarine deposits. Shortly after heavy rains in early December 1977, a slide occurred taking at least eight lives and carrying 67 single-family and row houses into a shallow ravine. Damage was over $7 million. The concluding statement in the article: "Last week's slide is expected to spark tighter controls of construction in questionable areas."

1.5.2 Regional and Total Slope Studies

Preliminary Phases

Objectives and Scope

The objectives of the preliminary phases of investigation, for either regional studies or for the study of a particular area, are to anticipate forms, magnitudes, and incidences of slope failures.

The study scope includes collection of existing data, generation of new data through terrain analysis, field reconnaissance, and evaluation.

Existing Data Collection

Regional data to be collected include: slope failure histories, climatic conditions of precipitation and temperature, seismicity, topography (scales of 1:50,000 and 1:10,000), and remote-sensing imagery (scales 1:250,000 to 1:50,000).

At the project location, data to be collected include topography (scales of 1:10,000 to 1:2,000, depending upon the area to be covered by the project, and contour intervals of 2 to 4 m, or 5 to 10 ft), and remote-sensing imagery (scale of 1:20,000 to 1:6,000). Slope sections are prepared at a 1:1 scale showing the proposed cut or fill in its position relative to the entire slope.

Landform Analysis

On a regional basis, landform analysis is performed to identify unstable and potentially unstable areas, and to establish preliminary conclusions regarding possible failure forms, magnitudes, and incidence of occurrence. A preliminary map is prepared, showing 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 items given above, and including points of slope seepage.

Field Reconnaissance

The region or site location is visited and notations are made regarding seepage points, vegetation, creep indications, tension cracks, failure scars, hummocky ground, natural slope inclinations, and exposed geology. The data collected during landform analysis provide a guide as to the more significant areas to be examined.

Preliminary Evaluations

From the data collected, preliminary evaluations are made regarding slope conditions in the region or project study area, the preliminary engineering geology and hazard maps are modified, and an exploration program is planned for areas of particular interest.

Explorations

Geophysical Surveys

Seismic refraction profiling is performed 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. Typical seismic velocities from the weathering profile that develops in igneous and metamorphic rocks in warm, humid climates are given in Figure 1.100. 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 is performed to determine the depth to groundwater and to rock. Profiling is generally only applicable to depths of about 15 to 30 ft (5 to 10 m), 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 Boring Program

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. The depth and number of borings depend on the stratigraphy and uniformity of conditions, but where the slope consists of colluvial or residual soils, borings should penetrate to rock. In other conditions, the borings should extend below the depth of any potential failure surface, and always below the depth of cut for an adequate distance.

Sampling should be continuous through the 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, although the conditions existing at the time of investigation are not likely to be those during failure.

In Situ *Measurements*

Piezometers yield particularly useful information if in place during the wet season. In clayey residual profiles, confined water-table conditions can be expected in the weathered or fractured rock zone near the interface with the residual soils, or beneath colluvium. A piezometer set into fractured rock under these conditions may disclose artesian pressures exceeding the hydraulic head given by piezometers set into the overlying soils, even when they are saturated (Figure 1.130).

Instrumentation is installed to monitor surface deformations, to measure movement rates, and to detect the rupture zone if the slope is considered to be potentially unstable or is undergoing movement.

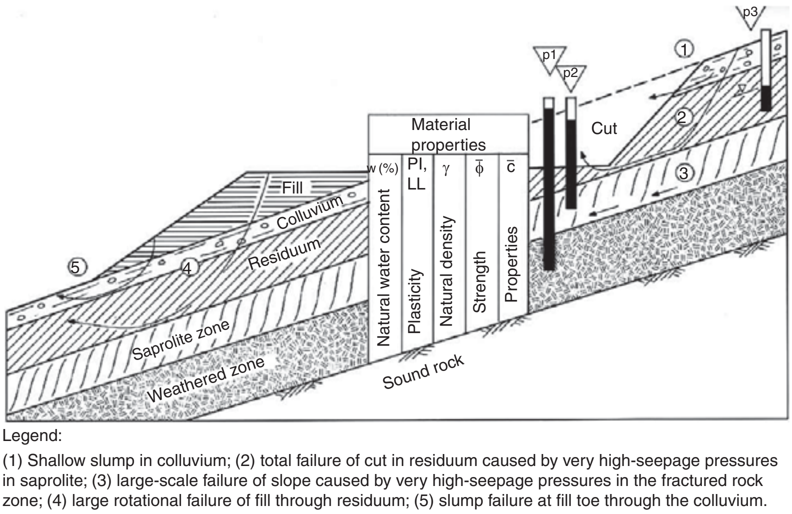


FIGURE 1.130
Schematic of typical section prepared at scale of 1:1 showing tentative cut and fill imposed on 30° slope in residual soil profile as basis for analysis. Piezometers P1 and P2 show excess water pressures in saprolite and fractured rock compared with saturated zone in residuum at P3. Several possible failure conditions requiring evaluation are shown.

Nuclear probes lowered into boreholes measure density and water content, and have been used to locate a failure surface by monitoring changes in these properties resulting from material rupture. In a relatively uniform material, the moisture and density logs will show an abrupt change in the failure zone from the average values (Cotecchia, 1978).

Dating Relict Slide Movements

Radiometric dating of secondary minerals in a ruptured zone or on slickensided surfaces, or of organic strata buried beneath colluvium, provides a basis for estimating the age of previous major movements.

Growth ring counts in trees that are inclined in their lower portion and vertical above also provide data for estimating the age of previous major slope movements. The date of the last major movement can be inferred from the younger, vertical-growing segments (Cotecchia, 1978). Slope failures cause stresses in the tree wood which result in particular tissues (reaction or compressed wood), which are darker and more opaque than normal unstressed wood. On the side toward which the tree leans there is an abrupt change from the growth rings of normal wood to those of compression wood. By taking small cores from the tree trunk it is possible to count the rings and estimate when the growth changes occurred and, thus, to date approximately the last major slope movement.

Slope Assessment

Data Presentation

A plan of the slope area is prepared showing contours, drainage paths, seepage emerging from the slope, outcrops, tension cracks and other failure scars, and other significant information. Sections are prepared at a 1:1 scale illustrating the stratigraphy and groundwater conditions as determined from the explorations, as well as any relict failure surfaces.

Evaluations and Analyses

Possible failure forms are predicted and existing failures are delineated as falls, slides, avalanches, or flows, and the degree of the hazard is judged. Depending upon the degree of risk, the decision is made to avoid the hazard or to eliminate or reduce it. For the cases of falls, avalanches, flows, and failures by lateral spreading, the decision is based on experience and judgment. Slides may be evaluated by mathematical analysis, but in recognition that movements may develop progressively.

Preliminary analysis of existing or potential failures by sliding includes the selection of potential failure surfaces by geometry in the case of planar slides, or analytically in the case of rotational slides, or by observation in the case of an existing slide. An evaluation is 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 (Table 1.6) as well as to changes that may occur with time (reduction from weathering, leaching, solution). Other transient conditions also require consideration, especially if the safety factor for the entire slope is low and could go below unity with some environmental change.

1.5.3 Detailed Study of Cut, Fill, or Failure Area

General

Detailed study of the area of the proposed cut or fill, or of the failure, is undertaken after the stability of the entire slope is assessed. The entire slope is often erroneously neglected in studies of cuts and side-hill fills, and is particularly important in mountainous terrain.

Explorations

Seismic refraction surveys are most useful if rock is anticipated within the cut, and there are boulders in the soils that make the delineation of bedrock difficult with test and core borings.

Test and core borings, and *test pits* are made to recover samples, including undisturbed samples, for laboratory testing. In colluvium, residuum, and saprolite, the best samples are often recovered from test pits, but these are usually limited to depths of 10 to 15 ft (3 to 5 m) because of practical excavation considerations.

In situ testing is performed in materials from which undisturbed samples are difficult or impossible to procure.

Laboratory Testing

Laboratory strength testing should duplicate the field conditions of pore-water pressures, drainage, load duration, and strain rate that are likely to exist as a consequence of construction operations, and samples should usually be tested in a saturated condition. It must be considered that conditions during and at the end of construction (short-term) will be different than long-term stability conditions. In this regard, the natural ability of the slope to drain during cutting plays a significant role.

Evaluation and Analysis

Sections illustrating the proposed cut, fill, or failure imposed on the slope are prepared at a 1:1 scale. The selection of cut slope inclination is based on the engineer's judgment of stability and is shown on the section together with the stratigraphy, groundwater conditions measured, and the soil properties as shown in Figure 1.130.

Analyses are performed to evaluate stable cut angles and sidehill fill stability, and the necessity for drainage and retention. Consideration must be given to the possibility of a number of failure forms and locations as shown in Figure 1.130, as well as to changing groundwater and other environmental conditions.

1.5.4 Case Study

Background

A roadway was constructed during the early 1990s beginning on the western coastal plain of Ecuador, crossing over the Andes Mountains, and terminating after 110 km at the city of Cuenca. Landslides began at numerous locations where the roadway climbed the steep mountain slopes, usually 35° or steeper. The general landform along a portion of the roadway is illustrated on the 3D diagram in Figure 1.131. Slope failures increased significantly during the El Niño years of 1997 and 1998.

Investigation

Initially, investigation included a number of trips along the roadway during which the slope failures were photographed and cataloged. Pairs of aerial photos were examined stereoscopically. Eventually a helicopter fly-over was made and the roadway continuously photographed. Debris avalanches, occurring on the upslope side of the roadway, were the most common form of slope failure (Figure 1.5 and Figure 1.132). More than 125 failure locations were identified. The landslide debris was bulldozed from the roadway onto the downslope side (Figure 1.132) further destabilizing the slopes and contributing to erosion and choking of streams downslope.

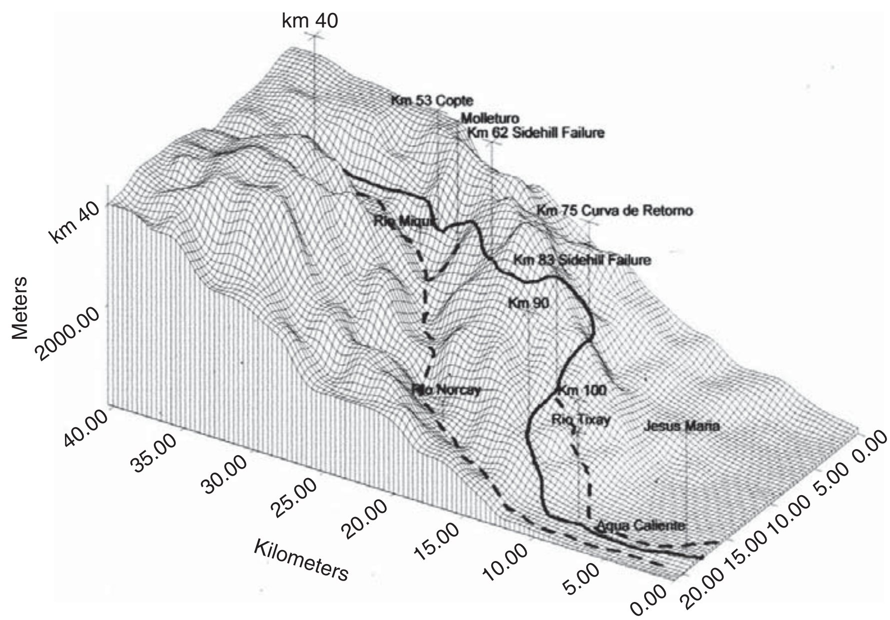


FIGURE 1.131
Three-dimensional diagram of a portion of roadway over the Andes Mountains in Ecuador. Side-hill failure at km 62 is shown in Figure 1.6. Debris avalanche at km 88 shown in Figure 1.132. Large slump slide shown in Figure 1.133.

FIGURE 1.132
Debris avalanches, km 83 along roadway in Figure 1.131.

Other slope failures included a few large "slump" slides (Figure 1.133) and numerous failures on the downslope of the roadway resulting from erosion from discharge of roadway storm drains (Figure 1.6).

FIGURE 1.133
Large slump slide at km 75 located in Figure 1.131. Large tension cracks appeared upslope and the two roadways in the photo center have dropped several meters.

Budget limitations required that detailed investigations using seismic refraction surveys and test borings be limited to four of the more critical locations.

Evaluations and Treatments

Debris Avalanches

The debris avalanches were occurring along 80 km of roadway beginning near the lowlands, where the dominant conditions are relatively soft volcanic rocks with a moderately thick cover of residual soils, and in places, colluvium. The natural slopes usually were inclined at about 35° or steeper. Initially, vegetation was removed and roadway cuts were inclined at 53°, much steeper than the original slopes. Subsequently rainfall and seepage resulted in the residual soils sliding along the fractured rock surface.

Unsupported cuts were made because the large number of cuts would cause support with retaining walls to make roadway construction prohibitive. A more stable alternate slope design could have included 10-m-high 45° cuts with benches to result in an overall cut slope inclination of 38°. Construction costs would be increased but roadway slope failures and maintenance costs should be decreased.

In many cases where failure has occurred the residual soil has been removed and the slope is now self-stabilized, although in some cases, there remains potentially unstable material upslope of the failure scar. Future failures will be removed from the roadway and disposed of in designated spoil areas, rather than dumping over the sides of the roadway.

Downslope Roadway Drains

It was recommended that all roadway drains be relocated so that discharge downslope is where erosion will not endanger the roadway. Drainage channels should be lined to prevent erosion. The failure shown in Figure 1.6 resulted from storm water discharge eroding the slope below a shotcreted gabion wall that was supporting the roadway. Storm water entered the catch basin shown on the upslope side of the roadway and discharged through a pipe exiting on the downslope side. The failure in Figure 1.6 was corrected with the construction of an anchored wall and relocating the storm water discharge point.

Large Slump Slide

At km 75 the roadway makes an abrupt switch-back as shown in Figure 1.133. At this location is a large slump slide evidenced by tension cracks upslope and roadway movement. When first visited during 1999 the portion of the roadway in the middle of the photo (Figure 1.133) had dropped about 2 m; when visited the following year, the roadway had dropped an additional 2 m and was almost impassable. This was considered a priority site for remediation.

Explorations with seismic refraction surveys and test borings determined geologic conditions to include about 15 m of colluvium overlying 5 to 20 m of fractured rock grading to hard rock.

Treatments recommended for stabilization included:

1. Surface drains constructed upslope to collect runoff and to discharge away from the failing area.
2. Subhorizontal drains installed along the toes of the cut slopes areas shown on the photo.
3. Upslope roadway cut slope to be reshaped with benches and covered with shotcrete.

4. Shotcrete placed to cover the shallow rock slope between the upper and lower roadways.
5. Low anchored concrete walls to be installed along the toe of the upslope and downslope side of the roadways.
6. The roadway grade is not to be raised as this would add load to the unstable mass.

1.5.5 Instrumentation and Monitoring

Purpose

Instrumentation is required to monitor changing conditions that may lead to total failure where slope movement is occurring and safety factors against sliding are low, or where a major work would become endangered by a slope failure.

Slope-stability analysis is often far from precise, regardless of the adequacy of the data available, and sometimes the provision for an absolutely safe slope is prohibitively costly. In this case, the engineer may wish to have contingency plans available such as the installation of internal drainage systems or the removal of material from upslope, etc., if the slope shows signs of becoming unstable.

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 failure. In cut slopes, instrumentation monitors movements and changing stress conditions to provide early warning and permit invoking remedial measures when low safety factors are accepted in design.

Instrumentation Methods Summarized

Surface movements are monitored by survey nets, tiltmeters (on benches), convergence meters, surface extensometers, and terrestrial photography. Accuracy ranges from 0.5 to 1.0 mm for extensometers, to 30 mm for the geodimeter, and to 300 mm for the theodolite (Blackwell et al., 1975). GPS systems are showing promise for continuously monitoring and recording slope movements.

Subsurface deformations are monitored with inclinometers, deflectometers, shear-strip indicators, steel wire and weights in boreholes, and the acoustical emissions device. Accuracy for extensometers and inclinometers usually ranges from 0.5 to 1.0 mm, but the accuracy depends considerably on the deformation pattern and in many instances cannot be considered better than 5 to 10 mm.

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.

GPS Installations

Mission Peak Landslide, Fremont, California

On the Internet during 2003, the U.S. Geological Survey (USGS) reported on a global positioning system (GPS) installation to monitor the Mission Peak Landslide in Fremont, California. Installed in January 2000, the system included a field station with a GPS antenna, receiver, controller card, and radio modem that sent data to the base station which included a radio modem, personal computer connected to a phone line or the

Internet for graphical output. The massive block at the head of the landslide was found initially to be moving at less than 1cm/week, then accelerating to 2 cm/week apparently in response to rainfall. At the cessation of seasonal rains it remained moving at the rate of 1 mm/week for a 4-month period from February to June 2000. GPS measurements were reported to typically show repeatability ±1 cm horizontally and ±2 cm vertically.

Lishan Slope, Xian, China

Orense (2003) describes the landslide hazard threatening the Huaqing Palace, in Xian, China. Built during the Tang dynasty (618–907), the Palace is located at the foot of the Lishan slope that shows visible deformation. The potential failure mass is a large-scale rock slide. Although in an area of earthquake activity, it is believed that subsidence in the valley from extensive groundwater withdrawal has resulted in activating slope movements. Geologic conditions generally consist of a layer of loess overlying gneiss bedrock. A site plan is given in Figure 1.134. The potential failure mass has been divided into three possible blocks. The dashed line represents the limits of a thick loess deposit that has already slid.

Studies were begun in 1991 by the Disaster Prevention Institute of Kyoto University, Japan, and the Xian Municipal Government. An extensive automated monitoring system was installed as shown in Figure 1.135. Included in the system were short- and long-span extensometers (lines A and B, Figure 1.134), total station surveying, GPS survey, borehole inclinometers, and ground motion seismographs. Data are transmitted periodically to Kyoto University via satellite.

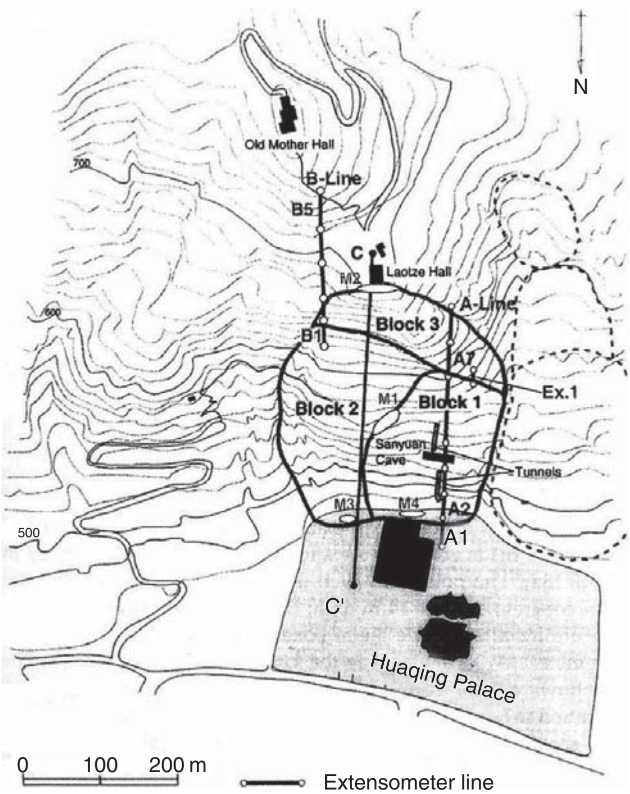


FIGURE 1.134
Plan map of the Lishan slope in China with potential landslide blocks and extensometer lines (From Orense, R. P., *Geotechnical Hazards: Nature, Assessment and Mitigation*, University of the Philippines Press, Quezon City, 2003. With permission. After Sassa, K. et al., *Proceedings of the International Symposium on Landslide Hazard Assessment*, 1–24, Xian, China, 1997.)

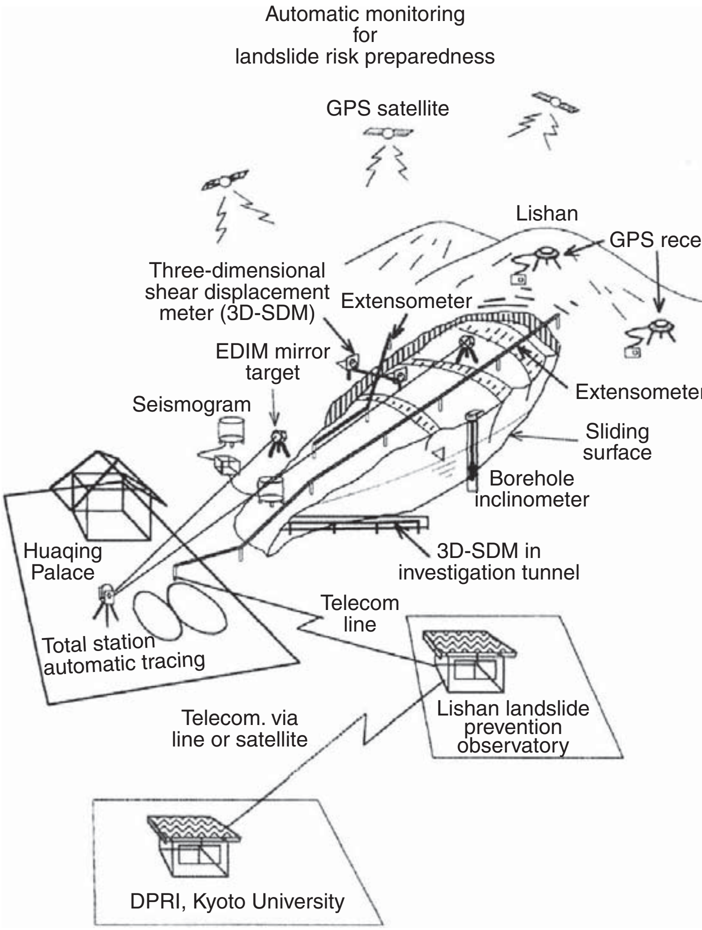


FIGURE 1.135
Monitoring system for the Lishan slope prepared for landslide risk assessment. (From Orense, R. P., *Geotechnical Hazards: Nature, Assessment and Mitigation,* University of the Philippines Press, Quezon City, 2003. With permission. After Sassa, K. et al., *Proceedings of the International Symposium on Landslide Hazard Assessment,* 1–24, Xian, China, 1997.)

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