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Highway Slope Maintenance and Slide Restoration

Reference Manual




NATIONAL HIGHWAY INSTITUTE

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16. Abstract

As focus changes toward the asset management of our existing infrastructure, the value of maintaining and managing our embankment and cut slopes becomes more apparent. This 132081 course provides the essentials to slope maintenance and slide restoration for transportation field personnel with an asset management perspective. This course is not meant to be highly technical, and explains conceptually the conditions and factors affecting slope movement, stability and deterioration, and the cost considerations of maintenance, stabilization and of slope failures. The course also provides the fundamental aspects of slope management systems and discusses the rationale of slope management considering the legal implications of slope failures and rock fall.

This Reference Manual provides the technical content and reference text used for the training course and is intended that the participants will use it after attending the course as a reference document in everyday

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CONVERSION FACTORS

Approximate Conversions to SI Units			Approximate Conversions from SI Units		
When you know	Multiply by	To find	When you know	Multiply by	To find
(a) Length					
inch	25.4	millimeter	millimeter	0.039	inch
foot	0.305	meter	meter	3.28	foot
yard	0.914	meter	meter	1.09	yard
mile	1.61	kilometer	kilometer	0.621	mile
(b) Area					
square inches	645.2	square millimeters	square millimeters	0.0016	square inches
square feet	0.093	square meters	square meters	10.764	square feet
acres	0.405	hectares	hectares	2.47	acres
square miles	2.59	square kilometers	square kilometers	0.386	square miles
(c) Volume					
fluid ounces	29.57	milliliters	milliliters	0.034	fluid ounces
gallons	3.785	liters	liters	0.264	gallons
cubic feet	0.028	cubic meters	cubic meters	35.32	cubic feet
cubic yards	0.765	cubic meters	cubic meters	1.308	cubic yards
(d) Mass					
ounces	28.35	grams	grams	0.035	ounces
pounds	0.454	kilograms	kilograms	2.205	pounds
short tons (2000 lb)	0.907	megagrams (tonne)	megagrams (tonne)	1.102	short tons (2000 lb)
(e) Force					
pound	4.448	Newton	Newton	0.2248	pound
(f) Pressure, Stress, Modulus of Elasticity					
pounds per square foot	47.88	Pascals	Pascals	0.021	pounds per square foot
pounds per square inch	6.895	kiloPascals	kiloPascals	0.145	pounds per square inch
(g) Density					
pounds per cubic foot	16.019	kilograms per cubic meter	kilograms per cubic meter	0.0624	pounds per cubic feet
(h) Temperature					
Fahrenheit temperature(^oF)	$5/9(^oF - 32)$	Celsius temperature(^oC)	Celsius temperature(^oC)	$9/5(^oC) + 32$	Fahrenheit temperature(^oF)

Notes: 1) The primary metric (SI) units used in civil engineering are meter (m), kilogram (kg), second(s), newton (N) and pascal ($Pa=N/m^2$).
2) In a "soft" conversion, an English measurement is mathematically converted to its exact metric equivalent.
3) In a "hard" conversion, a new rounded metric number is created that is convenient to work with and remember.

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CHAPTER 1
INTRODUCTION

1.1 PURPOSE AND SCOPE OF MANUAL

Driving down almost any road, you will come across a section of roadway that was built on a fill embankment or cut through a section of natural earth material. It probably comes as no surprise that in order to economically build roads that are both safe and efficient for traveling vehicles requires, that roads are commonly constructed using earth materials at relatively gentle grades. As a result, slopes are necessary when grading our embankments and cut sections. A slope in the context of this manual is any soil mass with an inclined surface. An embankment is a man-made change in elevation created by the use of back to back slopes. This manual is focused on maintenance of all types of slopes (i.e., soil and rock) and embankments in highway engineering practice.

The maintenance of our slopes and embankments is crucial to their long term performance. Without proper care, the stability of a slope may deteriorate to the point of failure. A slope failure can occur rapidly or slowly in many different types and forms. A severe and untimely slope failure (landslide) which occurs adjacent to a highway alignment can cause enormous economic losses and even casualties. The economic losses incurred by slope failures (landslides) include both direct and indirect costs that affect public and private properties. The direct costs include the repair, replacement, or maintenance resulting from damage to highway structures and public/private properties. The indirect costs are difficult to determine, and sometimes can be higher than the direct costs (TRB, 1996). The primary indirect costs include:

- Loss of industrial, agricultural, and forest productivity and tourist revenues as a result of damage to land or facilities or interruption of transportation systems
- Measures to prevent or mitigate additional landslide damage
- Loss of human or animal productivity.

Figure 1-1 Photo of Common Slope Failure Example

This manual focuses on the common stability and deformation problems for slopes and embankments, general engineering properties of earth materials and their effect on slope performance, identifying failure prone conditions, best practices for maintenance, stabilization methods and economics of repair. Major slope failures which require extensive geotechnical evaluation and analysis are discussed in this manual in limited manner. The emphasis is on the more commonly occurring minor slope failures for which the maintenance supervisor is typically responsible.

1.2 SLOPE MAINTENANCE PROBLEM

Slopes are not stable; mountains flatten and erode to be hills and plains. To build a slope is to over-steepen what exists naturally. As a result, erosion of man-made slopes can be expected to be faster than erosion of natural slopes, as nature tries to bring these things back into balance. Slope maintenance can slow this process and, in so doing, save (life cycle) investments of time and money, and increase the life and utility of a road network. This is important to recognize because maintenance needs are present throughout the life of a slope, not just after a slide, a conspicuous crack or a gulley. In this respect, slopes are no different from other engineered features on a highway system (pavement, bridges, walls, etc.) in that conditions continually deteriorate and maintenance is absolutely needed to ensure the design life is achieved. Maintenance before a "problem" occurs includes maintaining vegetation without over-irrigating, maintaining ditches and culverts to avoid ponding or erosion gullies, sealing pavement cracks that are the result of poor pavement performance, not slope performance. The maintenance of slopes and the restoration and correction of slides on highways has been identified by a number of state transportation agencies as a major and continuing problem involving considerable expenditures of funds. Each year, highway departments in the United States spend millions of dollars in maintaining highway embankments and slopes; removing rock falls and debris from roadways; and repairing landslides. These activities often cause restrictions/disruptions of traffic flow, creating serious safety hazards and economic loss due to the lost time and inconvenience of the traveling public. Maintenance crews must take corrective action to return slopes to a usable and stable condition, to restore the facility to normal operating conditions, and to minimize hazards to the traveling public.

The extent to which slope failures impact upon highway agencies in this country is not fully known. There is no doubt that large slope failures, which are reported in the newspapers, on television, and the internet, are a major item in highway budgets. Major slope failures often cost millions of dollars to restore the highway facility to a safe and usable condition. These failures significantly affect the highway budget.

User costs are often immeasurable. Direct cost to highway agencies related to slope maintenance and slide restoration may not be accurately reflected in maintenance records. This is largely due to procedures currently used for allocating costs to maintenance activities. For example, the repair of a roadway surface due to settlement may be charged to asphalt patching when, in fact, the settlement is a result of movement of the earthen materials within the slope or embankment. In addition, the costs associated with detours, decreased capacity, delays, increased wear on detour routes, etc. are often hard to quantify. Tracking costs for maintenance and restoration of slopes as well as user costs associated with such failures is extremely difficult.

There are approximately 4 million miles of roadway under the jurisdictions of various State and local highway agencies in the United States. In many jurisdictions, there are large numbers of slope failures, some of which may not require any significant maintenance attention while others must be given attention to maintain the facility in a safe and operating condition. Since most highway agencies do not maintain an inventory of slope failures, it is almost impossible to develop a reliable measure of the problem of slope failures. It is understood, however, that the problem is significant.

1.2.1 Economics Associated with Slope Maintenance and Restoration

Not only are records relating to the number of slope failures inadequate and sparse, information relative to the expenditures to remedy slope failures is also generally unknown. Expenditures of time, personnel, equipment, and materials to correct and remedy slope problems are not maintained by most highway agencies. Even if such a cost-accounting system were in place in a highway agency, the problem of assigning all costs associated with the maintenance of a slope is extremely difficult. Many activities are often more directly related (from a cost accounting point of view) to other aspects of highway maintenance, such as guardrail maintenance, drainage maintenance, etc.; while in many cases, maintenance activities on those elements of the highway are a result of a slope instability or failure.

The magnitude of the slope maintenance problem faced by the maintenance supervisor is unknown but is estimated to be extremely large. In addition to the cost of maintenance in terms of personnel, equipment, and materials mentioned above, there are other social and economic costs to the highway agency as well as the general public. It is obvious to most that there is destruction of public property when a highway slope fails. Not only is the slope itself damaged, but other adjacent elements of the highway also may be affected in a negative manner. Often when a slope fails and maintenance activities require closure or restriction of traffic flow in order to restore the slope to a safe and operable condition, there are significant inconveniences and user costs such as delays and increased vehicular costs.

1.2.2 Role of Maintenance with Respect to Slope Maintenance and Slide Restoration

The role of maintenance with respect to slope maintenance and slide restoration is to control the natural deterioration of highway slopes and embankments. This includes:

- Insure the safe passage of motorists and keep the road open if possible
- Perform routine slope maintenance
- Implement temporary measures in an attempt to halt, or delay, the deterioration of the slope
- Monitor and report the deterioration of the slope
- Perform restoration of slopes with small slide(s).

The costs associated with the prevention of deterioration of a highway slope may be assigned to one of the following categories:

<u>Routine Maintenance:</u> The slope problem is considered minor and only routine maintenance is required. For example, the problem may involve the removal for the first time of small quantities of rock or soil debris which has fallen onto the roadway or the roadway has settled or sagged and requires patching for the first time. Other routine actions might include clearing of a blocked drainage ditch, sealing cracks, or repairing slope erosion and re-establishing vegetation.

<u>Extraordinary Maintenance</u>: The deterioration of the slope has reached a stage where the slope problem continues to recur and extraordinary and temporary measures and costs are incurred to halt or delay the slope movement and maintain traffic flow until more permanent slide restoration measures are implemented. As an example, slope movements of a particular highway may have reached a stage where more than one patching has been performed, pavement cracks have been sealed on several occasions, and the guardrail has been repaired on different occasions. Another example might include a situation where large rock debris has fallen onto the highway on different occasions and required expensive removal.

Slide Restoration: When the deterioration of the slope has reached a condition where routine and extraordinary maintenance costs approach or represent a sizable portion of the cost of restoring the highway section to its original condition, or to a condition better than the original condition, then remedial repairs should be implemented. However, corrective actions may be taken in cases where a real danger threatens the traveling public, although extraordinary costs may not be involved. At this stage, the highway slope has failed or moved to such a state that the situation poses a real danger to the traveling public and the traffic lanes have to be closed or the roadway completely closed.

Each agency is encouraged to develop an accounting system that monitors maintenance costs for any section of their highway system that is experiencing slope problems so that a rational decision on the best course of action to maintain/remediate the slope can be made.

1.2.3 Communication and Coordination

Communication and coordination between the maintenance personnel, the geotechnical specialist, and the maintenance supervisor is crucial for the success of any slope maintenance program. This communication should be open in both directions. The maintenance personnel may have invaluable experience of which the geotechnical specialist is unaware, and the geotechnical specialist may have engineering insight and expertise that is crucial to the success of the maintenance program.

Routine maintenance is the responsibility of the maintenance supervisor (for the purposes of this manual, the supervisor of one or more maintenance crews). The maintenance supervisor will also typically be involved with extraordinary maintenance. However, extraordinary maintenance may normally be performed under the direction of the maintenance engineer (for the purposes of this manual, the head of the maintenance department). Slide restoration may not directly involve the maintenance supervisor. Whether the maintenance supervisor becomes directly involved and assumes the responsibility of slide restoration depends on the following factors:

- The skills and capabilities of the maintenance personnel and available equipment
- Whether or not the maintenance supervisor feels he thoroughly understands the problem
- If the cost of implementing a slide stabilization method is prohibitive
- The size of the slide and the depth and position of the failure plane or surface of sliding.

Each of the factors listed above must be considered in making a decision on who should be involved in the slide restoration. To aid the maintenance supervisor in making this decision, it is useful to classify the slide problem according to the size of the slope. For example, the height of the slope, as measured from the toe of the slope to the top of the shoulder, may be classified according to one of the following categories:

- Small Slope – 20 feet or less
- Medium Slope – 20 to 40 feet
- Large Slope – greater than 40 feet

In many instances, from the viewpoint of available maintenance equipment and materials' handling capabilities, slides that occur in slopes measuring less than approximately 20 feet could probably be repaired by many maintenance supervisors and crews. However, there are situations where the maintenance supervisor may not be able to handle a small slide because of budgetary constraints. For example, suppose that the supervisor has determined that a particular stabilization method can be applied. Suppose that the technique cost $450 per lineal foot (as measured parallel to the centerline of the roadway). Although the slide may only be 20 feet in height, the slide may be 700 feet in length. The total cost to repair the slide is $315,000 (700 feet x $450 per lineal feet). If the maintenance supervisor's total yearly budget is $1,000,000, then the slide restoration may not be feasible. In terms of budgetary restraints, the slide restoration may represent a major investment. However, suppose the slide is only 200 feet in width. The cost, in this case, to repair the slide is $90,000. In this case, the supervisor may feel that he can repair the slide with his forces. **It is suggested that a small slide may be defined as one that is less than 20 feet in height and the cost of the repair estimated to be less than an amount to be determined by the maintenance supervisor and maintenance engineer.** The maximum amount to be spent at a particular location should be determined by the maintenance supervisor, maintenance engineer, and geotechnical specialist. For example, it may be predetermined by the maintenance supervisor, maintenance engineer, and geotechnical specialist that the maintenance supervisor will not spend more than $50,000 to repair a slide along a certain stretch of highway. However, at a different location the value may be set at $100,000. If the amount of the slide exceeds this predetermined value, then the geotechnical specialist should be contacted. In the case of county or city maintenance supervisors, the local political unit should be contacted and given the necessary information concerning the cost of the slide restoration. The local political unit should seek the advice and assistance of a consulting geotechnical engineer.

Slides in medium and large slopes and embankments may or may not be repaired by the maintenance supervisor. For example, deep-seated failures should be examined by the geotechnical specialist, and maintenance engineer as well as the maintenance supervisor (Figure 1-2). In a deep-seated failure, the failure plane is typically located at least 10 feet below the surface of the slope but may extend several tens of feet below grade. Slides of this size will probably require a major geotechnical investigation and analysis to determine the most appropriate repair method. However, if the slide is less than 20 feet in height, although the slope height is greater than 20 feet, the maintenance supervisor and crew may still be capable of repairing the slide. For example, suppose an embankment measures 80 feet in height: a failure measuring 10 feet in height and 50 feet in width occurs at the toe of the embankment (Figure 1-3). Although the slope/embankment is classified as large, the maintenance crew could probably repair the failure based on advice from the geotechnical specialist and maintenance engineer.

Figure 1-2 1995 Landslide in La Conchita California (R. L. Schuster, 1995)

The decision to include the maintenance engineer and geotechnical specialist should also be based the consequences of the slope failure. If a subsequent failure of the slide restoration will potentially jeopardize the life and safety of the public then the maintenance supervisor should contact the geotechnical specialist and maintenance engineer so that an engineered solution may be developed.

It is recommended that the maintenance supervisor seek the advice of the geotechnical specialist and maintenance engineer before implementing a slide restoration. By working, with and seeking the advice of, the geotechnical specialist who is familiar with the local conditions and types of slides that occur in a locality, pre-engineered restoration methods may be developed for many small slides. Hence, the maintenance supervisor may be able to repair many small slides without needing the assistance of the geotechnical specialist.

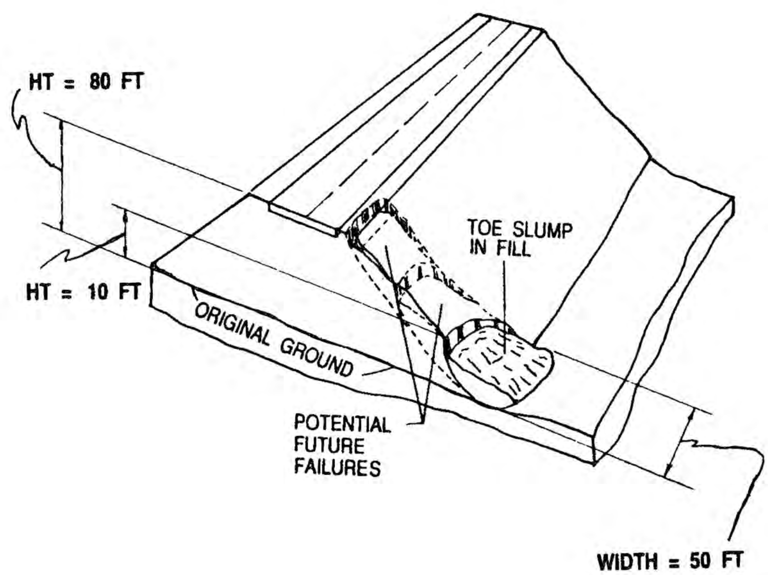

Figure 1-3 Example of Toe Slide in a Large Highway Embankment (Figure from Previous Manual page 9)

1.3 ORGANIZATION OF THE MANUAL

This manual is intended to be a stand-alone document and is geared towards providing guidelines for state, county, and city maintenance supervisors who are responsible on a daily basis for making decisions related to the maintenance of soil and rock slopes and embankments. The organization of the manual is presented below.

- Chapter 2 describes problems commonly encountered in highway engineering such as embankment deformation (settlement related to the embankment itself and the foundation soil) and slope stability of natural and man-made slopes.
- Chapter 3 presents discussions of the geotechnical and geological factors, including hydrostatic forces, affecting the performance of slopes and embankments.
- Chapter 4 discusses the both geological and geometrical conditions that make a slope and or embankment prone to stability issues.

- Chapter 5 reviews currently identified best practices for maintenance and monitoring of slopes and embankments.
- Chapter 6 discusses methods to inventory and rate slope failures, landslides and rockfall, as well as the benefits of slope management systems.
- Chapter 7 presents common alternatives for slope stabilization.
- Chapter 8 discusses the general economics, cost categories, and procedures for comparing estimated costs for various stabilization alternatives.
- Chapter 9 provides a brief overview of some of the legal issues associated with slope failures, landslides and rockfalls.

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CHAPTER 2

COMMON STABILITY AND DEFORMATION PROBLEMS FOR HIGHWAY SOIL AND ROCK SLOPES AND EMBANKMENTS

2.1 INTRODUCTION

Slopes and embankments are a common and complex part of highway facilities, yet are often overlooked. If a slope or embankment is not properly designed, constructed and maintained, problems can occur. The most complicated problems for slopes and embankments are associated with slope stability. Landslides, slips, slumps, mudflows, and rockfalls are simply various names given for slope failures. For embankments/fills, satisfactory performance depends not only on their internal stability, but also the foundations on which they rest. Slopes and embankments are designed elements whose foundation soils and subsurface conditions should be considered during design. Poor performance of embankments and slopes could be due to any number of problems, including unconsidered compressible foundation soils, planes of weakness in the foundation, improper control of infiltration and/or groundwater at interface, improper benching, improper control of embankment materials, oversteepened embankments (common to side-hill embankments, or where right-of-way is insufficient), inadequate/improper surface water control details, etc.

Stability problems can occur when a highway alignment is located adjacent to areas of potential landslides, sidehill cuts and fills, abandoned mining operations, soft foundations, buried stream channels, and landfills. Section 2.2 describes typical slope movements/failures. Section 2.3 introduces the basic concepts of slope stability. Section 2.4 presents the triggering mechanisms of slope movement and distress.

2.2 TYPES OF SLOPE MOVEMENT AND INSTABILITY

There are many types of slope movement, as described in the subsequent subsections. Sometimes they act alone, and sometimes they are combined. The importance of understanding the type of movement is twofold. First, it provides clues as to what might be causing the movement, and second, it provides an understanding of the mechanism necessary to design/construct mitigation measures.

2.2.1 Creep

Creep, as the name suggests, is a type of movement that takes place slowly. Terzaghi in 1950 identified the creep as continuous slope movement, which proceeds at an average rate of less than a foot per decade, which is about 30 mm per year. Typically the rate of creep movement is relatively faster at the surface and decreases with depth (Figure 2-1). Because the process is slow, most slopes that experience creep may remain stable for a very long period of time. Continuous creep is more common in clayey soils than in granular soils. Once creep has started, it may progress over a large area and is very difficult to stop.

There are two classes of creep. One is the seasonal creep which affects only the upper crust (usually less than 2 meters thick) of a slope due to seasonal freezing and thawing, or wetting and drying. This creep movement can vary from season to season but is always present. The second is massive creep, which produces almost constant movements in the upper crust ranging up to several meters thick. The process of massive creep movements is not very well understood.

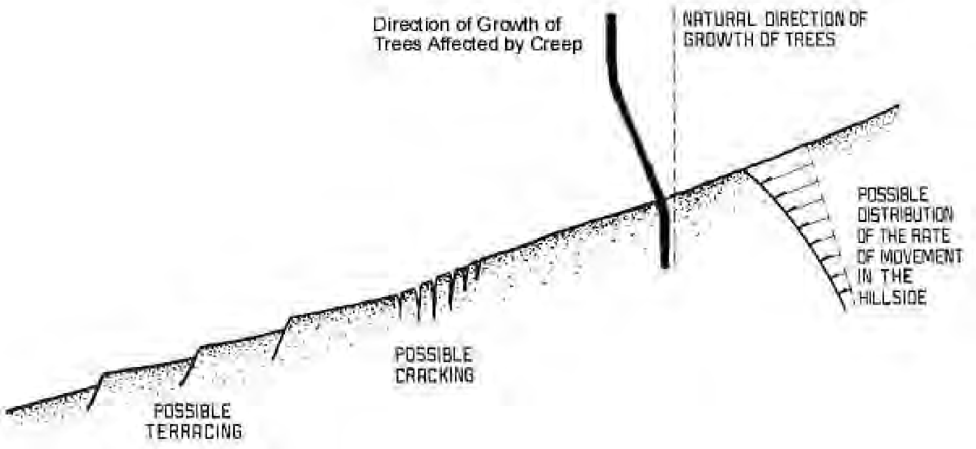


Figure 2-1 Signs of a Surficial Slide or Creep (After Rodriguez et al., 1988)

2.2.2 Fall and Topple

A fall is commonly associated with rock or highly overconsolidated cohesive materials (i.e., clay) and normally only occurs in very steep slopes (Figure 2-2). The steepness of the slopes may be caused by careless excavation, material weathering, or scour at the toe of the slope by wave action, river/stream erosion and seepage. However, in some cases, the steepness of the slope may be designed to prevent more severe stability problems (e.g. vertical faces in formations that differentially weather, cohesive formations in arid climates, etc.). In these cases, the steep faces are preferred and the fall is controlled and managed. Prior to failure, a portion of the mass pulls apart from the rest of the mass at the slope's face where it is unsupported. As the mass continues to move away from the face of the slope, a fall follows. In rock and cohesive material, tension cracks are likely to exist at the top of steep slopes. Water can accumulate in these cracks and the resulting thrusts can ultimately force the material to separate and fall. Ice wedging in these cracks and seismic forces can lead to similar falls.

Falls are not as common in soil slopes as they are in rock. This is because falls occur only on steep slopes and many soils would simply roll or ravel down to a less steep slope relatively quickly, where a rock slope can stand at a very steep angle, even vertical, for a long time. As the strength of soils are generally weaker than those of rock, soil slopes are not as steep as rock slopes and consequently rockfalls are much more common than soil-falls.

A topple is the forward rotation out of a natural or cut slope of a mass of rock or soil. Topples may lead to falls or slides of the rotated mass as shown in Figure 2-3.

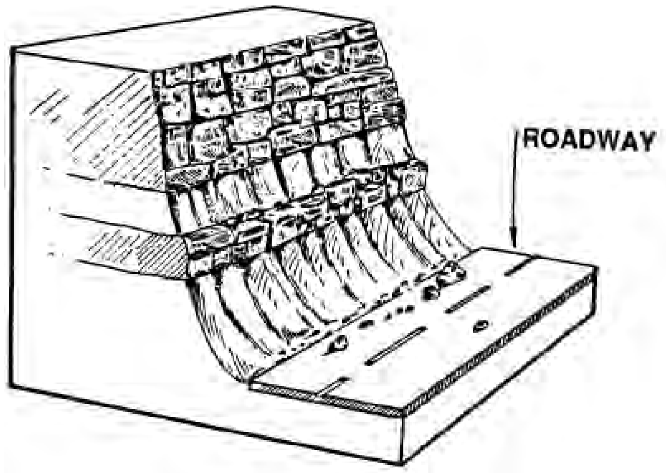


Figure 2-2 Typical Rockfall Movement Due to Weathering Process

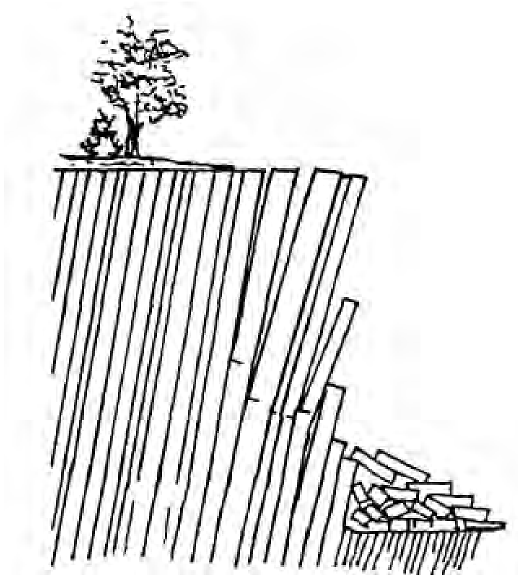


Figure 2-3 Typical Toppling Failure (132035 Manual)

TABLE 2-1 AIDS TO RECOGNITION OF ACTIVE OR RECENTLY ACTIVE SLIDE

Type of Movement	Stable Zones Around the Slide			Parts That Have Moved			
	Crest or Head of Failed Zone[1]	Principal Zone (Behind failed zone)[1]	Flanks[1]	Head[1]	Body[1]	Base[1]	Toe[1]
Falls							
Soil Falls	Cracks behind the failure line	Almost vertical, moist soil, very cracked on the surface	Often almost vertical	Usually unclear, the fallen material forms a pile of rocks near escarpment	Irregular	Is usually buried, if visible, generally shows causes of failure such as weak underlying rock or strata undermined by water	Irregular
Slides							
Rotational	Numerous cracks, the majority concave towards the slide	Inclined, clean, concave towards slide, often high, may have striations and grooves on surface, from crest to head, upper portion of slope behind failure may be vertical	Have large vertical components near the head, and notable horizontal components near the base. Height of flanks reduced towards base. Flank can be higher than original surfaces of ground between toe and base. In initial stages slides surrounded by terraced cracks	Upper part of failure conserves parts of the natural ground before failure. Pools form at toe of principal slope. Entire head of failure furrowed by cracks. Trees in fallen zone point uphill	The moving part of the soil breaks up and disintegrates. Longitudinal cracks, heave, generally pools develop just above the base	Transversal heave and cracks normally on the base. Upheaval zone, absence of large individual blocks. The trees lean downhill	Often a lobular earth flow zone, material that has been rolled over and buried. The trees are flattened or at different angles among the material at the toe

Continued on next page

TABLE 2-1 (CONTINUED) AIDS TO RECOGNITION OF ACTIVE OR RECENTLY ACTIVE SLIDE

Translational	Most cracks almost vertical, tend to follow outline of slope	Almost vertical in the upper portion, in lower almost flat with gradual transition	Lateral flanks very low vertical cracks, downhill the cracks generally diverge	Relatively intact, no rotation	Generally intact units except for tension cracks. Little or no vertical displacement in cracks	No base and no upheaval zone	Sliding over the surface of the ground
Flow of Dry Material							
Sand Flow	No cracks	Funnel-shaped when reaches angle of repose	Form continuous curve starting at crest	Generally no head	Cone-shaped mound of sand, same volume as portion emptied from head	No base	No toe or a wide, hardly perceptible fan
Flow of Moist Material							
Mud Flow	Few cracks	Upper part notched or V-shaped, long and narrow, smooth, often striated	Sloping, upper portion irregular, heaps of material in the lower portion of the flanks	May have no head	Moist to very moist, may contain large blocks in fine matrix. Flow lines, follow drainage lines, may have sharp bends. Long and narrow	No base or buried in debris	Extends sideways in lobes. When toe dries, may have a low step at the front
Earth Flow	May be some cracks	Concave to slide, sometimes almost circular, slide occurs through a narrowed portion	Curves, steep sides	Usually consists of a sunken block	Broken into numerous small pieces, moist, shows flow structure	No base	Extends in lobes
Sand or Silt Flow	Few cracks	Sloping, concave to slide, variety of shapes (almost straight, arc, bottle-shaped)	Flanks often converge in the direction of movement	Generally under water	The body extends like a fluid	No base	Extends in lobes

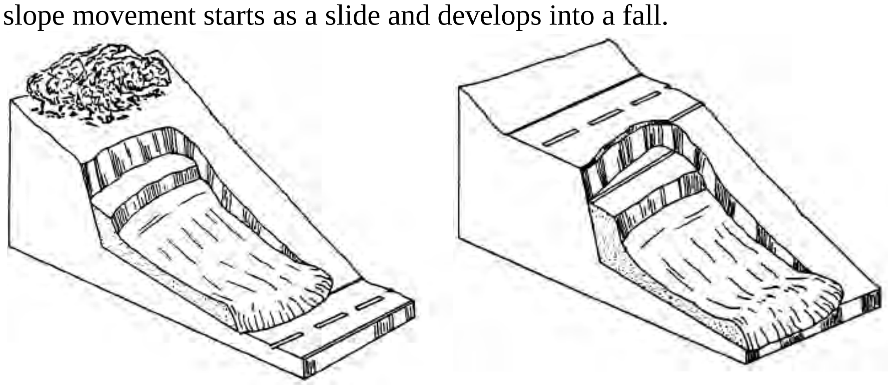
1. For a definition of terms (i.e., Head body, toe, etc.) see Section 3

2.2.3 Slide

A slide is a downslope movement of a soil or rock mass occurring predominantly on surfaces of rupture or on relatively thin zones of intense shear strain. Slides can be categorized into three common modes: translational, rotational, and compound slides.

2.2.3.1 Translational Slide

Most slides are translational in nature. This simply means that the failure mass moves, or translates, along a surface boundary that is more or less linear. This linear movement is especially true when the slide is shallow. The sliding surface is usually shallow and along a plane of weakness (a thin weak layer or stratum boundary). Therefore, translational slides are most common in thinly bedded soil slopes with soft clays, fine sands or loose non-plastic silts. Transitional slides formed by two intersecting planar discontinuities (joints) are common in exposed rock mass along roadway as shown in Figure 2-5. They are often referred to as rock wedge or block slides. A lateral force exerted by water or ice-filled cracks or a seismic event can also trigger such movements. This mechanism is quite similar to that of a fall. It is possible that a slope movement starts as a slide and develops into a fall.



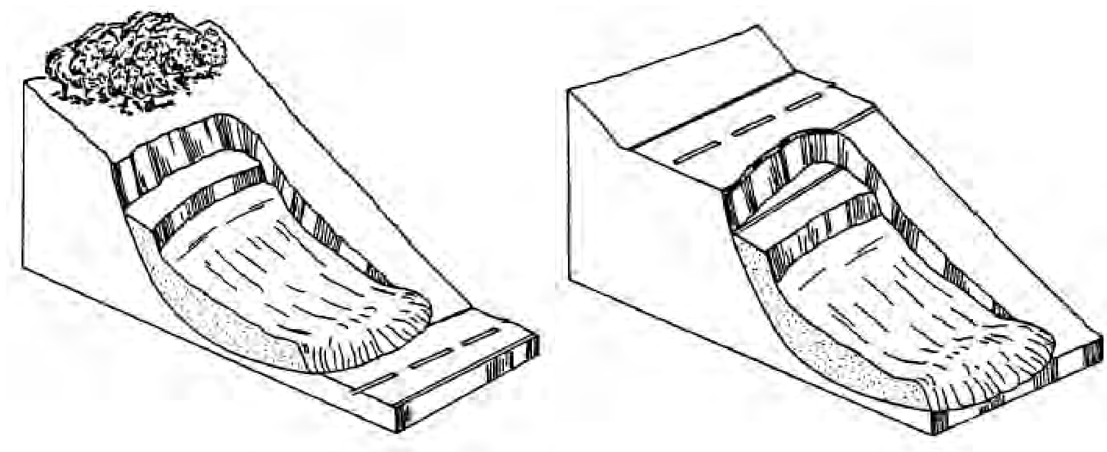
(a) (b)

Figure 2-4 Typical Translational Slides Above or Below Roadway; (a) Transitional Slide above Roadway; (b) Transitional Slide below Roadway

Figure 2-5 Typical Translational Slides – Rock Wedges above Roadway (132035 Manual)

2.2.3.2 Rotational

In weaker soils, the shearing surface can occur through the soil mass and the layer boundaries in the form of an arc (Figure 2-6). In soft relatively uniform cohesive materials, a deep-seated rotational slide into the foundation (a base of the slide) is more common, while in loose cohesionless materials, the circular slip tends to be shallow. The resistance that the moving soil mass is required to overcome in a rotational slide is the maximum (peak) strength of the soil. Therefore, when a rotational failure occurs, it is usually quite rapid. Rotational slides typically occur in road cuts, embankments, and embankments on weak soil foundations. The exposed slip surface is called a scarp. Before the rapid failure of a rotational slide, telltale signs such as cracks can often be observed at the head of the failing slide mass.



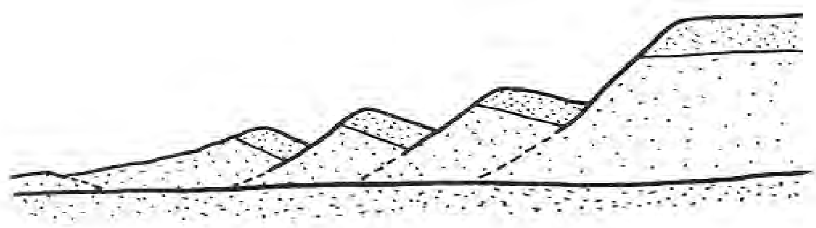
(a) (b)

Figure 2-6 Typical Rotational Slides Above or Below Roadway; (a) Rotational Slide above Roadway; (b) Rotational Slide below Roadway

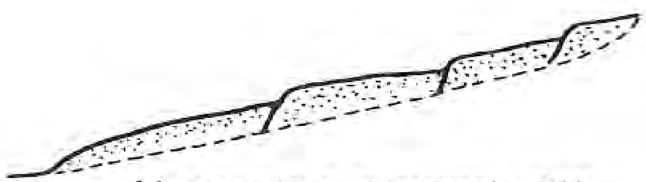
2.2.3.3 Retrogressive Slides

Occasionally a slide can be retrogressive, meaning that it can keep cutting back uphill into the slope (

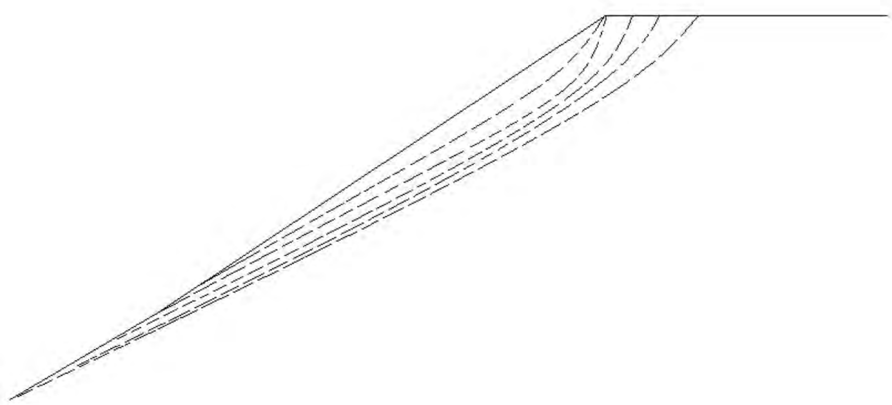
Figure 2-7). After a single failure, the material behind the failure surface becomes unstable and causes another failure uphill. This usually results in a series of slides and all the slides tend to converge in one extended surface. They can be rotational or translational.



a) Rotational retrogressive slide



b) Translational retrogressive slide



c) Rotational/Translational Retrogressive Slide of a Steepened Slope

Figure 2-7 Multiple Retrogressive Slides

2.2.3.4 Progressive (Advancing) Slides

Sometimes a large slide can trigger additional movement in the disturbed material in its toe. These are progressive slides which occur below the original slide surface (Figure 2-8). The falling material from the newly steepened scarp could be the source of such progressive movements. Very small step-like failures are sometimes formed.

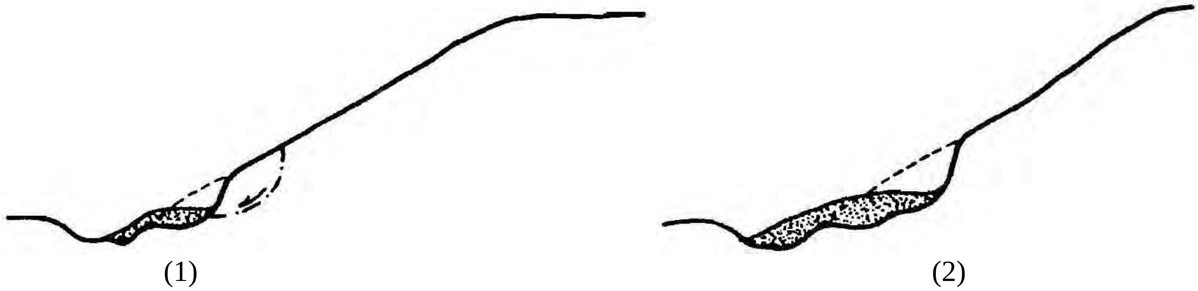


Figure 2-8 Multiple Progressive Slides

2.2.4 Flow and Spread

A flow is a slope failure that flows like a liquid and is heavily influenced by water. The flow characteristic is due to both the amount of water in the soil and the way the soil behaves in the presence of changing water conditions. The displacement rate of a flow can vary from a few centimeters per year to meters per second. In clay soils, a flow can occur when the amount of water in the soil (i.e., moisture content) is much higher than the liquid limit of the soil (the amount of water in the soil required to change the behavior of the soil from a solid to a liquid state). In non-cohesive soils, flows can take place in loose, saturated, silts and sands. Flow type movement, which can be referred to as mudflow, earthflow, debris flow, etc, are often caused by a combination of sliding and individual particle movements. Earthflow is usually used to describe a flow that has a lower moisture content than a mudflow, although the distinction is not well defined. Figure 2-9 presents the common mudflow types above and below roadway.

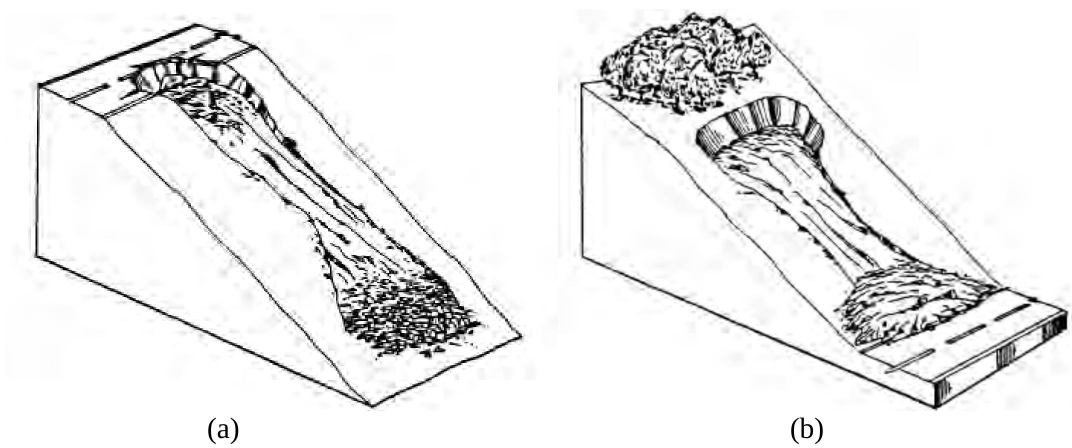


Figure 2-9 Typical Mudflow Slides Below and Above Roadway; (a) Mudflow Slide below Roadway; (b) Mudflow Slide above Roadway

Other flows may result from cuts in a special class of clays referred to as *sensitive* clays. Sensitive clays are stable in their undisturbed condition, but once it is disturbed, either by physical forces or chemical changes, their strength will decrease dramatically and they will become quite unstable.

In slow moving flows the movement rates are often affected by the seasonal climate changes. They commonly occur in shallow clayey materials that are either fissured or finely interstratified with layers of sand with high moisture contents and the slope angle will be close to the angle of residual strength of the material.

2.2.5 Settlement

The settlement of an embankment or side hill fill slope is a combination of the settlement of the embankment/slope fill, and the foundation on which it is resting. In general, the settlement of the foundation is more difficult to predict than that of the embankment/slope fill and quite often it is much larger. This is particularly true when the embankment/slope is resting on soft compressible soils.

2.2.5.1 Embankment/Slope Fill Settlement

The embankment/slope fill settlement is due to the reduction of the voids in the fill caused by the weight of the embankment/slope fill, the traffic loading, and possibly the weather (Figure 2-10). This settlement consists of an elastic component (immediate settlement) that occurs over a relatively short period of time, and a long-term consolidation component if cohesive soils (i.e., clays) are used for the embankment construction. In addition, some time-based settlement may result from the compaction of material placed below specified density requirements (for all soil types).

The elastic settlement of an embankment/slope can usually be ignored because by the time the embankment/slope is completed, most of the elastic settlement has already occurred. The additional elastic settlement due to traffic loading (typically a much smaller load than the embankment load) is insignificant when compared to that caused by the loading of the embankment.

Soil fill embankments/slopes generally exhibit greater magnitudes of fill settlement than do embankments constructed of compacted rockfill due to the higher compressibility of the fill material. Sherard, et al. (1963), based on data from dams constructed by the United States Bureau of Reclamation, estimated that the long-term settlement of a well-compacted earthfill embankment ranges between 0.2 and 0.4 percent of the embankment height.

Figure 2-10 Dips in Grade for Side Fill Slope

2.2.5.2 Embankment/Slope Foundation Settlement

If an embankment/slope is constructed over a weak and compressible foundation (e.g. soft clay or peat) the total settlement of the embankment may be substantial (Figure 2-11). This settlement will typically occur over an extended period of time.

Figure 2-11 Embankment Foundation Failure

2.3 SLOPE STABILITY CONCEPT

The stability of a slope or embankment is a function of the shear strength of the foundation and slope or embankment soils, the geometry (physical shape and dimensions of the slope), the groundwater, and any external loads. For translation failures, the factor of safety of the slope is typically defined as the sum of the resisting forces divided by the driving forces.

Figure 2-12 shows the failure mechanism for a simple non-circular failure surface. The driving force is the component of the weight of the soil above the failure surface that is parallel to the failure surface (W_p) and the resisting force is the sum of the shear strength along the failure surface (S)

$$\text{Factor of Safety} = \frac{\text{Resisting Force}}{\text{Driving Force}} = \frac{S}{W_p} \tag{2-1}$$

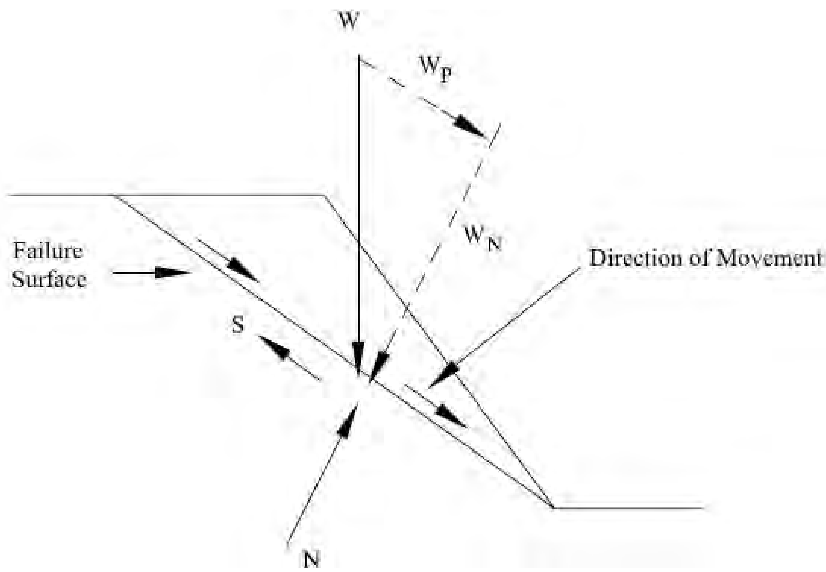


Figure 2-12 Simplified Planar Failure Mechanism

In the case of a rotational slide (i.e., circular), the factor of safety of the slope is defined as the sum of the resisting moments divided by the sum of the driving moments. Figure 2-13 shows a typical rotational failure of an embankment on soft clay.

The failure force (driving force) consists of the weight (W) of the soil above the failure surface. The centroid of this force is located a distance, L_w, from the center of the failure circle. The driving moment (destabilizing moment) is the product of the weight (W) of the failed soil mass times the moment arm (L_w).

The resisting force, against movement, is the sum of the soil shear strength along the failure surface. The resisting moment is the product of the shear force times the radius of the failure circle (R).

The factor of safety against a circular failure is equal to the ratio of the resisting moment divided by the driving moment.

$$\text{Factor of Safety} = \frac{\text{Resisting Moment}}{\text{Driving Moment}} = \frac{S \times R}{W \cdot L_w} \qquad 2\text{-}1$$

When the factor of safety is less than 1, failure will occur.

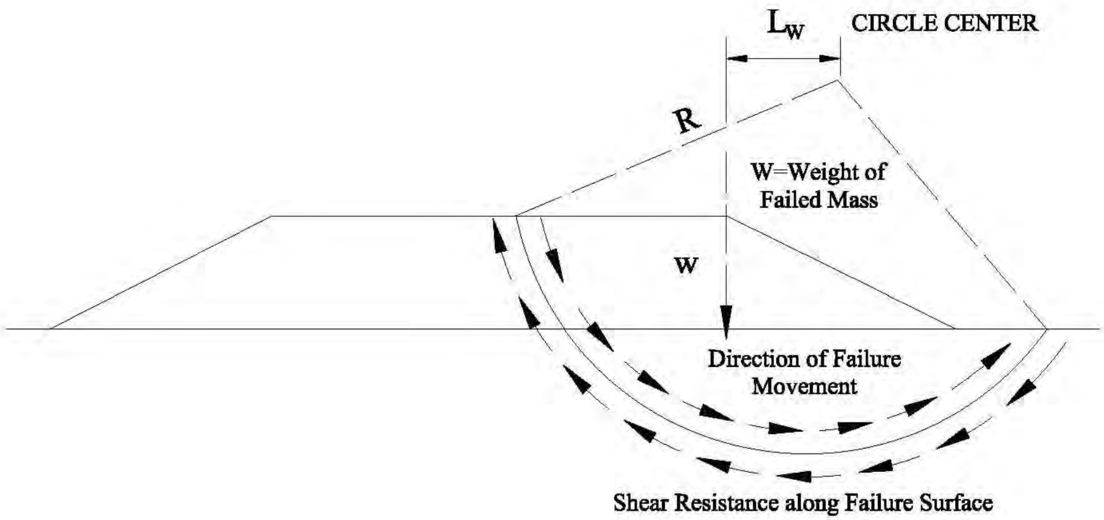


Figure 2-13 Typical Circular Failure Mechanism

An understanding of the role of the factor of safety is vital in the rational design and/or maintenance of slopes. A slope is typically designed for a factor of safety greater than one (between 1.3 and 1.5) to account for uncertainty about the reliability of the items that enter into the analysis, (i.e., strength of the soil layers, groundwater, stratigraphy, and loads). In general, the lower our confidence, in our ability to assess these items accurately, the higher the desired factor of safety should be.

2.4 COMMON CAUSES AND TRIGGERING MECHANISMS OF SLOPE MOVEMENT AND DISTRESS

The geological, topographical, climatic, and other factors that cause slope instability and landslides for highway projects include:

- Excessive slope angle or height
- Low strength foundation
- Removal of materials that provide resistance
- Increased loading
- Environmental factors
- Poor handling of fill materials
- Normally occurring high groundwater table
- Unsuitable materials
- Undetected geologic features
- Liquefaction
- Wildfires

Although there may be many causes for a slope failure or landslide, there usually is only one trigger. The trigger event, by definition, causes a near-immediate response (Varnes, 1978). The common triggering mechanisms for landslides in transportation projects include the following:

- Intense rainfall
- Rapid snowmelt
- Water level changes

- Earthquakes
- Human Activity
- Stream erosion
- Volcanic eruption (not discussed)

2.4.1 Intense Rainfall

Most slope and embankment failures and landslides occur after or during intense rainfall produced by storms. During 1997-98, numerous storms linked to El Niño triggered hundreds of landslides in California that caused casualties, damaged residences, or shut down major highways and local roads.

Slope failures in soils and weathered rock often are generated on steep slopes when both the intensity and duration of the rainfall reach a limiting point that causes the slope to fail. Ellen et al. (1988) stated that during 1982, intense rainfall lasting for about 32 hours in the San Francisco Bay region of California, triggered more than 18,000 predominantly shallow landslides involving soils and weathered rock, which blocked many primary and secondary roads.

2.4.2 Rapid Snow Melt

Rapid snow melt has been responsible for numerous landslides and slope stability problems across the United States. In January of 1995, the Washington, D.C. metropolitan area had over three feet of snow that melted in approximately 24 hours as a result of an uncharacteristic warm spell and thunderstorm. Dozens of shallow slope failures occurred as a result of the rapid snow melt and inundation of water on the slopes and embankments in the region.

2.4.3 Water Level Changes

Water contributes greatly to many slope failures. The sudden lowering of the water level (rapid drawdown) against a slope can trigger a failure, especially along coastlines and on the banks of lakes, reservoirs, canals, and rivers.

An increase in the groundwater table (i.e., following periods of prolonged above-normal precipitation) may increase the weight of the soil mass and at the same time reduce the effective strength of the saturated slope materials, triggering a failure. Rising groundwater levels can also accelerate active landslide movement. An increase water level may also result from seepage in areas below septic systems, ponded depressions, reservoirs, irrigation canals, and diverted surface channels. Such circumstances are sometimes overlooked on the ground because water sources may be far above or below the area under investigation, but they often become obvious in air photos. Leaking utilities are another potential source of excess water.

Intense rainfall and rapid snow melt (Sections 2.4.1 and 2.4.2) in and of themselves do little to affect slope stability unless they change the water level. Intense events, generally, lead to shallow failures and a long wet winters will typically lead to deeper (larger) failures. This is because intense events are usually over before infiltration reaches great depths and because water level changes from a wet month, for example, are still present months later at depth, but have long since dissipated near the surface. This relationship even carries on beyond one year. At depths of several meters or more, the soil moisture content has a large lag time. This means that if several generally wet years occur in a row, the moisture content will be incrementally higher each year, and closer to saturation and slope destabilization. Thus, one can expect increased likelihood of deep movement, even where none has been observed in the past, after a period of wet years.

2.4.4 Earthquake Shaking

Loose, saturated sands are particularly vulnerable to liquefaction during earthquakes, which leads to flow slides or unstable foundation conditions for overlying steep sloping deposits. Case histories indicate that during earthquakes, banks of well-compacted fill constructed over weak foundations are more prone to complete failures or severe slumping as compared to those founded on firm foundations. The Loma Prieta earthquake in October 1989 triggered thousands of slope failures throughout an area of 5,400 square miles. In addition to causing at least tens of millions of dollars of damage to houses, other structures, and utilities, slope failures blocked many transportation routes, greatly hampering rescue and relief efforts. Other examples of earthquake induced failures were found in many sections of highway fills in the Alaskan and Niigata Earthquakes of 1964 (Seed, 1970).

2.4.5 Human Activity

Human activity can often be the cause of serious slope stability problems. For example, plowing snow over the side of an embankment can increase the live load and when the snow melts, raise the groundwater table. Examples of how human activity can alter site conditions and reduce the stability of natural slopes are given below:

- Snow plowing
- Weak strata or stratification planes overloaded with fill or new structures.
- Thin stratum of permeable material, which acts as a natural drainage blanket for soft clay, removed by excavation.
- Seepage pressure increases or orientation of seepage forces change the direction of seepage as a result of cuts or fills or other adjacent construction.
- Hard fissured clays exposed to air or water because of cuts.
- Existing slope toes removed for construction of retaining structures.
- Possible leakage of water in pipes and sewers on crest of slopes.
- Natural slope vegetation removed by construction of access roads for equipment and trucks without provisions for adequate drainage systems.

2.4.6 Stream Erosion

Running water from streams and/or rivers may erode the toe of a slope. Where the banks are made of soil or unconsolidated material, the weakest and the most favorable slide prone position is often located at the point of maximum curvature of the stream. Here, the toe of the soil slope is under constant erosion by the running water, resulting in undermining the toe and an ensuing failure.
Bank erosion from groundwater movement is also common in loess deposits, the processes of which include:

1. Subsurface seepage with tunnel formation and ultimate collapse into gullies.
2. Surface soil creep on steeper slopes that generates slips and shallow slides.

All these erosive actions are time dependent and are influenced by factors such as climate, topography, stratigraphy, physical properties, chemical properties, mineralogy of the soil, and human activities. For engineering evaluation, particle size distribution and density are two properties that can be measured relatively easily and may, in some cases, have an influence on erosion.

2.4.7 Summary

As we have discussed in this section, water is the trigger to most slope movement and/or distresses. Surface water can cause problems by infiltrating into a slope, or by moving overland causing erosion. Both of these scenarios decrease the stability of a slope. Changes in water levels in streams and reservoirs, insufficient surface and groundwater control design details, and improper management of water as otherwise encountered during or after construction, can also significantly and adversely affect the stability of slopes. Properly addressing surface and groundwater will enhance the long term performance of our slopes and embankments. The next chapter will address, from a fundamental engineering perspective, how water affects slope stability.

CHAPTER 3
GENERAL ENGINEERING PROPERTIES OF EARTH MATERIAL AND INFLUENCES OF WATER

3.1 INTRODUCTION

For slope maintenance and restoration purposes, we shall consider the earth to be made up of soil and rock. ***Soil*** is considered uncemented or partially cemented, and will be defined as naturally occurring solid particles which are fairly readily separated into relatively small pieces, and in which the mass may contain air, water, or organic materials (derived from decay of vegetation). The mineral particles of the soil mass are formed from decomposition of the rock by weathering (by air, ice, wind, and water) and chemical processes. ***Rock*** is considered cemented and consolidated, and is that naturally occurring material composed of mineral particles so firmly bonded together that relatively great effort (i.e., blasting or heavy crushing forces) is required to separate the particles.

This chapter provides a basic understanding of the geotechnical properties of soil and rock, effective stress, and laboratory and field tests to determine the engineering properties of these materials. Having a basic understanding of these concepts is an essential tool to effectively manage and maintain soil and rock slopes located on our highways. For most rock slopes on highway projects, the stability is governed by the structural geology consisting of the orientation and characteristics of the joints, bedding and faults that occur behind the rock face. If the rock mass is weathered and fractured, the rock slopes may behave more similarly to the soil slopes. This chapter provides the fundamental concepts and basic geological processes necessary to understand the shear strength of soil and rock.

3.2 GEOTECHNICAL PROPERTIES – SOIL SLOPES

3.2.1 Basic Weight-Volume Relationships

Soil is a three phase system that consists of solid particles, liquid and gas. The liquid and gas phases occupy the voids between the solid particles as shown in Figure 3-1a. For practical purposes, the liquid may be considered to be water (although in some cases the water may contain some dissolved salts or pollutants) and the gas as air. Soil behavior is controlled by the interaction of these three phases. Due to the three phase composition of soils, complex states of stresses and strains may exist in a soil mass. Proper quantification of these states of stress, and their corresponding strains, is a key factor in understanding the performance of slopes.

The first step in the quantification of the stresses and strains in soils is to characterize the distribution of the three phases of the soil mass and determine their interrelationships. The interrelationships of the weights and volumes of the different phases are important, since they not only help define the physical make-up of a soil, but also help determine the in-situ geostatic stresses, i.e., the states of stress in the soil mass due only to the soil's self-weight. The volumes and weights of the different phases of matter in a soil mass shown in Figure 3-1a can be represented by the block diagram shown in Figure 3-1b. This diagram is also known as a phase diagram. A block of unit cross sectional area is considered. The symbols for the volumes and weights of the different phases are shown on the left and right sides of the block, respectively. The symbols for the volumes and weights of the three phases are defined as follows:

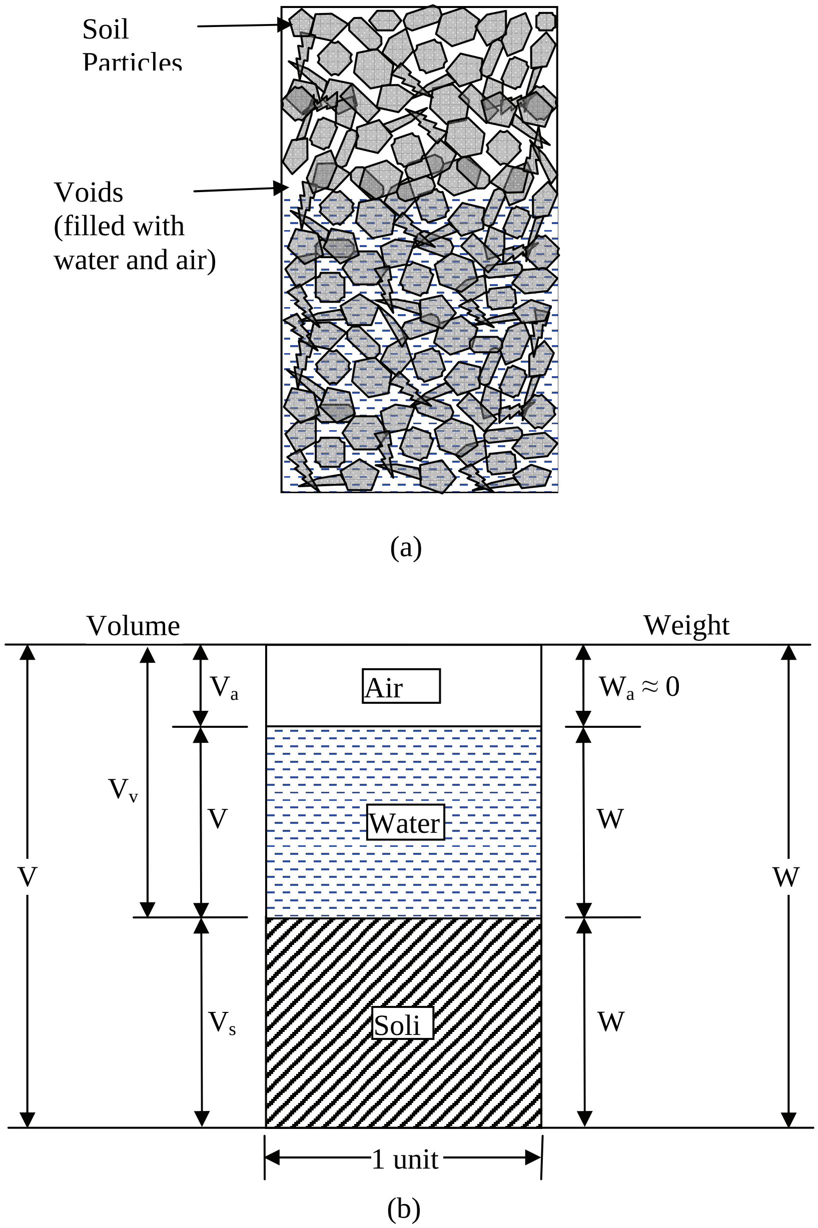


Figure 3-1 A Unit of Soil Mass and Its Idealization

V_a, W_a:	Volume, weight of air phase. For practical purposes, $W_a = 0$.	3-1
V_w, W_w:	Volume, weight of water phase.	3-2
V_v, W_v:	Volume, weight of total voids. For practical purposes, $W_v = W_w$ as $W_a = 0$.	3-3
V_s, W_s:	Volume, weight of solid phase.	3-4
V , W:	Volume, weight of the total soil mass.	3-5

Since the relationship between V_a and V_w usually changes with groundwater conditions as well as under imposed loads, it is convenient to designate all the volume not occupied by the solid phase as void space,

Vv. Thus, $V_v = V_a + V_w$. Use of the terms illustrated in Figure 3-1b, allows a number of basic phase relationships to be defined and/or derived as discussed in the following paragraphs.

3.2.1.1 Unit Weight

The simplest relationship between the weight and volume of a soil mass (refer to Figure 3-1b) is known as the **total unit weight, γ_t,** and is expressed as follows:

$$\gamma_t = \frac{W}{V} = \frac{W_w + W_s}{V} \qquad 3\text{-}6$$

The total unit weight of a soil mass is a useful quantity for computations of vertical in-situ stresses. For a constant volume of soil, the total unit weight can vary, since it does not account for the distribution of the three phases in the soil mass. Therefore, the value of the total unit weight for a given soil can vary from its maximum value when all of the voids are filled with water (S=100%) to its minimum value when there is no water in the voids (S=0%). The former value is called the **saturated unit weight, γ_{sat}**; the latter value is referred to as the **dry unit weight**, γ_d. In terms of the basic quantities shown in Figure 3-1b and with reference to Equation 3-6, when $W_w = 0$ the **dry unit weight, γ_d,** can be expressed as follows:

$$\gamma_d = \frac{W_s}{V} \qquad 3\text{-}7$$

For computations involving soils below the water table, the buoyant unit weight is frequently used where:

$$\gamma_b = \gamma_{sat} - \gamma_w \qquad 3\text{-}8$$

where, γ_w equals the unit weight of water and is defined as follows:

$$\gamma_w = \frac{W_w}{V_w} \qquad 3\text{-}9$$

In the geotechnical literature, the buoyant unit weight, γ_b, is also known as the effective unit weight, γ', or submerged unit weight, γ_{sub}. Unless there is a high concentration of dissolved salts, e.g., in sea water, the unit weight of water, γ_w, can be reasonably assumed to be 62.4 lb/ft^3 (9.81 kN/m^3).

3.2.1.2 Moisture Content

The moisture (or water) content test is one of the simplest and least expensive laboratory tests to perform. Moisture content is defined as the ratio of the weight of the water in a soil specimen to the dry weight of the specimen. Natural moisture contents (w_n) of sands are typically $0 \le w_n \le 20$ %, whereas for inorganic and insensitive silts and clays, the typical range is $10 \le w_n \le 40$ %. However, for clays it is possible to have more water than solids (i.e., $w_n > 100\%$), depending upon the mineralogy, formation environment, and structure of the clay. Therefore, soft and highly compressible clays, as well as sensitive, quick, or organically rich clays, can exhibit water contents in the range of $40 \le w_n \le 300\%$ or more.

The expression for determining moisture content as a function of the weight of the water (w_w) and the weight of the solid (w_s) in a soil sample is shown as below:

$$w_n = \frac{w_w}{w_s} \times 100 \qquad \text{3-10}$$

3.2.2 Size of Grains in the Solid Phase

As indicated in Figure 3-1a, the solid phase is composed of soil grains. One of the major factors that affect the behavior of the soil mass is the size of the grains. The size of the grains may range from the coarsest (e.g., boulders, which can be 12- or more inches [300 mm] in diameter) to the finest (e.g., colloids, which can be smaller than 0.0002–inches [0.005 mm]). Since soil particles come in a variety of different shapes, the size of the grains is defined in terms of an effective grain diameter. The distribution of grain sizes in a soil mass is determined by shaking air-dried material through a stack of sieves having decreasing opening sizes.

Table 3-1 shows U.S. standard sieve sizes and associated opening sizes. Sieves with opening size 0.25 in (6.35 mm) or less are identified by a sieve number which corresponds to the approximate number of square openings per linear inch of the sieve (ASTME 11).

To determine the grain size distribution, the soil is sieved through a stack of sieves with each successive screen in the stack from top to bottom having a smaller (approximately half of the upper sieve) opening to capture progressively smaller particles. Figure 3-2 shows a selection of some sieves and starting from right to left soil particles retained on each sieve, except for the powdery particles shown on the far left, which are those that passed through the last sieve on the stack. The amount retained on each sieve is collected, dried and weighed to determine the amount of material passing that sieve size as a percentage of the total sample being sieved. Since electro-static forces impede the passage of finer-grained particles through sieves, testing of such particles is accomplished by suspending chemically dispersed particles in a water column and measuring the change in specific gravity of the liquid as the particles fall from suspension. The change in specific gravity is related to the fall velocities of specific particle sizes in the liquid. This part of the test is commonly referred to as a hydrometer analysis. Because of the strong influence of electro-chemical forces on their behavior, colloidal sized particles may remain in suspension indefinitely (particles with sizes from 10^{-3} mm to 10^{-6} mm are termed "**colloidal**.") Sample grain size distribution curves are shown in Figure 3-3. The nomenclature associated with various grain sizes (cobble, gravel, sand, silt or clay) is also shown in Figure 3-3. Particles having sizes larger than the No. 200 sieve (0.075 mm) are termed "**coarse-grained**" or "**granular**" while those with sizes finer than the No. 200 sieve are termed "**fine-grained**." The results of the sieve and hydrometer tests are represented graphically on a grain size distribution curve or gradation curve.

The shape of the grain size distribution curve is somewhat indicative of the particle size distribution as shown in Figure 3-3. For example, A smooth curve covering a wide range of sizes represents a *well-graded* or *non-uniform* soil.

A vertical or near vertical slope over a relatively narrow range of particle sizes indicates that the soil consists predominantly of only a small range of particle sizes and is called a *poorly-graded* or *uniform* soil.

A curve that contains a horizontal or nearly horizontal portion indicates that the soil is deficient in the grain sizes in the region of the horizontal slope. Such a soil is called a *gap-graded* soil.

Well-graded soils are generally produced by bulk transport processes (e.g., glacial till). Poorly graded soils are usually sorted by the transporting medium, e.g., beach sands by water; loess by wind. Gap-graded soils are also generally sorted by water, with certain sizes were not transported.

Figure 3-2 Example of laboratory sieves for mechanical analysis for grain size distributions. Shown (left to right) are sieve nos. No. 200 (0.075 mm), No. 40 (0.425 mm), No. 10 (2.0-mm), and 3/8-in (9.5-mm). Example soil particle sizes shown at the bottom of the photo include (left to right): clay (kaolin), silt, medium-coarse sand, fine gravel, and medium gravel (FHWA, 2002b)

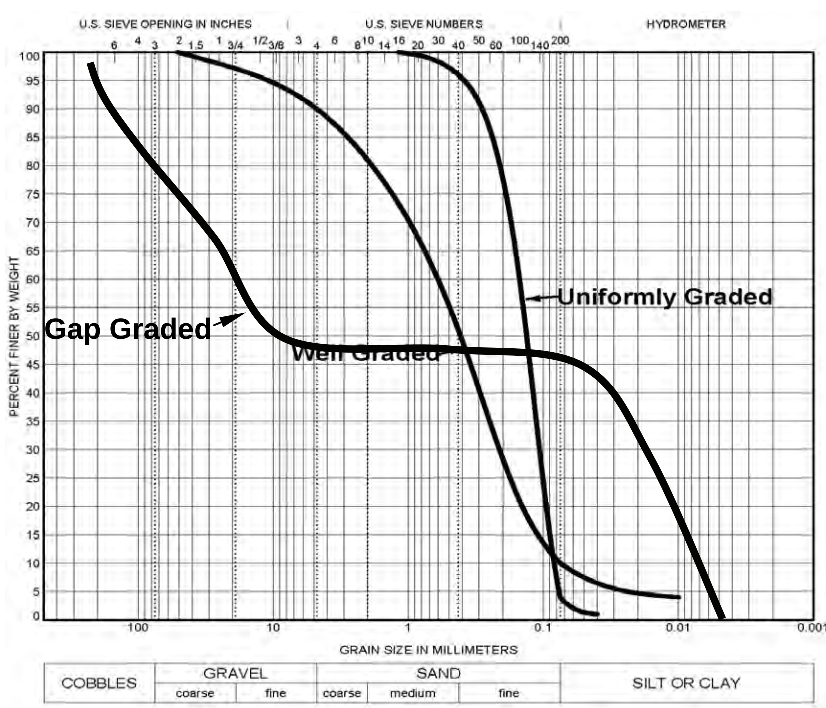


Figure 3-3 Sample Grain Size Distribution Curves

TABLE 3-1 U.S. STANDARD SIEVE SIZES AND CORRESPONDING OPENING DIMENSION

U.S. Standard Sieve No.[1]	Sieve Opening (in)	Sieve Opening (mm)	Comment (Based on the Unified Soil Classification System (USCS) discussed in Chapter 4)
3	0.2500	6.35	
4	0.1870	4.75	Breakpoint between fine gravels and coarse sands Soil passing this sieve is used for compaction test
6	0.1320	3.35	
8	0.0937	2.36	
10	0.0787	2.00	Breakpoint between coarse and medium sands
12	0.0661	1.70	
16	0.0469	1.18	
20	0.0331	0.850	
30	0.0234	0.600	
40	0.0165	0.425	Breakpoint between medium and fine sands Soil passing this sieve is used for Atterberg limits
50	0.0117	0.300	
60	0.0098	0.250	
70	0.0083	0.212	
100	0.0059	0.150	
140	0.0041	0.106	
200	0.0029	0.075	Breakpoint between fine sand and silt or clay
270	0.0021	0.053	
400	0.0015	0.038	

Note:
The sieve opening sizes for various sieve numbers listed above are based on Table 1 from ASTM E 11. Sieves with opening size greater than No. 3 are identified by their opening size. Some of these sieves are as follows:

4.0 in	(101.6 mm)	1-1/2 in	(38.1 mm)	½ in	(12.7 mm)
3.0 in	(76.1 mm)*	1-1/4 in	(32.0 mm)	3/8 in	(9.5 mm)
2-1/2 in	(64.0 mm)	1.0 in	(25.4 mm)	5/16 in	(8.0 mm)
2.0 in	(50.8 mm)	¾ in	(19.0 mm)**		
1-3/4 in	(45.3 mm)	5/8 in	(16.0 mm)		

* The 3 in (76.1 mm) sieve size differentiates between cobbles and coarse gravels. **The ¾ in (19 mm) sieve differentiates between coarse and fine gravels.

3.2.3 Shape of Grains in Solid Phase

The shape of individual grains in a soil mass plays an important role in the engineering characteristics (strength and stability) of the soil. Two general shapes are normally recognized, bulky and platy.

Bulky Shape

Cobbles, gravel, sand and some silt particles cover a large range of sizes as shown in Figure 3-2; however, they are all bulky in shape. The term bulky is confined to particles that are relatively large in all three dimensions, as contrasted to platy particles, in which one dimension is small as compared to the other two.

Platy Shape

Platy, or flaky, particles are those that have flat, plate-like grains. Clay and some silts are common examples. Because of their shape, flaky particles have a greater surface area than bulky particles, assuming that the weights and volumes of the two are the same. For example, 1 gram of bentonite (commercial name for montmorillonite clay) has a surface area of approximately 950 yd^2 (800 m^2) compared to a surface area of approximately 0.035 yd^2 (0.03 m^2) of 1 gram of sand. Because of their mineralogical composition and greater specific surface area, most flaky particles also have a greater affinity for water than bulky particles. Due to the high affinity of such soils for water, the physical states of such fine-grained soils change with the amount of water in these soils. The effect of water on the physical states of fine grained soils is discussed next.

3.2.4 Effect of Water on Physical States of Soils

For practical purposes, the two most dominant phases are the solid phase and the water phase. It is intuitive that as the water content increases, the contacts between the particles comprising the solid phase will be "lubricated." If the solid phase is comprised of coarse particles, e.g. coarse sand or gravels, then water will start flowing between the particles of the solid phase. If the solid phase is comprised of fine-grained particles, e.g., clay or silt, then water cannot flow as freely as in the coarse-grained solid phase because pore spaces are smaller and solids react with water. However, as the water content increases, even the fine-grained solid phase will conduct water and under certain conditions the solid phase itself will start deforming like a viscous fluid, e.g., like a milk shake or a lava flow. The mechanical transformation of the fine-grained soils from a solid phase into a viscous phase is a very important concept in geotechnical engineering since it is directly related to the load carrying capacity of soils. It is obvious that the load carrying capacity of a solid is greater than that of water. Since water is contained in the void space, the effect of water on the physical states of fine-grained soils is important. Some of the basic index properties related to the effects of water are described next.

The physical and mechanical behavior of fine-grained-soils is linked to four distinct states: solid, semi-solid, plastic and viscous liquid in order of increasing water content. Consider a soil initially in a viscous liquid state that is allowed to dry uniformly. This state is shown as Point A in
Figure 3-4 which shows a plot of total volume versus water content. As the soil dries, its water content reduces, and consequently so does its total volume as the solid particles move closer to each other. As the water content reduces, the soil can no longer flow like a viscous liquid. Let us identify this state by Point B in
Figure 3-4. The water content at Point B is known as the "Liquid Limit" in geotechnical engineering and is denoted by LL. As the water content continues to reduce due to drying, there is a range of water content at which the soil can be molded into any desired shape without breaking apart. In this range of water content, the soil is considered to be "plastic."

The liquid limit (LL), plastic limit (PL) and shrinkage limit (SL) are called Atterberg limits (after A. Atterberg (1911), the Swedish soil scientist who first proposed them for agricultural applications). By defining these points for a given soil, we are able to describe how the soil responds to water, which is very helpful in describing its behavior for engineering purposes. In fact, a standardized test has been developed to define both the LL and PL. The range between these two limits is known as the Plasticity Index.

- The ***Liquid Limit*** represents the moisture content at which any increase in moisture content will cause a plastic soil to behave as a viscous liquid. The LL is defined as the moisture content at which a standard groove cut in a remolded sample will close over a distance of ½-inch (13 mm) at 25 blows of the liquid limit device (Figure 3-4). The test is performed on material passing a US Standard No. 40 sieve (0.425 mm). During the test, the material is brought to various moisture contents, usually by adding water. The plot of moisture contents vs. blows required to close the groove is called a "flow curve" and the value of the liquid limit moisture content is obtained from the flow curve at 25 blows.

- The ***Plastic Limit*** represents the moisture content at which the transition between the plastic and semisolid state of a soil occurs. The PL is defined as the moisture content at which a thread of soil just crumbles when it is carefully rolled out by hand to a diameter of 1/8-inch (3 mm).

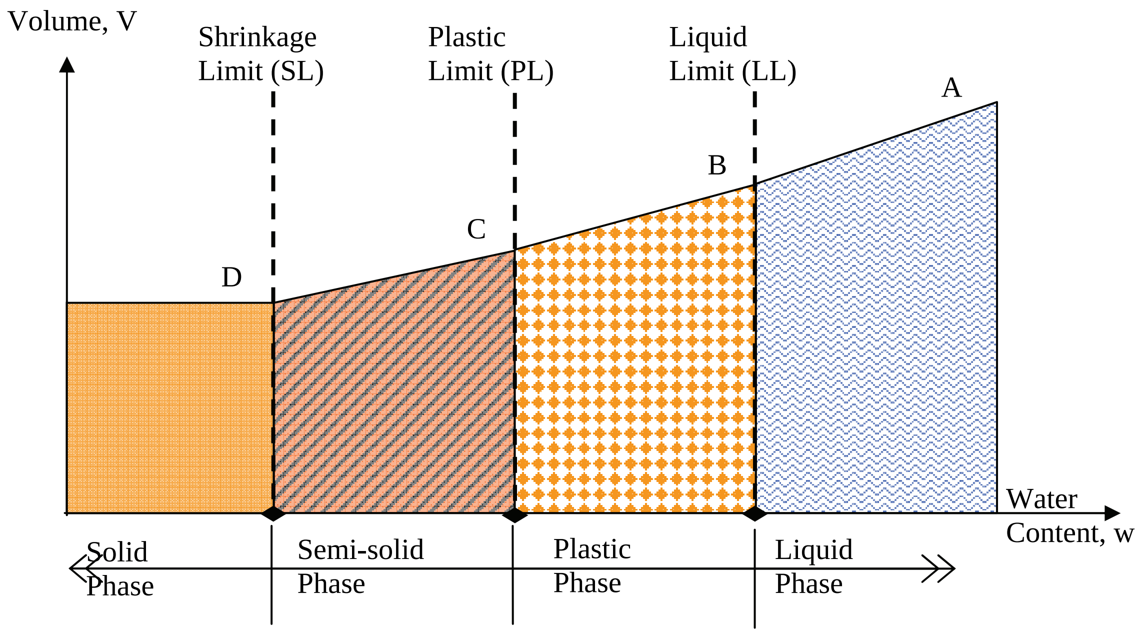


Figure 3-4 Conceptual Changes in Soil Phases as a Function of Water Content

Figure 3-5 Some of the Equipment Used for Atterberg Limits Testing of Soil.

If the soil is allowed to dry beyond the plastic state, the soil cannot be molded into any shape without showing cracks, i.e., signs of rupture. The soil is then in a semi-solid state. The water content at which cracks start appearing when the soil is molded is known as the "Plastic Limit." This moisture content is shown at Point C in Figure 3-4 and is denoted by PL. The difference in water content between the Liquid Limit and Plastic Limit, is known as the **Plasticity Index**, PI, and is expressed as follows:

$$PI = LL - PL \tag{3-11}$$

Since PI is the difference between the LL and PL, it denotes the range in water content over which the soil acts as a plastic material as shown in Figure 3-4.

It is useful to compare the natural moisture content of a soil with its Atterberg limits. By comparing the natural moisture content to the Atterberg Limits, one can see how the soil, at its natural moisture content will behave. The following relationship is known as the Liquidity Index (LI):

$$LI = \frac{w_n - PL}{PI} \tag{3-12}$$

where w_n is the natural (in-situ) water content of the soil. Note that when the Liquidity Index is greater than 1, the natural moisture content is greater than the Liquid Limit. When the Liquidity Index is less than 0, the natural moisture content is less than the plastic limit. Numerous other engineering correlations have been developed that relate PI and LI to clay soil properties, including undrained and drained strength to PI and compression index to LI.

As the soil continues to dry, it will be reduced to its basic solid phase. The water content at which the soil changes from a semi-solid state to a solid state is called the **Shrinkage Limit**, SL. Shrinkage Limit represents the moisture content corresponding to the change between the semisolid to solid state of the soil. The SL is also defined as the moisture content at which any further reduction in moisture content will not result in a decrease in the volume of the soil. No significant change in volume will occur with additional drying below the shrinkage limit. The shrinkage limit is useful for the determination of the swelling and shrinkage characteristics of soils.

For stability analysis, engineers are most interested in the load carrying capacity, i.e., strength, of the soil and its associated deformation. The soil has virtually no strength at the LL, while at water contents lower than the PL (and certainly below the SL) the soil may have considerable strength. Correspondingly, soil strength increases and soil deformation decreases as the water content of the soil reduces from the LL to the SL.

A valuable tool in assessing the characteristics of a fine-grained soil is to compare the LL and PI of various soils. Each fine-grained soil has a relatively unique value of LL and PI. A plot of PI versus LL is known as the **Plasticity Chart** (see Figure 3-6).

Casagrande proposed the inclusion of the A-line on the plasticity chart as a boundary between clay (above the A-line) and silt (below the A-line) to help assess the engineering characteristics of fine-grained soils. Once PI and LL are determined for a fine-grained soil at a specific site, a point can be plotted on the plasticity chart that will allow the engineer to develop a feel for the general engineering characteristics of that particular soil. The plasticity chart also permits the engineer to compare different soils across the project site and even between different project sites. The laboratory determination of the various limits (LL, PL and SL), are discussed later in this chapter.

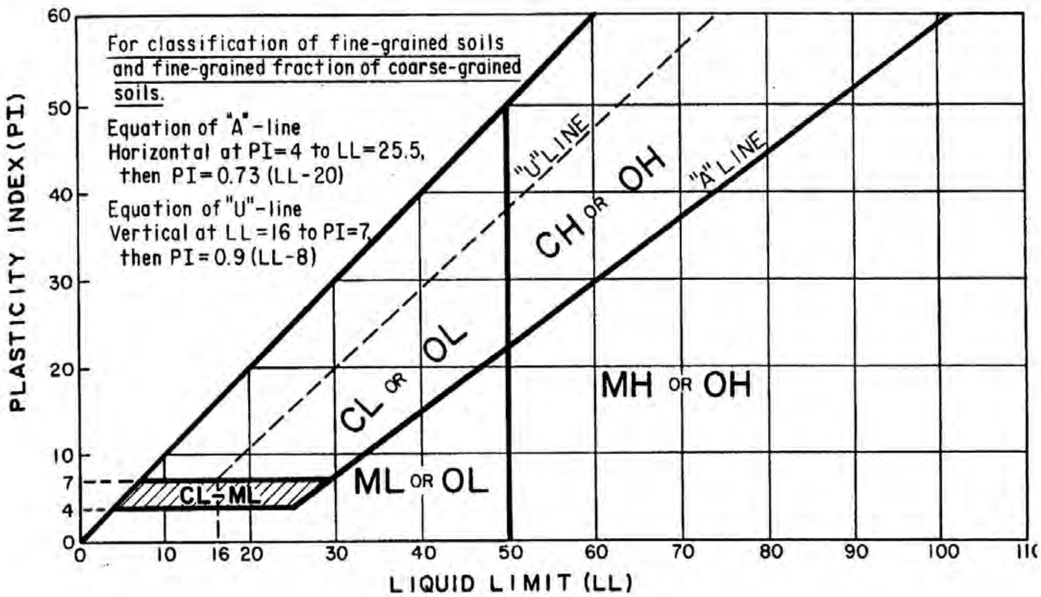


Figure 3-6 Plasticity Chart from Unified Soil Classification System (ASTM D-2487)

3.3 GEOTECHNICAL PROPERTIES – ROCK SLOPES

The strength of rock is generally controlled by the strength of the joints in the rock. In moderately strong to strong rock, the stress level due to gravity loads will be significantly less than the strength of the rock. Therefore, there will be little tendency for intact rock within the slope to fracture, and the compressive strength of an intact rock sample is a less important design parameter than the shear strength of the rock mass. Therefore, the design of rock cuts for highways is usually concerned with the details of the structural geology, that is, the orientation and characteristics (such as length, roughness and infilling materials) of the joints, bedding and faults that occur behind the rock face.

The durability of the rock may be of importance as well depending on the rock type and the weathering process.

3.3.1 Structural Geology

Figure 3-7 illustrates thirteen important geotechnical and geological properties of the rock mass that are necessary when evaluating a rock slope. A brief description of each property is presented (see Chapter 2 of the FHWA publication HI-99-007 "Rock Slopes" for a detailed discussion).

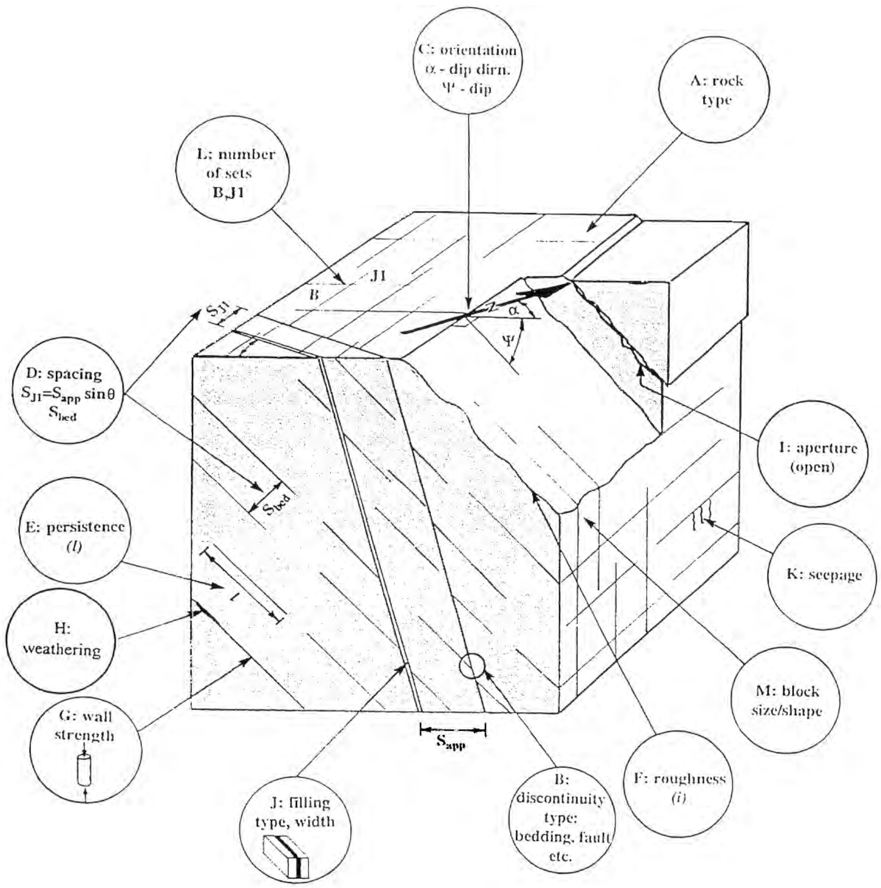


Figure 3-7 Definition of Structural Geological Terms of a Rock Mass (Wyllie, 1999)

Rock type - the rock type is defined by the origin of the rock (i.e. sedimentary, metamorphic or igneous), the mineralogy, the color and grain size (Deere and Miller, 1966).

Discontinuity type - discontinuity types range from clean tension joints of limited length to faults containing several centimeters of clay gouge and lengths of many kilometers.

Discontinuity orientation - the orientation of discontinuities is expressed as the dip and dip direction (or strike) of the surface. The dip of the plane is the maximum angle of the plane

Spacing - discontinuity spacing can be mapped in rock faces and in drill core, with the true spacing being calculated from the apparent spacing for discontinuities inclined to the face; spacing categories range from extremely wide (>2 m) to very narrow (<6 mm).

Persistence - persistence is the measure of the continuous length or area of the discontinuity; persistence categories range from very high (>20 m) to very low (<1 m).

Roughness - the roughness of a discontinuity surface is often an important component of the shear strength, especially where the discontinuity is undisplaced and interlocked.

Wall strength - the strength of the rock forming the walls of discontinuities will influence the shear strength where high stresses are generated at local contact points during shearing; rock strength categories range from very strong (>200 MPa) to very weak (1 to 25 MPa).

Weathering - reduction of rock strength due to weathering will reduce the shear strength of discontinuities as described in (G) above, as well as reduce the shear strength of the rock mass due to the diminished strength of the intact rock; weathering categories range from fresh rock to residual soil.

Aperture - aperture is the perpendicular distance separating the adjacent rock walls of an open discontinuity, in which the intervening space is air or water filled; categories of aperture range from cavernous (>1 m) to very tight (<0.1 mm). Aperture is thereby distinguished from the "width" of a filled discontinuity.

Filling/width - filling is the term for material separating the adjacent walls of discontinuities, such as calcite or fault gouge; the perpendicular distance between the adjacent rock walls is termed the **width** of the filled discontinuity.

Seepage - observations of the location of seepage from discontinuities provides information on aperture because ground water flow is confined almost entirely in the discontinuities (secondary permeability); seepage categories range from very tight and dry, to continuous flow that can wash out infillings.

Number of sets - the number of sets of discontinuities that intersect one another will influence the extent to which the rock mass can deform without failure of the intact rock.

Block size/shape - the block size and shape are determined by the discontinuity spacing and persistence, and the number of sets. Block shapes include blocky, tabular, shattered and columnar, while block size ranges from very large (>8 m^3) to very small (<0.0002 m^3).

3.3.2 Rock Durability

Widely occurring rock materials are prone to degradation when exposed to weathering processes such as wetting and drying, and freezing and thawing cycles. Rock types which are particularly susceptible to degradation are shale and mudstone which usually have a high clay content. The degradation can take the form of swelling, and weakening and disintegration and can occur within minutes to years after exposure. The effect of degradation on slope stability can range from surficial sloughing and gradual retreat of the face, to slope failures resulting from the loss of strength with time (Wu et al, 1981). In sedimentary formations comprising alternating beds of resistant sandstone and relatively degradable shale, the weathering process can develop overhangs in the sandstone and produce a rock fall hazard (see Figure 2-2).

The measurement of the durability of rock becomes an issue when it is to be subjected to the elements (e.g., flowing water, wetting and drying, wave action, freeze and thaw, etc.) in its proposed use. A simple index test of the tendency of rock to weather and degrade is the slake durability test (ISRM, 1981(a)). It is important that undisturbed samples are used that have not been excessively broken in the sampling procedure, or allowed to freeze. The test procedure comprises placing the sample in a wire mesh drum and drying it in an oven at 105 degrees for 2 to 6 hours. The drum is then partially submerged in water and rotated at 20 revolutions per minute for a period of 10 minutes. The drum is then dried a second time and the loss of weight is recorded. The test cycle is then repeated and the slake durability index is calculated as the percentage ratio of final to initial dry sample masses. A low slake durability index will indicate that the rock is susceptible to degradation when exposed. For highly degradable rocks, it is useful to perform soil classification tests such as Atterberg limits, and X-ray diffraction tests to identify clay mineral types and determine if swelling clays such as bentonites and montmorillonites are present.

3.4 PRINCIPLE OF EFFECTIVE STRESS

The contacts between soil grains are effective in resisting applied stresses in a soil mass. Under an applied load, the total stress in a saturated soil sample is composed of the intergranular stress and the pore water pressure. When pore water drains from a soil, the contact between the soil grains increases, which increases the level of intergranular stress. This intergranular contact stress is called the **effective stress**. **The effective stress, p_o, within a soil mass is the difference between the total stress, p_t, and pore water pressure, u**. The **principle of effective stress** is a fundamental aspect of geotechnical engineering and is written as follows:

$$p_o = p_t - u \tag{3-13}$$

In general, soil deposits below the ground water table will be considered saturated and the ambient pore water pressure at any depth may be computed by multiplying the unit weight of water, γ_w, by the height of water above that depth. The total stress at that depth may be found by multiplying the total unit weight of the soil by the depth. The effective stress is the total stress minus the pore water pressure. This concept is used to construct the profile of pressure in the ground as a function of depth and is discussed next.

3.4.1 Groundwater and Its Effect on Slope Stability

Within a slope or embankment, a change in the groundwater table will affect the stability of the structure. A rise in the groundwater table will increase the pore water pressure within the slope, resulting in a lower effective stress (i.e., due to increased buoyancy), which reduces the available shear strength within the soil mass. At the same time, the total unit weight of the soil (weight of the solids and water) increases, resulting in an increase in the driving force (destabilizing force). The combination of these factors (decreased shear strength and increased driving force) decreases the stability of the mass. Rapid increases in positive pore pressure can occur during and after a period of heavy rainfall. This is the main reason why the frequency of slope failure increases after heavy rainfall. The rate of increase, however, depends on many factors, such as the rate of rainfall, the nature of the ground surface, the catchment area, and the soil permeability.

The most significant effect of ground water on slope stability is the reduction in normal stress produced by water pressures acting within the slope. As shown in equation (3-6), a reduction in the effective normal stress results in a corresponding reduction in the shear strength of the rupture surface.

Generally, the strength properties of rock are the same whether they are wet or dry. However, some rocks, such as shales containing swelling clay and evaporites such as gypsum, exhibit a loss of strength in the presence of water.

3.5 SHEAR STRENGTH OF SOIL AND ROCK

If the imposed stress in a slope or embankment is increased until the deformations (movements) become unacceptably large, a "failure" is considered to have taken place. In this case, the strength of the soil/rock is considered to be insufficient to withstand the applied stress.

The strength of geologic materials is variable and is dependent on many factors, including material properties, magnitude and direction of the applied forces and their rate of application, drainage conditions of the mass, and the magnitude of confining pressure. Unlike steel, whose strength is usually discussed in terms of either tension or compression, and concrete, whose strength is generally discussed in terms of compressive strength only, the strength of soil is generally discussed in terms of shear strength. Typical geotechnical failures occur when the shear stresses induced by applied loads exceed the soil's shear strength somewhere within the soil mass.

3.5.1 Basic Concept of Shearing Resistance and Shearing Strength

The basic concept of shearing resistance and shearing strength can be understood by first studying the principle of friction between solid bodies. Consider a prismatic block B resting on a plane surface XY as shown in Figure 3-8. The block B is subjected to two forces:

- A normal force, P_n, that acts perpendicular to the plane XY, and
- A tangential force, F_a, that acts parallel to the plane XY.

Assume that the normal force, P_n, is constant and that the tangential force, F_a, is gradually increased. At small values of F_a, the block B will not move since the applied force, F_a, will be balanced by an equal and opposite force, F_r, on the plane of contact XY. The resisting force, F_r, is developed as a result of surface roughness on the bottom of the block B and the plane surface XY.

If the applied horizontal force, F_a, is gradually increased, the resisting force, F_r, will likewise increase, always being equal in magnitude and opposite in direction to the applied force. When the force F_a reaches a value that exceeds the resisting force (F_r), the block B will start sliding along the plane.

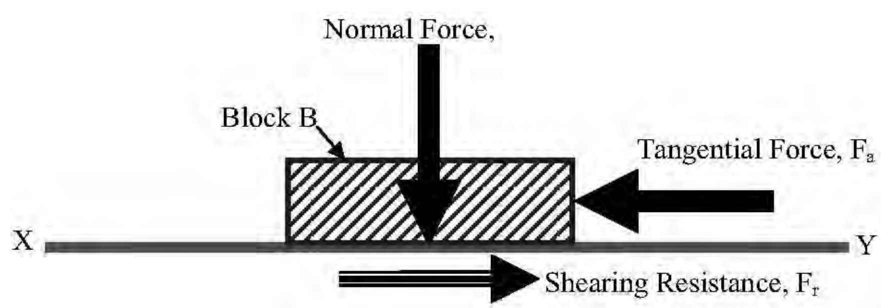


Figure 3-8 Basic Concept of Shearing Resistance and Strength (After Murthy, 1989)

If the same experiment is conducted with a greater normal force, P_n, the maximum frictional or shearing resistance $(F_a)_{max}$, will be correspondingly greater. A series of such experiments would show that for the case where the block and surface are made of the same material, the maximum frictional or shearing resistance is approximately proportional to the normal load P_n as follows:

$$(F_a)_{max} = P_n \tan \varphi \qquad 3\text{-}14$$

where φ is the effective friction angle of the block.

If A is the overall contact area of the block B on the plane surface XY, the relationship in Equation 3-14(a)may be written as follows to obtain stresses on surface XY:

$$\frac{(F_a)_{max}}{A} = \left(\frac{P_n}{A}\right)\tan\phi \qquad 3\text{-}14(a)$$

or

$$\tau = \sigma_n \tan \varphi \qquad 3\text{-}14(b)$$

The term σ_n is called the normal stress and the term τ is called the shear strength. A graphical representation of Equation 3-14 is shown in Figure 3-14a. In reality, the relationship is curved, but since most geotechnical problems involve a relatively narrow range of pressures, the relationship is assumed to be linear as represented by Equation 3-14 over that range.

The concept of frictional resistance explained above applies to soils that possess only the frictional component of shear strength, i.e., generally coarse-grained granular soils. But soils that are not purely frictional exhibit an additional strength component due to internal electro-chemical bonding between the particles. This bonding between the particles is typically found in fine-grained soils and is termed cohesion, c. Simplistically, the shear strength, τ, of such soils is expressed by two additive components as follows and can be graphically represented as shown in Figure 3-14(b):

$$\tau = c + \sigma_n \tan \varphi \qquad 3\text{-}15$$

Again, in reality, the relationship is curved. But, as noted above, since most geotechnical problems involve a relatively narrow range of pressures, the relationship is assumed to be linear as represented by Equations 3-14 over that range.

Equation 3-15 was first proposed by French engineer Coulomb and is used to express shear strength of soils. When plotted on arithmetic axes the resulting straight line is conventionally known as the Mohr-Coulomb (M-C) failure envelope. “Mohr” is included in “Mohr-Coulomb” because Equation 3-10 can also be derived based on the concept of Mohr’s circle.

As indicated previously, the deformation of soils occurs under effective stresses. In terms of effective stresses, Equation 3-16 can be re-written as follows:

$$\tau' = c' + (\sigma_n - u) \tan \varphi' = c' + \sigma' \tan\varphi' \qquad 3\text{-}16$$

where c′ = effective cohesion, σ′ is the effective normal stress and φ′ is the effective friction angle.

In geotechnical engineering, the normal stresses are commonly expressed using the overburden pressure concept. In terms of overburden pressure, the term σ_n in above equations is the same as p_t and the term σ′ is the same as p_o. Thus, Equation 3-17 and Equation 3-18 can be expressed in terms of overburden stresses as follows:

$$\tau = c + p_t \tan \varphi \qquad 3\text{-}17$$

$$\tau' = c' + (p_t - u) \tan \varphi = c' + p_o \tan\varphi' \quad \text{3-18}$$

The physical meaning of the M-C failure envelope shown in Figure 3-9(a) and Figure 3-9(b) may be explained as follows:

Every point on the M-C failure envelope represents a combination of normal and shear stress that results in failure of the soil, i.e., the Mohr failure envelope essentially defines the strength of the soil. In other words, any point along the M-C envelope defines the limiting state of stress for equilibrium.

If the state of stress is represented by a point below the M-C failure envelope, then the soil will be stable for that state of stress.

States of stress above the M-C failure envelope cannot exist, since failure would have occurred before that point could be reached.

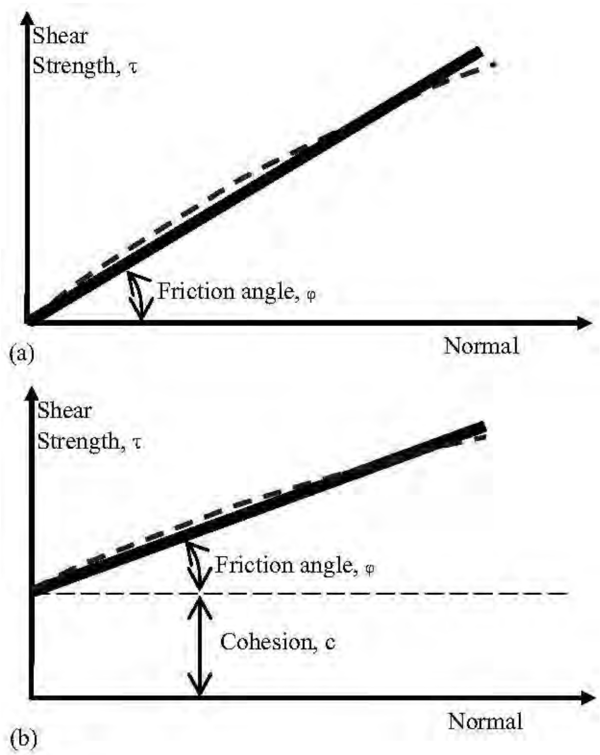


Figure 3-9 Graphical Representation of Shearing Strength

The shear strength of the soil along the failure surface is a function of the effective stress at failure as discussed previously. Furthermore, the effective stress can only be calculated indirectly if the pore water pressure at the state of failure is known, both in the laboratory and in the field. Excess pore water pressure in cohesionless (granular) soils are expected to dissipate rapidly during construction and, thus, as discussed previously, only drained shear strengths should be considered. Typically, however, loading (i.e. fill and embankment loading) and unloading (i.e. excavation) are applied much faster than the pore water can escape from the pores of a cohesive soil, and consequently excess pore pressure is generated. This is due to the fact that saturated fine-grained soils have a much lower permeability than granular soils, andthe tendency for water to move in and out of the pores is severely restricted in the clayey soils.

Commonly, based on the drainage and loading conditions as discussed above, stability problems have been analyzed based on three general categories:

- Drained (for long-term, or very slow loading condition),
- Undrained (for short-term or rapid loading condition), and
- Partially Drained (for intermediate, staged loading, or control rate loading condition)

In the laboratory, there are limiting conditions of drainage in triaxial tests (section 3.5.2) which can simulate the above three categories, which include consolidated-drained (CD), unconsolidated-undrained (UU), and consolidated-undrained (CU) conditions.

3.5.2 Laboratory Strength Tests for Soil and Rock Samples

Data generated from laboratory index tests provide an inexpensive way to assess soil consistency and variability among samples collected from a site. Information obtained from index tests is used to select samples for engineering property and strength parameter testing as well as to provide an indicator of general engineering behavior. For example, a soil with a high plasticity index (PI) can be expected to have high compressibility, low permeability, and high swell potential. Common index tests discussed in this section include moisture content, unit weight (wet density), Atterberg limits, and particle size distribution. Performance tests are tests that measure a material property that directly relates to performance. The performance tests that are important in evaluating the stability of slopes and embankments are the direct shear and triaxial shear strength tests.

The majority of strength tests conducted on cohesive soils are performed on relatively undisturbed samples obtained from thin walled sampling tubes. Non-cohesive soils are difficult to sample without considerable disturbance so testing is usually done on disturbed samples that are “remolded” or “reconstituted” prior to testing, or in-situ testing is performed. The value of test results for slope stability analysis or for other uses depends on how the sample was obtained or prepared, so this information should always be recorded and presented with the results.

The most common laboratory strength tests for soils are:

- Direct shear test
- Uniaxial compression test
- Triaxial compression test

The most common laboratory strength tests for rock samples are:

- Direct shear test
- Uniaxial compression test
- Point load test

3.5.3 Direct Shear Tests

The oldest form of shear test upon soil is the direct shear test, first used by Coulomb (1776). A schematic of the essential elements of the direct shear apparatus are shown in Figure 3-10. The soil is held in a box that is split across its middle; the bottom portion of the box is typically fixed against lateral movement. A confining normal force, N, is applied, and then a tangential shear force, T, is applied so as to cause relative displacement between the two parts of the box. The magnitude of the shear force is recorded as a function of the shear displacement, and the change in thickness of the soil sample is also recorded. The direct shear test is widely used in practice because it is a relatively simple test to perform and provides valuable information on the residual strength (i.e., strength after failure) of the soil. Many slope failures experience large deformations by the time they are recognized. At large deformations, many soils (i.e., soft clays and dense sands) lose strength. At small deformations, they reach their maximum strength (peak strength), and then with continued deformation, the strength drops to the residual strength. The direct shear test is an excellent test to evaluate the residual strength of the soil.

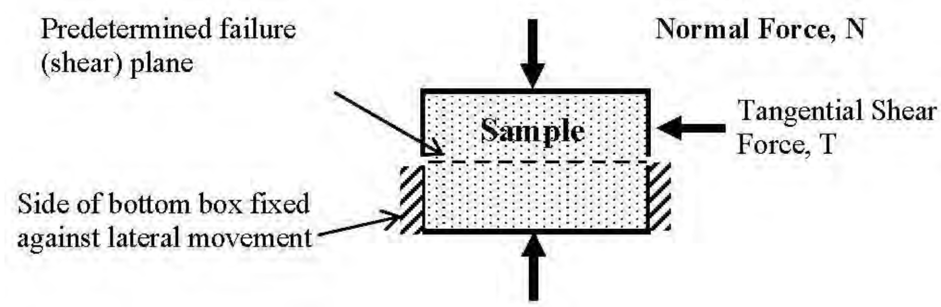


Figure 3-10 Schematic of the Direct Shear Test

3.5.4 Uniaxial Compressive Test

The uniaxial compression test is the simplest and fastest test for the determination of the rock strength and also can be used for cohesive soils. For soils, the test is not as accurate as the triaxial test discussed below: because of the influence of disturbance and uncertainty regarding confining pressure. The results of this test generally produce conservative values of intact rock strength. The results are influenced by the moisture content of the specimens. The geotechnical engineer and the geologist in charge need to make a decision as to the moisture content (i.e., natural, saturated, etc.) under which the rock should be tested. The rate of loading and the condition of the two ends of the rock will also affect the final results. The ends have to be nearly parallel to each other and clean of powder or small debris. The rate of loading should be closely controlled as per the ASTM test procedure. Inclined fissures, soft rock intrusions and other anomalies will often cause premature failures on those planes. They should be noted, so that where appropriate, other tests such as triaxial tests can be required.

This test is seldom used for slope stability analysis because for materials where this test is suitable, the stability is often governed by the shear strength of discontinuities. One exception is where the unconfined compressive strength is an important input parameter into rock mass classification systems that are sometimes used to evaluate slope stability directly, or to develop rock mass strength parameters for analysis.

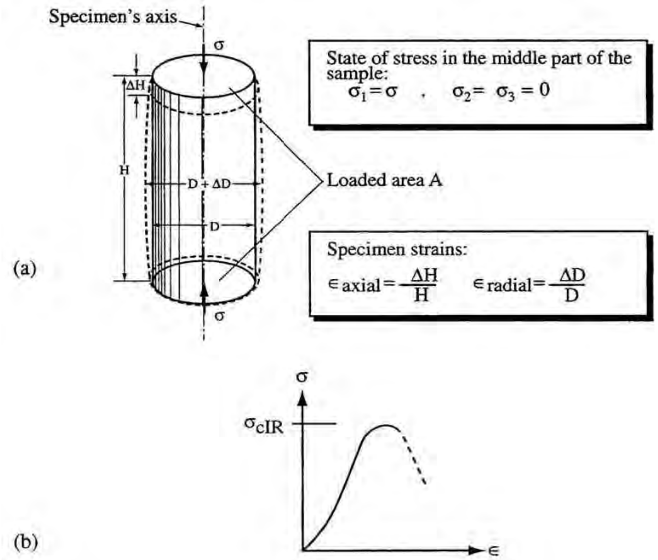


Figure 3-11 (a) Principle of an Uniaxial Compression Test on Rock; (b) Evaluator of Uniaxial Compressive Strength for Intact Rock (subscript IR) with Random Grain Fabric (Wittke,1990)

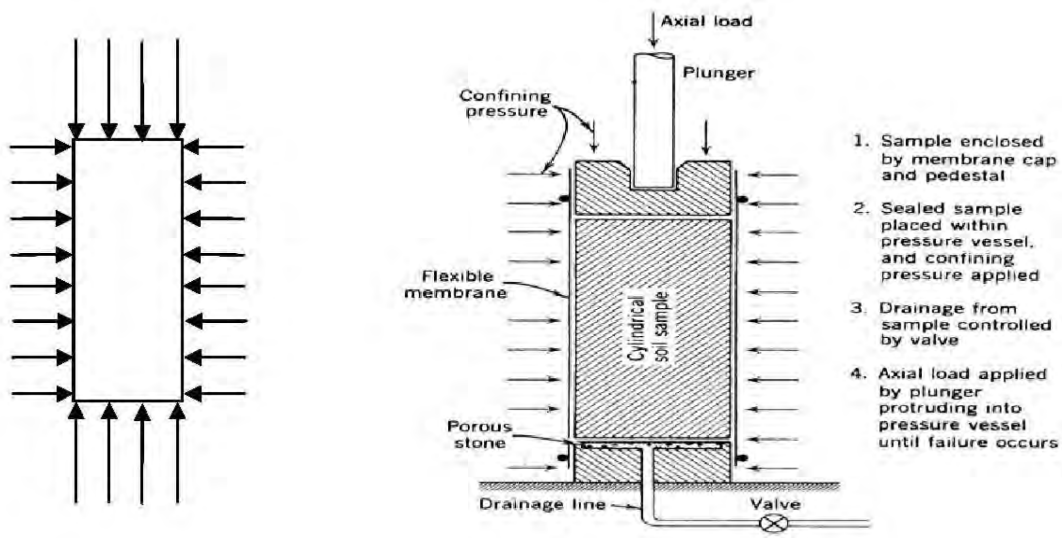
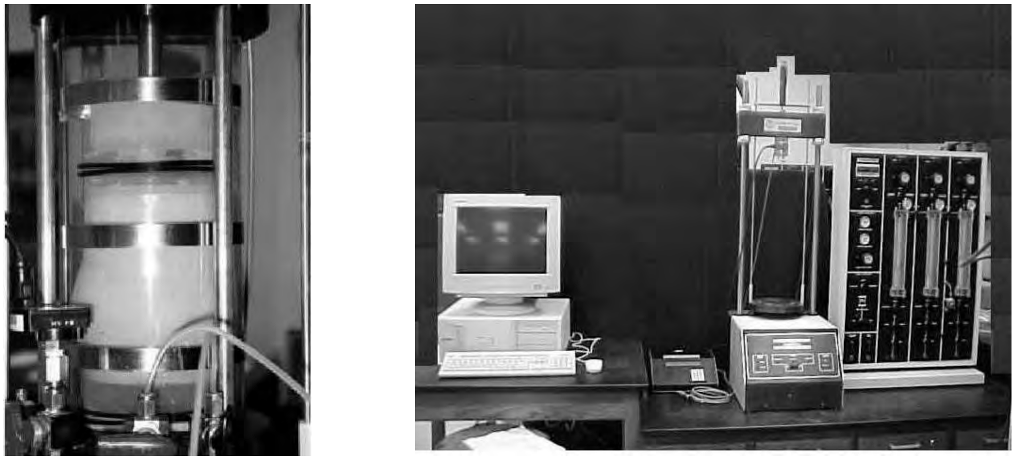


Figure 3-12 Schematic of a Triaxial Compression Test (Lambe and Whitman, 1979)



(a) (b)

Figure 3-13 (a) Failure of a loose sand specimen in a triaxial cell; and (b) Load frame, pressure panel, and computerized data acquisition system (Photographs courtesy of GeoComp Corporation)

3.5.5 Triaxial Tests

The triaxial test is very versatile in the sense that the shear strength can be evaluated under compression as well as extension loading conditions. A schematic of a triaxial compression test is shown in Figure 3-12, where the axial stress is greater than the confining stress. Lateral pressures at various depths below the ground surface can be simulated by confining pressures. Note that the confining pressure acts on the entire sample and is equal to the axial stress before the application of an axial stress increment. Typically, failure of the sample is caused by increasing the axial stress (compression) until a shear failure takes place.

3.5.6 Point Load Tests

The point load test is an appropriate method to estimate the compressive strength in which both core and lump samples can be tested (ISRM, 1981(c)). The equipment is portable, and tests can be carried out quickly and inexpensively in the field (Figure 3-14). Because the point load test provides an index value for the strength, standard practice is to calibrate the results with a limited number of uniaxial compressive tests on prepared core samples.

It has been found, on average, that the uniaxial compressive strength is about 20 to 25 times the point load strength index. However, tests on many different types of rock show that the ratio can vary between 15 and 50, especially for anisotropic rocks. Consequently, the most reliable results are obtained if a series of uniaxial calibration tests are carried out.

Point load test results are not acceptable if the failure plane lies partially along a pre-existing fracture in the rock, or is not coincident with the line between the platens. For tests in weak rock where the platens indent the rock, the test results should be adjusted by measuring the amount of indentation and correcting the distance D.

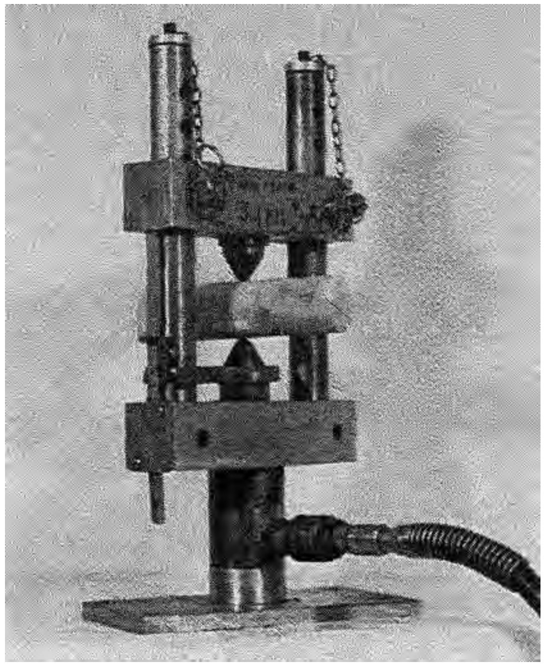


Figure 3-14 Point Load Testing Equipment and Point Load Strength Index

3.6 SHEAR STRENGTH OF SOILS AND ROCKCORRELATIONS OF SHEAR STRENGTH PARAMETERS WITH INDEX PROP ERTIES

This section presents some useful correlations between shear strength parameters and other index values or field conditions. These correlations may be used by the maintenance supervisor to estimate shear strength characteristics of the soils on the project site to perform a preliminary assessment of the criticality of the problem. It must be emphasized that predictions based on correlations alone should never be used for final design and that any assumptions regarding shear strength parameters should always be verified through testing.

3.6.1 Undrained Shear Strength of Cohesive Soils

For most saturated clays tested under quick undrained conditions, the angle of shearing resistance, φ_u, is zero. This means that the shear strength of the clay is a fixed value and is equal to the apparent cohesion, c_u, at a specific moisture content and preconsolidation pressure.

For most normally consolidated clays the apparent cohesion (c_u) also referred to as the undrained shear strength (s_u) is proportional to the effective overburden pressure (σ'_v). For such soils, Skempton (1957) proposed the relationship shown in Figure 3-15 between the c_u/σ'_v and plasticity index (PI). Figure 3-15 also includes results obtained by a number of other researchers. As can be seen in the figure, the composite of all findings varies so much that such relations should be used with caution. However, such correlations, particularly the correlation by Skempton (1957), are useful for obtaining preliminary estimates and for checking laboratory results of project-specific tests performed on normally consolidated clays.

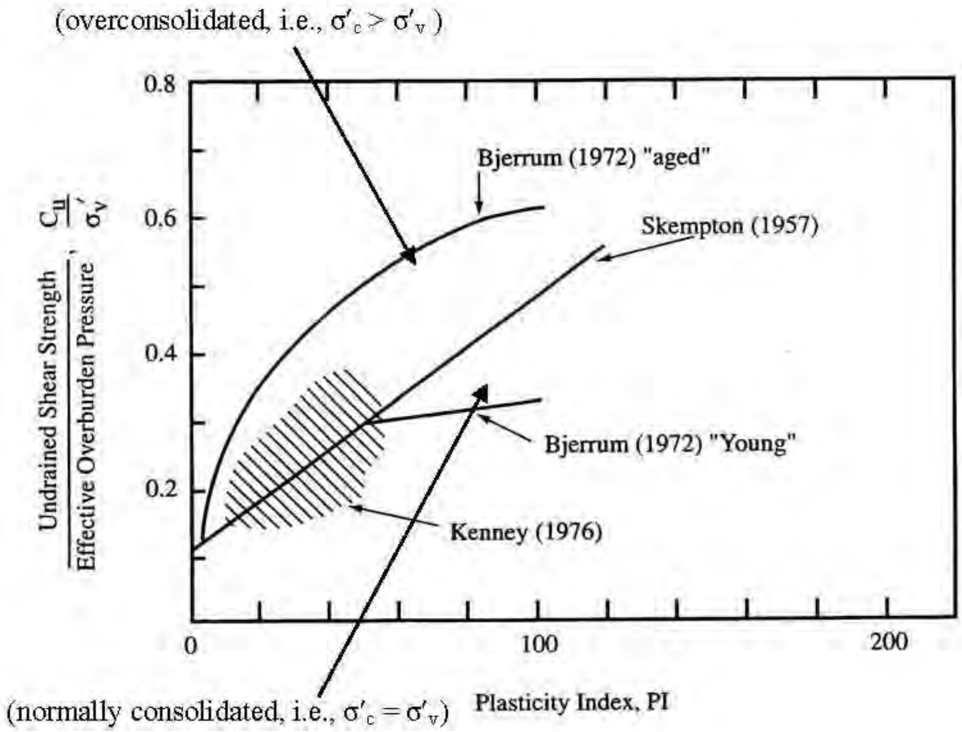


Figure 3-15 Relationship between the ratio of undrained shear strength to effective overburden pressure and plasticity index for normally consolidated and overconsolidated clays (after Holtz and Kovacs, 1981)

3.6.2 Drained and Effective Shear Strength of Cohesive Soils

It is often important to carry out stability calculations in terms of effective stresses. The soil strength parameters, c′ and φ′, used in these calculations should be obtained from either drained direct shear box or drained triaxial tests or from CU triaxial tests with pore water pressure measurements (giving φ'_{cu} and c'_{cu}). Generally, there is a minor difference in the results obtained from these two tests for saturated clays because the soil is being tested under different boundary conditions and stress paths. In-situ tests such as CPTs can also be used to estimate the drained and effective shear strength parameters of cohesive soils.

For clays, empirical correlations have been developed to relate φ′ to the plasticity characteristics of the soil. Figure 3-16 shows a slight trend of φ′ decreasing with increasing PI. The existence of these relationships arises because both PI and shear strength reflect the clay mineral composition of the soil; as the clay mineral content increases, the PI increases and the strength decreases. From Figure 3-16, it can be seen that the drained friction angle value can be ± 8^{o} in variance with respect to the dashed trend line.

Considering the overall importance of φ′ in stability calculations, foundations design, and landslide analyses, it is essential to assess φ′ directly by means of consolidated drained direct shear tests, consolidated drained triaxial tests, or consolidated undrained triaxial tests with pore water pressure measurements. The consequences of merely estimating φ′ can be economically unwise. As an example, in stability analyses for relatively long, shallow slip surfaces that may be associated with a landslide, the required forces that would need to be resisted by some form of stabilization system (e.g., retaining wall) would vary significantly depending on the drained friction angle of the soil. It is highly recommended that geotechnical designers develop historical data summaries of φ′ versus PI to check the validity of future test results.

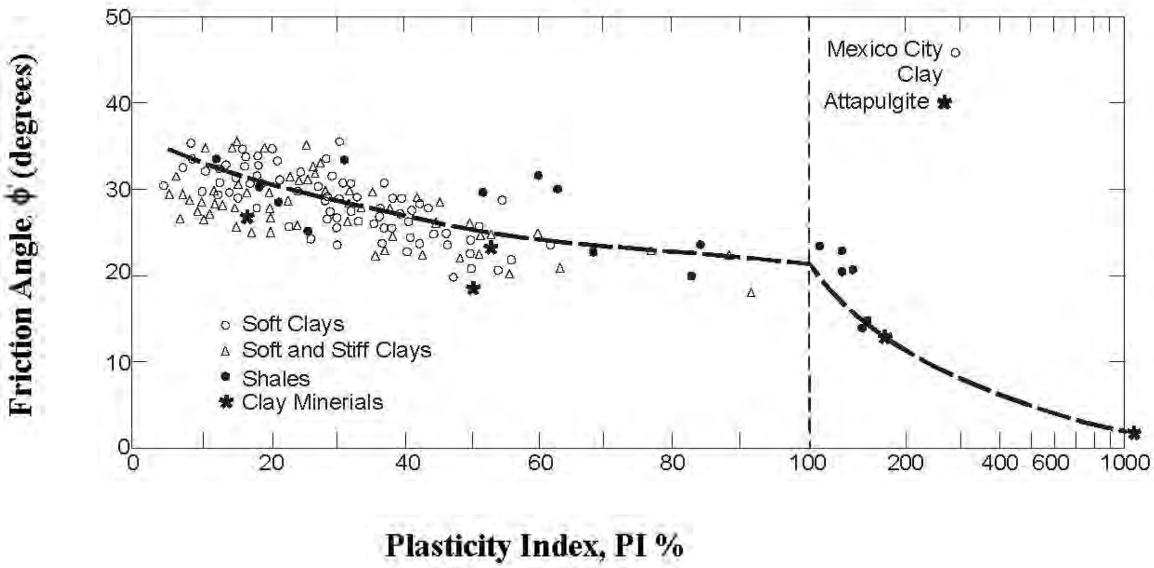


Figure 3-16 Relationships between φ′ and PI. (after Terzaghi, *et al.,* 1996).

3.6.3 Shear Strength of Cohesionless Soils

Because of their high permeability, pore water pressures do not build up significantly when cohesionless soils are subjected to shearing forces. The complication of total and effective stresses is therefore avoided and the phenomenon of apparent cohesion, or undrained shear strength does not occur. Consequently, the shear strength of cohesionless soils is defined exclusively in terms of frictional resistance between the grains, as measured by the angle of shearing resistance, φ′. Typical values of φ for sands and gravels are given in Figure 3-17 as a function of dry unit weight and relative density. The material types indicated in the figure relate to the Unified Classification System (USCS).

Figure 3-17 requires determination of relative density. A reasonable estimate of relative density can be obtained from Figure 3-18.

3.7 INVESTIGATIONS OF ACTIVE LANDSLIDE (SITE CHARACTERIZATION)

Investigations of stable slopes and stable ground, even those with inactive landslide deposits, may be more methodical than investigation of active slide areas because of time constraints and conditions of urgency. Once a landslide or slope instability has developed (activated), either during construction of a facility or subsequently, an investigation is undertaken to diagnose the factors affecting the movements and to determine what corrective measures are appropriate for preventing or minimizing further movements from occurring.

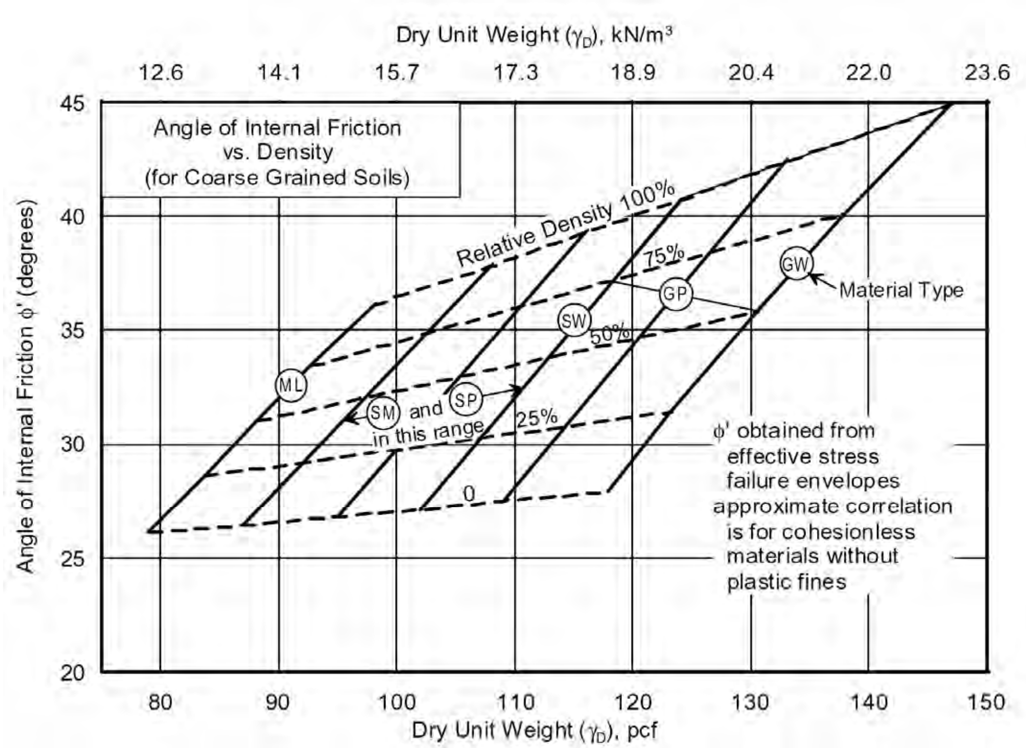
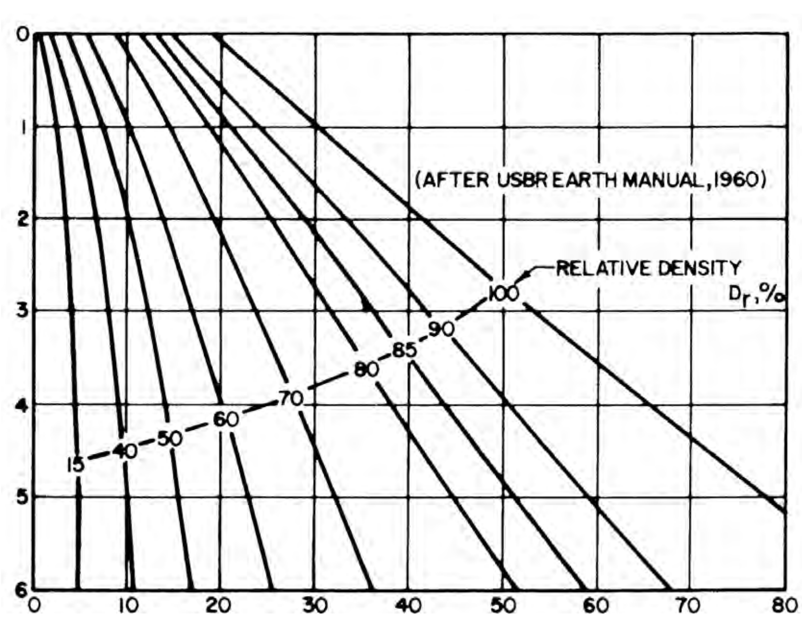


Figure 3-17 Correlation between Relative Density, Material Classification and Angle of Internal Friction for Coarse-Grained Soils (NAVFAC, 1986a)

Vertical effective overburden pressure, p_o, ksf



Energy corrected SPT N-value, N_{60}

1 ksf = 47.9 kPa

Figure 3-18 Correlation between Relative Density and SPT Resistance (NAVFAC, 1986a)

Basically, the information needed by the maintenance department and geotechnical specialist investigating an active landslide generally includes:

- Boundaries
- Landslide features and dimensions
- Landslide depths
- Landslide rate and direction of movement
- Probable cause(s) of movement
- Springs and seeps noticeable
- Groundwater level

3.7.1 Landslide Features and Dimensions

Landslide features and dimensions are the most important observations that must be made during the initial site reconnaissance and desk study before detailed analysis can proceed. Landslide features are best illustrated by the earth slide-earth flow diagram developed by Varnes, 1978 (Figure 3-19). The International Association of Engineering Geology (IAEG) Commission on Landslide and Other Mass Movement further defined each of the features and the dimension of a landslide schematically in 1990 Figure 3-20(a) and Figure 3-20(b), respectively). Definitions for the landslide features and dimensions are provided Table 3-2 and Table 3-2 respectively. Readers are referred to IAEG Bulletin "Suggested Nomenclature for Landslides" as listed in the references for detailed discussion.

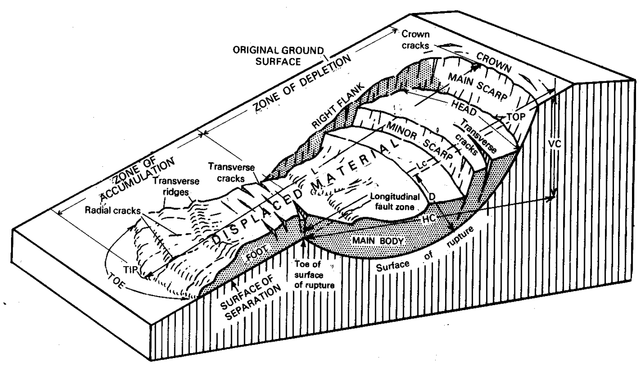
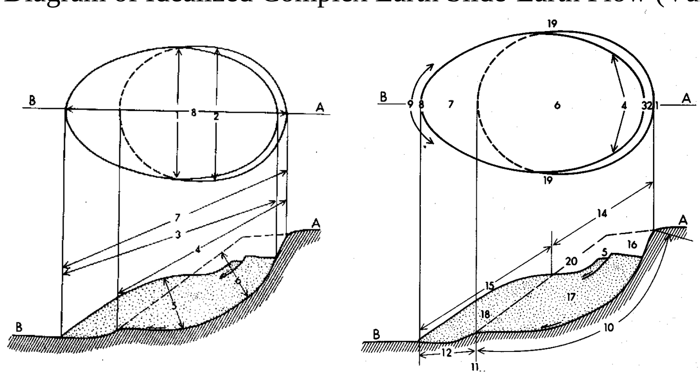


Figure 3-19 Block Diagram of Idealized Complex Earth Slide-Earth Flow (Varnes, 1978)



(a) (b)

Note: The numerical annotations for 3-16a and 16b are listed in Table 3-2 and Table 3-3, respectively.

Figure 3-20 Landslide Features and Dimensions (IAEG, 1990)

TABLE 3-2 DEFINITIONS OF LANDSLIDE FEATURES
(IAEG COMMISSION ON LANDSLIDE, 1990) (REFER TO FIGURE 3-19)

No.	Feature Name	Definition
1	Crown	The partially undisturbed material adjacent to the highest parts of the main scarp.
2	Main scarp	A steep surface on the undisturbed ground at the upper edge of the landslide caused by movement of the displaced material (13) away from the undisturbed ground. It is the visible part of the surface of rupture (10).
3	Top	The highest point of contact between the displaced material (13) and the main scarp (2).
4	Head	The upper parts of the landslide along the contact between the displaced material and the main scarp (2).
5	Minor scarp	A steep surface on the displaced material of the landslide produced by differential movements within the displaced material.
6	Main body	The part of the displaced material of the landslide that overlies the surface of rupture between the main scarp (2) and the toe of the surface of rupture (11).
7	Foot	The portion of the landslide that has moved beyond the toe of the surface of rupture (11) and overlies the original ground surface (20).
8	Tip	The point on the toe (9) farthest from the top (3) of the landslide.
9	Toe	The lower usually curved margin of the displaced material of a landslide; it is the most distant from the main scarp (2).
10	Surface of rupture	The surface that forms (or that has formed) the lower boundary of the displaced material (13) below the original ground surface (20).
11	Toe of surface of rupture	The intersection (usually buried) between the lower part of the surface of rupture (10) of a landslide and the original ground surface (20).
12	Surface of separation	The part of the original ground surface (20) now overlain by the foot (7) of the landslide.
13	Displaced material	Material displaced from its original position on the slope by movement in the landslide. It forms the depleted mass (17) and the accumulation (18).
14	Zone of depletion	The area of the landslide within which the displaced material (13) lies below the original ground surface (20).
15	Zone of accumulation	The area of the landslide within which the displaced material lies above the original ground surface (20).
16	Depletion	The volume bounded by the main scarp (2), the depleted mass (17) and the original ground surface (20).
17	Depleted mass	The volume of the displaced material that overlies the rupture surface (10) but underlies the original ground surface (20).
18	Accumulation	The volume of the displaced material (13) that lies above the original ground surface (20).
19	Flank	The undisclosed material adjacent to the sides of the rupture surface. Compass directions are preferable in describing the flanks, but if left and right are used, they refer to the flanks as viewed from the crown.
20	Original ground surface	The surface of the slope that existed before the landslide took place.

TABLE 3-3 DEFINITIONS OF LANDSLIDE DIMENSIONS (IAEG COMMISSION ON LAND SLIDE, 1990)(REFER TO FIGURE 3-20)

No.	Dimension Name	Definition
1	Width of the displaced mass, W_d	Maximum breadth of the displaced mass perpendicular to the length, L_d.
2	Width of the rupture surface, W_r,	Maximum width between the flanks of the landslide, perpendicular to the length, L_r.
3	Length of displaced mass, L_d	Minimum distance from tip to the top.
4	Length of the rupture surface, L_r	Minimum distance from the toe of the surface of rupture to the crown.
5	Depth of the displaced mass, D_d	Maximum depth of the displaced mass, measured perpendicular to the plane containing W_d and L_d.
6	Depth of the rupture surface, D_r	Maximum depth of the rupture surface below the original ground surface measured perpendicular to the plane containing W_r and L_r.
7	Total length, L	Minimum distance from the tip of the landslide to its crown.
8	Length of center line, L_{cl}	Distance from crown to tip of landslide through points on original ground surface equidistant from lateral margins of surface of rupture and displaced material.

3.7.2 Landslide Investigations

In order to gather the above information, a landslide investigation usually consists of several of the following components:

- Site reconnaissance
- Geological mapping
- Aerial photo study
- Subsurface Exploration
- Geotechnical instrumentation

Initial Site Reconnaissance and Geological Mapping

Initial site reconnaissance and geological mapping should be performed as soon as possible after the landslide or instability occurs. Prior to requesting the involvement of engineering geologist or a geotechnical engineer, the maintenance supervisor should perform a preliminary site reconnaissance. In most cases, landslides or slope instability do not occur without some advance warning. Therefore, if a slide is discovered in the early stages, steps by the maintenance department might be taken to prevent further movement which in turn will limit the amount of sliding mass involved and the cost of remediation. Once a landslide is recognized, the general fundamental features, dimension, volume, and immediate damage of the landslide should be documented on a ***landslide report*** during the initial site reconnaissance by the maintenance department and before more detailed examination is undertaken. Several landslide report/inventory forms are available. Cruden et al. (1996) proposed a simple single-page form containing only basic information such as location, geometry, volume and estimated damages (Figure 3-21). This form may be used or the maintenance department may develop their own form. In any case, in order to maximize the collection of information during the initial reconnaissance, the following data should be collected and documented in a report:

- Date of observation
- Location of the observed landslide
- Observable features
- Geometry
- Type, modes and rate of slope movements
- Estimate position of ground water table and piezometric surface from observable springs and seeps
- Potential difficult accessing, drilling and/or excavation conditions
- Potential locations of subsurface explorations
- Possible causes of the movement
- Photographs and sketch

LANDSLIDE REPORT

Inventory Number: ________

Date of Report: ____________________ (day / month / year)

Date of Landslide Occurrence: ____________________ (day / month / year)

Landslide Locality: ____________________

Reporter's Name: ____________________

Affiliation: ____________________

Address: ____________________

Phone: ____________________

		Degrees	Minutes	Seconds
Position:	Latitude			
	Longitude			
	Elevation:	crown		m a.s.l.
	Surface of rupture	toe		m a.s.l.
		tip		m a.s.l.
Geometry:		Surface of rupture	Displaced Mass	
	Length	L_r =	L_d=	L =
	Width	W_r =	W_d =	
	Depth	D_r =	D_d =	
Volume:	$V= \pi L_d D_d W_d/6$	or V =	Swell factor =	
	V =	$m^3 \times 10^n$	n =	
Damage:	Value	________		
	Injuries		Deaths	

Figure 3-21 Proposed Standard Landslide Report Form

Ground Survey

Ground survey is essential to determine the geometry of the active landslide, extent of landslide activity and the rate of movement. Cross sections should be developed. As a rule of thumb, the area of investigation should be two to three times wider and longer than the observable slide area. It may also be necessary to investigate the top of the slope to a change in slope angle.

Boring and other direct investigative techniques for ***subsurface investigation*** for stable slopes can be tailored for landslide investigation. It is preferable that the investigation be performed after the site reconnaissance and ground survey. Cross sections should be developed in advance based on the surface observations through the center of the landslide depicting possible toe bulges and uphill scarps. Circular, translational or elliptical failure surfaces sketched through these limits can suggest the maximum depth of movement.

The depth of a landslide investigation, however, is difficult to plan in advance, and must be revised as the investigation proceeds to identify the surface of rupture (failure), and the materials underlying the rupture surface that have not been disturbed by the landslide. The specification should be flexible enough to allow additional depth of investigation when the data obtained suggest deeper movements. The objective of penetrating the soil underlying the rupture surface is to identify if the underlying materials may remain stable or may also be subjected to potential future landslide activity. However, at least one boring should extend far below the suspected depth of shear: sometimes deep, slow movements are masked by the greater activity at shallower depths.

Experience demonstrates that the depth of movement below the ground surface at the center of a landslide is seldom greater than the width of the zone of surface motion. McGuffey (1991) suggests that the maximum depth of the rupture surface is often approximately equal to the distance from the break in the original ground surface slope to the most uphill crack or scarp. Subsurface exploration of landslides is frequently performed under less-than-idea environmental or working conditions. These constraints and details of the exploration program should be provided and considered in the overall geotechnical evaluation.

CHAPTER 4
IDENTIFYING FAILURE PRONE CONDITIONS

4.1 INTRODUCTION

As mentioned in Chapter 1, all slopes are to some degree unstable and need maintenance to remain as designed. For some slopes this maintenance is minimal and for others it is more significant. This chapter will help you identify slopes with more significant needs. To effectively manage the maintenance of soil and rock slopes and embankments for our highways it is important to understand the regional geology and climate and their affect on the performance of our slopes and embankments. It is also extremely helpful to understand situations that are prone to failure and be able to recognize tell tale signs of movement. This chapter will focus on these topics.

4.2 GEOLOGIC CONDITIONS

The geology of a region has a paramount effect on the performance of soil and rock slopes and embankments. Local geological details, such as (1) geometry of the subsurface, (2) soil and rock characteristics (i.e., shear strength of soil, strike and dip of bedding planes in rock), and (3) groundwater (which are sometimes called as the "3-rule" of slope stability), considerably influence the performance of individual slopes. Thus, the performance of slopes and embankments is an interdisciplinary endeavor requiring concepts and knowledge from geology and soil/rock mechanics. Awareness of geology is necessary for appropriate idealization of ground conditions and the subsequent development of realistic stabilization measures.

4.2.1 Types and Characteristics of Geologic Soil Deposits

Soils are sediments either transported to their present place by water, glacier, and air, or are formed in place from local bedrock (residual soil). The different transporting agents have different sedimentation characteristics and affect the properties of their soils in different ways. Soils must be recognized by the means of their transportation and by the manner of their deposition. Different types of soils include (1) alluvial deposits (by water), (2) glacial deposits (by glaciers), (3) eolian deposits (by wind), (4) alteration (residual) deposits, (5) colluvial/talus deposits (by gravity), and (6) marine deposits. Each of these soil types has unique engineering characteristics that will affect the performance of a slope or embankment.

<u>Alluvial Deposits</u>

Alluvial deposits are transported by running water and deposited when the velocity of the water flow was no longer sufficient to carry them. Deposits formed in river valleys are fluvial, and those in lakes are lacustrine. The deposits generally are of relatively narrow particle size range, regardless of whether they consist of cobbles and gravels from rushing rivers and creeks, sands from moderately moving rivers, clays from sluggish rivers, or from precipitation water moving in sheets down the sides of gentle slopes. Since river flow and location vary considerably over time, multiple zones of varying grain size are frequently present within one deposit. These soils do not exhibit distinct horizontal strata and usually are unconsolidated, unless subject to removal of overburden. These characteristics provide a favorable situation for landslides and slope instability.

Glacial Deposits

Glacial deposits were transported by glaciers, whose action resembles a giant bulldozer. Materials are both pushed forward and dropped on the side and crushed when overridden by the glacier. Warm temperatures cause general thawing of the glaciers, which stops their forward movement and permits settling out or further movement of suspended rock particles by flowing water. Glacial deposits may vary in grain size composition from boulders to clays. Glaciers often produce a disordered landscape, with inhibited drainage and the development of bogs. These soils are commonly found in the midwest, pacific northwest, northeastern and eastern United States, and Alaska.

There are four types of glacial deposits, (1) glacial drift, (2) till, (3) glaciofluvial deposits, and (4) glacio-lacustrine deposits. Glacial drift is rock debris that has been transported and deposited by glaciers. It is generally used to describe undifferentiated glacier deposits and does not imply any specific mode of transport or depositional environment.

Till is unsorted, unstratified, and unconsolidated heterogeneous material deposited directly from the ice (ice has no sorting power) and generally consists of clay, silt, sand-gravel, and boulders intermingled in varying proportions. Till is usually dense to very dense and is of high strength and low compressibility except, perhaps, where till has high plasticity, being derived from shale bedrock. Standard Penetration Test (SPT) values tend to be high because of boulders and gravels. High SPT values are not a reliable indicator of density in till. Typically slopes and embankments founded on glacial till do not experience deep seated (within the till foundation soil) instability.

Glaciofluvial deposits are materials moved by glaciers and subsequently sorted and deposited by streams flowing from the melted ice. They are of two distinct types, advance outwash and the recessional outwash, each of which has potentially significant differences in engineering properties (unconsolidated versus overconsolidated). The deposits usually are stratified and may occur in the form of outwash plains, deltas, and terraces, which tend to vary from loose to medium dense.

Eolian Deposits

Eolian deposits are transported by the wind. They range from sand dunes to loess deposits of particles that are predominantly silt-sized with a certain amount of fine sand and aggregated clay particles. Dune sand deposits are recognizable as low, elongated or crescent-shaped hills, with a flat slope windward and steep slope leeward of the prevailing winds. Usually, these deposits have very little vegetation cover. The material is very rich in quartz and typically consists of a limited grain size range, usually fine- or medium-grained sand, no cohesive strength, moderately high permeability, and moderate compressibility. In the United States, dunes are common to Nebraska, Kansas, Iowa, Mississippi, Indiana, and Idaho.

Loess deposits include extensive areas in the temperate plains regions. Loess consists mainly of angular particles of silt and/or fine sand and sometimes particles of clayey material. Although they have low density, naturally dry loessial soils have a fairly high strength because of the clay binder. However, they are easily eroded when close to the groundwater table. When the soil is flooded or rained on, the soil structure can collapse and large settlements may occur. It is important to protect the foot of a cut against saturation during heavy rainstorms. Otherwise, landslides may be triggered by erosion at the toe of the slope. Cuts in loess are often set back from the roadway more than usual to prevent damage or interruption of traffic because of local instabilities.

Seepage below the groundwater table can cause slope failures in loess (for example, bank erosion) when the binding material is removed. The binding material, which is frequently calcium carbonate, can be dissolved within a few weeks or months. When this occurs, the loess assumes the character of a supersaturated rock flour, which flows like molasses (Terzaghi, 1950). Consequently, subsurface piping occurred within the soil mass that eventually lead to ultimate collapse of overburden soils (Figure 4-1).

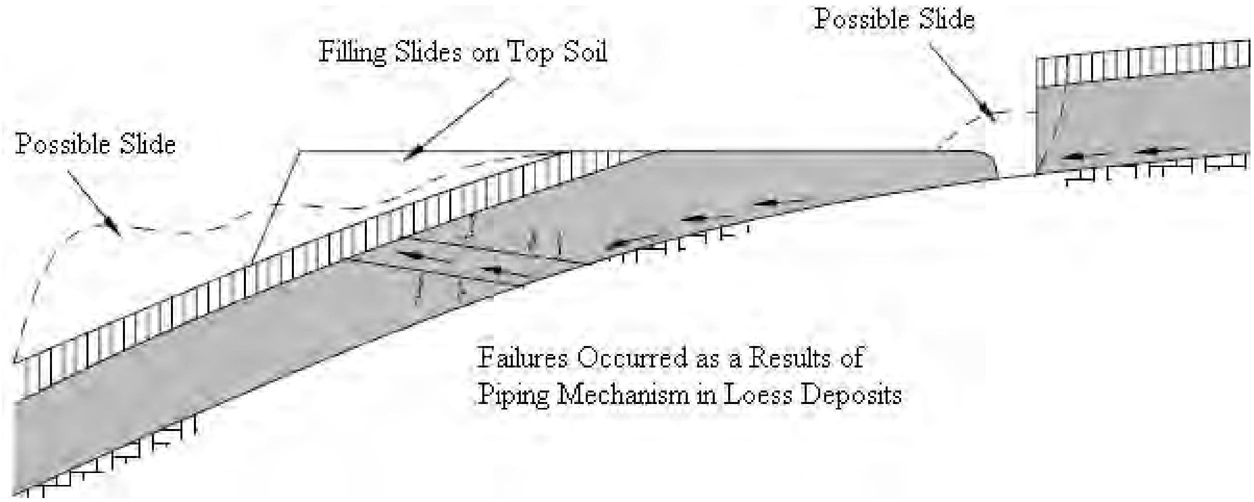


Figure 4-1 Schematic Diagram Showing Landslides as a Result of Piping Mechanism in Loess Deposits (After Evans, 1977)

Residual Deposits

Residual deposits are formed in place by mechanical and chemical weathering of the parent bedrock. Figure 4-2 depicts a typical form of the deep tropical weathering profile (Little, 1969). These soils are found over much of the eastern part of the United States, east of the Appalachian Mountains, in the southeastern United States, and tropical areas, such as Hawaii and Guam.

Laterites and saprolites are two types of residual soils. The laterites typically are bright red to reddish brown soils, which are formed initially by weathering of igneous rocks with the subsequent leaching and chemical erosion from high temperature and rainfall. The colloidal silica is leached downward, leaving behind aluminum and iron which become highly oxidized (hence the red color) and are relatively insoluble in the high pH environment (greater than 7) that is typical when laterites weather. Well-developed alteration soils are generally porous and relatively incompressible. Saprolites are zones consisting of completely weathered or highly weathered bedrock that contain soil-like materials but retain the original relic rock structure.

The stability of slopes in residual soils is very difficult to predict on the basis of field or laboratory tests. The engineering properties of such soils can appreciably differ from those of sedimentary soils with the same composition and grain size distribution. Their properties can vary considerably laterally and with depth because of differential weathering patterns. A residual soil deposit will frequently contain unweathered boulders (corestones) and unweathered rock layers.

Landslides and slope instability are common in deep residual soils, particularly during periods of intense rainfall. The rainfall-induced landslides are usually shallow.

Figure 4-3 shows the typical landslides in residual soil and underlying weathered rock.

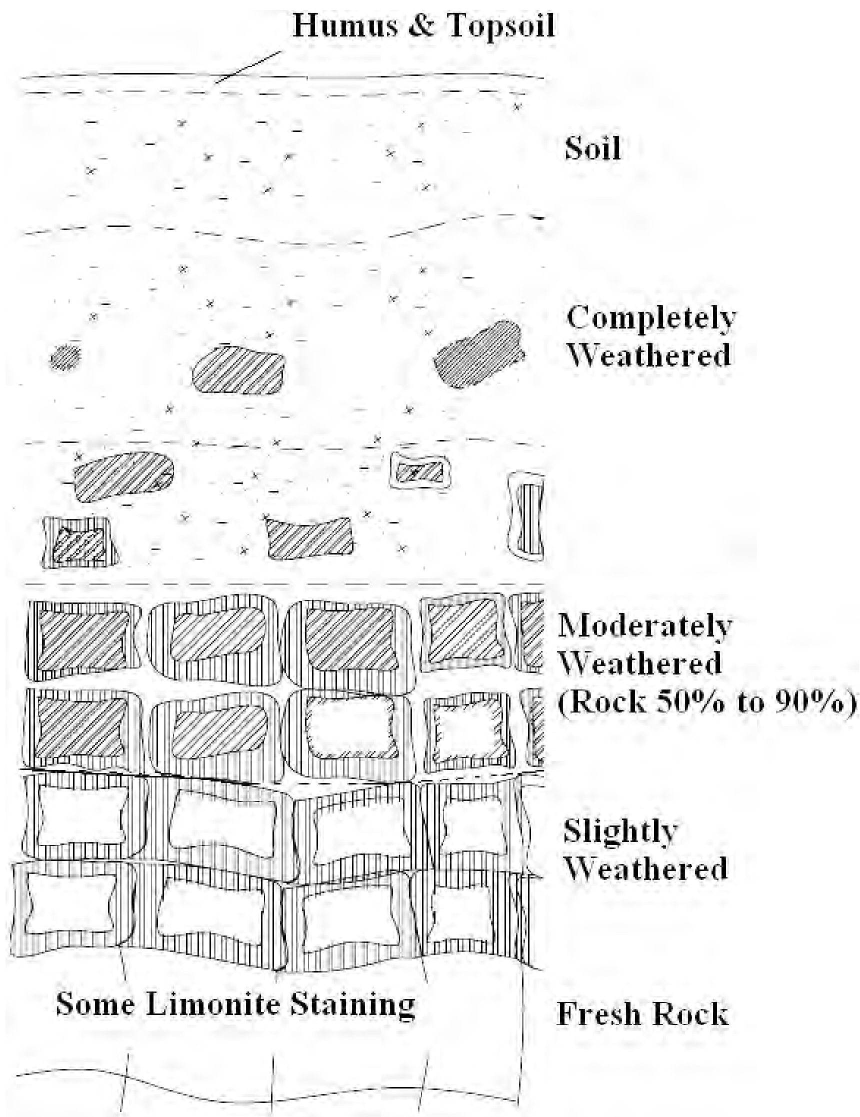


Figure 4-2 Schematic Diagram of Typical Residual Soil Profile (Little, 1969)

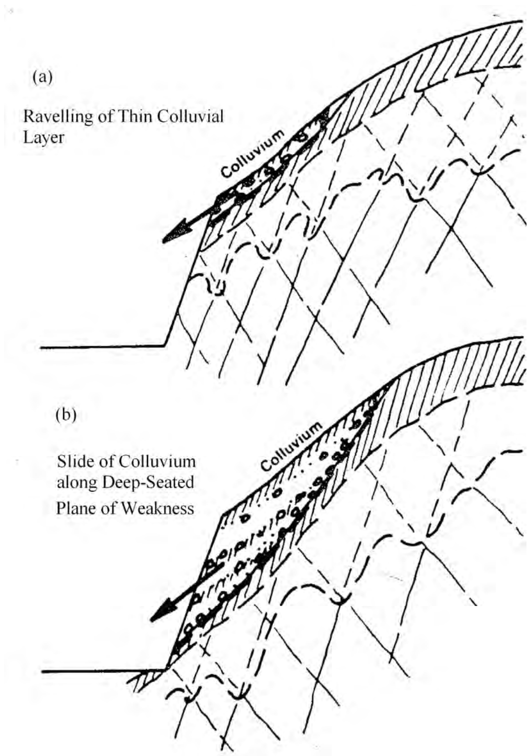


Figure 4-3 Common Types of Landslides in Residual Soil and Weathered Rock. (After Deere and Patton, 1971)

Colluvial/Talus Deposits

Colluvial/talus deposits are soil deposits that have been moved down slope by gravity on steep slopes (Figure 4-4). A typical example is the residual soils (formed by weathering of parent soil and rock materials) that have traveled down slope. These materials are easily identified in air photos as bare slopes in mountainous areas, but they are not obvious on vegetated lower slopes. Colluvial/talus deposits often are loose and unconsolidated. An unstable condition can exist when colluvial/talus deposits rest on slopes and further slope movements are likely. Slope movements, before total failures, range from the barely perceptible creep movements to the more discernible movements of several cm per week (Hunt, 1984) However, colluvium in the ravines of appalachia is typically not loose and may consist of clays, silts, sands, gravels, cobbles and boulders.

The natural causes of these movements are weathering, rainfall, snow and ice melt, earthquake-induced vibrations, and changing water levels as a result of floods or tides. Cuts made in colluvial or talus slopes are expected to become less stable with time and, unless retained or removed, usually lead to failure.

Marine Deposits

Marine deposits originate from two general sources: (1) terrestrial sediments from rivers, glaciers, wind action, and slope failures along the shoreline, and (2) deposits of organic and inorganic remains of dead marine life, and by precipitation from over-saturated solutions.

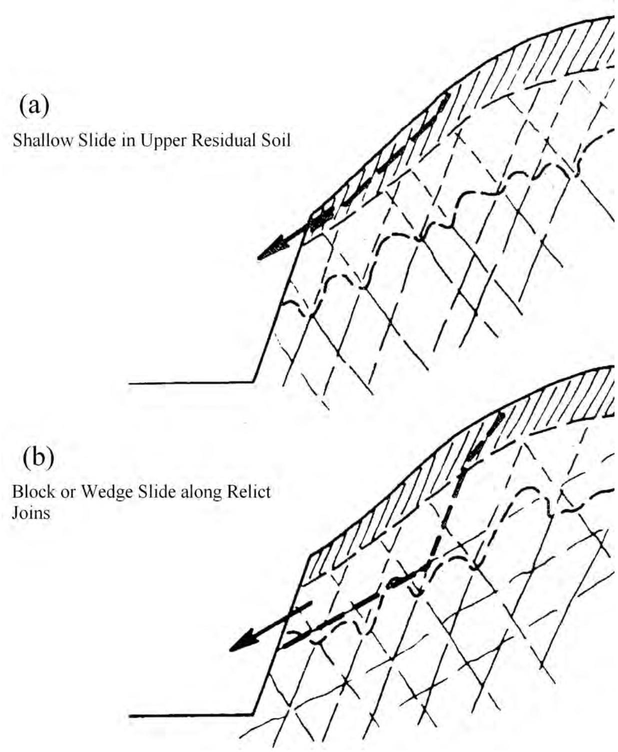


Figure 4-4 Common Types of Landslides in Colluvial Soil (After Deere and Patton, 1971)

Marine deposits consist of sands, silts, and clays. Marine sands normally are composed of quartz grains, which are very hard even though the deposit may be compressible because of the loose arrangement of the quartz particles. Marine clays usually are normally consolidated and soft.

Marine clays from glacial runoff deposited in marine estuaries along coastlines are called glacial-marine clays. As a result of leaching, they are often of high sensitivity. Sensitivity tends to increase with time as groundwater continues to leach the salt, which slowly weakens the deposit until it can no longer retain its natural slope and a failure results, often in the form of a flow.

4.2.2 Types and Characteristics of Rock

There are generally three types of rock that are common, igneous, metamorphic and sedimentary. Igneous rocks are rocks that are formed by cooling molten lava or magma. Metamorphic rock are formed within the earth when previously existing rocks are subjected to temperatures and pressures different from those under which they were first formed. Sedimentary rock, unlike igneous rocks and metamorphic rocks were formed near the earth's surface by deposition and lithification of particles erodes from previously existing soil and rock or dissolved ions.

The stability of rock cuts for highways is a function the structural geology (i.e., the orientation and length, roughness and infill materials of the joints, bedding plans and faults that occur behind the rock face). For most rock cuts, the stresses in the rock are much less that the strength of the intact rock so there is little concern that fracturing of the intact rock will occur. The stability of the cut is, therefore, typically dependent on the stability of the blocks of rock formed by the discontinuities in the rock mass.

Figure 4-5 shows a range of geologic conditions and their influence on slope stability. Slopes a) and b) show typical conditions for sedimentary rock, such as sandstone and limestone containing continuous beds, on which sliding can occur if the dip (i.e., the measure in degrees of how much a planar geologic structure is displaced from the horizontal) of the beds is steeper that the friction angle of the discontinuous surface. In a) the beds daylight on the steep cut face and blocks may slide on the bedding, while in b) the face is cut coincident with the bedding planes and the face is stable. In c) the overall face is also stable because the main discontinuity set dips into the face. However, there is some risk of instability of surficial blocks of rock formed by the conjugate joint set (i.e., joints perpendicular to the main discontinuity joint set) which dips out of the face. In d) the main joint set, also dips into the face but at a steep angle to form a series of thin slabs that can fail by toppling where the center of gravity of the block lies outside the base. Slope e) shows a typical horizontally bedded sandstone/shale sequence in which the shale weathers considerably faster than the sandstone to form a series of overhangs that can fail suddenly along vertical stress relief joints. Slope f) is cut in weak rock containing closely spaced but short, impersistent joints that do not form a continuous sliding surface; a steep cut in this material may fail along a shallow circular surface partially along joints and partially through intact rock.

Landfills

Highway alignments sometimes cross over landfills. These fills may contain organic material, tree limbs, refuse, and a variety of debris that are commonly dumped, pushed, and spread by bulldozers, and then compacted by refuse compactors. Compaction of landfills is somewhat different from the compaction of soils, particularly with respect to crushing. A significant part of landfill compaction crushes (collapsing) hollow particles, such as drums, cartons, pipes, and appliances. The compaction process also brings the crushed particles closer together i.e., decreases the volume of voids. The crushing is the important part of the compaction of landfills.

Relatively little is known about the engineering properties of landfills. The paucity of data results in part from the difficulties in credible sampling and testing of refuse. This difficulty is further compounded by the fact that refuse composition and properties are likely to change erratically within a landfill and are also likely to decompose with time.

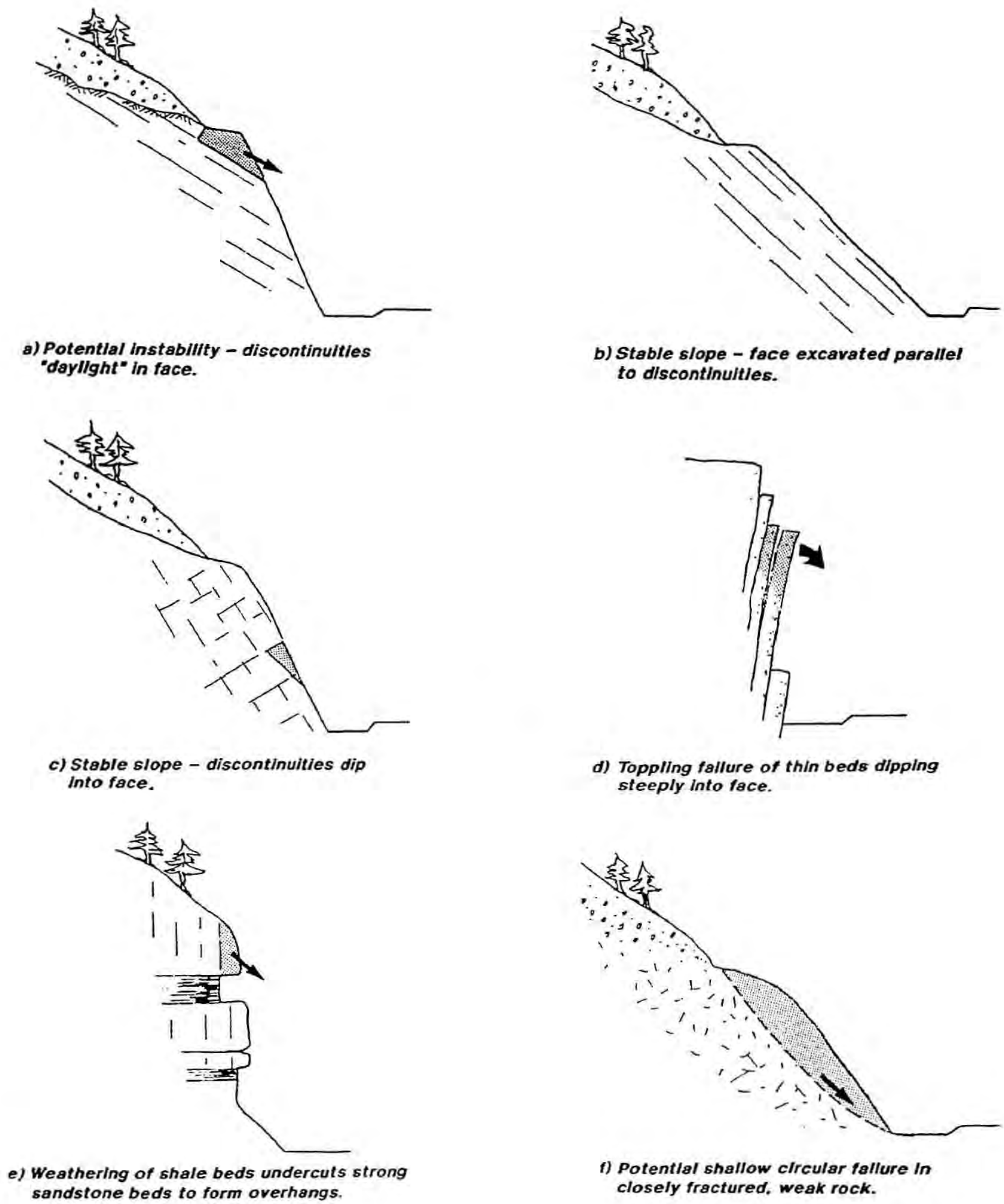


Figure 4-5 Influence of Geological Conditions on Stability of Rock Cuts

4.3 PROCESSES THAT AFFECT SHEAR STRENGTH AND SHEAR STRESS

Stability failures of natural and man-made slopes are often attributed to: (1) ground causes, (2) morphological causes, (3) physical causes, and (4) man-made causes. Ground characteristics can be identified by site reconnaissance and subsurface investigation techniques. Changes in site ***topography*** over time are apparent from review of old surveys, maps, and air photos. Physical causes require the application of instruments such as seismographs, rain gauges, flow gauges, inclinometers and piezometers. Manmade causes can be documented by site records, plans, or other observations. **TABLE 4-1** is a list of conditions that may lead to instability.

4.3.1 Processes that Increase Shear Stress

The following are processes that will increase shear stress:

1. Removal of support - External disturbance in the form of erosion or steepening of a slope or embankment, or of ground adjacent to it, alter the balance between forces tending to cause instability and the forces tending to resist it. Typical natural agents are streams, rivers, waves, currents, internal piping, and slope movements. Man-made landslides can be caused by excavations for cuts, quarries, pits and canals, and the drawdown of lakes and reservoirs. Large landslides have been caused by very small cuts along highways. When cleaning ditches and dressing slopes, care needs to be taken not to oversteepen the slope.

2. Surcharge - External disturbance from filling, stockpiling, dumping waste on top of a slope, or adding water by rain or snow precipitation, by the growth of glaciers; and by the flow of surface and groundwater into the displacing mass. In addition, materials can be added by landslides from above the slope, by volcanic activities, and by the growth of vegetation. When crossing a steep, high, or otherwise potentially unstable slope, snow or road debris should not be dumped over guardrail unless advised otherwise by the Geotechnical Specialist.

3. External disturbance - Seismic activities (earthquakes) and volcanic explosions can alter the local stress field within a slope or embankment. Man-made disturbance includes pile driving and the passing of heavy vehicles.

4. Uplift - Tectonic forces, melting of the ice sheets, or volcanic processes can cause uplift of an area's surface or steepening of slopes or embankments as drainage responds by increased incision. The cuttings of valleys in the uplifted area may cause valley rebound and accompanying fracturing and loosening of valley walls with inward shear along flat-lying discontinuities. The fractures and shears may allow pressure buildup in the loosened mass and eventually lead to landslides.

As can be seen here, removal of support or adding surcharge increases shear stress and can lead to instability. **TABLE 4-1** presents the factors that commonly lead to an increase in shear stresses in natural or man-made slopes and embankments.

TABLE 4-1 CAUSES OF INSTABILITY AND DEFORMATIONS OF SOIL SLOPES AND EMBANKMENTS (AFTER CRUDEN AND VARNES, 1992)

Ground Causes	
A.	Weak materials
B.	Sensitive materials
C.	Weathered materials
D.	Sheared materials
E.	Jointed or fissured material
F.	Adversely oriented mass discontinuity (for example, bedding, schistosity)
G.	Aversely oriented structural discontinuity (fault, unconformably, contact)
H.	Contrast in permeability
I.	Contrast in stiffness (stiff, dense material over plastic materials)
Morphological Causes	
A.	Tectonic or volcanic uplift
B.	Glacial rebound
C.	Fluvial erosion of the slope toe
D.	Wave erosion of the slope toe
E.	Erosion of the lateral margins
F.	Subterranean erosion (solution, piping)
G.	Deposition loading in the slope crest
H.	Vegetation removal (for example, forest fire, drought)
III.	Physical Causes
A.	Intense rainfall
B.	Rapid snow melt
C.	Prolonged exceptional precipitation
D.	Rapid drawdown (of floods and tides)
E.	Earthquake and Soil Liquefaction
F.	Volcanic eruption
G.	Thawing
H.	Freeze and thaw weathering
I.	Shrink and swell weathering
IV.	Man-made Causes
A.	Excavation
B.	Loading of slope crest
C.	Drawdown (of reservoirs)
D.	Deforestation
E.	Irrigation
F.	Mining
G.	Artificial vibration
H.	Water leakage from utilities and septic systems
I.	Poor Selection of Embankment Materials
J.	Poor Compaction Effort

TABLE 4-2 FACTORS THAT MOST COMMONLY CAUSE AN INCREASE IN SHEAR STRESS IN A SLOPE OR EMBANKMENTS

1.	Removal of support
A.	Erosion
1.	By streams and rivers
2.	By glaciers
3.	By action of waves or marine currents
4.	By successive wetting and drying (for example, winds, freezing)
B.	Modification of the initial slope by falls, slides, settlements or other causes.
C.	Human activity
Cuts and excavations	
Removal of retaining walls or sheet piles	
Drawdown of lakes, lagoons, or bodies of water	
2.	Overloading
A.	Natural causes
1.	Weight of rains, snow, etc.
2.	Accumulation of materials because of falls, slides or other causes
B.	By human activity
Construction of fill	
Buildings and other overloads at the crest	
Possible water leakage in pipes and sewers	
3.	Transitory effects, such as earthquakes
4.	Removal of underlying materials that provided support
A.	By rivers or seas
B.	By weathering
C.	By underground erosion due to seepage (piping) solvent agents, etc.
D.	By human activity, (excavation or mining)
By loss of strength of the underlying material	
5.	Increase in lateral pressure
A.	By water in cracks and fissures
B.	By freezing of the water in the cracks
C.	By expansion of clays

Extracted from Highway Research Board, *Landslides and Analysis and Control*, Schuster, et al., Ed., Special Report No. 176, Washington D.C., 1978.

4.3.2 Processes Contributing to Reduced Shear Strength

The following processes will reduce the available shear strength of the soil:

1. Weathering - It is now widely recognized that weathering or other physiochemical reactions may occur at a rate rapid enough to cause concern in the design of slopes and embankments. Therefore, it is important to consider not only the existence of weathering and other physiochemical reactions

that have occurred but also the possibility of continued and even accelerated weathering and physiochemical reactions.

Weathering of soils tends to weaken ionic bonds and reduce shear strength. Weathering can be accelerated by slope disturbance caused by construction or an earthquake, by fresh exposure to the atmosphere and other agents such as stream action. Fissuring of clays may be caused by drying or by release of vertical and lateral restraints from erosion or excavation. The exchange of ions within clay minerals and within the pore water of the clay may lead to substantial changes in the physical properties of some clays. Electrical potentials set up by these chemical reactions or by other processes may attract water from the weathering front (Cruden and Varnes, 1992)

2. Pore water pressure increase - Increase of pore water pressures within a slope can occur because of significant changes in the surrounding areas, such as deforestation, filling of valleys, disturbance of natural drainage characteristics, urbanization, and exceptional rainfall.

3. Progressive decrease in shear strength of slope materials – This may be caused by significant deformation, which does not appear to constitute instability but does lead to it. Such deformation may result from sustained gravitational forces and slope disturbances of intensity not high enough to cause complete failure. Deformations often occur along a slope with major natural discontinuities, ancient slip surfaces, and tectonic shear zones.

4. Progressive change in the stress field within a slope - Every natural geological formation has an initial stress field that may be significantly different from one considered in terms of the weight of the material alone. Lateral stresses may occur that do not bear any predictable relationship with the vertical stress computed from gravitational considerations. The unique initial stress field of any slope depends on its geological background and other natural factors. The stress history of the slope material is of tremendous importance. A change in the initial stress field may occur from causes similar to those that produce a progressive decrease of shear strength. Release of stresses may accompany or follow most forms of slope disturbance. Often, this leads to changes in the magnitude and orientation of the stresses.

Table 4-3 lists the factors that commonly cause a reduction in the shear strength of the materials.

TABLE 4-3 FACTORS THAT REDUCE SHEAR STRENGTH OF NATURAL AND MAN-MADE SLOPES

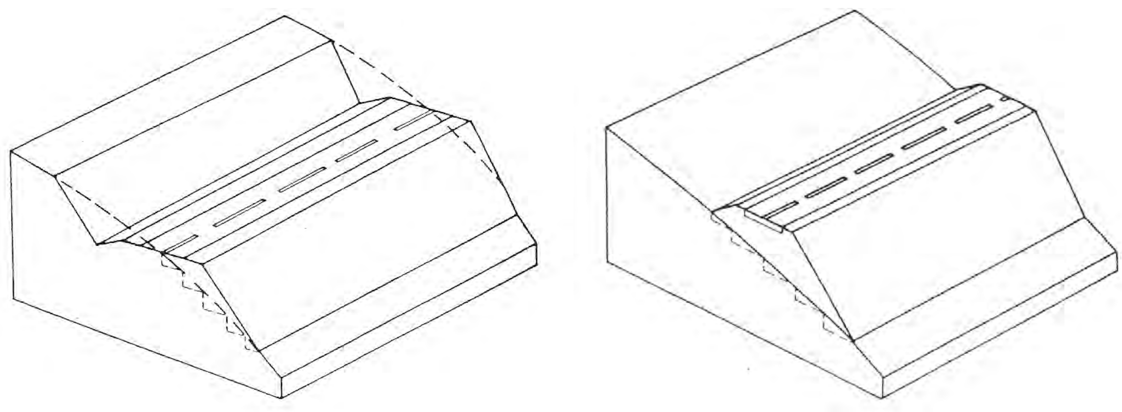
I.	Factors inherent in the nature of the materials	
	A.	Composition
	B.	Structure
	C.	Secondary or inherited structures
Stratification		
II.	Changes caused by weathering and physiochemical activity	
	A.	Wetting and drying processes
	B.	Hydration
Removal of cementing agents		
III	Effect of pore pressures, including those due to seepage	
IV	Changes in structure, including figuration caused by stress release and structural degradation under the acting shear stresses	
Extracted from Highway Research Board, *Landslides and Analysis and Control*, Schuster et al., Ed., Special Report No. 176, Washington D.C., 1978.		

4.4 VULNERABLE LOCATIONS AND CONDITIONS

In general, most materials used in highway construction become less dense and weaken with displacement. This is why it is important to catch and slow early signs of movement/distress. Some sites are vulnerable to slope instability or landslides because of conditions existing at that particular location. Often these conditions are related to a geometry of the location, to prevailing drainage patterns, or to a very localized geologic or soil or rock conditions.

Side-Hill Cut and Fill Sections

Side-hill cut and fill sections are particularly prone to instability. The toe of the cut slope on the uphill side is subject to erosion and loss of toe support (undercutting). The side-hill fill portion of a cut and fill section may be weakened by groundwater saturation. Also, if the interface between the original ground and the fill material is not properly benched and a drainage blanket added, failure of the fill may occur along that plane. Figure 4-6 (a) shows a side-hill cut and fill and Figure 4-6 (b) shows a side-hill fill.



(a) (b)

Figure 4-6 Side-Hill Fill (a) Cut and Fill Side-Hill Fill; (b) Side-hill Fill

4.4.1 Very High Fills

When highway fills for side-hill slopes and or embankments are over approximately 20 feet in height, the embankment or slope may creep or slump (settled) under its own weight. This happens over a very long period of time (10 to 20 years) and at decreasing rates. If rate is observed to increase then sliding is initiating and a landslide may develop. Otherwise, settlement will eventually slow and stop. Usually the sides of the embankment develop a noticeable bulge and the surface of the roadway may have a slight "dip" (Figure 4-7).

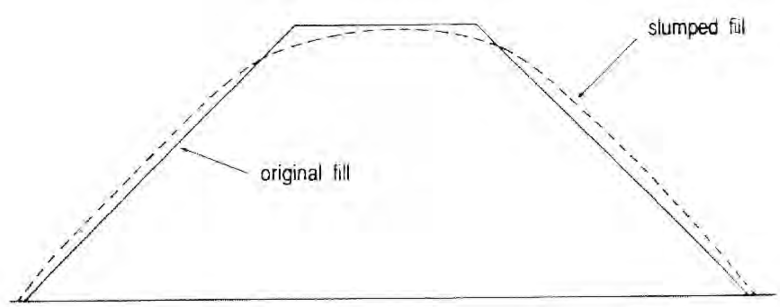


Figure 4-7 Settlement of Very High Embankment

4.4.2 Vertical or Near Vertical Rock Faces

Near vertical to vertical rock faces are always a hazard because of the proximity of the rock face to the roadway and the potential for falling rocks. Ditches may become clogged and debris will often fall onto the driving surface. Many rock falls are due to weathering, either from freeze thaw or from differences in the rate of weathering between soft soil and rock layers and competent rock layers (Figure 4-8).

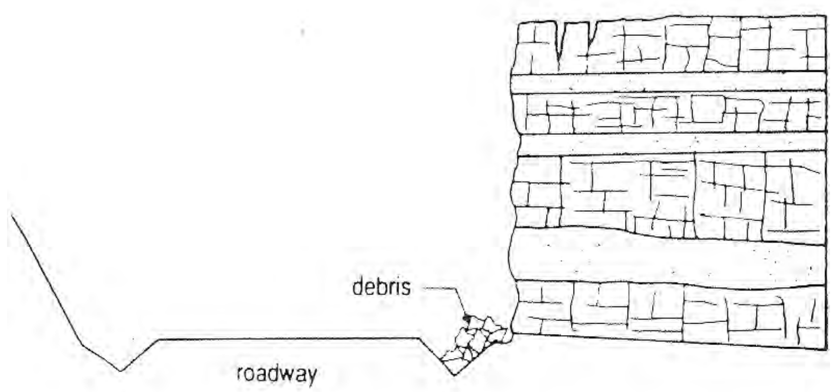


Figure 4-8 Near Vertical Rock Cut

4.4.3 Poorly Drained Locations

Drainage is one of the most important factors involving slope stability and landslides. Subsurface water may saturate and weaken the soils of the embankment or slope, and the natural soils (Figure 4-9). The result is often a slope failure. Chapter 5 addresses the best maintenance practices to prevent slope instability caused by poor drainage. Surface water, it not properly drained away from the earth structure, also may saturate the soil or infiltrate rock structures.

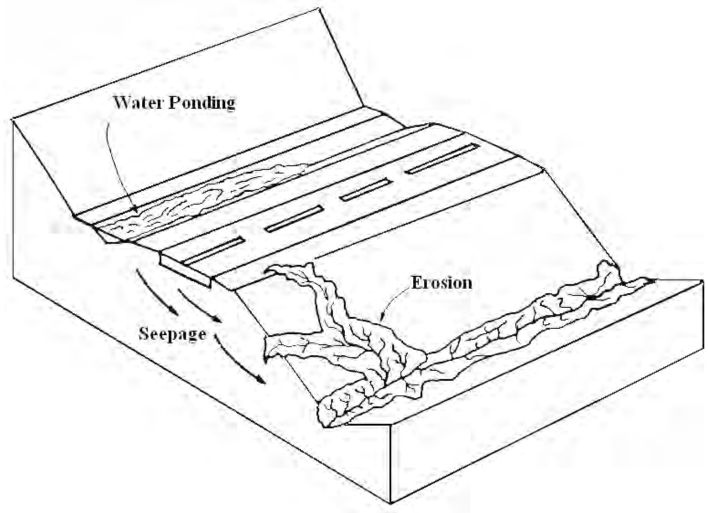


Figure 4-9 Example of Poorly Drained Highway Location

4.4.4 Naturally Occurring Springs

Springs located at the toe of slopes or embankments may soften the soil, causing it to lose strength and allowing the embankment or slope to fail. If springs occur at the toe of a cut slope, on the uphill side of an embankment, the side-hill embankment may become saturate and eventually cause the embankment to fail (Figure 4-10).

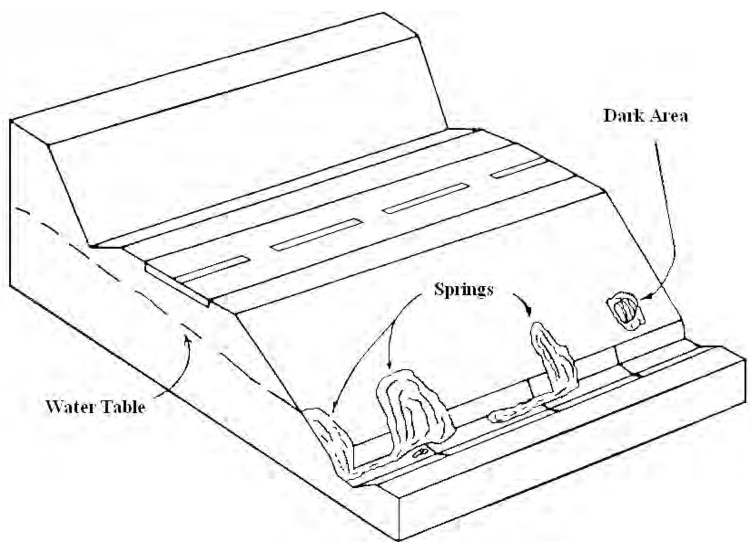


Figure 4-10 Naturally Occurring Springs

4.5 SIGNS OF MOVEMENT

Most slope failures do not occur without some advanced warning. Maintenance personnel should be trained to look for these signs. If a slide is discovered in the early stages, steps often may be taken to prevent further movement, prevent major failure, and save the cost of extensive repair.

One of the early tasks of the maintenance supervisor is to determine the location of the failure or slip plane so as to assess the extent (horizontally and vertically) of the unstable mass. Attempts to identify the source of the problem and mode of failure are important since the corrective action must be related to eliminating the source of trouble.

<u>Tension Cracks</u>

Soil is very weak in tension and it only takes a small amount of movement at the top of a slope before the soil “breaks and a crack is formed. Tension cracks in the roadway indicate movement has started. These cracks permit water to enter and further soften material along the failure plane as well as add additional water pressure to the moving mass. Tension cracks above the roadway indicate the slope is in the early stages of movement (Figure 4-11).

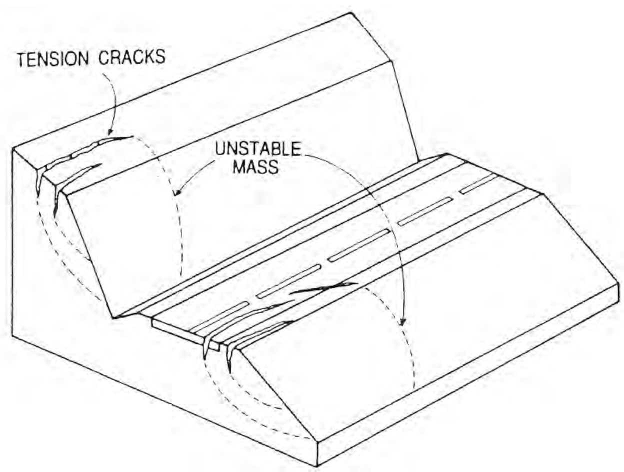


Figure 4-11 Tension Cracks

4.5.1 Sunken Guardrail

Guardrails are installed to match the grade of the roadway. If there is an obvious dip in the guardrail, but none is observed in the roadway, this probably indicates that shallow movement is occurring within the slope or embankment and involves only the shoulder but not the driving lanes (Figure 4-12). However, if there also is an obvious dip in the roadway, this would indicate a major portion of the embankment is involved in the movement. Dips in the guardrail at bridge approaches indicate that the approach embankment and/or foundation have settled or that the embankment is creeping.

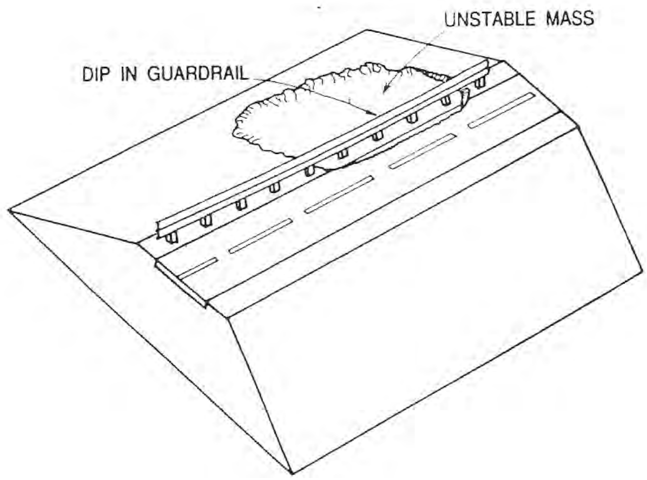


Figure 4-12 Dip in Guardrail

4.5.2 Dips in Grade

For long and high embankments, dips in the grade usually involve all driving lanes. This type of movement may be associated with slumping or creeping of the embankment under its own weight. Dips in grade may also be associated with culverts located under large fills. In many cases, these dips may be attributed to settlement of the backfill around the culvert and are not related to slump or creep.

<u>Debris in Roadway</u>

Debris of soil or rock on the roadway may indicate an unstable slope above the roadway. This debris could be the forerunner of a massive rockfall or slide. A continuing problem at a particular site would require maintenance personnel to take measures to protect the motoring public.

<u>Bulges in Slope Face</u>

Most slides in soil masses will have a bulge at the toe of the slide where the sliding mass has accumulated. This bulge indicates considerable movement already has occurred and that movement will probably continue until complete failure occurs (Figure 4-13).

<u>Poor Drainage</u>

<u>Surface Water</u>

- Blocked Culverts – A culvert that does not permit water to flow properly may cause water to pond next to the toe of a slope or embankment. This condition will facilitate saturation of the toe of the slope, cause the soil to lose strength and hinder the ability of the soil at the toe to resist the weight of the soil higher up on the slope. Consequently a slope failure may result.

- Broken Paved Ditches – Paved ditches that are broken permit surface water to flow under the remaining portions of the ditch. This may erode the embankment or slope or it may permit surface water to saturate portions of the slope.

- Water Ponding above, below, on, and in the Median of a Roadway – Ponding water is always an undesirable source of saturating water. Water ponding above the roadway may cause a cut slope to become saturated and slide onto the roadway. Water ponding in a ditch or in a median may saturate the entire embankment or further saturate a potential (i.e., weakened) failure plane in an embankment or slope. Water ponding at the toe of the embankment will weaken the toe and may cause instability.

- Drainage Structures Discharging onto Slopes – Pipes, culvert, ditches, or other drainage structures that permit water to flow onto an unprotected embankment or slope may be a major factor in causing a slope failure. Water from these structures may saturate a soil structure or severely erode the slope.

Subsurface Water

- Springs on or at Toes of Slopes – Springs indicate the presence of the groundwater table as it intercepts the ground surface. Springs also may indicate where water from a water-bearing rock formation has saturated a portion of an embankment of cut slope. Areas around springs will be particularly vulnerable to landslides or instabilities.

- Light and Dark Areas on Slope Face - Differences in color may indicate distinct differences in the amount of water from one area of a slope to another. The darker areas, areas containing the greater amount of water, will be more prone to instability.

- Soft or Muddy Areas on Slopes – Although no free or flowing water may be present, a soft or muddy area on a slope could indicate an area that is saturated and is subject to movement.

- Vegetation – The type or condition of vegetation growing on slopes may indicate the presence of subsurface water. Cattails or willow trees are particularly good plants to warn maintenance personnel of subsurface water. Grassy areas on a slope that stay green in dry seasons are sometimes indications of subsurface water.

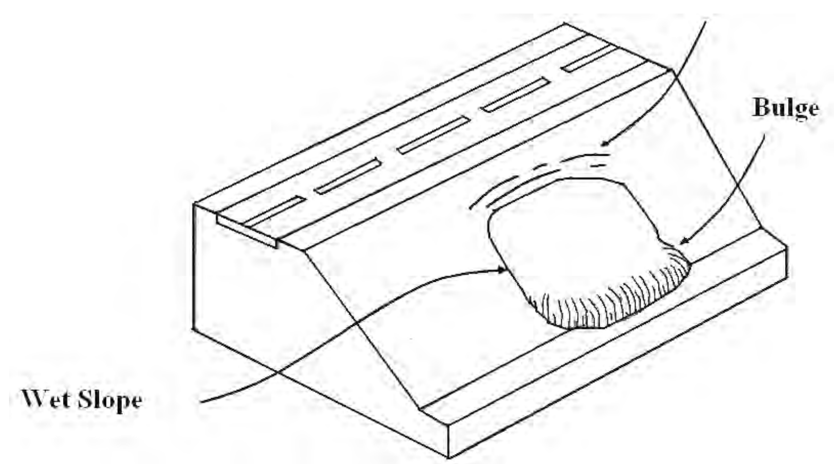


Figure 4-13 Bulging at the Toe of the Slope

4.5.3 Erosion

- Toe of Embankment or Slope – Surface water from paved ditches or other drainage structures may erode the toe of an embankment or slope, removing supporting soil and cause a failure.

- Toe of Cut Slope – Rapidly flowing water in drainage ditches often cause severe erosion at the toe of cut slopes. Also, poor practices when cleaning ditches may undercut the toe of cut slopes and cause instability.

- On Slopes or Embankments – Surface water from broken paved ditches or other drainage structures often is the source of this form of erosion. Poor maintenance practices are often the cause of this type of erosion.

<u>Changes in Features</u>

More subtle signs of earth movement may include trees that are tilted from vertical. Titled trees at the toe of a slope that are now growing vertically indicate an old landslide that moved many years previously. However, the movement has stopped and the tree is now growing vertically again. A tree growing in a continuous gentle curve may indicate a very gradual and slow creeping movement. Telephone poles and fences that have sunk or are titled out of alignment are also good indicators of earth movement.

<u>Changes in Structures</u>

Bridge abutments that tilt in relation to the bridge beams or abutments that move toward the ends of the bridge beams indicate that the approach embankment is moving or creeping toward the bridge. Settlement of bridge approach pavement slabs indicates that the approach embankment is settling or slumping.

Retaining Walls are structures used to hold back soil slopes. However, if the soil continues to move excessively, the wall will tilt from the vertical and in severe cases will actually overturn (Figure 4-14). Cracks in retaining walls also may be evidence of soil movement behind the wall.

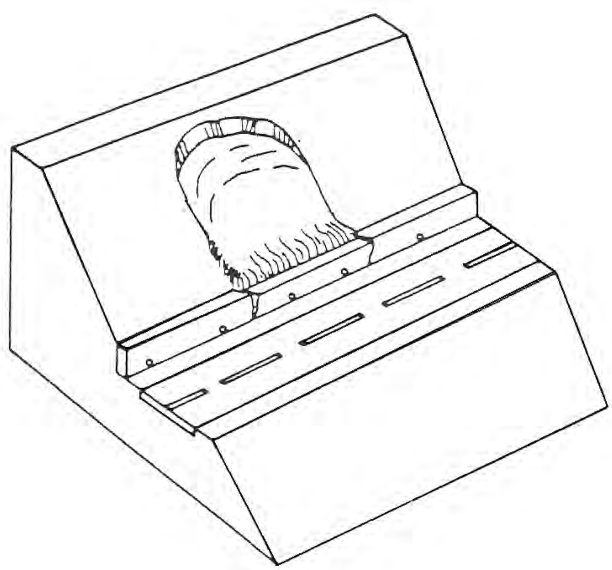


Figure 4-14 Overturning of Cantilever Retaining Wall

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CHAPTER 5

BEST PRACTICES FOR MAINTENANCE AND MONITORING

Many landslides and related problems can be prevented, controlled, or minimized by careful maintenance practices. This chapter provides guidance for maintenance supervisors and staff on best maintenance practices to address conditions that can lead to slope instability, guidance on when maintenance supervisors and staff should seek additional assistance from maintenance engineers and geotechnical specialists, and guidance on techniques that can be used for effectively monitoring soil and rock slopes in cases where monitoring is judged to be an appropriate tactic for addressing specific slope problems. The practices described are intended to minimize the number of slide problems, reduce the impact of slide problems that may occur, or delay the need for more expensive repairs. Effective application of this guidance will lead to more responsive and effective maintenance efforts, to reduced costs to the maintenance agency, and reduced inconvenience and danger to the public.

5.1 ROLES AND RESPONSIBILITIES

The vast majority of maintenance activities described in this chapter will generally be undertaken by maintenance crews as routine work. However, it is important that such activities be documented and periodically shared with maintenance engineers and geotechnical specialists to ensure that maintenance tasks are being appropriately performed, to make them aware of recurring problems that may require further study or more extensive repair, and to allow accounting for costs associated with slope maintenance and slide repair activities. Deliberate tracking of maintenance activities will help the agency develop proper programming for maintenance expenditures, may help to avoid more significant and costly problems, may lead to avoiding use of ineffective or even harmful practices, and may lead to improved design procedures in cases where designs may be leading to the need for excessive maintenance. Furthermore, knowledge of what is working and what is not will help designers better understand the mechanics of failures and to design mitigation and new projects more efficiently.

There will also be instances where required maintenance and repair activities are beyond the capabilities of maintenance staff. In such cases, maintenance engineers and geotechnical specialists should be called in to assist with evaluation of conditions and selection of appropriate maintenance and repair activities.

5.2 MAINTENANCE OF SLOPES

Three common characteristics of slopes that tend to produce failures are excessive slope angle, excessive slope height, and excessive groundwater. Activities to improve these characteristics (i.e. reducing slope height, reducing slope angle, and removing or controlling groundwater) will almost always improve the stability of a slope. Initial considerations for maintenance activities therefore generally focus on one or more of these conditions. However, maintenance of soil and rock slopes is seldom this easy because of right of way constraints and environmental or other restrictions. Furthermore, implementing changes to the slope geometry or groundwater conditions may improve stability in one area but at the same time reduce stability in another (effectively moving the problem, but not correcting it). For example, use of a slope buttress may improve the stability of the slope above the buttress, but decrease the stability of the slope below the buttress. Similarly, drainage trenches designed to remove groundwater from a slope may carry it to other areas, which may then experience problems that have not been present prior to the "remediation". It is therefore crucial to look at the bigger picture of things beyond the boundaries of individual slides to ensure that unintended consequences do not result from slide maintenance activities. Geotechnical specialists can assist with such evaluations.

This section describes best maintenance practices regarding activities directly involving soil and/or rock slopes. These activities primarily relate to maintenance of erosion features and erosion control elements, maintenance or repair of small slides, and maintenance of rockfalls.

5.2.1 Erosion

Erosion and erosion features can lead to slope problems in a number of ways. Excessive erosion in natural or engineered drainage channels can produce slope instability by reducing resistance forces Figure 5-1). Excessive eroded material from either channel or sheet erosion can also collect in drainage channels causing them to clog and produce drainage problems that can lead to slope failures (Figure 5-2). Excessive erosion is therefore an important cause, or precursor to many slide problems and must be corrected in a timely manner to limit the potential for producing larger slide problems.

While all soils are subject to erosion, some are more susceptible to erosion than others. In general, erosion results when the velocity of water (or wind) becomes great enough to move soil particles. Larger, and therefore heavier soil particles such as gravel and course sands, are generally less susceptible to erosion than finer grained soil particles. On the other hand, very fined grained soils like clays (especially high plasticity, fat clays) have substantial "cohesion" between particles that acts to hold particles together and reduces the potential for erosion. Soils with intermediate particle sizes and little "cohesion", such as fine sands and silts, therefore tend to be most prone to significant erosion problems. Such soils are present across the U.S., but are particularly prevalent in certain geographic regions such as the Mississippi River valley.

Figure 5-1 Example of Significant Erosion Producing Slope Instability Concerns

Figure 5-2 Example of Eroded Soil Accumulating in Drainage Ditch

Control of erosion can be accomplished using several different general approaches:

- Reducing the velocity of water passing over or through the soil
- Increasing the resistance of soil particles to being moved
- Avoiding contact between moving water and soil

Table 5-1 summarizes several specific techniques for controlling erosion within each of these general categories.

TABLE 5-1 GENERAL AND SPECIFIC METHODS FOR CONTROL OF EROSION

General Approach	Specific Techniques
Reducing Velocity of Water	• Grading ditches to appropriate slope • Use of ditch checks and other measures to control velocity of water within ditches • Use of retention basins to control velocity of water
Increasing soil resistance to erosion	• Maintaining vegetation (roots act to hold soil particles together) • Use of synthetic erosion control products such as geotextiles, erosion control mats or blankets • Use of soil amendments (e.g. lime, polyacrylamides or PAMs, etc.) or soil mixtures (e.g. mixing sand and clay) to make soil more cohesive • Use of soil or geosynthetic filters to prevent migration of soil particles
Avoiding soil-water contact	• Routing water through hardened drainage channels (e.g. interception ditches on slopes) • Diverting water through pipes

In many cases, several different approaches may be suitable for a particular instance and specific actions for particular erosion problems must be selected on a case by case basis. While the primary motivation for selection is likely to be cost and expected effectiveness, secondary factors such as appearance may also be important in some cases. Maintenance personnel should consult with geotechnical specialists in instances where the most appropriate technique for a particular situation is unclear.

Additional recommendations for controlling and reducing erosion problems:

- Regularly inspect erosion-control elements including paved ditches, ditch checks, pipe and culvert headwalls, slope paving, rip-rap, geosynthetic erosion control products, and all vegetation. Promptly repair deteriorated or damaged erosion control elements to maintain their effectiveness.
- Reseed disturbed soil as soon as possible after repairs, especially when the disturbed soil is on a significant slope. When reseeding, use straw or geosynthetic erosion control products to control erosion until vegetation is established.
- Maintain curbs, dikes, or berms intended to prevent surface water from flowing onto soil slopes.
- Regularly inspect and clean surface ditches and pipes to prevent accumulation of debris that may compromise drainage elements intended to keep water off of slopes.
- Maintain appropriate vegetation in unlined channels to help hold soil in place.

5.2.2 Maintenance/Repair of Small Slides

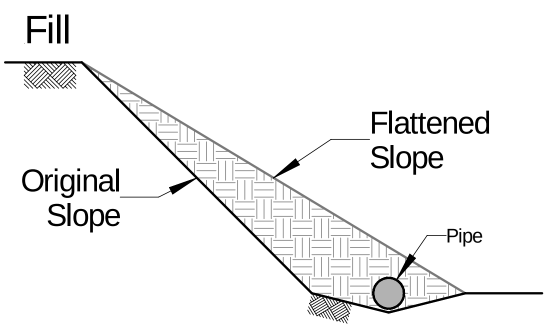
When a small slide has occurred, maintenance personnel can do a number of things to prevent the slide from becoming worse, to slow or stop slide movement, or to repair the slide. As discussed in more detail later in this chapter, such activities should only be undertaken by maintenance personnel in cases where the slide is relatively small and the consequences of additional sliding are deemed to be limited. If questions regarding the cause or effectiveness of a particular repair technique exist, maintenance personnel should seek assistance from a geotechnical specialist to ensure that the proposed actions will be effective and will not make matters worse.

While a large number of general and specific techniques are available for repair and stabilization of soil and rock slope failures (Chapter 7), techniques amenable to implementation by maintenance staff are generally limited to those that can be undertaken with common maintenance equipment. These techniques will generally be limited to those involving limited earthwork to improve or maintain the stability of the slope. Specific examples include regrading the slope to repair the slope to its original (pre-slide) topography, flattening or benching of the slope to improve stability, and potentially removal and replacement of the failed slope with materials having better characteristics. Maintenance crews may also implement elements to control flow of surface or subsurface water, which can improve the stability of a slope if applied correctly in appropriate conditions.

Regrading of small slides is a common technique used to “repair” small slides. This technique seldom produces a permanent fix because slopes are generally more prone to failure once they have initially failed. However, it can be an appropriate action on a temporary basis to prevent expansion of a slide and reduce the potential for additional damage. Slides that require repeated regrading should be repaired using techniques described in Chapter 7 as repeat failures can substantially degrade the strength of the soil and may lead to more significant failures if not properly repaired. Because regrading seldom produces a permanent repair, strong consideration should generally be given to use of one or more techniques for monitoring slope movements as described later in this chapter. Erosion control techniques should also be applied to the regraded area to control erosion.

If sufficient right of way and materials are available, small slides can often be repaired by adding material to produce a flatter slope. Additional material used to flatten a slope should generally have strength properties that are equal to or better than those of the existing slope and it is preferable if the material is freely draining (e.g. clean sand or gravel). In cases where slope flattening is to be used, the additional fill must be placed in a controlled fashion and cannot be simply dumped from the crest of the slope. Care must also be taken to ensure that flattening of the slope will not interrupt important drainage features, or that accommodations are made to maintain drainage if the flattened slope does intercept drainage ditches. Figure 5-3 shows a technique used to flatten a slope while maintaining drainage through the roadside ditch.

In some instances, effective repair of small slides can be accomplished by removing the soil involved in the slide and replacing it with materials that have better engineering characteristics as illustrated in Figure 5-4. A common implementation of this technique is to remove poor and often wet silty or clayey soils and replace them with clean dry aggregate materials that have higher strength and are generally better draining than silty/clayey soils. The failed soil can also be amended with admixtures such as cement, lime, fly ash, or other cementitious produces and then placed back into the failure area although it is generally advisable to consult with appropriate engineering or geotechnical specialists for such repairs as the effectiveness of different amendments can vary substantially.



(a) Schematic Diagram

(b) Photograph

Figure 5-3 Illustration and Photograph of Technique to Maintain Drainage through Ditch at Slope Toe while Flattening Slope to Improve Stability; (a) Schematic Diagram of Technique, and (b) Photograph of Technique in Use in the Field

(a)

(b)

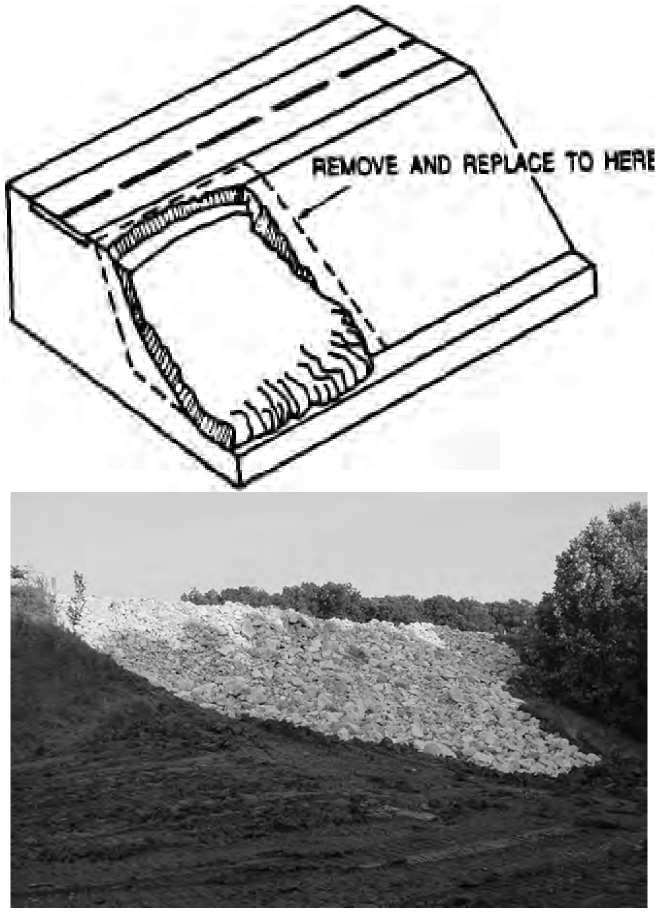


Figure 5-4 Illustration showing common removal and replacement technique for repairing small slides: (a) Schematic illustration; and (b) photograph of technique applied in the field.

When implementing removal and replacement for small slides, it is critical that existing material be excavated beyond the existing sliding surface to ensure that the technique is effective. Removal and replacement of material above the existing sliding surface will have very little effect on the stability of the slope. Removal and replacement should also not be confused with practice of "sidecasting" illustrated in Figure 5-5. While these techniques can at times appear similar, the practice of sidecasting generally increases the driving forces on the slope and does nothing to increase resistance to sliding so the net effect of this practice is to <u>reduce</u> stability rather than improve it. Sidecasting should therefore be avoided.

(a)

(b)

Figure 5-5 Illustration Showing the Poor Practice of “Sidecasting”: (a) Schematic Illustration; (b) Photograph of Technique Unsuccessfully Applied in the Field.

Removal and replacement of failed areas on a slope also poses a significant, although temporary challenge that must be carefully considered prior to undertaking removal and replacement. The challenge with this technique is that excavation of the failed material necessarily involves creating a slope that is temporarily much steeper than the original slope (which failed!). The steepened slope has much greater chances of failing. If the steepened slope fails, the consequences are often much greater than those of the original slope and the failure is very likely to involve the roadway surface itself. Because of this issue, removal and replacement is generally best accomplished during dry times of year when the soil will likely have additional strength that can help maintain the stability of the temporary slope. Removal and replacement of larger slide areas should also generally be undertaken in short sections (i.e. excavating a short section of the slope and compacting the replacement soil back into that section before excavating the next section) to minimize the risk of failures during construction. Regardless of whether this activity is undertaken during dry periods or in sections, careful consideration must still be given to the potential consequences of excavating the failed soils prior to undertaking such actions. Consultation with geotechnical specialists is strongly advised before such activities are undertaken.

Additional recommendations for improving the stability of slopes with small slides include:

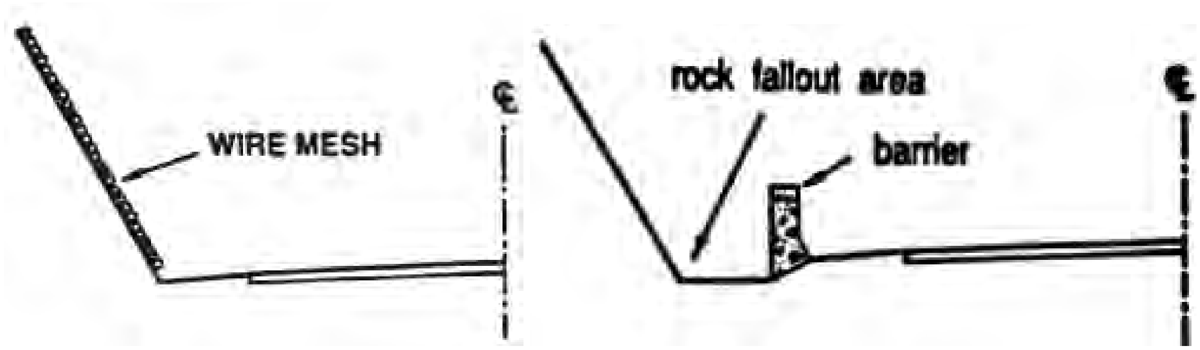
- If possible, direct surface waters away from the slide area using pipes or ditches to reduce water reaching the slope.
- If there is excess water in the slide area, providing some mode of drainage is generally beneficial. Horizontal drains can be helpful where subsurface water is a problem (Fig. 5.11). Ditches and pipes can also be used for draining ponded surface water.
- Establishing vegetation may help to absorb excess water and prevent erosion.
- All surface cracks should be sealed to prevent water from rapidly infiltrating the slope.
- The movement of all known slides or unstable areas should be regularly monitored to determine if maintenance efforts are effective.

Activities to be avoided include:

- Avoid excavating the toe of a slope or removing lateral support for a slope or embankment.
- Avoid excavation or earthwork that would permit water to pond in the slope area.
- Avoid adding load to the top of the slope (e.g. side casting, repeated patching of pavement or shoulder). Areas requiring frequent patching should be evaluated to determine the cause of settlement and properly repaired.
- Avoid blocking drainage structures including ditches, pipes, or culverts.

5.2.3 Rockfalls

Regular maintenance is also required to control rockfalls to limit exposure of the public to potential safety hazards. The primary objective of rockfall control measures is generally to minimize the potential for falling rock to enter the roadway. As illustrated in Figure 5-6, some common measures to accomplish this can include use of catchment ditches or areas to collect falling rock, use of wire mesh or grid placed directly onto the rock slope, or use of barriers to prevent falling rock from reaching the roadway.



(a) (b)

Figure 5-6 Common Measures for Preventing Falling Rock from Reaching the Roadway: (a) Wire Mesh Placed Directly onto Rock Slope, and (b) Rock Fall Catchment Area and Barrier.

All rockfall control measures should be inspected regularly to ensure they are functioning properly. Fallen rock collecting in catchment areas or behind barriers should be regularly removed because collection of substantial amounts of rock can reduce the effectiveness of the control measures and allow falling rock to reach the roadway. When clearing catchment areas, it is important that the rock slopes not be undercut (Figure 5-7), which can promote instability and may lead to more serious problems in the future. The quantity of fallen rock should be monitored over time because increases in quantity of falling rock may indicate more serious problems. Slopes exhibiting increasing quantities of falling rock should be considered for scaling or other more effective control measures to limit the potential for catastrophic events. Rock slopes with significant potential for large rock slides should also be continually monitored by appropriate geotechnical specialists.

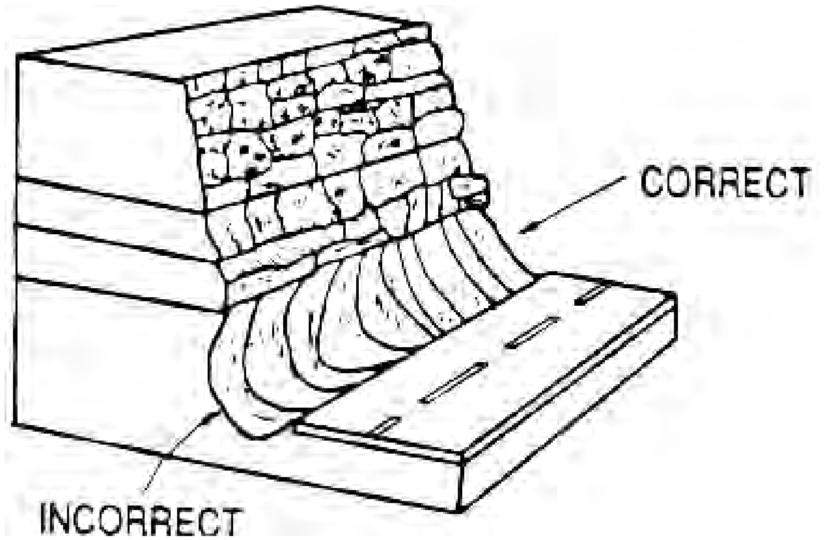


Figure 5-7 Illustration Showing Correct and Incorrect Methods for Clear Rockfall Catchment Areas

5.3 MAINTENANCE OF DRAINAGE FEATURES

As discussed in previous chapters, water is one of the most frequent triggering mechanisms for slides in soil and rock slopes. As such, control of surface and subsurface drainage can be one of the most effective means to limit slides along transportation routes. However, drainage features must be appropriately maintained if they are to remain effective, continuing to perform their intended function. In fact, in some cases drains and ditches that are not maintained can do more harm than if none were there. The following describe recommended practices for maintaining surface and subsurface drainage measures and techniques that can be used to improve drainage conditions in areas that have experienced slides or appear prone to future sliding.

5.3.1 Maintenance of Surface and Subsurface Drainage Features

All surface and subsurface drainage elements such as lined and unlined ditches, pipes, detention basins, pavement edge drains and other features should be regularly inspected to ensure they are working properly. Accumulated soil, vegetation, or other debris in drainage ditches and pipes should be immediately cleared to prevent water from backing up and potentially saturating the slope (Figure 5-8), which can lead to sliding. Damaged sections or open joints in engineered drainage channels (Figure 5-9) should also be promptly repaired to avoid excessive erosion and provide for free flow of surface water.

Similarly, buried pipes with open joints or damage must also be immediately repaired to avoid introducing excessive amounts of water into slopes and potentially producing severe erosion. Particular attention should be given to head walls and inlets/outlets for pipes and drainage channels to ensure that erosion in these areas is controlled. Curbs and other features intended to control surface water along roadways and bridges should also be inspected and maintained to ensure that water is directed away from slopes to control erosion and limit saturation of slopes.

Figure 5-8 Example of Soil Impeding a Drainage Ditch Resulting in Ponded Water Within Ditch

Figure 5-9 Example of Damage to Engineered Drainage Ditch that Requires Repair to Prevent Erosion and Ensure that Water is Directed Away from the Slope and Roadway

Inspections should also include observation of areas where water is ponded, or may pond. Several potential areas of this type are illustrated in Figure 5-10. Where such areas are identified, corrective measures such as regrading of ditches, smoothing of ruts, or removal of debris should be undertaken as quickly as possible to reduce the potential for ponded water to cause sliding. In particularly troublesome cases, installation of drop inlets and underground piping may be necessary for conveying water away from low lying areas that have propensity for ponding water. Cracks observed in slopes, roadway shoulders, or pavements should be also be sealed to eliminate preferential flow paths for water to infiltrate into the slope.

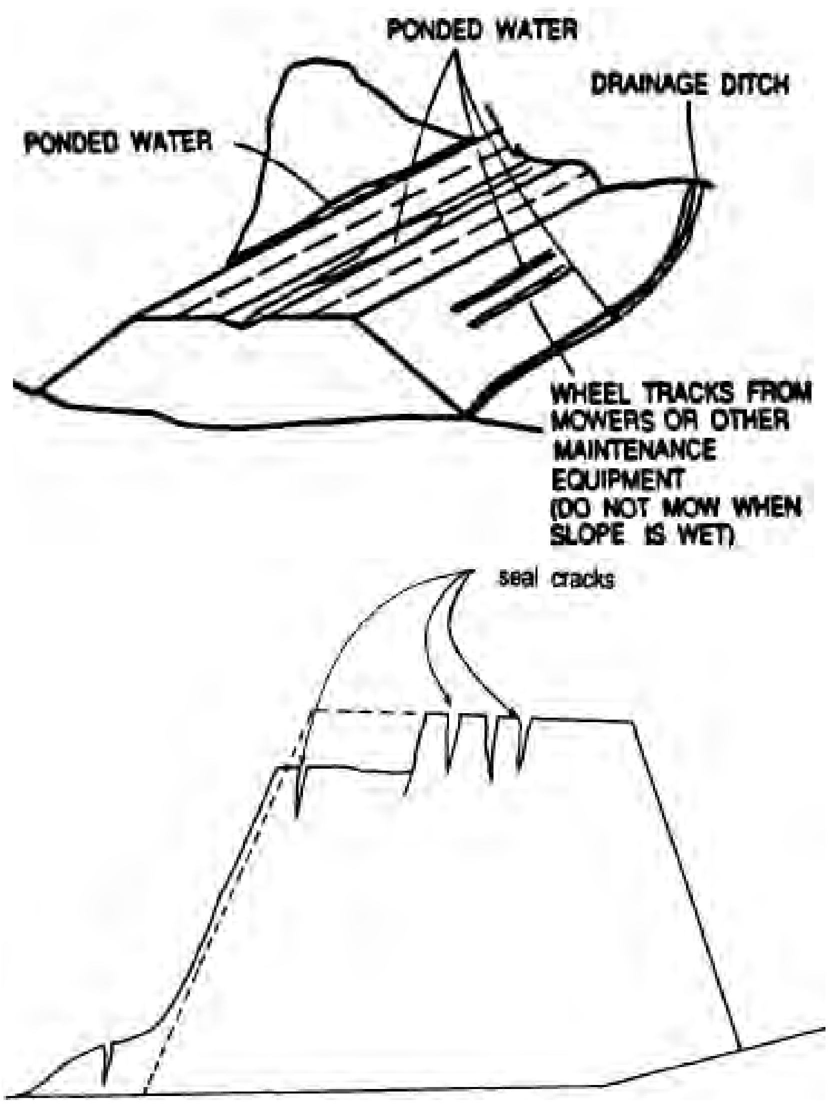


Figure 5-10 Illustration of Drainage Problems that Can Lead to Slope Instabilities and Should be Repaired

General recommendations for repair of damaged drainage channels and pipes include:

- Avoid use of rigid pipe in a landslide or other unstable area as these pipes cannot accommodate the movement that is occurring in the area without separating at the joints or failing in other areas.
- Use head walls and slope protection on the inlets and outlets of drainage pipe to prevent erosion.
- Avoid use of pipe that is too small to allow water to freely flow.
- Avoid diverting water towards a slope, particularly if the area is prone to sliding.
- Avoid use of drainage pipes that are too short and will allow water to discharge onto the slope to prevent erosion or saturation of the slope.
- Avoid over-excavation in ditches as this may remove toe support of an embankment or cut slope and lead to instability. Over-excavation may also permit water to pond in the ditch.
- Use suitable geotextile (or soil) filters around all drainage gravel behind walls and in trench backfills to prevents fine clays and silts from clogging the void spaces in the gravel.
- Use pavement edge drains in slide prone areas to help collect surface water that gets into the pavement structure through cracks in the surface and through paving joints.
- Use only pipe that is resistant to degradation (e.g. by acidic conditions) in areas where soil or water may degrade piping and lead to premature failure.
- Provide access/cleanouts at regular intervals in all drainage systems and avoid using features on any subsurface system that may hinder or prevent maintenance activity.
- Monitor all utility installations ensure that damage to existing drainage systems does not occur.
- Avoid constructing structures with a granular backfill that will not drain (bathtub design).

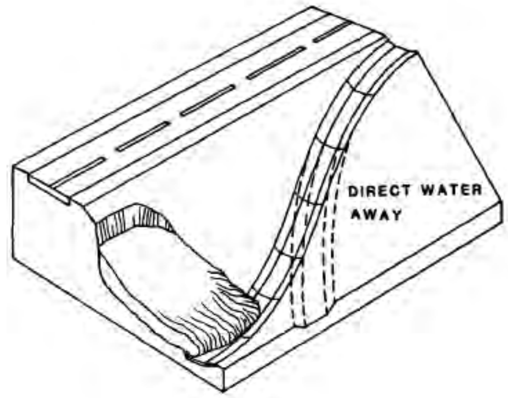
5.3.2 Use of Surface and Subsurface Drainage to Improve Stability

Improving or controlling surface and subsurface drainage can also be an effective means for improving stability in slide prone areas and can be used as a "repair" method for small slides when water has contributed to sliding. Control of drainage can be used alone, or in combination with other techniques such as regrading, flattening, and removal and replacement.

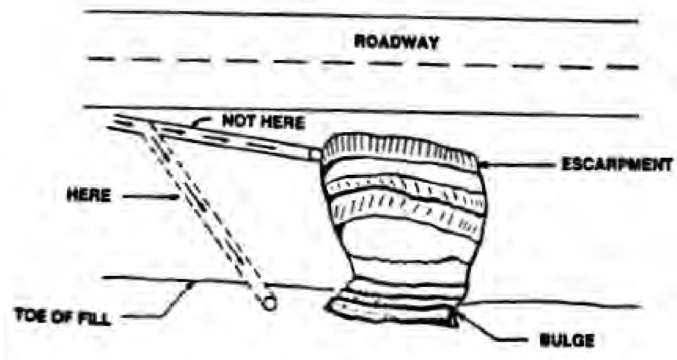
If at all possible, water should be diverted away from a slide area or potentially unstable slopes as illustrated in Figure 5-11. Diversion of water serves to help prevent erosion and to reduce saturation of the slope, both of which will tend to improve stability. Use of horizontal drains can also be used to improve stability of slopes where high water tables or springs may be contributing to instability as illustrated in Figure 5-11. In cases where water is being introduced into a slope through cracks in the pavement or through the pavement drainage layers, use of pavement edge drains may also be an effective means for limiting flow of water into a slope.

5.4 MAINTENANCE OF PAVEMENTS

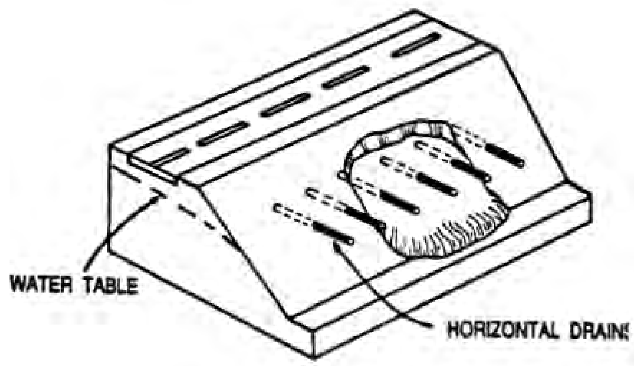
Pavement surfaces can have a significant impact on the stability of soil and rock slopes depending on their structure and condition. On one hand, pavements serve as a load on the slope that must be resisted to maintain stability. Loads from the pavement itself are generally small and not of major consequence. However, repeated patching of pavements adjacent to a slope as a result of continuing settlement can add additional load to the slope. Repeated patching should therefore be avoided. If repeated patching is necessary to maintain the smoothness of the pavement, additional repair activities should also be undertaken to control the movement of the slope – in essence treating the cause of the slope problems rather than simply treating the symptoms.



(a)



(b)



(c)

Figure 5-11 Example of Use of Horizontal Drains to Improve Stability of Slopes When Subsurface Water is Contributing to Instability.

Properly functioning pavements also generally serve as a barrier to prevent infiltration of water into a slope, which facilitates stability as long as care is taken to control runoff from the pavement to prevent saturation and erosion of the slope. However, improperly maintained pavements can serve as a conduit for water to enter a slope if the pavement has large or excessive cracks either from slope movements or from deteriorated pavement materials (Figure 5-12). The additional water entering the slope can lead to saturation of the slope, worsening of slope problems, and potential failures. Pavement bases and subgrades can also serve to convey water to the slope if pavement edge drains are not used or functioning properly. Maintenance activities associated with pavements should focus on remedying these potential problem conditions.

The following activities are specifically recommended:

- All surface joints, cracks, and edges that have a potential to permit water to enter the subgrade or embankment should be routinely sealed to prevent unnecessary infiltration of water into the slope.
- Similarly, all potholes and areas of uneven settlement should be patched to prevent standing water that could saturate the subgrade.
- Pavement overlays covering the entire pavement surface should be considered when the pavement has many cracks from fatigue or temperature changes.

- Existing pavement edge drains should be routinely inspected to ensure they are functioning properly and repaired or replaced when found to be deficient.
- In locations without pavement edge drains, base courses should "daylight" to ensure that the pavement section does not hold water ("bathtub" effect), which can contribute to slope instability.

Figure 5-12 Example of Pavement Cracking that Should be Sealed to Prevent Worsening of Slope Stability Problem

5.5 OTHER MAINTENANCE PRACTICES

In addition to maintaining slopes themselves, drainage features, and pavements, there are a number of additional practices that can contribute to limiting slide problems as well as a number of practices that should be avoided. Several of these are summarized here.

Good practices to maintaining stable slopes and embankments include:

- Appropriate drainage should be provided for bridge abutments and paved spill slopes beneath bridges to prevent erosion around bridge abutments and minimize high pore water pressures behind abutment walls.
- Formal drainage structures such as pumping stations, diversion structures, siphons, and irrigation systems should be routinely inspected and maintained to ensure proper performance.
- Expansion joints in pavements and at bridge ends should be periodically inspected and maintained to ensure they are not serving as conduits for water into the pavement slope that can saturate the soils and potentially lead to erosion.
- Backfill for utility construction on or near slopes should be properly compacted to prevent settlement that may lead to ponding of water near the slope.
- Use proper filtration materials around trench backfill to prevent erosion of soils.

Practices that should be avoided include:

- Avoid watering of vegetation on slopes and in drainage channels unless necessary to maintain vegetation; if watering is required, keep it to a minimum.
- Avoid mowing slopes when soil is very wet to avoid rutting which can concentrate water flow and increase erosion.
- Avoid use of impermeable slope paving in cases where water may get behind it. Such surfaces prevent subsurface flow from escaping and can produce high pore pressures or cause frost heaving in the winter.
- Avoid channeling water behind headwalls, abutments, or retaining walls.
- Avoid severing subsurface drainage measures during installation of utilities.
- Avoid excavation on or near slopes that may cause instability.
- Limit the time that excavations are left open, or provide positive drainage to prevent water from standing in the excavation and saturating the surrounding soil.
- Avoid installing utilities in trenches without providing for escape of water from the trenches. Enclosed utility trenches can serve as a conduit for water into a slope.

5.6 WHEN TO SEEK ASSISTANCE

An important issue facing maintenance staff charged with maintaining soil and rock slopes is when to seek assistance from other agency staff or external consultants. In many cases, maintenance staff are hesitant to seek outside assistance because it often requires additional funds beyond what is needed for the maintenance itself. However, it is also important to realize that use of ineffective maintenance practices can lead to much greater costs if large slides occur or if repeat and continuing maintenance is required because of poor practice.

Establishing broadly applicable rules for when assistance should be sought is difficult because of the widely varying nature of slope problems, and the different relationships among maintenance and technical staff within different agencies. However, the following general guidelines can serve as a simple basis for such decisions, with consideration given to project and agency specific issues and constraints. In general, assistance from geotechnical specialists or outside consultants should generally be pursued when any of the following conditions are present:

- Any time maintenance personnel feel uncertain about the causes and/or effective solutions associated with a particular problem;
- When repeated recurrences of a slide or related problem have been observed or past practices have been unsuccessful;
- When consequences of failure of maintenance and repair measures are substantial (i.e. when costs of failure will be very high or when public safety may be compromised); and
- When the slide or related problem is costly enough to justify more thorough evaluation.

While these conditions are necessarily vague, it is possible for specific agencies to develop more specific guidance on when assistance should be pursued. For example, an agency may choose to require assistance from geotechnical specialists if the overall costs of maintenance and/or repair for a particular site exceeds $20,000, or some other figure developed in consultation with relevant parties.

5.7 MONITORING AND OBSERVATION

There are many occasions where symptoms of sliding (e.g. cracks, guardrail settlement, bulging on slope surface) are observable but significant consequences have yet to have occurred. In such cases, it may be prudent to monitor and observe the slope for a period of time to identify whether the movement is likely to produce significant consequences and, if so, to produce information to assist in establishing the likely cause of the movements, which will generally provide useful information for selecting the most appropriate means for repairing the slide.

In general, the objectives for monitoring and observation include:

- Establishing whether movements are occurring and continuing
- Establishing the rate of movement or changes in the rate of movement, which may indicate impending sliding
- Establishing potential causes or triggering events to movement
- Establishing details regarding the nature of the movement, such as sliding depth and/or sliding rate.

Monitoring can be accomplished using a variety of different techniques that range from simple visual observations to use of elaborate field instrumentation. Several inexpensive techniques can also be used to improve upon simple visual observation to help evaluate slope movements. Several such techniques are described in the following.

5.7.1 Simple Methods for Qualitative Monitoring

A number of simple, inexpensive techniques can be used to facilitate monitoring of slopes to improve upon visual observations alone. While these techniques will not provide quantitative data of the type provided by more elaborate field instrumentation, they can help to observe small movements that would otherwise be difficult to identify. One simple technique that can be used to identify movement that is affecting pavements is to paint lines across small cracks in the pavement to help distinguish movement that has already occurred from new movements as shown in Figure 5-13. "Spot marks" can also be placed on the pavement and measured periodically to help identify the magnitude and direction of movements.

Figure 5-13	Paint Lines across Cracks in Pavement Used to Help Identify Slope Movements

Another simple technique that can be useful for observing slope movements is to place a row of marker stakes along the slope, across the area where movement is suspected (Figure 5-14). If slope movements occur, the stakes will generally help to identify the extent of slope movement and may help to observe small movements by observing the line of stakes from the end of the line. If cracks are present in a slide, one may obtain information about the rate and growth of a slide by measuring the length and width of cracks.

Figure 5-14	Marker Stakes Placed on Slope to Help Identify Slope Movements

5.7.2 Quantitative Monitoring Methods

In many case, the simple techniques described in the previous section will not be sufficient for measuring small movements. The techniques also do not provide information regarding the rate or depth of sliding and it is often difficult to establish the cause(s) of sliding with such simple methods because no information is provided regarding water levels or other information that contributes to slope instability. In cases where more precise measurements are required, various types of field instrumentation are available to help determine the size of a landslide, how rapid it is moving, the depth of failure plane, the direction of movement, the location of ground water or the water pressure in the unstable slide material. While comprehensive coverage all available field instrumentation is beyond the scope of this manual, the follow describe some of the most commonly used field instrumentation for slide investigations. In all cases, geotechnical instrumentation should be programmed in consultation with geotechnical specialists and installed by personnel experienced in installation of geotechnical instrumentation.

Slope Inclinometers

Slope inclinometers are the most commonly used geotechnical instrumentation for monitoring lateral ground movements. The device consists of an inclinometer probe that is placed down special inclinometer casing to establish a profile of lateral movement with depth below ground surface. Figure 5-15 shows a schematic of inclinometer casing and probe. The casing is installed by boring a hole through the moving landslide mass and usually 5 to 10 feet into stable soil or bedrock. A special hollow plastic casing is installed in the entire length of the hole and the hole is backfilled with gravel, sand, or grout. The bottom portion of the casing (5 to 10 feet) is located in stable soil or bedrock so that it cannot move. The upper portion of casing is free to move as the landslide moves. The casing has vertical grooves on the interior wall in which the probe "rides" to maintain the orientation of the probe. The probe measures the deviation of the casing from the vertical at various depths in the hole. If the landslide is still moving, the casing will also move. Periodic readings will indicate if the slide is moving, the depth at which it is moving, and the rate of movement.

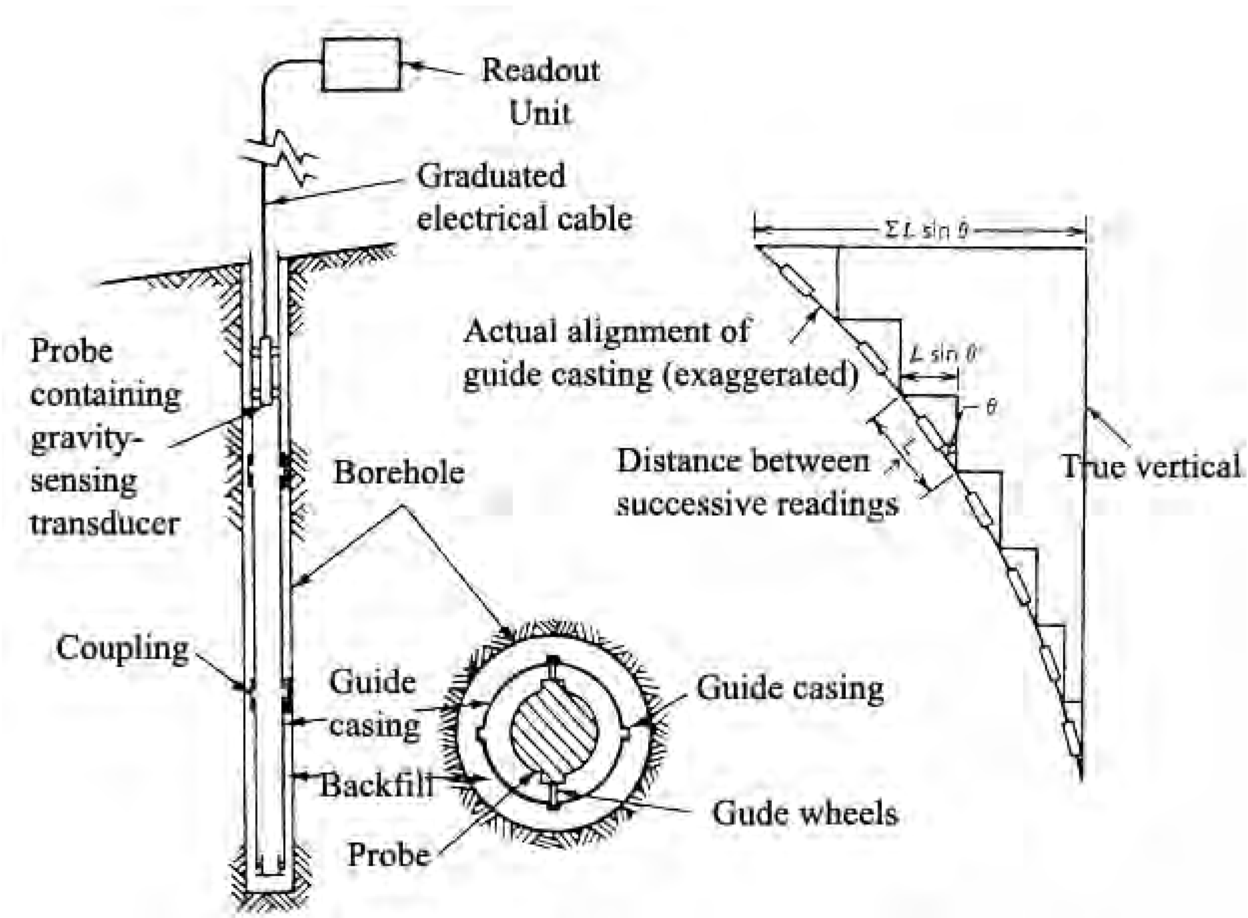


Figure 5-15 Principles of Inclinometer Operation (Dunnicliff, 1988 and 1993)

<u>Drain Tile</u>

In cases where funding or equipment is not available for slope inclinometers, drain tile or downspout may be installed in a landslide as “poor mans” substitute for slope inclinometers to provide some indication of sliding depth as illustrated in Figure 5-16. By lowering a rope or tape in the tile until the rope or tape can go no further because the tile or downspout has closed off, an indication of the location of the failure plane may be obtained. Unlike slope inclinometers, these methods will not yield the rate of movement.

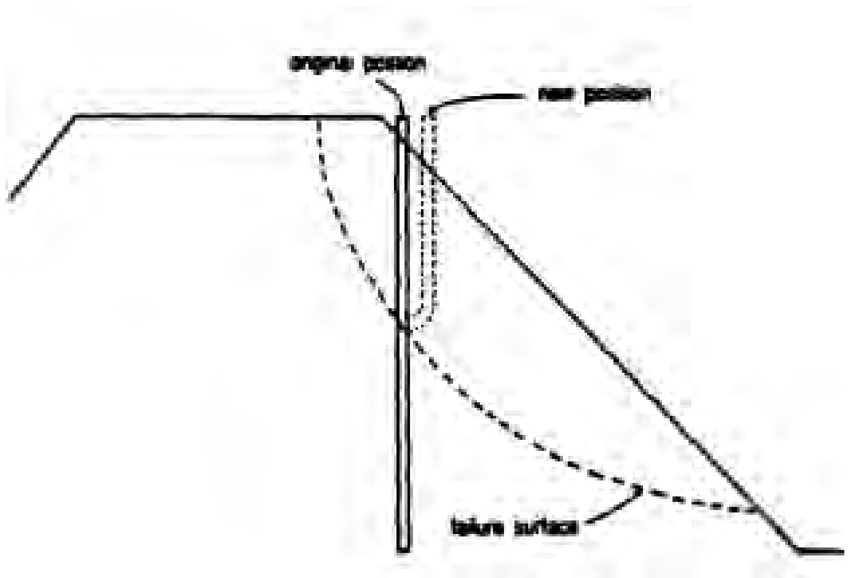


Figure 5-16 Cross-Sectional View of Movement of Drain Tile or Downspout

Time-Domain Reflectometry (TDR)

A relatively new technique that can be used to identify the depth of sliding for moving soil is time-domain reflectometry, or TDR. In this TDR technique, a coaxial cable is installed in a borehole through soil that may be moving. Once in place, the sliding depth is measured using special instruments which send an electrical signal down the coaxial cable as illustrated in Figure 5-17. If sliding is occurring along a discrete surface, the sliding will create a crimp in the coaxial cable, which will cause a reflection of the electrical signal when it reaches the crimp. The location of the crimp is then determined by measuring the time between sending the electrical signal and measurement of the reflection using the TDR instrument (the instrument sends the signal and measures the reflection). The primary advantage of using TDR is that costs are relatively low once the TDR instrument is acquired. The disadvantages are that it is still a relatively new technique so there are few people that have experience with the system. Like the drain tile approach described above, the technique also only provides an indication of the sliding depth and does not provide information regarding the magnitude or rate of movement.

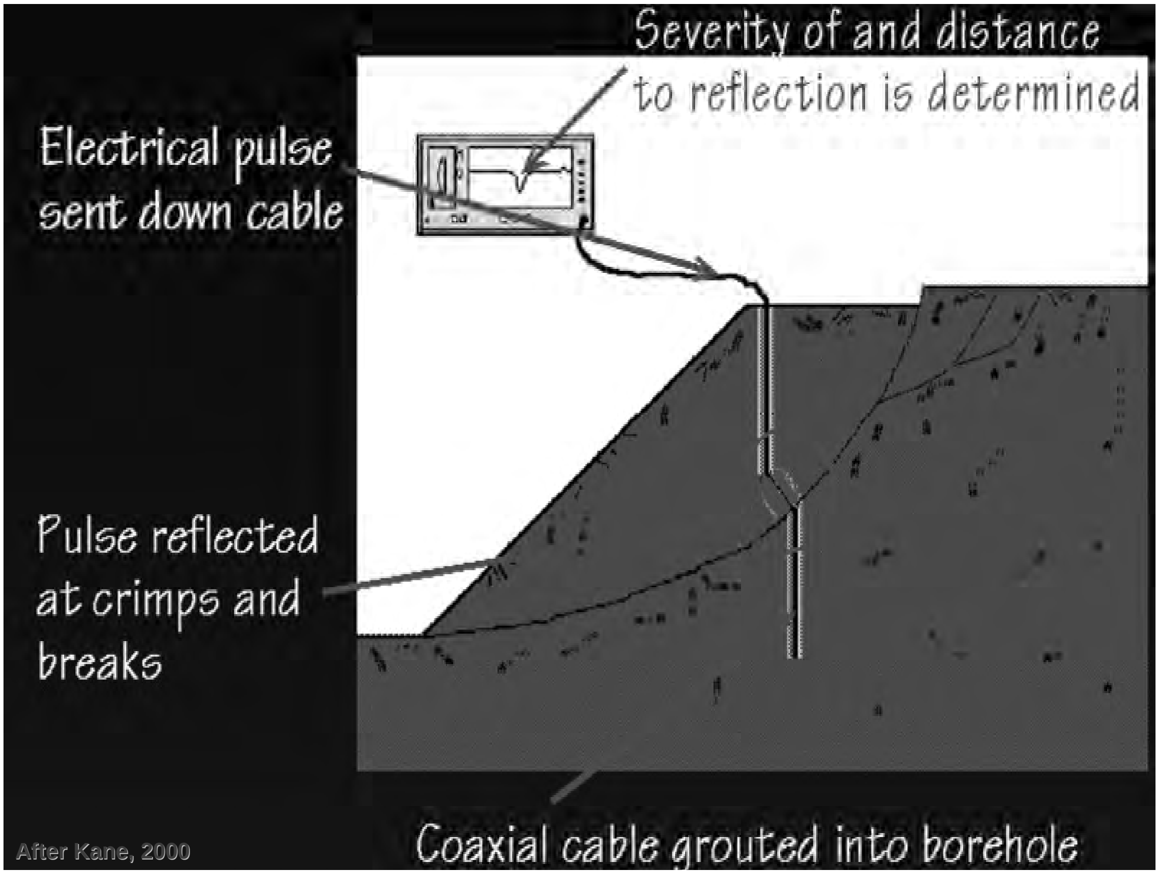


Figure 5-17 Illustration of Time-domain Reflectometry (TDR) Technique as Used to Identify Sliding Surface for Slope Sailures (after Kane, 2000)

Ground Surveys

Establishing a series of survey monuments in the vicinity of a slide (Figure 5-18) can be helpful for determining surface movements in unstable area. A series, network, or grid of survey hubs, stakes, or iron pins may be precisely located and elevations and alignments taken on each monument. Subsequent surveys can then reveal the relative horizontal movement of the monuments to each other as well as any elevation changes. This information defines or indicates the aerial extent of a slide and the rate at which it is moving. When implementing ground surveys over slide areas, it is very important to have a stable (i.e. non-moving) baseline or reference point from which to reference the slide measurements. Identifying such a point can be difficult in slide prone terrain so careful attention must be given to selection of the reference point. In many cases, it is good practice to use several redundant reference points in case one or more of them are found to be moving.

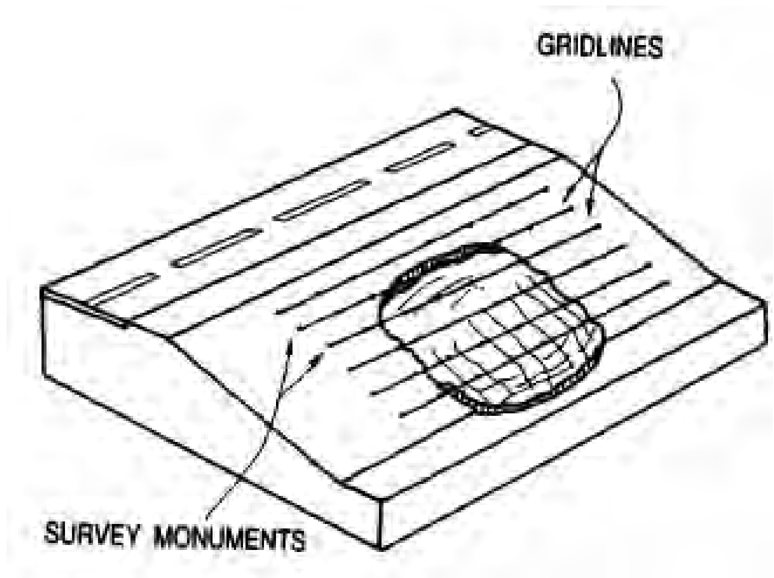


Figure 5-18 Tracking Slope Surface Movements Using Survey Monuments Placed in a Pattern of Gridlines

Piezometers

Piezometers are used to measure water pressure in the soil, which can be a significant contributor or trigger to slope failures. While a number of different types of piezometers are available, they are most commonly installed in a borehole at some predetermined depth. The borehole is sealed below and above the depth at which the piezometer is installed (Figure 5-19) so that the piezometer measures water pressure only at the desired depth. Piezometer readings can either be taken periodically using a readout device, or can be connected to a data acquisition system to permit readings to be taken automatically at regular intervals. Automated data collection has the distinct advantage that it allows for more frequent readings to be taken, therefore increasing the likelihood of recording data when water pressures are their highest, which is when movement would be most likely to occur.

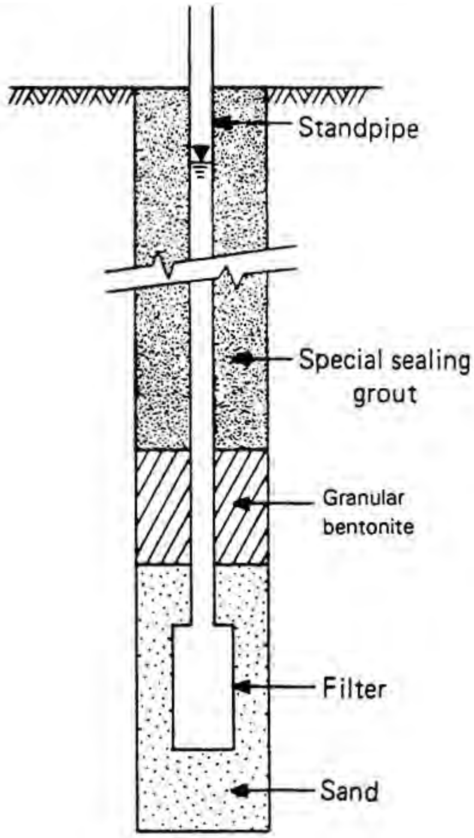


Figure 5-19 Schematic of Open Standpipe Piezometer Installed in a Borehole (Dunicliff, 1998)

Observation wells can also be utilized to obtain information about the free water table in the subsurface. Observation wells are similar to piezometers, except that they are generally screened over their entire length rather than over a limited length of the hole. After sufficient time is allowed for the water level in the standpipe to reach equilibrium, the elevation of the water table is determined by lowering a measuring device (stringline, chain, etc.) to the water surface to establish the depth of water at the time of reading. Like borehole piezometers, observation wells can also be fitted with automated means for measuring water level. The primary disadvantage of observation wells is that because they are continuously screened, they are not effective for identifying special groundwater conditions like perched water tables or artesian conditions.

Regardless of the type of instrumentation installed, it is critical that it be carefully protected from damage. Often instrumentation must provide months or even years of data; and if they are destroyed, valuable data may be lost. Instruments may be protected by stakes, flagging, fence posts, or any other appropriate means as illustrated in Figure 5-20. Maintenance and/or construction personnel should be fully informed about the location, purpose, and operation of instrumentation.

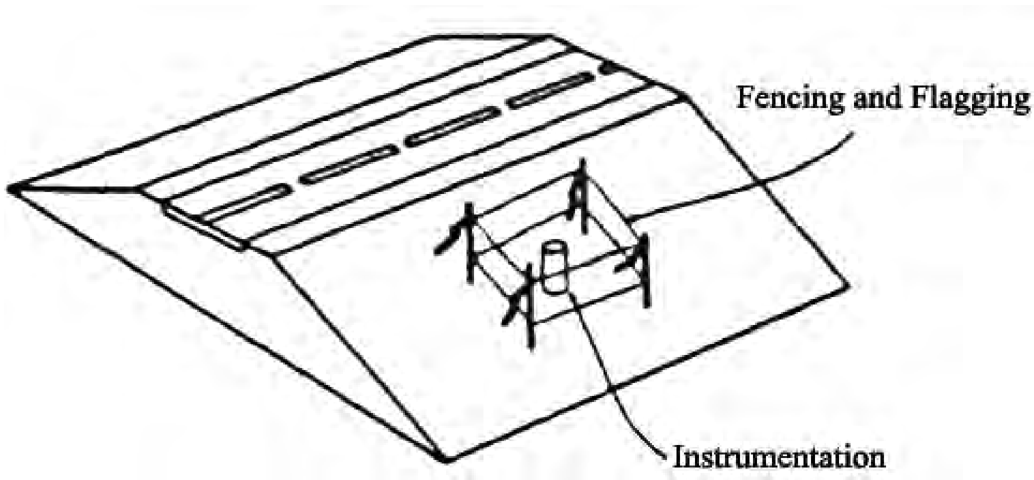


Figure 5-20 Example Illustration of Protection for Field Instrumentation

CHAPTER 6
SLOPE MANAGEMENT SYSTEMS

6.1 MOTIVATION FOR MANAGEMENT OF LANDSLIDES AND ROCKFALLS

Slope failures and landslides constitute significant hazards to all types of public and private infrastructure. Total direct costs for maintenance and repair of landslides involving major U.S. highways alone (roughly 20 percent of all U.S. highways and roads) were recently estimated to exceed $100 million annually (TRB, 1996). In the same study, indirect costs attributed to loss of revenue, use, or access to facilities as a result of landslides were conservatively estimated to equal or exceed direct costs. Costs for maintaining slopes for other highways and roads maintained by government and private agencies such as county and city governments, the U.S. Forest Service, the U.S. Army Corps of Engineers, and the National Parks Service significantly increase the total costs for landslide repairs. Another significant, but largely neglected, toll of landslides is the costs associated with routine maintenance and repair of “minor” slope failures, or “nuisance slides”. Individually, costs for repairs of these slides are quite low. However, cumulative costs for repair of nuisance slides are extremely large because of their high frequency. Unfortunately, little data is available to define costs for nuisance slides. However, the Transportation Research Board suggests that maintenance costs incurred for repair of minor slope failures by several state departments of transportation (DOT’s) that have kept “good maintenance records” generally equaled or exceeded the costs incurred for repair of major slopes. Available evidence therefore suggests that total annual costs for landslide repair and maintenance nationwide is several hundred million dollars.

Because of the great costs involved, there is significant challenge in effectively managing the maintenance and repair of small and large landslides and rockfalls. Financially speaking, the problem is similar to other maintenance problems in that sufficient funding is seldom available to fully repair all problems. Maintenance managers and staff must therefore make judgments to prioritize problems according to some criteria in order to plan and execute appropriate maintenance activities. This issue is compounded because there are multiple approaches to selecting the “most appropriate” maintenance and repair tactics. For example, one strategy that can be adopted is a “fire fighting” strategy wherein maintenance and/or repair is only undertaken when a problem occurs that is substantial enough to warrant rapid or emergency repair. An alternative strategy is to focus on addressing problems early, before they become severe. This approach is appealing in that costs associated with emergency response are avoided, but also introduces the potential for performing maintenance/repair at sites that may not eventually lead to a substantial problem. Other intermediate strategies or tactics can also be adopted. Unfortunately, there has been limited study of alternative strategies to evaluate which are most effective. When coupled with the general lack of financial data upon which to perform evaluations, there is simply insufficient information upon which to establish the “best solution” to these problems.

Even if such data were available (it is to varying degrees within different agencies), it is very likely that the “best” strategy for one agency may not be what is best for another due to different geographic and geologic conditions; different design, construction and maintenance practices; different organizational relationships within different agencies; and different funding mechanisms and funding constraints from agency to agency. In fact, the most appropriate strategy or tactic may even vary within a single agency depending on what part of the agency is being represented. For example, from a statewide perspective minimizing long term, or life cycle costs may be the most appropriate strategy whereas from the perspective of a individual maintenance area, an alternative strategy such as maximizing potential to acquire future maintenance funds may be most appropriate. While one can argue that policy should be set to provide for consistent strategy and equitable funding across an agency, it is practically difficult to accomplish this objective.

Despite these challenges, maintenance specialists and other agency staff must still respond to slope maintenance and repair needs and must do so with the objective of providing the most benefit to the agency within the constraints of time and budget. Fortunately, development and implementation of soil and rock slope management systems can assist agencies in developing the most appropriate strategies and tactics for maintenance and repair of landslides and rockfalls. Such management systems can also serve to support public accountability for landslide repair and may even serve as effective legal defense against some claims from landslides and rockfalls as discussed in Chapter 9. While landslide and rockfall management systems do not eliminate the challenges of programming slope maintenance and slide repairs with limited funding, they do provide knowledge, tools, and systems needed to more effectively evaluate slide problems from a "big picture" perspective, and to allow better strategies and policies to be developed and implemented so that the most effective use of limited funds can be made.

Specific benefits of slope management systems include:

- Provides for more proactive handling of slide and rockfall problems
- Provides means for collecting and tracking of expenditures, costs, and consequences associated with slide problems
- Provides a justifiable and defendable basis for making selection decisions
- Provides for more consistent strategy across agency for dealing with slides
- Provides valuable data to support public awareness of landslide and rockfall issues and requests for additional slide repair and maintenance funding so that programming can be improved
- Provides information that may lead to improvements to current design specifications to potentially lessen need for slide maintenance and repair

While slope management systems provide substantial benefits, they do have the following limitations:

- Do not "solve problem" in sense that they do not ensure sufficient funding, nor do they address the issue of technical viability for specific repairs of specific slides or rockfalls;
- Do not establish "optimum" strategy for addressing slide maintenance and repair; rather they enable effective implementation of a given strategy
- They require funding and manpower to maintain and improve system over time

6.2 COMMON FEATURES OF SLOPE MANAGEMENT SYSTEMS

A number of slope management systems have been developed, or are being developed by state departments of transportation and other federal, state, and local agencies. While there are certainly differences among the different systems, all share similar features including:

- A inventory of slopes or landslide/rockfall sites;
- Means and methods to collect data regarding the condition of inventoried sites and other relevant data necessary for prioritizing sites
- A "rating system" for prioritizing sites according to established criteria

These systems also generally establish procedures for periodically reviewing inventoried slopes and updating the database to reflect changes in conditions (e.g. for removing slopes that have been effectively repaired, for adding new slopes not previously included in the inventory, and/or for reprioritizing sites based on updated policy, changes in funding, or other influences). In general, the inventory and data collection/inspection aspects of the different slope management systems are pretty similar. In contrast, the rating systems differ substantially. The following provide additional information regarding each of these primary components of slope management systems.

6.2.1 Inventory

Inventories of slope failures and rockfalls are key components of slope management systems and all management systems include some form of inventory. Most inventories include only sites where substantial landslides or rockfalls have occurred. These systems are really "problem systems" in that they are primary intended to prioritize problem conditions (unlike pavement or bridge management systems which generally include pavements/bridges that are performing well in addition to those that are not). Others systems may include more minor problems such as small slides and erosion/drainage problems. No systems in current use truly include all slopes, although that would be beneficial. Regardless of the scope of the inventory, all systems utilize some form of database for storing appropriate data about each entry (e.g. slope characteristics, geologic information, traffic information, etc.) and to facilitate inquiries regarding the inventoried sites to develop information for making programming decisions. Specific details of the databases vary from agency to agency, but all share relatively common capabilities.

It is important here to distinguish between a landslide/rockfall inventory system and a slope management system. Some agencies may maintain inventories of soil slides and rockfalls, and may perform periodic inspections of slide prone sites or sites that have experienced failures. While such inventories can be useful, an inventory alone does not constitute a slope management system. A slope management system includes additional capabilities to prioritize and rank sites according to established criteria, which are used to make programming decisions.

6.2.2 Data Collection

One of the practically challenging aspects of implementing and maintaining slope management systems is collection of data to initially populate the inventory, and subsequently to routinely maintain the data so that it is kept up to date. Collection of appropriate data generally involves field inspections of slope sites to gather information about the slope or slide problem in addition to desk studies to collect other relevant information such as traffic volume, accident history, etc. The purpose of field inspections is twofold: first, to collect data that will be useful for selecting appropriate remedial measures; and second, to collect data that is necessary to rate the specific site as compared to other sites. In general, these two purposes are complimentary and can generally be accomplished in the same inspection.

Because of the importance of data collection for slope management systems, many agencies have developed specialized slope inspection forms to ensure that all necessary data is collected. Such forms vary from agency to agency, but share many similarities as well. An example inspection form is shown in Figure 6-1. More recently, some agencies are also moving to electronic site inspections using laptop or handheld computers. These systems serve the same purpose as hard copy forms, but make the task of entering data into the database much more efficient.

6.2.3 Rating Systems

The final major component of all slope management systems is a rating system that is used to prioritize different sites according to some established criteria. This component generally differs from agency to agency for a variety of reasons including different scale of problems, different agency priorities, or different agency funding, etc. Despite their differences, existing rating systems are generally based on a numerical ranking system whereby each site is given a numerical score. Sites with higher scores are usually considered to be higher priority sites than sites that produce lower scores. The resulting scores are used to produce programming for which sites should be remediated first based on available funding levels.

2/06

ROCKFALL REPORT

NYSDOT TRANSPORTATION MAINTENANCE DIVISION

1. Rockfall Location:

Reference MarkerInformation

Line 1 ____________________

Line 2 ____________________

Line 3 ____________________

2. Route number____________________

3. Town / Village ____________________

4. County ____________________

5. Residency ____________________

6. First reported by ______________ ___don't know

7. Side of Road ______ Right Side ______ Left Side

8. Date of rockfall: _____Mo. _____Day _____Yr. _____don't know

9. Approximate time of rockfall: ____: ____ AM-PM or ____ don't know

10. Weather conditions at time of rockfall: (circle all appropriate)
a) raining c) thawing e) freezing g) don't know
b) snowing d) sunny f) other ____________________

11. Volume of rockfall in cubic yards:______________(estimate)

12. Material landed in (circle all appropriate):
a) ditch area
b) shoulder area
c) driving lane

13. Size of largest fallen block:
a) baseball
b) waste basket
c) desk size or larger

14. Traffic accident ?	____ yes	____ no	_____ don't know
15. Vehicle damage ?	____ yes	____ no	_____ don't know
16. Personal injury ?	____ yes	____ no	_____ don't know

17. Comments ____
____
____
____
____

18. Residency contact person: ____________________phone number ( ) ____________

19. Form completed by:____________________________________Date:_____/____/____

20. Distribution: 1 copy to Regional Geotechnical Engineer
1 copy to Engineering Geology Section

Geology Fax Number (518) 457-8080 Geology Phone number (518) 457-4731

Figure 6-1 Sample Rockfall Inspection Form

The specific factors considered in the ranking systems differ from system to system, and some systems use additional “weighting factors” to place more importance on some factors (e.g. safety) than others. Table 6-1 shows a summary of different factors used in several different slope management systems. In the table, the factors have been placed into four different categories:

- Slope characteristics – information about the geometry and geology of the slope/site
- Highway/traffic characteristics – information about traffic passing near the slide area and potential consequences to traffic if the slide impacts travel;
- Slide characteristics – information about the size and/or character of the landslide or rockfall if it has already occurred;
- Consequence characteristics – information about existing or potential consequences of the landslide/rockfall if it is not repaired

Of all factors shown in Table 6-1, only Average Daily Traffic (ADT) and decision site distance are included within each of the rating systems shown in the table. It is worth noting that some of the factors listed in the table represent different ways of incorporating a specific issue. For example, some rating systems include consideration of “slide volume” in ranking sites, whereas others consider the vertical and horizontal dimensions. Both of these options are ways to represent the fact that larger slides are going to be given higher priority than slides that are much smaller. They are just different ways to accomplish the same general objective. It is also worth noting that none of the systems shown in the table include all potential factors that may play a role in prioritizing sites. The minimum number of criteria for the systems shown is 8; the maximum number is 15. These represent a reasonable range of different parameters to consider in a rating system to keep it simple, while still maintaining enough parameters to distinguish between sites that should be ranked highly and sites that should be lower ranked.

6.3 EXAMPLE SLOPE MANAGEMENT SYSTEM

To illustrate the general approach utilized in most existing slope management systems, the system developed by the Oregon Department of Transportation (2001) is described in more detail here. The Oregon DOT system was selected for this example because it includes all major components of a slope management system and because it includes specific consideration of both potential risk and economic impacts. Selection of this system is not meant to imply endorsement of the system, or that this system is “better” than others that have been developed. It is only used as an example to demonstrate a typical system and its operation.

The Oregon DOT system is based on development of an initial hazard rating that is indicative of the risks involved with a slide or rockfall. This hazard score is subsequently modified to account for economic considerations to achieve the final “score”, referred to as the STIP (Statewide Transportation Improvement Plan) score, that is used for ranking and prioritization.

TABLE 6-1 SUMMARY OF PARAMETERS INCLUDED IN SEVERAL STATE LANDSLIDE RATING SYSTEMS (MODIFIED AFTER LIANG, 2006)

Information Category	Information	Landslide Rating System					
		Oregon DOT – 1992	Oregon DOT – 2001	NYSDOT – 1988	NYSDOT – 1992	WSDOT – 1993	Ohio DOT – 2006
Slope	Slope Height	✓		✓	✓		
	Geology Prone to Slides	✓		✓	✓		
	Ground Water or Surface Water	✓		✓	✓	✓	
Highway/Traffic	Average Daily Traffic	✓	✓	✓	✓	✓	✓
	Highway Classification		✓			✓	
	Traffic Speed	✓		✓	✓	✓	✓
	Detour Time					✓	
	Decision Site Distance	✓	✓	✓	✓	✓	✓
	Travel Distance			✓	✓		✓
Slide	Slide Volume	✓		✓	✓		
	Relative Emergency						✓
	Failure Frequency		✓	✓	✓		✓
	Rate of Movement						✓
	Scarp Vert/Horiz. Dimensions		✓			✓	✓
Consequences	Expected Fatalities			✓	✓	✓	
	Risk to Vehicles	✓			✓	✓	✓
	Expected Damage				✓	✓	✓
	Impact to Road Structures/Features		✓			✓	✓
	Annual Maintenance Costs		✓			✓	✓
	Accident History		✓			✓	✓
	Cost-benefit Ratio		✓			✓	✓
	Potential Future Impact				✓	✓	✓

The initial hazard rating is developed using five different factors (or categories) that include:

- Failure Type/Hazard
- Roadway Impact
- Annual Maintenance Frequency
- Average Daily Traffic
- Accident History

For each category, a value between 0 and 100 is assigned based on criteria defined for each factor, as summarized in Table 6-2. For example, for the “Failure type/Hazard” factor, a score of 9 is given for slides that are slow moving and unlikely to cause a roadway hazard, a score of 27 is given for slides that have not moved suddenly in the past but may affect the roadway, and higher scores for slides that have created a road hazard in the past. In this system, no score is given to slides that are small and do not affect the roadway because such slides are excluded from the ranking (other agencies may choose to include such problems). For slides falling into the high hazard category, scores between 81 and 100 are given based on the site distance present in the field and the AASHTO decision sight distance established based on speed limit.

TABLE 6-2 SUMMARY OF HAZARD SCORE SYSTEM USED IN OREGON DOT LANDSLIDE HAZARD RATING SYSTEM

Hazard Score

1.	**Failure Type/ Hazard**	Very small or insignificant failures that do not affect the roadway. (Not Scored)	**LOW HAZARD**; Slower slides with low potential for causing a road hazard. (9 Points)	**MEDIUM HAZARD**; Slides that have not moved suddenly in the past, but have the potential to cause a road hazard. (27 Points)	**HIGH HAZARD**; Rapid slides that have created a road hazard in the past. Includes debris flows and rockfalls. (81-100 Points based on sight distance)

		LOW hazards receive 0 points	**MEDIUM** hazards receive maximum of 54 points				**HIGH** hazards can receive full point range		
2. **Roadway Impact (pick one)**	**Landslides:**	All **LOW HAZARD** slides above (0 Points)	Would only affect shoulder during major failure (3 Points)	Two-way traffic would remain after major failure (9 Points)	One-way traffic would remain after major failure (27 Points)	Total Closure in the event of major failure; 0-3 Mile Detour (54 Points)	Total Closure in the event of major failure; 3-10 Mile Detour (70 Points)	Total Closure in the event of major failure; 10-60 Mile Detour (85 Points)	Total Closure in the event of major failure; >60 Mile Detour (100 Points)
	OR								
	Rockfalls:	Rocks are completely contained in ditch (3 Points)	Rocks fall into shoulder only (9 Points)	Rocks enter roadway (27 Points)	No ditch, all falling rocks enter roadway (81 Points)	Rocks occasionally fill part or all of a lane (100 Points)			

3.	**Annual Maintenance Frequency**	0-5 Failures Per Year Sliding Scale from 0-100 Points			
4.	**Average Daily Traffic**	0 - 40,000 Cars per day Sliding Scale from 1 - 100 Points			
5.	**Accident History**	No accidents (3 Points)	Vehicle or Property Damage (9 Points)	Injury (27 Points)	Fatality (100 Points)

500 Total Possible Points

In a similar fashion, scores for the remaining categories are established using pertinent data for the particular site in question. The total hazard score is then taken to be the sum of all individual category scores, with a maximum possible score of 500. It is noteworthy that the Oregon DOT system uses an "exponential scoring" scale that produces progressively higher scores for greater hazards for several of the scoring categories to help distinguish between sites that may otherwise appear to have similar scores. Such scales are relatively common among the different landslide management systems because they promote a wider range of final scores that tends to make programming of slide repairs more straightforward.

Once a total hazard score is established, the score is modified to account for cost-benefit considerations and highway importance. The cost-benefit factor is established based on the ratio of established maintenance costs to the estimated repair costs as shown in Table 6-3. The highway importance factor is established from Table 6-4. Once these factors are established, the final STIP score is computed as:

$$\text{STIP Score} = (\text{Hazard Score}) \times (\text{MC20 factor}) \times (\text{Highway factor}) \qquad 6\text{-}1$$

Where MC20 factor is the cost-benefit factor from Table 6-3 and Highway factor is the factor associated with highway type (Table 6-4).

TABLE 6-3 MAINTENANCE COST-BENEFIT FACTORS FOR OREGON DOT LANDSLIDE RATING SYSTEM

$\frac{\text{20-Yr Maintenance Cost}}{\text{Repair Cost}}$	Maintenance Benefit-Cost Factor (MB/C)
> 0.0 - 0.2	0.5
≥ 0.2 - 0.4	0.75
≥ 0.4 - 0.6	1
≥ 0.6 - 0.8	1.06
≥ 0.8 - 1.0	1.12
≥ 1.0 - 1.2	1.18
≥ 1.2 - 1.4	1.24
≥ 1.4 - 1.6	1.3
≥ 1.6 - 1.8	1.36
≥ 1.8 - 2.0	1.42
≥ 2.0	1.5

TABLE 6-4 HIGHWAY IMPORTANCE FACTORS FOR OREGON DOT LANDSLIDE RATING SYSTEM

Highway Type	Highway Factor
District Highway	1.0
Regional Highway	1.05
Statewide Highway	1.1
Interstate Highway	1.2

The final STIP score is then used as the primary measure for ranking landslide or rockfall sites included in the inventory. However, because the system does not include all possible factors that contribute to prioritization (no system can possibly do this) and because the developers recognized the need for subjective judgment to be included in the final prioritization, the system allows additional "non-scored impacts" to be included in establishing the final prioritization of sites. Other non-scored impacts included in the Oregon DOT system include:

- Traffic delay costs
- Emergency lifeline priority
- Existing culvert damage
- Environmental impacts
- Impacts to adjacent structures or properties
- Interagency funding opportunities

The ability to invoke subjective judgment is an important feature in landslide management systems that provide some flexibility to respond to situations that are not properly represented by the systems. However, such flexibility also has the potential for abuse so these features should be constrained to the extent possible, or subject to independent review, to ensure that the flexibility is not abused by users of the management system. In the end, the true power of the management systems is to provide guidance on selected the highest priority sites and not to enforce mandatory actions per se.

6.4 ALTERNATIVE MANAGEMENT STRATEGIES

As with other forms of maintenance, different management strategies can be used for repair and maintenance of soil slides and rockfalls. The specific strategy selected will likely depend on the types of slide and rockfall problems that are present within a region, upon the relative prominence of slides and rockfalls compared to other maintenance issues, upon the current state of other transportation assets such as bridges and pavements, and upon the availability of funding among other factors. As stated earlier, it is likely that different agencies and potentially even different groups within a single agency (e.g. districts) may be best served by adopting different management strategies.

Two general management strategies that are generally suitable for management of landslides and rockfall are:

- Risk-based strategy – strategy where sites are repaired in decreasing order of risk to the traveling public so that highest risk sites are repaired first
- Cost-benefit strategy – strategy where sites are repaired in decreasing order of a cost-benefit ratio that indicates where the most economical benefits can be achieved.

In some case, these strategies may produce similar ranking, but it is not necessarily the case. Arguments for both strategies are provided in the following.

6.4.1 Minimization of Risk Strategy

Perhaps the most common strategy for management of soil slides and rockfalls is to prioritize sites based exclusively on risk to traveling public. Sites would then be repaired in order from highest risk sites to lowest risk sites, with as many sites being repaired as possible within available funding. In subsequent years, sites would be reprioritized with new high risk sites being added, and repaired sites being demoted in the priority list and the process repeated.

Given that most slide and rockfall management systems deal only with problem conditions (as opposed to all slopes), there is a lot of motivation for adopting this strategy because safety concerns are generally considered paramount for transportation agencies. The strategy generally results in high risk sites being quickly addressed. This approach is also quite defendable to the public. The approach has several limitations however, including:

- Because many high risk sites require costly repairs, this strategy can lead to relatively few sites being repaired in a given year which can lead to higher overall risks because more sites may have intermediate risk. In other words, the strategy does not necessarily lead to achieving the lowest overall risk across the transportation system.
- If all repaired sites are "fully repaired" (i.e. such that risks are dramatically reduced), the strategy can lead to the situation where relatively few sites are in excellent condition while numerous others maintain a relatively high risk.
- The strategy can lead to ignoring or avoiding problems with intermediate or future risks that could potentially be remedied more cost effectively with proactive action prior to conditions worsening.

Despite these limitations however, the strategy is a logical one that is easily defended to the public and is likely to remain popular. This strategy is generally best suited for agencies that have a large number of sites with high risk to the traveling public (e.g. agencies with numerous large landslide and rockfall problems).

6.4.2 Cost-Benefit Strategy

An alternative strategy, referred to here as the "cost-benefit strategy", is to prioritize sites according to criteria that measure the relative benefits of maintenance and repair expenditures. This strategy seeks to maximize the return on dollars spent for maintenance and repair. With this strategy, the ratio of expected maintenance costs over some period are compared to estimated costs for complete repair of the problem (generally presumed to be a "permanent fix"), and sites with the greatest benefits are given priority for maintenance and repair. This strategy is also defendable as it seeks to make the best use of tax dollars for maintenance and repair. However, since risk level is not explicitly considered, it could potentially lead to some sites with significant risk being missed in prioritization efforts. Because of this fact, the cost-benefit strategy is generally better suited for agencies that have significant slide/rockfall maintenance and repair problems, but relatively few sites that pose significant risks to the traveling public (e.g. agencies that primarily face "nuisance slide" problems rather than large and potentially catastrophic landslides or rockfalls).

6.4.3 Alternative or Balanced Strategies

For most agencies, neither of the above strategies is completely satisfactory. Most agencies therefore attempt to prioritize sites based on some balance of risk and cost-benefits. This is precisely what is done in the Oregon DOT management system described in Section 6.3. The preliminary rating is based exclusively on risk, but then this rating is modified by a cost-benefit factor so that sites with greater benefits will be rated higher if similar risk levels are found.

The relative importance of risk issues and cost-benefit issues in combined strategies can be altered by simply using different scales for factors related to risk and cost-benefits as dictated by conditions that are typically encountered for a specific agency. In the Oregon DOT system, for example, primary weight is given to risk, while secondary consideration is given to cost-benefit issues. Such weighting is appropriate for areas like the pacific northwest, where high risk conditions are commonplace. In agencies where risks to the driving public are less acute, it may be more reasonable to give preference to cost-benefit issues, with secondary consideration given to risk level.

One issue that is seldom included in current slope management systems is the idea of partial or temporary repairs. Rather, most existing management systems presume that a repair is sufficient to completely resolve the problem condition such that future maintenance and repairs will not be needed (or will at least be greatly reduced). This approach is unique when compared to management systems for other transportation facilities such as pavements and bridges. A common management strategy in pavement and bridge management systems is to try to bring all pavements or bridge up to some minimally accepted level of performance rather than bringing a few pavements/bridges up to “new condition”. For these systems, a much broader range of possibilities for maintenance and repair are generally considered (e.g. from simply sealing cracks, to pavement overlays, to complete replacement of pavement). Such strategies are generally not considered in existing slope management systems, although they could be.

Regardless of the strategy or strategies that are adopted for a given agency, it is important that the slope management system properly reflect the adopted strategy and that it include all information necessary to effectively evaluate alternative sites according to the strategy. Simply put, the slope management system in general, and the rating system component of the slope management system in particular, must be tailored to the specific management strategy selected.

6.5 CHALLENGES FOR IMPLEMENTING SLOPE MANAGEMENT SYSTEMS

While it is clear that use of effective slope management systems can lead to more effective use of limited maintenance funds and better understanding of slope maintenance and repair issues, there are a number of challenges that are present in getting the systems implemented and used to their full effectiveness. These challenges include:

- Substantial effort is required to initially develop a management system that is appropriate for an individual agency;
- Persistent effort must be maintained over time if the systems are to be utilized to their full capabilities
- Technologies utilized for maintaining databases of problem sites and other components of slope management systems tend to change frequently, which makes maintaining up to date systems a challenge,
- Adapting the systems to reflect changes within an agency (e.g. funding levels, personnel changes, policy changes), can also create challenges

As a result of these challenges, previous efforts by different agencies have achieved varying degrees of success. Some agencies that have developed slope management systems have been able to successfully utilize and maintain the systems over time, while other agencies have not been able to maintain efforts necessary to keep the systems useful. The lesson from these experiences is that slope management systems must be promoted and sufficiently funded over time to maintain the quality and effectiveness of system and to adapt the system to changing conditions. Without such dedicated support, implementation of slope management systems is unlikely to be successful in the long run (of course the same is true of bridge and pavement management systems as well).

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CHAPTER 7
STABILIZATION AND REPAIR METHODS

7.1 INTRODUCTION

Soil and rock slope maintenance decisions need to consider the cost of mitigation versus the cost and associated risks of continued maintenance, especially since the cost of slope/rockfall mitigation can be quite high. A benefit/cost analysis can be performed to guide these decisions, but unless the costs of potential injuries, damage and interruption of use to vehicles are included along with the annual maintenance costs, it is unusual to obtain a ratio greater than one. More generally, organizations rely on an on-going maintenance program and allocate construction dollars only towards the most hazardous sites.

Commonly, a combination of maintenance and mitigation is needed since few mitigation measures result in a maintenance-free end product. Mitigation measures are typically grouped into two main categories as either engineered or non-engineered solutions. Engineered solutions are active interventions used to diminish the occurrence or affects of landslides and or rockfalls, while non-engineered solutions are more passive measures that include continued or increased maintenance, warning signs and slope monitoring systems. Many of the described "engineered" solutions can be implemented as part of a slope maintenance program if maintenance personnel possess the necessary specialized skills and equipment to initiate and complete these types of mitigation measures or if a discretionary slope maintenance budget is available to apply to specialty maintenance contracts.

7.2 SOIL SLOPES

There are five basic categories for the repair/stabilization of soil slopes. They include:

1) Modify slope profile
2) Increase the shear strength of the soil
3) Control surface and subsurface water
4) Retaining Structures
5) Relocation to avoid the problem

A summary of these techniques is included on Table 7-1.

7.2.1 Modify Slope Profile

The slope may be altered so the driving forces tending to cause failure are decreased or the resisting forces tending to stabilize the slope are increase. For example, removing the top portion of a fill decreases driving forces. Placing a soil mass at the toe of the failure increases the resisting forces. Re-grading and flattening the slope of a highway is the primary and typically the most economical method used to repair small failures. In the case of fill slopes, this method usually causes minimal disturbance to the existing pavement and fill. In the case of cut slopes, generally sufficient material may be removed to permit the passage of traffic during the repair. Proper signage is essential for the safety of the traveling public as well as the maintenance crew.

TABLE 7-1 SUMMARY OF SOIL SLOPE REPAIR/STABILIZATION TECHNIQUES

MITIGATION MEASURE	DESCRIPTION/PURPOSE	LIMITATIONS
ENGINEERED MEASURES		
1. MODIFY SLOPE PROFILE		
Flatten Slope	The slope may be altered so the driving forces tending to cause failure are decreased or the resisting forces tending to stabilize the slope are increase. This is the primary method used to repair small failures.	If additional right of way is not available this solution is not practical.
Soil or Rock Berms at toe of slope	Earth and rock berms are used to provide a counterweight in the toe area of a failed slope. Berms usually are applied to correct deep rotational failures (which usually occur in cohesive soils, clays and silts).	If additional right of way is not available this solution is not practical.
Shear Keys	Shear Keys or shear trenches provide increases shear resistance to failure and may also serve as a subsurface drain.	Since excavation of the trench may be near the toe of the failure, stability of the standing embankment is critical. Excavation of the trench should not be performed in segments exceeding approximately 50 feet. Immediately backfill each 50 foot segment before excavating the next 50 foot segment.
Benching	Benching is appropriately applied to steep slopes in weathered rock where flattening is difficult and sloughing occurs.	To highly overconsolidated soils and weathered rock.
2. INCREASE SHEAR STRENGTH		
Mechanically stabilize - compaction	Compaction is a means of reducing the volume of a soil mass by application of loads, such as rolling, tamping or vibrating. Compaction generally increases the shear strength of the soil and reduces the permeability of the soil.	Compaction equipment must be selected based on the type of soil to be compacted.
Mechanically stabilize – densification and drainage	Densification and drainage of soils at depth may be achieved through a suit of ground improvement techniques (i.e., Vibro-replacement, Vibro-compaction, deep dynamic densification, wick drains, sand drains and surcharge loading, etc.).	These techniques require a specialty contractor and an engineered design.
Chemically stabilize - lime	Lime stabilization is typically applicable for plastic clays and silts and dirty sands. To be effective the plasticity index of the soil should be greater than or equal to 10%.	Lime is a caustic material which requires special consideration during placement and mixing.
Chemically stabilize - flyash	Flyash treatment has been used successfully with sands, and gravels. Flyash may be combined with lime or cement. Silts are generally considered to be the most suitable fine grained soil for lime flyash treatment.	Generally when used alone flyash is not appropriate for use in fine grained soils.
Chemically stabilize - cement	Cement stabilization is predominately used for granular soils. Soils with less than 35% passing the number 200 sieve and a plasticity index less than 20% are suitable for cement stabilization.	Not suitable for fine grained soils.

TABLE 7-1 (CONTINUED) SUMMARY OF SOIL SLOPE REPAIR/STABILIZATION TECHNIQUES

3. DRAINAGE		
SURFACE WATER		
Surface Ditches	Open surface ditches may be used to reduce ponding and to control runoff within the distressed area.	Caution should be used when using open ditches within the slide area as this could aggravate the situation and contribute to further distress.
Seal joints, cracks, fissures	Surface water flowing into an open crack may significantly affect the stability of a highway slope failure in progress. Sealing these cracks is extremely important.	Continued moved of the failed mass will require resealing periodically.
Vegetation	Vegetation may be used to control erosion of unstable masses. Vegetation may also help prevent small shallow sloughs from occurring.	
SUBSURFACE DRAINAGE		
Horizontal Drains	Horizontal drains are a good early treatment to intercept subsurface water at side-hill fill and cut slopes. This technique is often the most economical drainage measure and is regularly used where deep failures have occurred.	Drains may be difficult to install when boulders are present. Drains may clog.
Blanket Drains	The blanket consists of pervious drainage material wrapped in a geotextile filter and are used at the interface between the existing soils and the reconstructed soil mass to facilitate drainage at this interface.	Drainage blankets placed along the back slope of the interface between new and existing fill will reduce the friction at the interface which could cause additional movement.
4. RETAINING STRUCTURES		
Walls, Soil nailing, etc.	Earth retaining structures have been used extensively to increase the stability of slopes and embankments. However, engineering analysis and design are required for the successful use of this technology.	This technology should not be implemented without design assistance from the geotechnical group.
NON ENGINEERED MEASURES		
5. AVOIDANCE		
Tunnels	Avoids slope hazards by moving the roadway inside the rock mass away from external rockfall sources.	Hazards associated with traffic in confined space. Long tunnels require lighting and special ventilation. Expensive.
Realignment	Full road realignment or facility relocation to move away from rockfall area.	Often the old road must be maintained for existing accesses. Commonly there is limited space for this option. Expensive.
Elevated Structures	Used to span the anticipated rockfall paths allowing rockfalls to pass beneath.	The structure must completely span the active area to avoid being damaged by rockfalls. Expensive.
Monitoring	The use of instruments to detect incipient rockfalls.	Lead time to events can be short.

Slope flattening is often easier to construct and less expensive than other remedial methods. However, if additional right of way is required, the cost may be prohibitive. Also, the time delay to acquire the right of way may contribute to an aggravation of the instability. When a cut slope or a slope of the original ground under a fill is very steep, it may be impractical to flatten the slope.

Slope flattening is normally used to correct deep rotational failures in fine grained soils (i.e., clays) (Figure 7-1). Typically the failed slope is regarded so the new slope is 3 or 4 horizontal to 1 vertical. The slope should not be regarded to the original slope angle, since the slope failed at this angle re-grading to the same angle will usually insure that a failure will occur again. Since failure occurred, the soil in the failure zone is weaker than the soil above and below the failure plane.

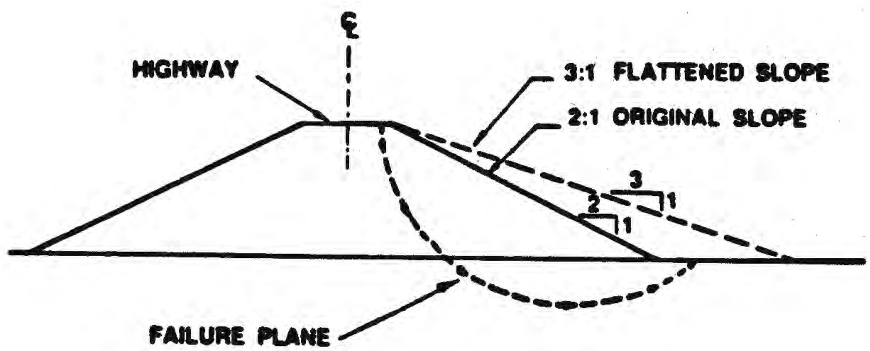


Figure 7-1 Cross Section Illustrating the Concept of Flattening a Slope

Slope flattening also is useful for repairing small slides where the slope in the toe area is steep (i.e., 2 horizontal to 1 vertical or steeper) (Figure 7-2). The over steepened slope may be the result of erosion or may have been constructed that way originally.

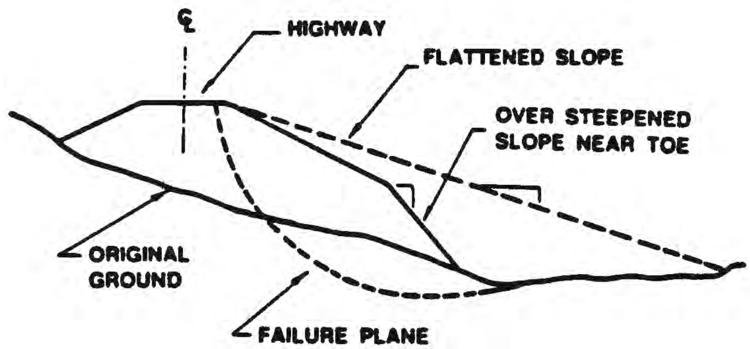


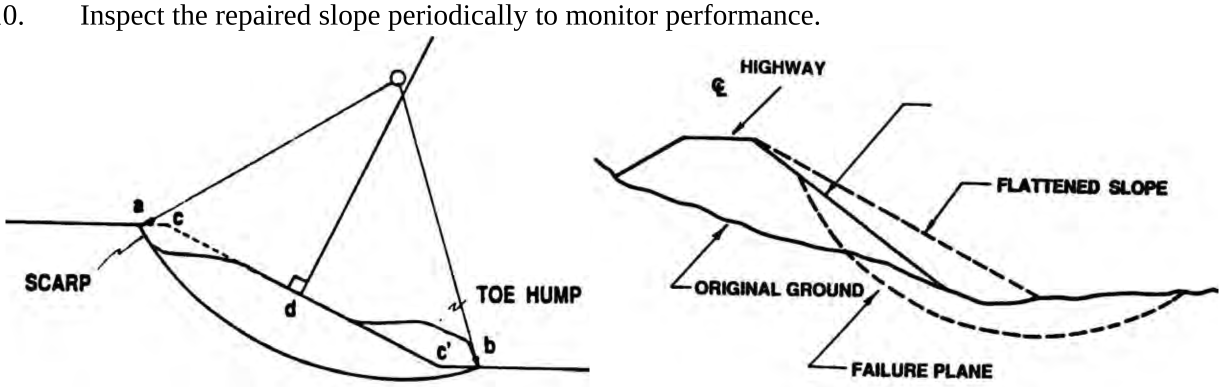
Figure 7-2 Over Steepened Slope near Toe and Use of Flattened Slope

7.2.1.1 Basic Steps for Flattening Slopes

The basic steps that should be following when repairing a failed slope by flattening the slope face are provided below:

1. Obtain slope profile when feasible. Locate the failure scarp at the top of the failed mass (Figure 7-3A, point a) and the bulge at the toe of the slope (point B), when the slope profile is obtained. On a plotted cross section, sketch a circular arc through these two points and any other points know or estimated to be on the failure surface. It is this arc that approximates the location of the failure plane. The circular failure arc that passes through these two points (a and b) is located by trial and error. The center of the circle is moved around and the radius of the circle is varied until a circle is located that passes through the two failure points.

2. Divide the slope line segment cc’ into two equal parts. Construct a perpendicular line at point d, the center of line cc’. The center of the circle, typically, is located slightly above the line perpendicular to the slope.
3. Determine flattened slope angle. Generally, a 3 or 4 horizontal to 1 vertical slope may be used. Use the flatter slopes for the more clayey or silty slopes. Make sure that the flattened slope covers the toe area of the failed slope. When the material of the flattened slope is placed as shown in Figure 7-3B most of the additional weight is to the left of the vertical line through the center of the circular failure arc. This adds to the driving forces and will further destabilize the slope.
4. Look on site (within the right of way) for suitable borrow material. Clean, durable stone, such as limestone, sandstone or washed bank run gravel, is preferred. However, if these higher quality material are not available, use what is available, except for plastic clays and organic soils. Waste materials from other slope failures, ditch cleaning, slope cleaning, etc. may be used. It is good policy during ditch and slope cleaning to stockpile waste materials for future use. Stockpiles might be located close to highway sections showing signs of distress as determined from the inventory.
5. Remove failed material partially or completely (Cut slightly below the failure surface to remove as much soil as possible). Suggested limits of excavation are shown in Figure 7-4. Caution should be used when removing failed material as the process of removal may lead to a larger failure. Work quickly and around the clock if necessary to place material back at the toe of the failure. If practical, do not remove more than 50 to 100 lineal feet of the failed material at a time (this is about the minimum distance that equipment can operate). Replace the material immediately.
6. If the material removed is wet (i.e., equipment sinks into the soil), then dry the material by grading back and forth or using a plow or disc until the water content is about equal to the plastic limit. Adding a small amount of hydrated or quick lime when available can aid in drying the soil and improve workability. This is effectively mainly in clayey soils (i.e., soils that stick to your hands). Use two to six percent lime by weight of failed mass
7. If the material removed is wet, place a drainage blanket on the lower portion of the distressed area when soil is used to construct the flattened slope. A drainage blanket is a relatively thin course of free-draining material and is usually 1 to 3 feet thick. The drainage blanket should be protected from clogging by enclosing it with a geotextile filter, or by designing the blanket as a filter. Another internal drainage option is shown in Figure 7-6. After excavating the unstable material trenches are excavated into the slope at approximate 5 to 10 foot intervals. The sides and bottom of the trench are lined with geotextile and backfilled with free-draining stone.
8. Regrade and place the flattened slope against the drainage blanket.
9. Reseed the flattened slope.
10. Inspect the repaired slope periodically to monitor performance.



(a) (b)

Figure 7-3 Method for Approximating the Failure Surface and Position of the Flattened Slope

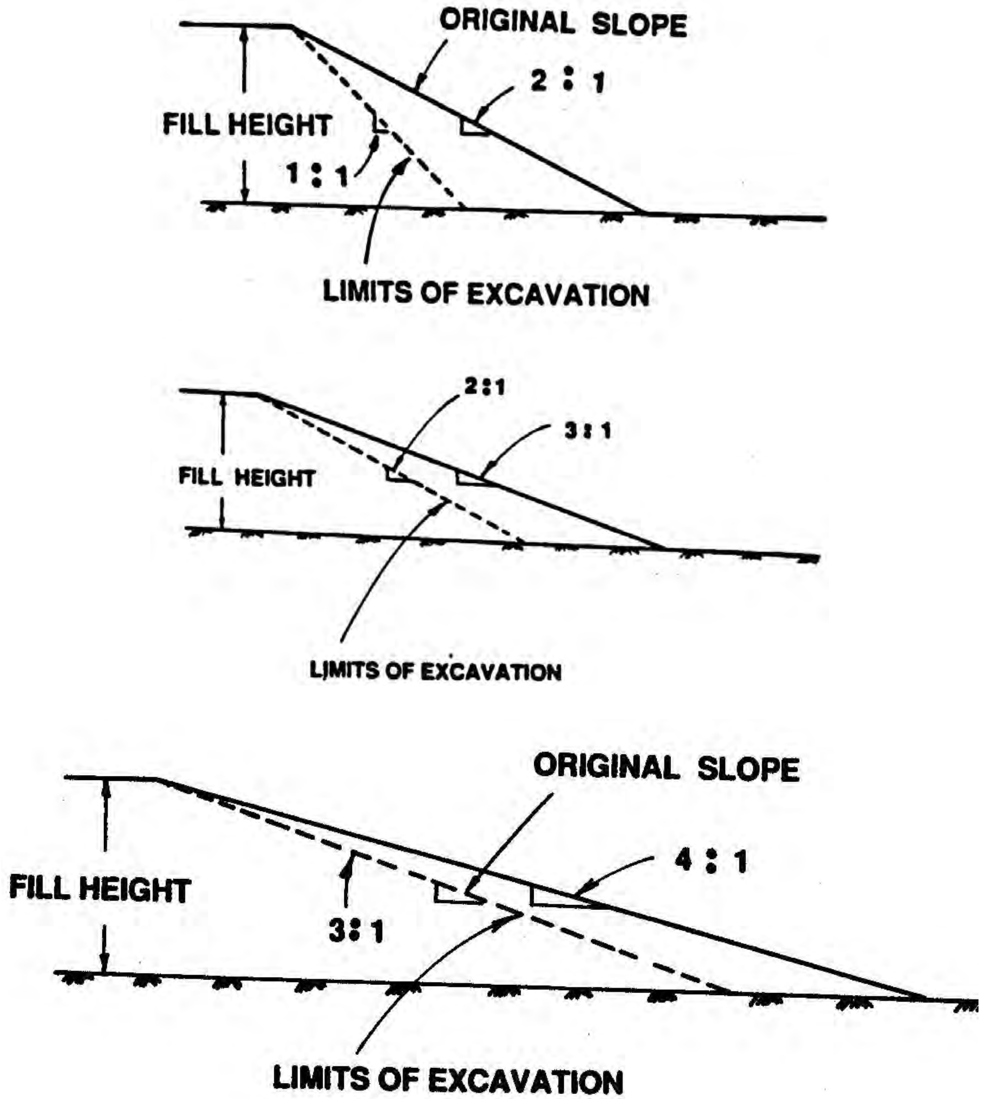


Figure 7-4 Limits of Excavation

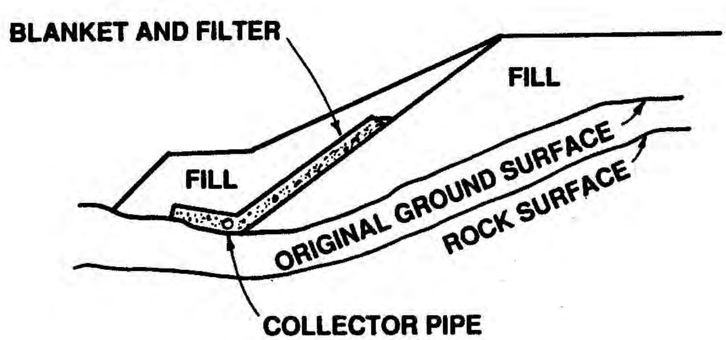


Figure 7-5 Drainage Blanket

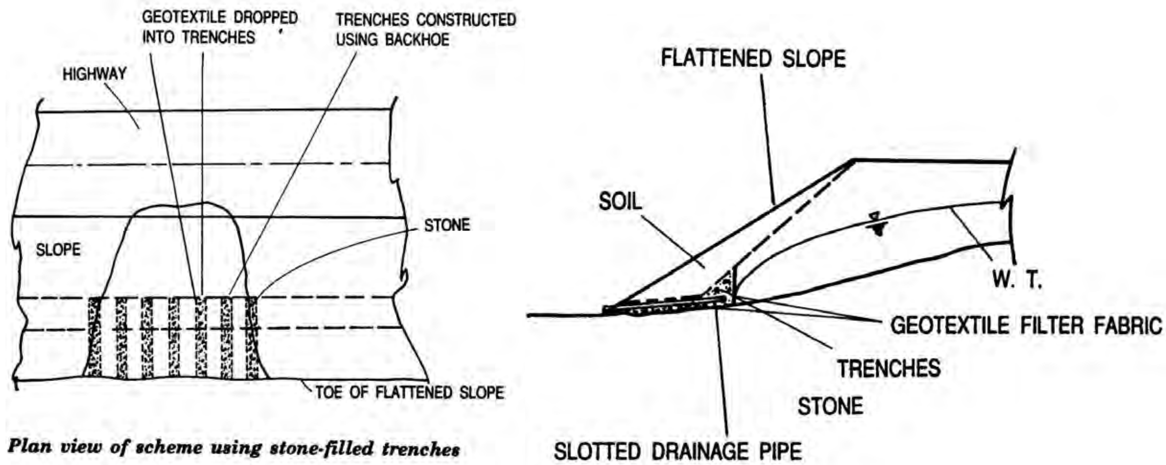


Figure 7-6 Slotted Drainage System

7.2.1.2 Earth and Rock Berms

Earth and rock berms are used to provide a counterweight in the toe area of a failed slope. Berms usually are applied to correct deep rotational failures (which usually occur in cohesive soils, clays and silts). Also berms are used to repair small slides where the toe area of a slope may be over steepened as a result of erosion or poor construction. Small toe failures that may lead to larger failures, may be corrected by earth or rock berms (Figure 7-7).

Typically, the volume of the berm should be approximately one quarter to one half of the unstable soil mass and should extend beyond the toe area of the failure. Caution should be exercised in placement of the berm. Steps involved in construction the berm are essentially the same as those in constructing flattened slopes. Caution should be exercised when the berm must be constructed on soft soils since there is the danger a failure may occur in the berm. The factor of safety of a slope with a berm is usually higher that that of a flattened slope without a berm.

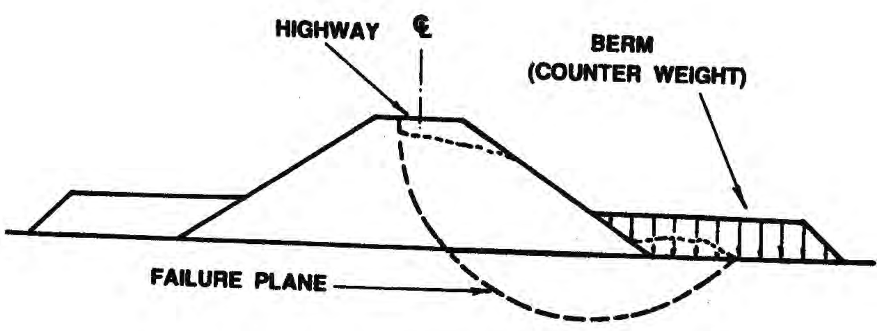


Figure 7-7 Use of Berm as a Counterweight

7.2.1.3 Shear Keys

Shear Keys or shear trenches provide increased shear resistance to failure and may also serve as a subsurface drain. A shear key is frequently a good supplement to flattened slopes and berms. Shear keys should extend the entire length of the failure, and should be keyed into firm competent soil or rock. Course non-degradable rock should be used to backfill the shear trench. The rock fill should not have more than 5% passing the number 200 sieve when used as a drainage course.

Trench back slopes may usually be constructed between 1 to 1.5 vertical to 1 horizontal. Since excavation of the trench may be near the toe of the failure, stability of the standing embankment is critical. It is best to construct shear keys during the dry season of the year. Excavation of the trench should not be performed in segments exceeding approximately 50 feet. Immediately backfill each 50 foot segment before excavating the next 50 foot segment.

7.2.1.4 Benching

Benching is appropriately applied to steep slopes in weathered rock where flattening is difficult and sloughing occurs as shown in Figure 7-8. Benching helps to control erosion and catch debris of small slides. The benches should be slopes so that runoff is collected and conveyed from the slide area.

7.2.1.5 Increase Shear Strength

There are various methods of stabilization that may be used to increase the shear strength of the soils. These methods may be divided into two groups, mechanical stabilization and chemical stabilization. Mechanical stabilization is achieved through either compaction or densification and drainage. Chemical stabilization consists of lime, flyash and cement stabilization.

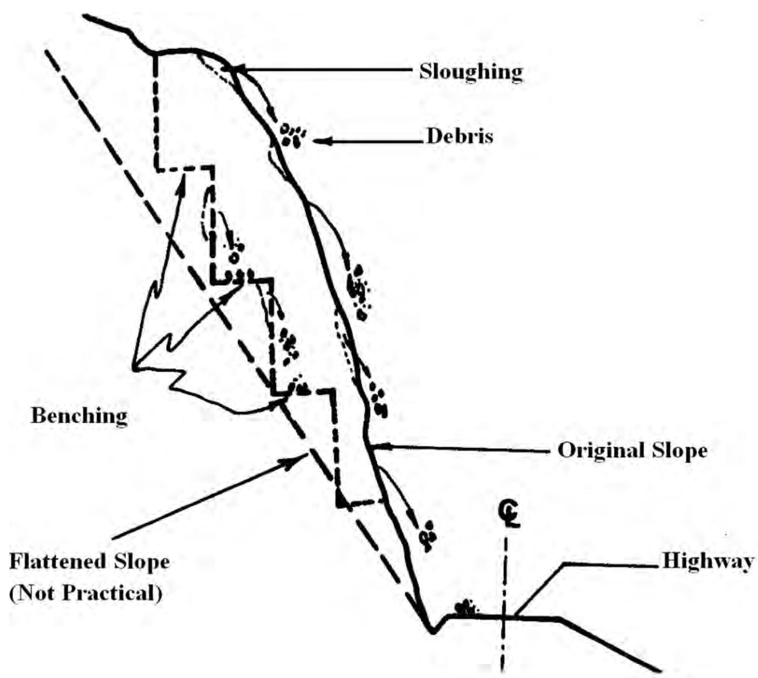


Figure 7-8 Benching a Steep Slope

7.2.1.6 Mechanical Stabilization

Compaction

Compaction is a means of reducing the volume of a soil mass by application of loads, such as rolling, tamping or vibrating. When a soil is compacted, air is expelled from the soil mass without significantly changing the amount of water in the soil. When compacting a soil the following changes generally occur:

- Strength increases
- Density increases
- Permeability decreases
- Compressibility decreases
- Swell potential decreases
- Shrinkage decreases

Compaction is generally used in most all methods for repair of highway slope failures.

Densification and Drainage

Densification and drainage of soils at depth may be achieved through a suit of ground improvement techniques (i.e., Vibro-replacement, Vibro-compaction, deep dynamic densification, wick drains, sand drains and surcharge loading, etc.) that require a specialty contractor and an engineered design. These methods are generally beyond the scope of this manual.

7.2.1.7 Chemical Stabilization

Chemical stabilization is the altering of soil properties by adding certain chemical additives to the soil. The addition of chemicals creates a cementing action, in some cases, that binds soil particles together to increase their shear strength. Chemical stabilization may be used to repair small slides in the following situations.

Working Platform at the toe of the slope – In this situation, a chemical additive such as quick lime may be used to dry wet cohesive soils at the toe of the slide. A working platform is constructed using the in-place wet soils mixed with quicklime.

Flattened slope and/or berm built entirely of partially of soil-chemical mixture - Depending on the position of the failure plane, the soil –chemical mixture could be placed in the lower portion of the reconstructed slope or the entire slope could be reconstructed with soil chemical mixture.

Grouting is one form of chemical stabilization. However, grouting typically involves specialty contractors and design from the geotechnical group and is therefore not covered in this manual.

Lime Stabilization - Lime included all classes of lime and hydrated lime. Lime stabilization is typically applicable for plastic clays and silts and dirty sands. To be effective the plasticity index of the soil should be greater than or equal to 10%. As a general guide, soils that classify as A-4 through A-7, according to AASHTO classification system or CH, CL, MH, ML, SC, SM, SW-SC, SP-SC, and SM-SC according to the Unified Classification system are potentially susceptible to stabilization with lime. The percent lime (based on dry weight of soil) ranges from 3 to 8%.

Hydrated lime or quicklime is especially applicable to situation where clayey soils of failed slopes are saturated. In cases where berms or flattened slopes may be used and the foundation soils and the failed soils are wet, the addition of 3 to 8% lime aids in drying the wet soils quickly. Lime accelerates the breakup of clay clods during mixing, makes the clayey soil friable, improves workability, and aid compaction. Since the strength of a soil lime mixture is generally much larger than the soil without lime, soil lime mixtures may be used in a variety of ways with other repair methods.

There are numerous tests that may be conducted to estimate the percent lime required to stabilize a soil. They include the following:

- pH Test- Soil specimens are mixed with different percentages of lime. The pH value of each lime soil mixture is determined and platted as shown in Figure 7-9. The point at which the pH value does not increase significantly with additional amounts of lime is the optimum percent of lime to be added to the soil. These tests usually may be performed in less than a day.
- Unconfined Compression triaxial tests – Soil specimens are molded using different percentages of lime. The specimens are cured either 7 or 14 days. Unconfined compression strength tests are performed and results plotted as a function of percent lime. The optimum lime content is that percentage at which there is no significant increase in shear strength. This method is generally more reliable then the pH test. Generally, about one percent of lime is added to the optimum percent to account for losses of lime during mixing in the field.
- National Lime Association method (Liquid Limit Method) – Determine the liquid limit (LL) of the soil. If the liquid limit is less than 50%, then the optimum percent of lime 12% of the LL (Percent Lime = .12LL). If the LL is greater than 50%, the percent lime usually required for stabilization is between 5.6 and 6%.

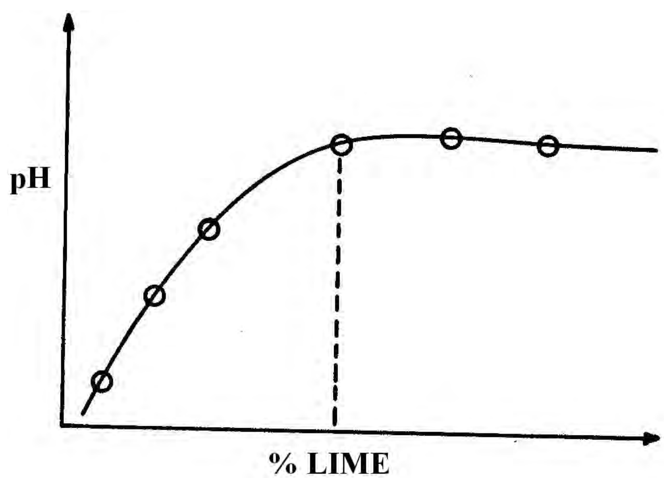


Figure 7-9 Relationship Between pH and Percent Lime

The generalized steps in lime stabilization are presented below:

1. Determine percentage of lime required – For quick estimates on small slides, use National Lime Association method.
2. Scarify and pulverize the soil – a grader and or disc harrow may be used to initially scarify and pulverize the soil.
3. Spread lime- Special mechanical spreaders are typically used to place the lime. Lime may be spread either dry or in slurry form. Granular quicklime may be spread by dump trucks.

4. Water- Water is spread on the layer of lime using a water truck (if necessary).
5. Mixing – Rotary mixing is the preferred method. ON small jobs, blade mixing or disc harrow mixing may be more practicable.
6. Curing – lime soil mixtures should cure from 12 to 48 hours. The curing periods aids in breaking down clods of clay.
7. Final mixing and pulverization – Grader scarifier is used to loosen the layer and rotary mixer is used to perform final mixing.
8. Compaction – The lift of soil lime mixture should be compacted to 95% of standard density.

Flyash - Flyash is a by product of coal burning. Generally when used alone flyash is not appropriate for use in fine grained soils. However, flyash treatment has been used successfully with sands, and gravels. Flyash may be combined with lime or cement. Silts are generally considered to be the most suitable fine grained soil for lime flyash treatment.

Cement - Cement stabilization is predominately used for granular soils. Soils with less than 35% passing the number 200 sieve and a plasticity index less than 20% are suitable for cement stabilization. Steps involved in constructing soil cement stabilized soils are similar to those described for lime stabilized soils.

Drainage - Water is the most common trigger to failures of highway slopes. It is therefore, extremely important that the control of surface and subsurface water is address in any remediation program. There are a variety of surface and subsurface measures that may be used to control water. Surface treatments include surface ditches, seal joints, cracks and fissure, re-grade slope to eliminate ponding, and vegetation. Subsurface drainage measures include horizontal drains, and drainage blankets.

7.2.1.8 Surface Drainage

Surface Ditches – Open surface ditches may be used to reduce ponding and to control runoff within the distressed area. Open ditches also may be used to surround a slide area to prevent the entry of water into the unstable mass. Caution should be used when using open ditches within the slide area as this could aggravate the situation and contribute to further distress. The ditches should be layed out to convey the water to a point outside the distressed area. Surface ditching may be as simple as cutting or making an unlined channel to constructing a channel lined with clay, rock of geosynthetic.

Small streams of surface water running into the distresses area or onto the failed slope may be diverted by cutting an open drainage ditch around the head of the slide so that runoff is diverted away from the slide. A French drain (Figure 7-10) may be used to intercept subsurface water and convey it away from the distressed area.

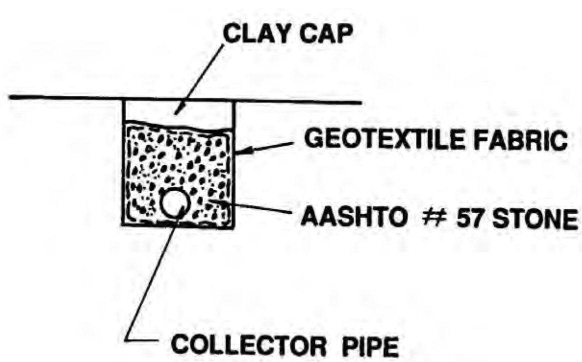


Figure 7-10 French Drain

Seal Joints, Cracks and Fissures – Surface water flowing into an open crack may significantly affect the stability of a highway slope failure in progress. Tension cracks develop at the head of slopes because soil cannot withstand large tensile forces over an extended period of time. Tension cracks may extend several feet vertically into the slope. The horizontal driving force exerted by water in a tension crack can be substantial. An 18 foot deep crack filed with water will exert approximately 10,000 pounds of force per lineal foot against the failure mass. It is, therefore, essential to seal all joints cracks and fissures observed in the distressed area of the slide. In emergency situation, use plastic sheeting to cover open cracks and joints until the maintenance crew can provide a more permanent solution. Cracks may be sealed with asphalt and they should be inspected periodically and repaired again as necessary. Reopening of the repaired crack is a sign that the movement of the slope is continuing.

Vegetation – Vegetation may be used to control erosion of unstable masses. This method provides a protective surface on the slope and is used as a means of lining ditches. Roots of plants and grasses absorb moisture and aid in preventing infiltration of water into the unstable mass. **Seeding and the establishment of vegetation always should be used in the reconstruction of slope failures.** In some cut slopes, seeding may not be practical because of the steepness and the nature of the material on the slope. On fill slopes, vegetation may also help prevent small shallow sloughs from occurring. Roots form a matrix or mat that acts as soil reinforcement holding the top few feet of soil in place.

7.2.1.9 Subsurface Drainage

Horizontal Drains – Horizontal drains are a good early treatment to intercept subsurface water at side-hill fill and cut slopes. This technique is often the most economical drainage measure and is regularly used where deep failures have occurred.

Installation of horizontal drains typically consists of inserting a slotted PVC pipe or perforated metal pipe (typically 1.5 to 2.0 inches in diameter) into cased horizontally drilled holes. The pipe is usually placed on a 3 to 20 % grade to allow gravity drainage (Figure 7-11). The casing prevents collapse of the hole.

Horizontal drains usually are positioned in the lower portion of the slope and below the toe of the slope in natural ground. Frequently, drains are installed in rows. In difficult terrain, they may be installed in a fan shaped pattern. Usually, at least tow rows of drains are needed to be effective. To prevent clogging by fine materials and roots the drains are wrapped with a geotextile filter. Routine maintenance and monitoring of horizontal drains should be performed periodically.

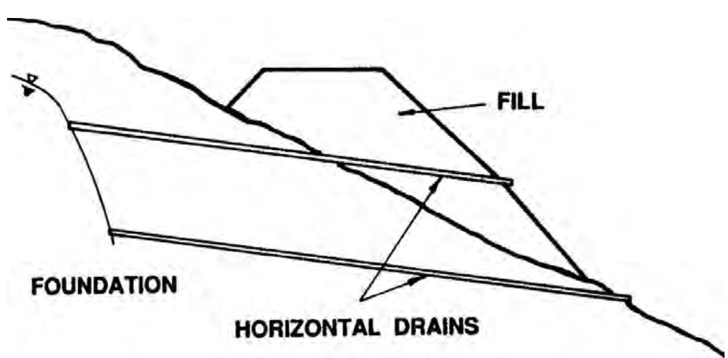


Figure 7-11 Horizontal Drains

Blanket Drain – Drainage blankets are used to collect and control ground water. The blanket consists of pervious drainage material. Sound, durable rock fill should be used to construct the blanket. The rock fill should contain less than 5% material passing the number 200 sieve. Drainage blankets are normally placed directly on the existing soil. Fines from the soil may eventually infiltrate the rock/gravel fill and clog the drain. Small amounts of fines, as low as 5% may significantly reduce the permeability or capacity of the drainage blanket. An alternative to the conventional drainage blanket is a geosynthetic drainage composite consisting of a geotextile filter sandwiching a plastic drainage core. The drainage blanket should be wrapped with a geotextile filter. Figure 7-12 shows the typical location of the drainage blanket.

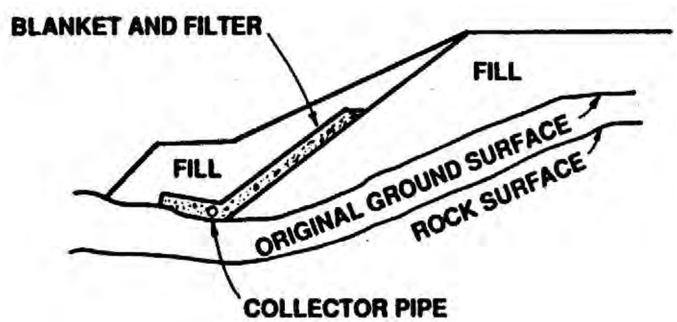


Figure 7-12 Typical Drainage Blanket

7.2.2 Retaining Structures

Retaining structures are used to stabilize slope failures by increasing the forces tending to resist the sliding mass. Generally, the retaining structure is placed at the toe of the distressed area or slope failure Figure 7-13). Types of structures used in highway slide corrections include:

- Cast in place concrete cantilever walls
- Crib Walls
- Gabion Walls
- MSE Walls
- Tieback Walls
- Reinforced Slopes
- Piles and Caissons

The use of retaining structures will require an engineered design solution. Without the proper design the retaining structure may be inadequate to withstand the driving forces from the slide mass. Therefore, it is recommended that earth retaining structures are not used without design assistance provided by a geotechnical engineer.

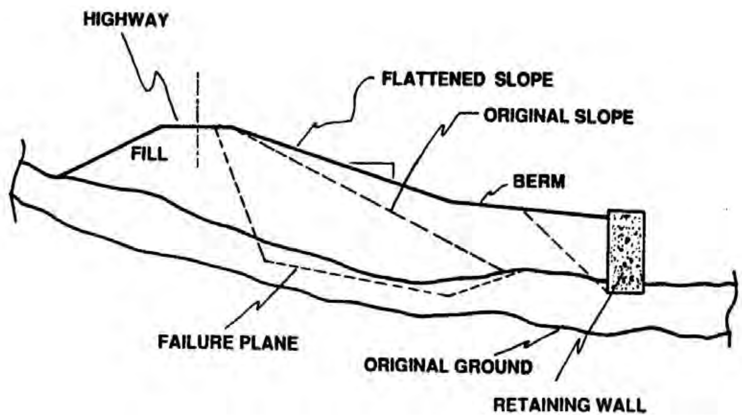


Figure 7-13 Earth Retaining Structure used to Stabilize a Slope Failure

7.3 ROCK SLOPES

As with other types of slopes adjacent to highways, rock slopes require maintenance. The issue of rockfall affecting highways has been a recognized concern for decades. The United States highway systems has evolved from dirt tracks to paved roads over many years; and, in mountainous areas, these roads were commonly located at the base of steep natural slopes. In these locations and in part because of past construction practices, it is very common that the natural and manmade slopes produce rockfall.

Rockfall is a natural occurrence and the extent of this is evident from the extensive accumulations of colluvial debris found at the base of slopes and in adjacent streams and rivers. The size of the debris ranges from sand to in some cases house-sized boulders. Also, until quite recently, it was standard construction practice to use very aggressive construction techniques (over blasting and/or ripping with heavy equipment) to build highway cuts into these slopes. These practices facilitated rock excavation, but they also damaged the rock cuts and left them more prone to rockfall.

The results of this are many thousands of rockfall problem sites throughout North America. When combined with the significant increases in populations and numbers of automobiles on the roads today, this has resulted in an unfortunate increase in the occurrence of vehicles striking a rock in the road or being struck by a falling rock. The efforts required to keep roadways clear and improve safety can place significant demands on maintenance personnel and budgets. At some point, the organization may elect to invest in additional rockfall mitigation measures to address the rockfall problem through either a maintenance or construction contract and allow rockfall maintenance to be more of a scheduled activity rather than an emergency response.

Each organization will plan for these expenditures and respond to safety responsibilities based on established policies. Regardless of what guides the decision to augment the typical on-going slope maintenance activities with additional capital expenditure, the project must compliment the long-term slope maintenance goals. Rockfall mitigation, whether intended to reduce, control or avoid rockfalls, can vary from a modest safety improvement to a complete correction of the problem.

On-going maintenance measures include road patrols, ditch cleaning and slope monitoring along with proper signage to warn highway users of hazardous driving conditions. Slope and fallout area modifications and improvements are more involved and can require more planning and design and in some cases specialty contractors to implement. However, depending on the extent of the modifications and the staff resources and capabilities available, some enhancements can be implemented by the agency. In either case, several of the described solutions can be implemented as part of a slope maintenance program or constructed through a maintenance contract if the required discretionary maintenance budget is available. A summary of the rockfall maintenance practices and mitigation measures including the limitations associated with each is included in Table 7-2.

7.3.1 Maintenance Practices

On-going maintenance practices that include activities to keep roadside ditches cleaned to improve ditch performance and roadways clear to remove roadway hazards, and slope monitoring to identify incipient as well as warning signs should be considered in any rock slope maintenance program. These techniques are summarized below.

Warning Signs - Warning signs are an inexpensive but important measure. Their presence has played a significant role during several rockfall litigation actions. Where possible, signs with flashing lights that attract attention and overcome a driver's tendency to ignore warning devices should be used.

Maintenance Road Patrols - Road patrols can be intermittent or continuous depending on the situation. Rockfalls are removed from the road and, if rockfall conditions worsen, the road can be closed. In addition, maintenance can observe roadside ditches and clean them out as necessary to maintain their effectiveness in capturing rockfalls.

Monitoring - Slope monitoring uses surveying (both traditional, and terrestrial and aerial Lidar techniques) as well as installed geotechnical instruments to detect block movements that may indicate incipient failure. Automated road closures (drop gates) and/or notification communications can be included with electronic monitoring systems.

7.3.2 Rockfall Mitigation

Rockfall mitigations are designed to reduce the potential for or the affects of rockfalls. The three main categories of these types of solutions include stabilization, protection and avoidance measures. The goal of stabilization is to reduce the potential for rockfalls to initiate. Protection measures are intended to control/capture rockfalls once they fall. Avoidance measures are an attempt to move the road or facility away from the rockfall problem without necessarily doing anything to reduce the rockfall potential. It is not uncommon for a rockfall reduction program to include some elements from all three mitigation categories.

Stabilization - Stabilization measures are changes to the slope or individual rock blocks to reduce the likelihood of rocks falling. This can be accomplished by securing rocks in place using rock bolts or dowels, removing loose rocks in a controlled manner (scaling), or changing the slope configuration by reshaping or flattening the slope.

Protection - Protection techniques are used to control rockfalls once they initiate. The goal is to change the rockfall behavior by dissipating rockfall energy or by intercepting and capturing rockfalls in order to restrict them from entering a prescribed area such as road or other facility. Protection options include building or reshaping catchment areas/roadside ditches, or installing draped mesh, modified catch fences, rigid and flexible barriers, or a rock shed.

Avoidance - Avoidance measures rely on a relocation or realignment of a road or facility to move them away from the rockfall source. These techniques can require significant investment and right-of-way, but they may be the only way to completely eliminate rockfall risks. Options can vary from a slight shift in the road to a completely new roadway alignment using tunnels or elevated structures.

TABLE 7-2 ROCK SLOPE AND ROCKFALL MITIGATION ALTERNATIVES

MITIGATION MEASURE	DESCRIPTION/PURPOSE	LIMITATIONS
	MAINTENANCE PRACTICES	
Warning Signs	Alerts users to the potential for rockfall and for fallen rocks to be encountered on the roadway.	Users become accustomed to their presence and ignore warnings.
Road Patrols	Roadway inspections during periods of higher rockfall activity to find and remove fallen rocks.	May encompass too many miles of road to manage in a timely manner.
Ditch Cleaning	Removal of accumulated rockfall debris from ditches to maintain their effectiveness in capturing rocks.	Temporary measure requiring repeated action by maintenance crews and a suitable disposal site.
Monitoring	The use of instruments to detect incipient rockfalls.	Lead time to events can be short.
	STABILIZATION	
Removal:		
Scaling	Removal of loose rock from slope by means of hand tools and/or mechanical equipment. Commonly used in conjunction with most other design elements.	It is a temporary measure that usually needs to be repeated every two to ten years as the slope face continues to degrade.
Rock Removal/ Blast Scaling	Removal of loose rock or large rock blocks from slope by means of blasting or chemical expanders.	Damage from fly rock and rockfall, and possible undermining or loss of support by key block removal.
Trim Blasting	Used to remove overhanging faces or protruding knobs that may act as launch features on a slope.	Difficulties with drilling, debris containment and safety.
Re-sloping	Cutting the rock slope at a flatter angle to improve slope stability and rockfall trajectories.	May have right-of-way or environmental issues.
Slope Benching	Creating steps in the cut slope to accommodate variable rock layers where differential weathering of softer layers can otherwise undermine the slope.	Increases excavation and right-of-way. Can create a launch feature if accumulated debris is not removed.
Reinforcement:		
Dowels	Untensioned steel bars/bolts installed to increase shear resistance and reinforce a block. Increases normal force friction once block movement occurs.	Passive support requires block movement to develop bolt tension. Slope access difficulties.
Shear Pins	Provides shear support at the leading edge of a dipping rock block or slab using grouted steel bars.	Cast-in-place concrete needed around bars to contact leading edge of block. Access difficulties.
Rock Bolts	Tensioned steel bars/bolts used to increase the normal force friction and shear resistance along potential rock block failure surfaces. Applied in a pattern or in specific block.	Less suitable on slopes comprised of small blocks. Difficult to access slope.
Shotcrete	Pneumatically applied concrete requiring high velocity and proper application to consolidate. Primarily used to halt the on-going loss of support caused by erosion and spalling. Also helps retain small supporting rock blocks.	Reduces slope drainage. Can be unsightly unless sculpted or tinted. Wire mesh or fiber reinforcement required. Needs a minimum two-inch thickness to resist freeze/thaw.

Continued on next page

TABLE 7-2 (CONTINUED) ROCK SLOPE AND ROCKFALL MITIGATION ALTERNATIVES

Buttresses	Provide support to overhanging rock or lateral support to rock face using either earth materials, cast concrete or reinforcing steel.	Height limitations. May form a roadside hazard and be unsightly.
Cable Lashing	Anchored, tensioned cable(s) used to strap a rock block in place. May be used in conjunction with cable nets or wire mesh. Also used as a temporary support during rock bolt/dowel drilling activities.	Due to slope and/or block geometry, typically movement must occur for full cable resistance to develop.
Whalers/Lagging	Anchored beams or steel straps used to hold rock blocks in place between bolts locations. Also used as a temporary measure to provide support during rock bolt/dowel drilling activities.	Unsightly as a permanent application. Movement must occur for full tensioning/resistance to develop.
Anchored wire mesh/cable nets/ high tensile strength steel mesh	A free-draining, pinned/anchored in place nets or mesh. Used to apply an active retention force to retain rocks on a slope.	May form pockets of loose rock as rockfall debris accumulates. Can be difficult to clean out.
Drainage:		
Weep Drains	Reduces water pressures within a slope using horizontal drains or adits. Commonly used in conjunction with other design elements.	Difficult to quantify the need and verify the improvements achieved.
PROTECTION		
Mesh/Cable Nets:		
Slope Protection (Draped Mesh)	Hexagonal wire mesh, cable nets, or high tensile strength steel mesh placed on a slope face to slow erosion, control the descent of falling rocks and restrict them to the catchment area.	Requires a debris collection ditch area. Must consider debris and snow loads on anchors. Typically limited to 4-foot minus rocks.
Suspended Systems (Curtains)	Wire or cable mesh draped by fence posts or suspended across chutes. The fence (impact zone) intercepts and attenuates falling rocks initiating upslope, and directs them beneath the mesh and into the roadside catchment area.	Requires a debris collection ditch area. Must consider debris and snow loads on anchors. Typically limited to 4-foot minus rocks.
Catchment Areas/Sheds:		
Ditches/Berms	A shaped catchment area at the base of a slope used to contain rockfall.	Tall slopes require wide fallout areas. May have right-of-way or environmental issues.
Barriers:		
Rockfall Sheds	A covered structure used to intercept and divert rockfalls.	Expensive, hazards associated with traffic in confined space. Must consider down slope issues.
Rigid Barriers (with or without fence extension)	Rigid barrier walls used to intercept falling rock and restrict them to a prescribed fallout area. Examples include Jersey/GM barriers, guardrails, and other concrete, gabion or MSE walls.	Rigid systems are more prone to damage by higher energy events. Complicates debris cleanout and snow plowing.
Flexible Barriers	Wire ring, high-strength wire mesh or cable net panels with high-energy absorption capacity supported by steel posts and anchor ropes with braking elements. Typically proprietary systems.	Requires room for barrier to deflect during impacts. Must be cleaned out periodically. Complicates snow plowing.
Continued on next page		

TABLE 7-2 (CONTINUED) ROCK SLOPE AND ROCKFALL MITIGATION ALTERNATIVES

AVOIDANCE		
Tunnels	Avoids slope hazards by moving the roadway inside the rock mass away from external rockfall sources.	Hazards associated with traffic in confined space. Long tunnels require lighting and special ventilation. Expensive.
Realignment	Full road realignment or facility relocation to move away from rockfall area.	Often the old road must be maintained for existing accesses. Commonly there is limited space for this option. Expensive.
Elevated Structures	Used to span the anticipated rockfall paths allowing rockfalls to pass beneath.	The structure must completely span the active area to avoid being damaged by rockfalls. Expensive.

7.3.3 Selection Considerations

Organizations responsible for managing rock slopes are faced with expectations and requirements to address rockfall hazards through maintenance efforts and if needed capital expenditures. The selection of which mitigation measures to adopt is highly dependent on agency policies, available budgets, and the effectiveness of the proposed measure relative to slope conditions and the desired safety improvements and reduction in maintenance.

The three types of mitigation alternatives previously described as stabilization, protection and avoidance make up the vast majority of rockfall mitigation work. As discussed below, a relative comparison using commonly considered criteria is provided in Table 7-3 for each of the mitigation measures listed in Table 7-2. The cost comparison shown is very general because construction costs vary significantly within agencies and from region to region depending on the ability to do the work in-house verses contracting out and the variability in local material costs, and the availability of specialized equipment and experienced local contractors.

7.3.4 Design Criteria

Once a site is selected for improvements (refer to the slope management systems described in Chapter 6), various design criteria should be evaluated before settling on a specific mitigation measure. Project goals need to be established. The goals can vary from controlling all possible rockfalls to just addressing the dominant sizes that are more common. To establish attainable project goals, it is important to quantify and clearly understand the scope of the rockfall problem before performing these evaluations and comparisons.

The selection of a particular measure should be based on an evaluation of its effectiveness and durability to contend with the anticipated rockfall volume, size and energies; various constructibility issues; contractor capability and availability; traffic restriction limitations; and construction time and seasonal requirements. In addition, except in extreme emergency situations, the environmental limitations and aesthetic goals would need to be considered as well. At times, the environmental considerations may be the overriding project limitations. Once the design goals and criteria are established and mitigation design concepts are developed, the design and construction costs should be evaluated. The construction cost should be compared to the cost of continued long-term maintenance.

This exercise will not yield one correct answer and it is likely that more than one measure or a combination of measures may be suitable. Experience with rockfall mitigation design and applicability can provide an important check on the reasonableness of this evaluation. Other agency or outside resources may be needed to assist with these decisions and to advance the project.

Effectiveness: The mitigation measures need to match the rockfall conditions. Obviously, it would serve little purpose to construct a measure that would be severely damaged or destroyed by the next rockfall event or one that would do little to reduce maintenance requirements or improve safety. Quantifying rockfall conditions requires information about the anticipated rockfall size or volume per event, trajectory, and recurrence interval. A well-documented rockfall history provided by maintenance can serve as the basis for this evaluation as an excellent indicator of what can be expected in the future. This information is critical to both maintenance and other project development decisions.

A measure's suitability must take into account the planned use of the roadway and the vulnerability of its users to rockfalls. A higher level of mitigation (greater investment) is generally considered appropriate where vulnerability is higher. However, few efforts will eliminate all rockfall risks, and for most projects, a hazard or maintenance reduction is the project goal. Typically the higher the demand for hazard elimination, the higher the construction cost, right-of-way needs, and/or environmental impacts. Some limitations may not allow construction of the most preferred option, but in all cases, the justification for the investment assumes that the end product will be a safer and/or more readily maintained facility.
In general, stabilization and protection measures reduce the affects of rockfalls by limiting their occurrence or by controlling their behavior. These measures are intended to reduce the likelihood of rockfalls reaching the road. Depending on the degree of stabilization or protection, they can vary from relatively inexpensive to costly. Avoidance measures shift or relocate the road to a safer position relative to potential rockfalls, but depending on how much realignment is needed they tend to require a greater investment.

If rockfalls are comprised of relatively small blocks (less than 2 to 3 feet in diameter), most stabilization measures other than re-sloping may be relatively ineffective and protection measures may be more appropriate to consider. Conversely, if expected rockfalls are comprised of larger blocks, stabilization measures may prove a more reasonable means of rockfall mitigation. Because of higher costs and other potential local effects, avoidance measures are usually considered only when any vulnerability is considered too high or more typically where the local conditions specifically accommodate this type of mitigation.

Durability: This criterion reflects how well a particular measure will withstand the rigors of exposure and use and still continue to serve as intended. The improper selection of a measure to match the actual rockfall problem can severely reduce service life, especially with measures that must withstand repeated rockfall impacts. Service life can also be shortened by corrosive environments where the soil pH and resistivity, acid rain or runoff or salt water spray come in contact with measures containing steel. In some applications, improper slope drainage or handling of runoff can lead to early failures. These conditions must be well understood in order to be properly addressed.

Poor construction quality due to a lack of adherence to specifications or inexperience can significantly contribute to a limited service life. Inexperience can lead to misjudging the difficulty in accessing remote slope locations, using improper equipment or procedures. It can also increase the risk of accidents, cause project delays, lead to construction claims, and possibly even to making the rockfall problem worse.

Constructability: Constructability is governed by accessibility to the area; the presence of nearby structures or utilities; the level of risk faced by maintenance or contractor personnel; the availability of specialized materials or rockfall mitigation systems, and specialty contractors and equipment. The measures described in this manual are standard mitigation measures that have been constructed worldwide over many years; yet, each rockfall site is unique and must be evaluated with constructibility in mind.

Worker safety can be affected by slope stability. Temporary stabilization measures may be needed before constructing more permanent design elements. In some cases, high wind areas may preclude the use of cranes, which can limit the access for installing stabilization measures. Adjacent buildings or structures (especially when historically significant), utilities, or protected streams or rivers may interfere with rock scaling or blasting operations. Road surfaces and retaining walls may need added temporary protection or follow-up repairs. If excavation is planned, the width of the excavation must be adequate to accommodate standard construction equipment and avoid narrow cuts that cannot be safely worked.

<u>Traffic Restrictions:</u> Many rockfall mitigation measures cannot be constructed unless traffic control is in place. In fact there is roughly a tenfold increase in the risk of rockfall-related accident during construction operations. Slope scaling and blasting are obvious causes, but any time workers are on the slope, the risk of causing a rockfall are heightened. In some cases, outrigger requirements for a crane can take up all available room.

An adequate detour or the ability to close the road periodically for short periods may be needed. Productive work can be significantly limited when road closures must accommodate both work and road clearing as in the case of slope scaling before traffic can move through the area. This limitation will increase project cost and duration.

<u>Time of Construction:</u> In some case, the duration or timing of construction is an important consideration. This can be especially true in high traffic areas or when environmental requirements limit available construction time in some cases for entire seasons. The seasonal limitations may be associated with in-water work periods or by the need to remain clear of certain areas during nesting periods or when protected fish, plant or animal species are present. Unrestricted local access may be required in tourist areas or other seasonally open areas when traffic restrictions could unacceptably impact the local economy.

<u>Environmental Considerations and Aesthetics:</u> Many potential rockfall projects are located in mountainous areas or steeply incised river valleys. These areas are commonly remote and may contain certain protected species of plants, animals, birds or fish. Some may also contain historically significant features that cannot be disturbed. These conditions may restrict the types of rockfall mitigation measures that will be acceptable to regulatory agencies or can be built within prescribed environmental or aesthetic limitations.

If necessary, certain measures can be modified to compliment the environment and be less obvious to road users. Modifications can vary from coloring draped mesh to match the ground color to more extreme efforts such as sculpting and tinting shotcrete to match the texture and appearance of the native rock. These requirements generally increase cost, sometimes significantly, but they may be the only way to gain acceptance for their use. Other items that should be considered are the ability to access upper slope areas, disturb vegetation including trees, and the need for an acceptable disposal sites and haul routes.

<u>Cost:</u> Cost studies are a routine part of all maintenance decisions. However, with rockfall mitigations, there are numerous levels of mitigation and, thus, the costs can vary considerably depending on the degree of rockfall control. At times, a very modest investment can yield a reasonable improvement such

as when minimal reshaping of a fallout area produces a marked improvement in containing typical rockfalls.

Avoidance measures requiring complete realignments, tunnels or elevated structures tend to be much more costly and beyond the realm of maintenance but regardless the cost may be justifiable depending on project goals. A benefit/cost analysis can be performed to compare different options and the cost of improvements versus continued maintenance.

Maintenance Requirements: Like cost, maintenance requirements can vary significantly between measures but not necessarily directly to the level of improvement. As with any constructed feature, none will be maintenance free; however, a reduction in road patrol requirements and the ability to schedule routine maintenance rather than remaining in a continuously reactive mode is typically a design goal.

7.3.6 Conclusion

The proper selection of an appropriate mitigation measure or approach requires a thorough understanding of the rockfall problem, and the project goals and limitations. Experience is perhaps the most valuable tool to use when evaluating and selecting an appropriate rockfall mitigation measure and some maintenance personnel have only limited experience with all the possible options. If this is the case, a comparison of the criteria discussed above can be beneficial to guide initial decisions. It is common that the final mitigation decision will require the input from several different disciplines.

Still, it is not just a design issue and the opinions of those who will construct the rockfall mitigation measure as well as those who will be responsible for the on-going maintenance should be included in the decision process. Levels of improvement and expenditures should be consistent with program goals and other highway maintenance needs and responsibilities.

The mitigation selection step occurs very early in the project process and, thus, it should not necessarily result in an absolute single decision. Often, a better understanding of the rockfall problem or unanticipated project limitations may become evident over time. A degree of flexibility in considering several appropriate measures and the willingness to change or adjust as the project develops is valuable.

TABLE 7-3 ROCKFALL MITIGATIONS

MITIGATION MEASURE	CRITERIA								
	Complexity	Effectiveness	Durability	Constructability/ Special Expertise	Road Closure/ Traffic Restrictions	Environmental Limitations	Aesthetic Impacts	Cost	Maintenance Requirements
MAINTENANCE PRACTICES									
Warning Signs	L	L-M	na	na	N	L-M	L	L	L-M
Road Patrols	L	L-M	na	na	N	na	na	L-H	H
Ditch Cleaning	L	L-H	na	na	Y	L-M	na	L-M	H
Monitoring	M-H	L	L-H	M-H	N	L	L	L-H	L-H
STABILIZATION									
Removal:									
Scaling	L-M	L-H	L-M	M	Y	L	L	L-M	L-M
Blast Scaling	M-H	L-H	M	M-H	Y	L-M	L	L	L-M
Trim Blasting	M-H	M-H	M-H	M-H	Y	L-H	L-H	M	L-M
Re-sloping	L-H	M-H	H	L-M	Y	L-H	L-H	M-H	L
Benching	M-H	L-H	M-H	M-H	Y	M-H	L-H	H	L-H
Reinforcement:									
Dowels	M	M-H	H	H	P	L	L	M-H	L
Shear Pins	M	M	M	H	P	M	M	M	L
Rock Bolts	M-H	M-H	H	H	P	L	L	M-H	L
Shotcrete	M-H	M-H	M-H	H	P	M-H	H	M-H	L
Buttresses	M-H	H	H	M	P	L-H	H	M-H	L
Cable Lashing	M-H	L-M	L-M	M	P	L-H	M	M	L-M
Whalers/Lagging	M	M	L-M	M	P	L-H	M-H	M	L
Anchored Mesh	M	M	M	M	P	M	H	L-H	M-H
Drainage:									
Weep Drains	L	L-H	M	L	P	L-H	L	L	H
PROTECTION									
Mesh /Cable Nets:									
Draped Mesh	L-M	M-H	M-H	M	Y	M-H	M-H	L	L-M
Suspended Curtain Systems	L-M	M-H	M-H	M	Y	M-H	M-H	L-M	L-M
Catchment Areas/Sheds:									
Ditches/Berms	L	M-H	H	L	P	L-M	L-M	L-H	H
Rockfall Sheds	VH	H	M-H	H	P	H	H	H	L-M
Barriers:									
Rigid Barriers	L	M-H	L-M	L	P	L	L	L	M-H
Flexible Barriers	M	M-H	M-H	M	P	L	L	M	M-H
AVOIDANCE									
Tunnels	VH	VH	H	H	P	L-M	L-H	VH	M-H
Realignment	M-H	M-VH	H	M	P	M-H	L-H	H-VH	M-H
Elevated Structures	M-H	M-H	L-H	M	P	L-H	L-M	VH	M-H

L = low, M = medium, H = high, VH = very high, N = no, Y = yes, P = possibly, na = not applicable

CHAPTER 8
ECONOMICS OF LANDSLIDE AND ROCKFALL REPAIR

8.1 GENERAL ECONOMIC ISSUES

As described in Chapter 7, numerous techniques are available for maintenance and repair of slope failures. The techniques range from simply replacing the failed material back on the slope and regarding, to installation of drainage measures or reinforcement, to construction of complete earth retaining structures. However, the costs and the long-term effectiveness of alternative repair measures can vary dramatically, both overall and on a case-by-case basis, and no single technique is suitable for all cases. While much work has been performed to develop guidelines and methods to prevent, maintain, and repair slides and rockfalls, limited guidance has been developed to help decision makers determine whether, when, or how a slope failure should be repaired so that limited funds are applied where the most benefit will be gained. One impediment to development of such assistance is that the economics of constructing and maintaining transportation slopes and embankments has not been thoroughly studied and is not well understood.

Economic issues are complicated by the fact that different sources of funds are generally involved and those in charge of the different sources are motivated to make those funds go as far as possible. Thus, construction personnel are motivated to limit construction costs even if it means increased maintenance costs. Similarly, maintenance personnel are often motivated to do inexpensive patching, even if it might not make sense from a long-term perspective, because problems that become acute may become eligible for special emergency repair funds or other programs that will not affect their limited maintenance budget. There is also a frequent tendency to consider the use of agency personnel and equipment as being "free", because salary and equipment budgets are often kept separate from other budgets, but to consider outside contracts to "cost money". In reality, both in house and contract work costs money to the agency, but they may affect individual budgets within an agency quite differently.

Economic issues are particularly challenging for nuisance slides where large consequences are often not present, but where the need for continuing maintenance can lead to significant economic burden for agencies responsible for maintaining them. For example, it is possible to effectively argue that many slopes where nuisance slides are prevalent are simply too steep and that constructing flatter slopes would reduce long-term maintenance costs. While constructing flatter slopes would undoubtedly reduce the number of nuisance slides, there are additional costs for constructing flatter slopes that have to be considered (e.g. additional earthwork, additional right-of-way, etc.). The appropriate question to be asked is whether it is more economical to construct slopes that will minimize long-term maintenance costs at the expense of higher initial costs or is it more economical to save money on initial construction at the risk of having to expend funds to maintain the slopes over the long-term. The answer to this question is not always clear. However, the prevailing perception is that the lifecycle costs (all costs incurred to construct and maintain something) for routinely maintaining and repairing nuisance slides are smaller than the costs associated with acquiring the additional right of way and materials necessary to construct flatter slopes.

In many cases, slopes being maintained are already constructed so there is little need to be concerned with how they were constructed – they are what they are. However, similar questions also arise when considering alternative maintenance and repair strategies. For example, is it better to select repair methods that are relatively expensive but likely to permanently repair problems or is it better to utilize "patches" such as regrading that are substantially less expensive but likely to only be temporarily effective? Answering these questions is difficult because the end result is uncertain (if we knew how much we'd spend on long-term maintenance we could answer the question) and because of generally poor record keeping on the part of many agencies (which is one of the motivations for maintaining slope management systems as described in Chapter 6). This uncertainty requires that consideration be given to the probability of failure, consequences, and risk, as discussed later in this chapter.

Regardless of whether considering new construction or maintenance and repair of existing construction, and whether considering large landslides and rockfalls or nuisance problems, the basis for evaluation of alternative actions should be life cycle costs. For soil and rock slopes, these will generally include costs in the following general categories:

- Initial construction costs (e.g. cut/fill, labor, retaining structures, drainage measures, right of way)
- Recurring maintenance costs (e.g. mowing/vegetation control, drainage maintenance, etc.)
- Unexpected or intermittent maintenance costs (e.g. repair of small slides, repair of drainage elements, guardrail, pavement sections, etc.)

In general, construction costs and even recurring maintenance costs can usually be predicted with reasonable accuracy. Unexpected or intermittent maintenance costs, on the other hand, are more difficult to predict because the frequency and severity of the maintenance requirements are generally unknown. Significant judgment is therefore required in making these decisions with consideration given to the potential risks and costs involved as discussed in more detail later in this chapter. Life cycle costs used for evaluating alternative maintenance and repair strategies should also include "internal" agency costs such as personnel and equipment, as well as costs that will come outside of the agency.

8.2 COST CATEGORIES AND VARIABILITY OF COSTS

Costs for maintenance and repair of slides and rockfalls depend upon a number of factors, including:

- Size and severity of problem condition
- Specific maintenance or repair technique utilized
- Location of specific site
- Availability of equipment and materials
- Whether the activity is performed in house or is contracted to outside groups
- The degree of stabilization achieved (e.g. temporary repair versus permanent repair, or stabilization to achieve one level of risk versus another)

Because each of these factors varies on a case by case basis, no attempt is made here to present cost estimates for different stabilization techniques. Rather, personnel are recommended to develop cost estimates for alternative repairs on a case by case basis. This is preferably done using historical costs that are tracked by an agency over time (slope management systems can greatly facilitate this). For repair methods for which historical data are not available, personnel can consult with private contractors, other state DOT personnel that have experience with the "new" technique, or agency construction personnel that can assist in developing cost estimates. There are also a number of national construction publications and internet web sites that can be used to develop estimated costs, although this will generally require breaking down costs in some detail in order to produce an overall cost estimate.

Regardless of the method chosen for developing cost estimates, it is important that the estimates include <u>all</u> costs associated with the problem condition, and not only the costs associated with the maintenance or repair activity itself. For example, for repairs involving removal and replacement of the failed material in a slide, estimates should generally include costs for:

- Site investigation and monitoring required prior to, during, or following construction
- Mobilization of personnel and equipment
- Traffic control (e.g. signs, barrels, flag personnel, etc.)
- Excavation of deleterious material
- Hauling and spoiling of excavated material
- Acquisition of replacement material (e.g. rock fill, "shot" rock, etc.)
- Placement of replacement material
- Repair/replacement of damaged signs, guardrail, pavement, shoulders, etc.
- Seeding and erosion control following construction
- Overhead or administrative costs

As well as any other costs deemed necessary. Cost estimates for other stabilization or maintenance techniques should generally include most of the same categories, with perhaps one or more additional categories for items specific to a particular technique (e.g. reinforcement, drainage materials, etc.). In some instances, these costs may be borne by different agency departments or from different funding accounts. Nevertheless, it is important that complete and accurate costs be estimated, and subsequently accounted for following construction so that the costs are appropriately assigned and tracked. In cases where appropriate accounting codes are not present in an agency's system to indicate expenditures as being associated with slope maintenance and repair, efforts should be taken to develop such codes to facilitate accurate tracking of these costs.

8.3 PROBABILITY OF FAILURE, CONSEQUENCES, AND RISK

As discussed previously, much of the challenge involved with evaluating alternative maintenance and repair actions is that the future performance of a slope is uncertain. Because of this uncertainty, future costs for maintenance and repair are also uncertain. While different maintenance and repair activities (e.g. regrading versus removal and replacement versus other techniques) will generally produce different chances of future failure, it is important to realize that there is some chance of future failure for <u>all</u> maintenance and repair techniques. We simply cannot achieve, or cannot afford to achieve absolute safety for any slope and there is always the potential for unforeseen loading conditions from extreme events or unknown behavior to occur because of incomplete understanding of slope performance.

Because we cannot achieve complete certainty, our objective is to try to balance the <u>probability of failure</u> and <u>consequences</u> involved with each case with the costs required to reduce the probability of failure to an appropriate level. In this context, "probability of failure" refers to the chance of having a failure or unacceptable performance of a slope. This probability of failure is often expressed as a percentage or as an equivalent decimal value. For example, the probability of failure could be equivalently expressed as 1 percent or 0.01. However, because probabilities of failure for earth slopes are often very small numbers, it is also useful to express probabilities of failure as a ratio. The equivalent ratio for the previous examples would be that the slope has a 1 in 100 chance of failure.

The term "consequences", as used here, refer to things that will occur, or are expected to occur, if failure actually occurs. Consequences certainly include costs associated with a future failure, including costs to repair the failure, but can also include human injuries or fatalities or other less tangible things like legal liability or even political consequences such as loss of faith by the traveling public. In most cases, we will choose to express consequences in terms of dollars to make our evaluations convenient.

Neither the probability of failure nor consequences alone are sufficient for judging selections regarding slope repair options. For example, if there is a very high probability of a failure occurring, but that failure will cause no significant consequences, there is little need to take significant action or spend significant funds to reduce the probability of failure. This is, in essence, like the tree that falls in the forest…if no one knows, then who cares. Conversely, if a slide with very high consequences (e.g. significant cost, or potential fatalities, etc.) is being considered, but the chances of that slide occurring are extremely small (i.e. very small probability of failure), then again there is little need to do anything about it because the chance of having the failure occur is so small. These two extreme cases illustrate the point that slope maintenance and repair decision should not be made based on consideration of probability of failure alone or consequences alone because the end result will inevitably be spending scarce resources fixing a "non-problem". Of course, most realistic cases fall somewhere in between these two extremes. Such cases require combined consideration of both the probability of failure and the consequences involved. The combination of probability of failure and consequences is frequently referred to as "risk". Risk is commonly computed as the product of the probability of failure and consequences, and thus is also expressed in units of dollars.

8.4 BALANCING OF RISKS AND COSTS

Two separate but related problems are often faced when trying to evaluate alternative methods for stabilization and/or repair of slides and rockfalls or when deciding whether action is necessary at all. The first is to decide what level of stabilization is appropriate for a given situation considering both the probability of failure and consequences involved (i.e. the risks involved). This issue is addressed in this section in the context of what is referred to as acceptable risk. The second issue is to decide which maintenance/repair method (usually out of several, or perhaps many methods) is most economical or effective for a particular situation. The approach to addressing the second issue can also assist in establishing the appropriate level of stabilization in addition to facilitating comparison of alternative actions, as described in more detail in the following section.

Most people have some intuitive appreciation for balancing risks with the cost of an activity and understand that situations that are truly risky (i.e. situations with both high probability of failure and significant consequences) generally warrant significant expenditures or efforts to reduce the level of risk, whereas situations with significantly less risk (i.e. situations with relatively low probability of failure or small consequences) warrant less attention to reduce the risk. In the context of landslides and rockfalls, slopes with a high probability of failure and significant consequences generally warrant relatively large expenditures to reduce the probability of failure or consequences whereas slopes with a low probability of failure or limited consequences warrant less effort and expenditures (generally speaking). However, while most people appreciate the general relationship, few have a firm grasp on exactly what levels of risk are "high" and what levels of risk are "low", particularly in the context of engineered structures and systems such as the national transportation system.

Figure 8-1 shows a graphic that is commonly presented to demonstrate what many people consider to be "acceptable risk" associated with several different activities or industries. The first thing to note about the figure is that it relates probability of failure to the potential consequences involved – shown in the figure in terms of potential loss of life or in terms of potential monetary costs. The figure thus deals with combined probability of failure and consequences (i.e. risk) by relating acceptable probabilities of failure to the consequences involved. This graphics shows that acceptable probabilities of failure generally decreases as the consequences (in terms of cost or lives lost) become greater. For relatively low consequence events (little chance of loss of life or significant costs), people are generally willing to accept greater probabilities of the event occurring (perhaps on the order of 1 in 10 or 1 in 100 chance). This situation is similar to many nuisance slides where consequences are generally low. Conversely, for relatively high consequence events (greater chance of personal injury or death and/or significant costs), people are generally only willing to accept much smaller probabilities of failure (perhaps 1 in 1000 or 1 in 10,000 chance). This situation is similar to larger landslides and rockfalls with much greater consequences.

While Figure 8-1 provides some general guidance on accepted levels of risk for many types of civil infrastructure, it should not be used as an absolute guide for establishing acceptable risk because people consider different types of risk differently, often depending upon whether some degree of personal choice is involved. For example, people are generally willing to accept less risk for things that are unintentional or for which they have little influence (e.g. plane flight, travel on highways, etc.) as compared to things which they can control or have some choice (e.g. skydiving). In the context of transportation applications, people are generally less willing to accept a bridge failure that seems to occur "out of the blue" than they are willing to accept a bridge failure that occurs because of some extreme event such as a barge impact or an earthquake *even if the consequences are similar*. In the context of landslides and rockfalls, people are generally more willing to accept a landslide or rockfall that appears to have occurred as a result of an earthquake or severe flooding than they are a landslide or rockfall that just appears to "happen" with no obvious cause. The most common example cited of this phenomenon is commercial aviation accidents, which have risk that is far lower than many other common activities (including car travel along the nation's highways) as shown in Figure 8-1. Nevertheless, the perception of the general public is that such incidents are dramatically more impacting despite the fact that commercial air travel is far safer than automobile travel. Because many landslides and rockfalls can happen without apparent cause, the general public is less willing to accept them and therefore they tend to have an acceptable probability of failure that is likely below the "generally accepted" line shown in the figure.

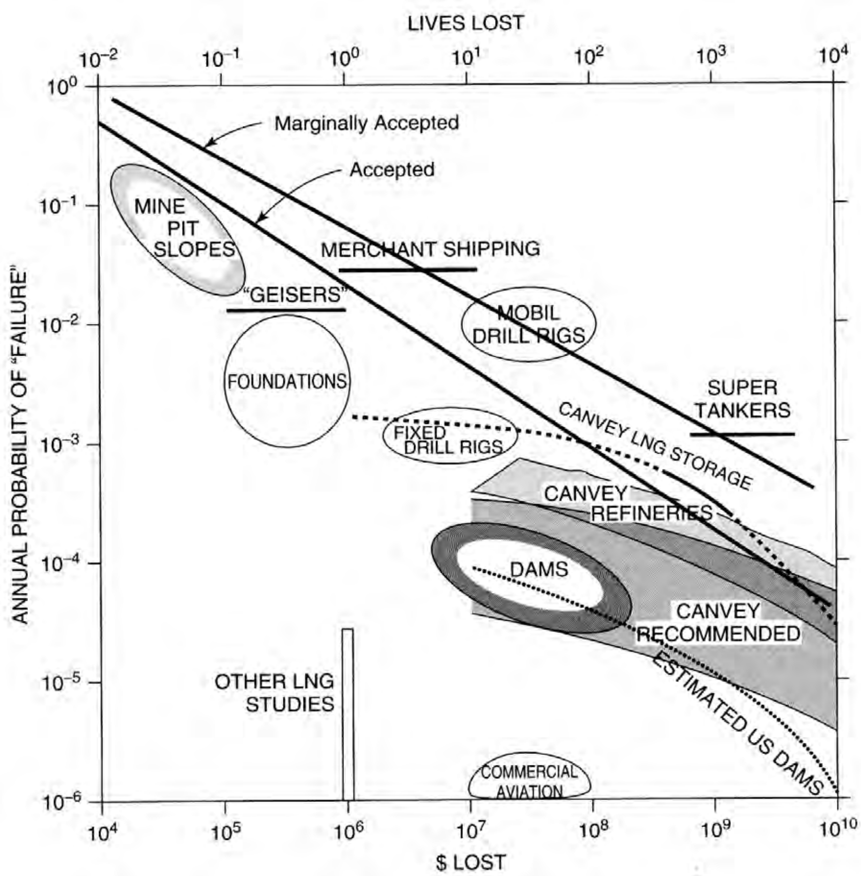


Figure 8-1 Graphical Representation of Relationship between Acceptable Risk as Function of Consequences (costs or lives lost) for Variety of Applications (from Baecher, 1982)

It is possible to develop quantitative estimates for the probability of failure as well as quantitative estimates for costs and consequences so that direct application of the information in Figure 8-1 can be made. However, it is still not common for designers and engineers to explicitly evaluate probabilities of failure for landslide and rockfall applications. Rather, it is more common to develop conventional "factors of safety" that reflect the relatively stability of a soil or rock slope. As discussed in Chapter 3, factors of safety of 1.0 indicate a slope is just on the verge of failure, while factors of safety greater than 1.0 indicate that there is sufficient resistance to prevent sliding, at least theoretically based on the different assumptions used in the models. Because the models used and the input soil or rock properties used in the models are not perfect, it is common to require that factors of safety be substantially greater than 1.0 for many problems to account for the potential for conditions to be different than assumed in the models. Common practice for many slope design applications is to require factors of safety of at least 1.3, and more commonly 1.5 for slope stability problems.

The factors of safety commonly computed have some relation to the probability of failure, although the relation between factor of safety and probability depends on site specific parameters so no general relation can be developed. Nevertheless, it is generally true that slopes that have higher factors of safety have a lower probability of failure while slopes with lower factors of safety have higher probabilities of failure, all else being equal. In the absence of specific knowledge regarding the probability of failure, or case specific estimates of the probability of failure, decision makers can make use of this general relation to help make slope stabilization decisions. Simplistically speaking, this relation suggests that higher factors of safety should be required for situations with greater costs or consequences, while lower factors of safety may be acceptable for cases where consequences are smaller.

8.5 COMPARISON OF STABILIZATION ALTERNATIVES

In most cases, decision makers are faced with evaluating two or more competing alternatives for repair and stabilization of soil and rock slopes. This situation is beneficial because it provides decision makers with alternatives that can help make effective use of available funding but it can also be challenging because it is frequently difficult to get an "apples to apples" comparison. Direct comparison of costs for different methods is only appropriate if the competing methods have similar probabilities of failure or risk. However, it is common that different stabilization alternatives may have substantially different factors of safety or probabilities of failure. One frequently ends up with several alternatives, some of which are less costly but provide marginal reduction in risk and some of which are more costly but provide greater reduction in risk. In the right circumstances, it is perfectly acceptable to select a less costly method, even if it produces greater overall risk than other options, but this has to be done with the realization that the less costly method carries greater risk than the more expensive and reliable methods.

Fortunately, there are several economic strategies than can be used to help decision makers select a "most appropriate" maintenance or repair method from among a number of options. Strategies that will be discussed here include:

- Minimum cost strategy
- Minimum risk strategy
- Minimum "total cost" strategy
- Maximum "value" strategy

Each of these strategies is described in more detail below.

8.5.1 Minimum Cost and Minimum Risk Strategies

The two most straightforward strategies that can be used to select the “most appropriate” slope maintenance or repair method from among a number of options are the minimum cost strategy and the minimum risk strategy. Each of these strategies is implemented by calculating the costs or risk (recall that risk is the product of the probability of failure and the consequences of failure, expressed in units of dollars) associated with the various options and then selecting the option that has the least cost or least risk.

Unfortunately, neither of these two strategies is very effective over the long-term because both neglect key issues that have a strong impact on the long-term effectiveness of the decisions. The minimum cost strategy, for example, tends to produce cost savings in the short term because stabilization options with the least cost are routinely selected. However, the minimum cost strategy ignores the fact that less costly methods generally produce much higher probabilities of failure, thereby leading to more future failures that introduce additional costs. There are times when it may be appropriate to get a temporary fix, and it may work out for a limited number of cases, but in general the minimum cost strategy tends to be short sighted and lead to much greater costs in the future that are not accounted for in the decision process.

In contrast, the minimum risk strategy exclusively considers the risks involved with different stabilization options but neglects the fact that methods that produce lower risk also tend to have greater costs. Thus, implementation of the minimum risk strategy tends to produce a few sites with extremely low risk but leaves many sites with higher risk because maintenance and repair budgets are limited.

When considered from an agency-wide perspective, both the minimum cost and minimum risk strategies tend to produce extreme decisions that are generally poor choices among available maintenance and repair options. The minimum cost strategy tends to minimize initial costs but produce high risk of future failures while the minimum risk strategy minimizes risk but produces high costs for stabilization of individual sites. The remaining strategies discussed in this section are generally preferable because they seek to balance risks and costs in a logical manner.

8.5.2 Minimum “Total Cost” Strategy

Another strategy that is generally preferable to the minimum cost and minimum risk strategies is the minimum “total cost” strategy. This strategy is somewhat similar to the minimum cost strategy (which ignores risk) except that it accounts for risk by prioritizing options according to a “total cost”. The “total cost” of a particular stabilization option is computed as the sum of the initial costs (the same costs that are considered in the minimum cost strategy) plus the risk (which you’ll recall is the product of the consequences and the probability of failure and is expressed in dollars). The total cost is thus computed as

$$\text{Total Cost} = \text{Initial Cost} + \text{Risk} = \text{Initial Cost} + p_f\left(\text{Consequences}\right) \qquad \text{8-1}$$

where p_f is the probability of failure of the particular maintenance or repair method and consequences is the cost of the consequences of failure should it occur (e.g. costs for additional maintenance/repair, costs for damage to signs, guardrail, structure damage, etc.). The total costs computed in this way include the cost of the stabilization option that is consider in the minimum cost strategy but also includes *potential* costs that will be incurred if failure occurs.

Once total costs are computed for each maintenance/repair option, the preferred option is selected as the option with the least total cost. The primary benefit of this option over the minimum cost and minimum risk strategies is that it accounts for both costs and risk and therefore tends to produce an appropriate balance between risk and costs. One challenge for implementing the minimum total cost strategy as well as the minimum risk strategy is that they require estimation of the probability of failure for each of the maintenance/repair options. This issue will be addressed further in the following section in the context of the maximum value strategy, which also requires estimation of the probability of failure.

8.5.3 Maximum "Value" Strategy

Another generally effective strategy that can be used for comparing maintenance and repair alternatives is the maximum value strategy. This strategy differs somewhat from the other strategies in that it seeks to maximize the "value" of a particular stabilization option rather than minimizing something. In essence, this strategy seeks to get the most "bang for the buck". The maximum value strategy also differs from the other strategies in that it is intended for comparison of two stabilization options in a head-to-head fashion to identify the preferred option rather than prioritizing a list of maintenance/repair options. However, this limitation can be addressed by sequentially comparing all possible combinations of maintenance/repair options in turn to identify the option that is preferable to all other options.

The basic decision to be made in the maximum value strategy is whether a more costly but more reliable maintenance or repair method is worth the additional cost over a less costly but less reliable method. The decision is based on comparison of the reduction in risk provided by the more costly but more reliable method with the additional costs required to achieve the reduced risk (Duncan, 2000; Loehr et al., 2004). This basis for selection can be expressed using the following inequality

$$\begin{pmatrix} \text{reduction in risk} \\ \text{for more} \\ \text{reliable design} \end{pmatrix} \geq \begin{pmatrix} \text{added cost of} \\ \text{more reliable design} \end{pmatrix} \qquad \text{8-2}$$

If the more costly and more reliable method reduces risk by an amount greater than the additional cost for the method, it is preferable to the less costly and less reliable method. However, if the reduction in risk is found to be less than the additional cost, then the less costly method is deemed to be preferable.

Recalling that risk is the product of the probability of failure and the consequences, the general relation in Eq. 8-2 can be expressed in a more useful computational form. If the two methods being compared are termed "method A" and "method B", and if the estimated cost of method A is denoted A, the estimated cost of method B is denoted B, the consequences of failure is denoted C, and the probability of failure for methods A and B are denoted as $p_{f\text{-}A}$ and $p_{f\text{-}B}$, respectively, Equation8-2 can be expressed as

$$C \times \left(p_{f-A} - p_{f-B}\right) \geq \left(B - A\right) \qquad \text{8-3}$$

This says that for method B (the more costly and more reliable method) to be preferable, the reduction in risk (the left hand side of the equation) must be greater than the added costs for method B (the right side of the equation). This simple relation can also be presented in the form of a chart shown in Figure 8-2.

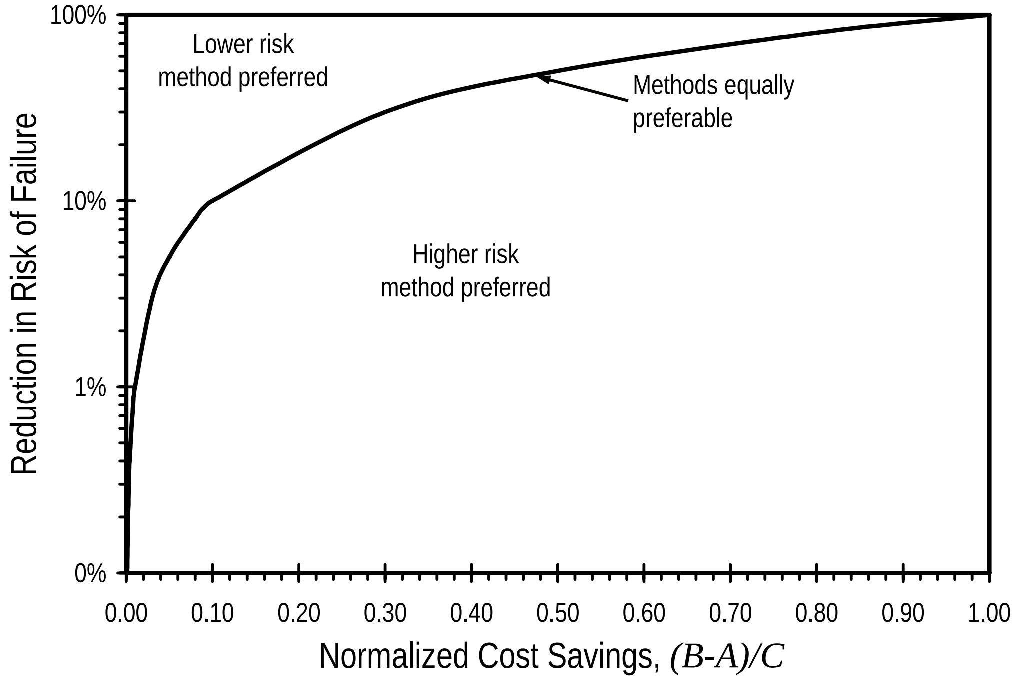


Figure 8-2 Chart Indicating Preferential Decision for Alternative Slope Stabilization Measures where A is the Cost of the More Risky Stabilization Method, B is the Cost of the Less Risky Method, and C is the Consequences of Failure (adapted from Loehr et al., 2004)

These simple relations or the chart shown in Figure 8-2 can be used to directly evaluate alternative stabilization techniques if the costs, consequences, and risks of the alternative stabilization techniques can be estimated. Estimation of costs for alternative stabilization techniques is generally straightforward, as is estimation of potential consequences. Estimates for probability of failure for the alternative techniques are not commonly made in current practice, although techniques are available for this purpose when knowledge of specific probabilities of failure is important (e.g. when consequences and costs are large). Maintenance personnel should consult with qualified geotechnical and/or construction specialists for assistance in estimating costs, consequences and probabilities of failure when accurate estimates of these values is important.

In many cases, however, highly accurate estimates for the probability of failure may not be necessary. The maximum value strategy can be used to facilitate effective decision making, by either using judgment to estimate the probabilities of failure associated with alternative techniques or to estimate the reduction in probability of failure between the two repair techniques. As an example, consider a case where *B*=\$150,000, *A*=\$50,000, and *C*=\$500,000. For this case, the normalized cost savings, (*B*-*A*)/*C*, is 0.20. Referring to Figure 8-2, this would suggest that Method B must reduce the probability of failure by at least 20 percent (0.2) more than method A for the additional costs to be justified. If method B was believed to provide substantially greater reduction in the probability of failure than 20 percent, it would clearly be the preferred alternative. Conversely, if method B were believed to provide substantially less reduction in the probability of failure than 20 percent, method A would be the preferred alternative. Or, if method B were believed to lower the probability of failure by approximately 20 percent as compared to method A, then the two alternatives would tend to produce similar overall results when balancing costs and consequences.

If the consequences for the above example were substantially lower, say C=\$125,000, the result changes significantly. In this case, method B must reduce the probability of failure by at least 80 percent compared to method A for the additional costs to be justified, a value that is very unlikely to be achieved unless method A is extremely risky or method B is extremely conservative. Thus, in this case, the less costly but more risky method A would be a strongly preferred alternative. These simple examples demonstrate that even if probabilities of failure cannot be accurately quantified, this technique can often provide information that is sufficient to make the appropriate decision.

In general, the maximum value strategy is consistent with general intuition. If a more expensive method provides little reduction in risk, that method provides little additional benefit and is unlikely to be preferable. Or, conversely, if the more expensive method provides a relative great reduction in risk, it is more likely to be preferred. What the maximum value strategy provides is quantifying this intuition and basing the actual decision on the consequences associated with failure. In addition to providing a quantitative means for comparing alternatives in an "apples to apples" form, this technique can also be an effective means for establishing what level of stabilization is appropriate by comparing methods that provide varying degrees of risk reduction and selecting the one that provides the most overall value.

It is important to note that both the maximum value strategy presented here and the minimum total cost strategy presented previously will produce "optimal" decisions in terms of balancing risks, consequences, and costs when consistently applied over a large number of cases. However, using these approaches does not guarantee that the selected alternative will not actually fail (regardless of the decision, there will be some probability of failure). Rather, the advantage of the approach is that it will tend to promote use of funds for cases where risk can be substantially reduced and use of less funds for cases with less risk so that, over time, the agency will be making the best use of its limited funds. The actual result may not work out as desired for individual cases, but if applied consistently over time the overall results will balance out for the benefit of the agency. This is similar to the common investment strategy of diversifying investments where portions of the investments are placed in high risk, high return investments and other portions are placed in lower risk, lower return investments to achieve a balance of both investment security and investment growth.

One final issue that deserves attention is the fact that these methods generally require that consequences be expressed in units of dollars. For tangible things like damage to highway signs, guardrails, and repair costs this is relatively straightforward. It can be more difficult to quantify less tangible things like legal consequences, personal injury, or loss of life. In such cases, judgment must generally be used to estimate some financially equivalent consequence. While this may seem crude, it is often not as difficult as it might seem because risk of personal injury or death is understandably considered to be a dire consequence and thus warrants using dollar equivalents that are much greater than other consequences to be considered. In such cases, the consequences used will be extremely large, which will make the relative savings relatively small so that methods with greater cost and lower risk are almost always preferable (as intuition would suggest).

CHAPTER 9
LEGAL ISSUES OF LANDSLIDES AND ROCKFALLS

It is an unfortunate reality that landslides and rockfalls do occur, and often despite our best efforts, these events can impact property of adjacent landowners and potentially cause personal harm to the public. When such events do happen, it is not uncommon for litigation to ensue. In such cases, transportation agencies are faced with providing legal defense for the agency to protect the agencies interest whether the legal issues are well founded or frivolously based. This chapter provides general guidance for transportation agencies to use to raise awareness of potential legal liabilities so that agencies can appropriately deal with them when they arise and develop protection from legal liabilities arising from landslides and rockfalls to the extent possible.

9.1 POTENTIAL LEGAL LIABILITIES

Although the number of legal cases resulting from landslides or rockfalls has been increasing over the years, few legal precedents have been established to guide courts in determining responsibility for these landslides or in assessing the damages caused by them. This lack of specific laws and legal decisions is perhaps due to two main factors: 1) cases that involve private companies are settled out of court and 2) many cases against state or federal agencies are settled out of court or the public agency exercises its sovereign right to refuse to consent to be sued. It is most important that those who undertake activities that involve the use of slopes have an understanding of the legal implications of that use. The situation is further complicated by the fact that most of these cases are under the jurisdiction of state laws. Thus, legal liabilities tend to vary substantially from state to state and legal precedent in one state may have little ramification for other states.

Since most litigation involving landslides relates to construction and maintenance of public roads, we will assume that a public agency is a defendant. It should also be understood that when liability for a landslide is discussed, it must be assumed that a landslide has caused personal injury and/or property damage thus enabling an action against a public agency. The legally protected interest of the injured party may be his or her personal property, real estate, or physical well being. It should also be assumed that the public agency is in some way responsible for the landslide. Responsibility or liability commonly relate to construction or maintenance operations that create or activate a landslide on public property. A more difficult legal situation is when one tries to argue that a landslide or rockfall has occurred because of a failure to maintain a soil or rock slope. In such cases, the legal argument is that the slide or rockfall occurred because of “neglect” rather than a direct act (even if unintentional). Such cases are generally more complex and difficult to prove, but may nevertheless have ramifications for transportation agencies.

There have been numerous cases in which private property has been damage and/or personal injury has resulted from landslides on public highways in the United States. In those instances, liability of the public agency having jurisdiction over the highways has varied from state to state. Some states prohibit suits against public agencies through sovereign immunity. Other states have established statutory provisions under which recovery may be realized. Such statutes generally delineate specific duties and responsibilities of public agencies, specific circumstances of the slope failure, procedural requirements for bringing action against a public agency, and specific defenses available to the public agency.

While it is true that states and the federal government, as owners, may invoke the protection of sovereign immunity, there are many indications that this sheltered position will not survive. Recently, numerous agencies have lost the right of sovereign immunity. While in some states, employees of governmental bodies appear to be held harmless from legal action, there are indications it is possible to bring personal suits against such employees for negligence. Again, this is a state specific issue so it is important that appropriate employees are made aware of the situation for their state.

With regard to landslides, it should be understood that the legal rights of private citizens against public agencies are divided into two categories: 1) a property owner's rights and response to the invasion of the property by sliding material or interference with the lateral support of the property by construction or maintenance of a public way and 2) a highway travelers rights in tort against a public agency for injury sustained from a landslide that resulted in part from the negligent construction or maintenance of a public way. Each of these two cases is described in more detail in the following sections.

9.1.1 Landslide Encroachment onto Adjacent Property

When a landslide results in damage to property either by invasion of the property or loss of its lateral support, the liability of a public agency is not necessarily based on statutes. Under the Fifth Amendment to the United States Constitution, just compensation must be paid when public works or other governmental activities result in the taking of private property. That concept can be extended to the damaging of property as a result of an action of a public agency. The owner of the property may bring an action known as an inverse condemnation suit to recover damages. Courts have held that a state or local government cannot take or damage private property for public use without just compensation.

In studying several cases, it may be concluded that if public works activities result in the creation of a landslide or the reactivation of an old landslide that causes damage to private property, the public agency is liable for the full extent of such damage. If the particular governmental agency has such a constitutional provision, even in jurisdictions that do not have a provision relating directly to damage of private property, the courts have tended to find that the damage that results to private property constitutes a taking for which just compensation must be paid.

9.1.2 Injuries Sustained from a Landslide

Although the courts have made it clear that a public agency is not an insurer of the safety of persons using its highways, in certain circumstances travelers are protected by law from landslides. In general, the public agency will not be held liable for injuries if it can be shown that the acts or omissions that created the dangerous condition were reasonable or that the action taken to protect against such injuries or the failure to take such action was reasonable. The reasonableness of action or inaction is determined by considering the time and opportunity that the public employees had to take the action by weighing the probability and gravity of potential injury to persons foreseeably exposed to the risk of injury against the practicality and cost of protecting against such injury.

Some states have statutes that impose liability for the dangerous condition of public property. In those states, the injured person must prove that the public property was in a dangerous condition at the time of the injury and that the injury resulted from that dangerous condition. This dangerous condition must be the result of negligence, a wrongful act, or failure of an employee of the public agency to act within the scope of his or her employment. The public agency must have had notice of this dangerous condition in sufficient time prior to the injury to have taken measures to protect against it. Thus, liability depends upon whether circumstance and conditions were such that the danger was reasonably foreseeable in the exercise

of ordinary care and if so, whether reasonable measures were taken by the public agency to prevent injury.
A public agency, since it is not an insurer of the safety of the travelers on its highways, need only to maintain highways in a reasonably safe condition for ordinary travel under ordinary conditions or under such conditions as should be reasonably expected.

Foreseeability has been held as a necessary element of contributory negligence. It must be shown that a reasonable person would have foreseen the exact consequences of negligence. The doctrine of contributory negligence has been replaced by comparative negligence in many states. Under comparative negligence doctrines, negligence is measured in terms of percentage. Any damages allocated are diminished in proportion to the amount of negligence assigned to the plaintiff.

9.2 RISK MANAGEMENT

Risk management recognizes the likelihood of lawsuits resulting from slides which cause human andlor property damage. To prevent or minimize this type of litigation and to help defend lawsuits, it is suggested that the following steps be followed:

1. Establish a regular record for inventorying and inspecting highways under your agencies responsibility.
2. Record all inspections and notice of change of any slide areas.
3. After noticing movement in a potential slide area, make regular inspections using inspection forms and diaries.
4. Document your actions relative to corrective measures taken after the inspection.
5. After identifying slide movement, take photographs during each inspection.
6. Erect warning signs at sites which cannot immediately be repaired.
7. Take routine corrective maintenance actions as soon as possible to improve safety at the site.
8. If routine maintenance does not provide a safe travel way for the public, program a design project immediately for reconstruction and/or rehabilitation.
9. The most important aspect…keep good maintenance records.

As stated earlier, while providing transportation services, the governmental agency is not the absolute insurer of the safety of a highway user. The total resources of any government are limited and it would not be realistic to expect that the bulk of all funding be devoted to keeping the roads in an absolutely sound and safe condition. However, the courts have consistently held that governments are required to maintain streets and roads in a reasonably safe manner.

Your attention is called to a book that was published in 1985. The title of the book is "Killer Roads from Crash to Verdict." Most county engineers live in terror because of it. This book was written by attorneys for attorneys. It is considered to be a "how to sue" book. It therefore behooves all governmental agencies and their employees to establish a standard of care for a given maintenance activity. It is also important that all employees seek to achieve a reasonable level of performance.

It should be understood that the function of governmental agencies is to provide security and services for its citizens. Transportation is one of the services which governmental officials and employees are charged with providing. The goal of transportation should be the safe and efficient movement of people and goods within reasonable physical constraint.

Finally, when the potential for legal liability exists, it is important to include possible legal liabilities with other possible consequences (e.g. repair costs, etc.) evaluating costs, consequences, and risks of slides and when programming or ranking sites for maintenance and repair. States with laws that are prone to rule for the traveling public may want to consider explicitly including legal liability with consequences when balancing risks and costs. Conversely, states with laws that strongly protect public agencies can be less concerned with legal issues.

9.3 RELATION WITH MANAGEMENT SYSTEMS

Use of slope management systems can also have an impact on legal liabilities, both positively and negatively. As stated above, public agencies are not liable for injuries if it can be shown that the acts or omissions that created the dangerous condition were reasonable or that the action taken to protect against such injuries or the failure to take such action was reasonable. In some states, use of a slope management system has been successfully used for defense of cases contending that actions were not reasonable. In essence, the fact that a slope management system had been developed, maintained, and applied when ranking and selecting sites, was found to be sufficient documentation that the agency was taking reasonable action to maintain the roadway within the available budget. In such cases, simple awareness of a problem has been found to be insufficient to introduce legal liability.

However, other states have avoided use of slope management and rating systems for fear of creating possible legal liabilities associated with knowledge of potential safety hazards. The logic of this stance is that, by their very nature, slope rating and management systems identify sites with potential safety hazards. If a landslide or rockfall were to cause an accident or damage to adjacent property, it is possible that a plaintiff could argue negligence on the part of the agency for not rectifying the "known problem", *even if could be shown that the state had insufficient funds to make the necessary repairs*. Such a concern may be well founded depending upon state specific law so agencies are encouraged to consider both positions with respect to the specific law of their jurisdiction.

9.4 LEGAL CASE HISTORIES

Two case histories of legal claims against state departments of transportation are described here to demonstrate some of the legal bases, defense strategies, and other issues that can arise. One case is a case where the state was found liable; the other is a case where the state was found not liable on the basis of sovereign immunity. Remember, liabilities from landslides and rock falls are governed by state laws so the decisions in other states might be different.

9.4.1 Domyan vs. Ohio Department of Transportation

This case involved an accident that occurred when a driver hit a rock that had fallen onto the roadway from an adjacent rock cut causing damage to the vehicle. The plaintiff filed a claim to recover costs for repair of the vehicle that were not covered by the plaintiff's insurance in the amount of $400. The plaintiff claimed that Ohio Department of Transportation was negligent in that it failed to maintain the roadway in a safe, drivable condition and that it should have installed a rockfall barrier to prevent falling rock from reaching the roadway.

In their defense, Ohio DOT claimed that it was not liable for the damage to the vehicle because it was unaware of the rock on the roadway and that it had exercised due diligence to maintain the roadway in a safe condition. Furthermore, it argued that it had installed "falling rock" warning signs along this stretch of roadway to serve adequate warning to drivers and, thus, the drive should have known of the potential for rocks in the roadway.

In the court's decision, relevant legal precedents were cited that state that Ohio DOT has a duty to keep its roads in a safe, drivable condition. However, the precedents also indicate that the burden of proof in the case is placed on the plaintiff to prove negligence by the agency – namely that the agency had notice of the "defect" (rock in roadway) and either failed to respond to the defect or responded negligently.

Based on the facts presented and the relevant legal precedents, the court found that Ohio DOT was negligent and liable in this case because it knew of the general potential for falling rock (it had, after all, installed warning signs) in the area and should have installed measures to provide adequate safety. However, the court also found the plaintiff negligent for failing to avoid striking a "stationary object" in the traveling lane. Ohio DOT's negligence was deemed to be 60%, as allowed by Ohio law. Ohio DOT was therefore found to be liable in the amount of $240 (60% of original $400 claim).

9.4.2 Goodrich vs. The State of New York

This case involved an accident that occurred when a "toaster oven" sized rock fell through the windshield of a vehicle traveling along a state highway. The driver in the vehicle suffered moderate injuries; a passenger in the vehicle was uninjured. The vehicle was also seriously damaged.

Like the Ohio case, the plaintiffs' claim argued that the state was negligent in that it had failed to rectify a dangerous situation. However, the plaintiffs also claimed that the state had improperly designed/constructed the roadway in the first place because the site did not satisfy current NYS DOT design criteria and that past evaluations of the site performed by the state had been insufficient.

In their defense, the State of New York argued that the site had been routinely inspected and rated the suspect rock slope following established protocols (3 inspections in 12 years) and that the site did not make the state's prioritized list of problem areas. The state further argued that stabilization of the site would have been prohibitively expensive.

Relevant precedents for this case indicate that the state has an absolute duty to maintain roadways in a <u>reasonably</u> safe condition, but that the state was NOT an insurer or roadways. Like the Ohio case, the burden of proof was placed on the plaintiff to prove the following:

- That the state's negligence had caused the accident
- That the state had adequate notice of the dangerous condition, but failed to take reasonable corrective action
- Without proof the these, the state was entitled to "qualified immunity"

Based on these precedents and the facts presented, the court dismissed the case stating that the plaintiff had failed to prove that the state failed to perform a reasonable study and that the state had negligently constructed the roadway/tunnel. Furthermore the course found that the state was not required to bring "old" sites up to new standards unless substantive construction was to occur at the site.

Interestingly, the courts decision also discusses the fact that the state had based its decision not to repair the site based on the relative risks and costs present at the site and indicated that this was "playing the odds". Ordinarily, this would be basis to assign liability to the state. However, in this case the state's immunity was found to supersede this liability so the case was dismissed.

Note however, that in a similar case but with more serious injury and one fatality, the state was found liable because in that case the state had had notice of actual falling rocks, boulders, and debris and had failed to take appropriate action.

CHAPTER 10
REFERENCES

AASHTO. (2002). Standard Specifications for Highway Bridges, 17th Edition, American Association State Highway and Transportation Officials, Washington, D.C.

Baecher, G.B. (1982), "Statistical methods in site characterization", *Updating subsurface samplings of soils and rocks and their in-situ testing*, Engineering Foundation, Santa Barbara, CA, pp. 463-492.

Collin, J.G., Leshchinsky, D., and Hung, C. J. (2005) Soil Slopes and Embankment Design Reference Manual, Federal Highway Administration, September, 2005.

Cruden, D.M. and D.J. Varnes, (1992). "Landslide Types and Processes," *Landslides: Investigation and Mitigation*, Transportation Research Board, National Academy of Sciences.

Cruden, D.M., Varnes, D.J. (1996) "Landslide Types and Processes", *Landslides Investigation and Mitigation Special Report 247,* (Turner A.K. and Schuster R.L. eds.) Transportation Research Board, National Academy of Sciences, Washington, D.C, Chapter 3, July.

Deere, D.U., and F.D. Patton. (1971). Slope Stability in Residual Soils. In *Proc., Fourth Pan American Conference on Soil Mechanics and Foundation Engineering*, San Juan, Puerto Rico, American Society of Civil Engineers, New York, Vol. 1, pp. 87-170.

Duncan, J.M. (2000). Factors of Safety and Reliability in Geotechnical Engineering. *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 126, No. 4: 307-316.

Dunnicliff (1988), *Geotechnical Instrumentation for Monitoring Field Performance*, John Wiley, 577 pp.

Dunnicliff (1993), Geotechnical Instrumentation for Monitoring Field Performance, John Wiley, 577 pp.

Dunnicliff, J. (1998), Geotechnical Instrumentation Reference Manual, Federal Highway Administration, National Highway Institute, Arlington, Virginia. Publication No. FHWA-NHI-98-034.

Holtz, R. D., and Kovacs, W. D. (1981). *An Introduction to Geotechnical Engineering*, Prenctice-Hall, Inc., Englewood Cliffs, NJ.

Hunt, R.E., (1984). *Geotechnical Engineering Investigation Manual*, McGraw Hill Book Company, New York.

IAEG Commission on Landslides. (1990). Suggested Nomenclature for Landslides. *Bulletin of the International Association of Engineering Geology*, No. 41, pp. 13-16.

Kane, W.F. (2000), “Monitoring Slope Movements with Time Domain Reflectometry,” presented at *Geotechnical Field Instrumentation: Applications for Engineers and Geologists*, April 1, ASCE Seattle Section Geotechnical Group and University of Washington Department of Civil Engineering.

Lambe, T. W., and Whitman, R. V. (1979). *Soil Mechanics: SI Version*, John Wiley & Sons, Inc., New York, 553 p.

Liang, R.Y. (2006), *Landslide Hazard Rating Matrix and Database*, a draft preliminary interim report to Ohio Department of Transportation Office of Research and Development, Vol. 1, 126 pp.

Loehr, J.E., K.L. Sanford Bernhard, D. Huaco (2004), *Decision Support for Slope Construction and Repair Activities: An Asset Management Building Block*, report to Missouri Department of Transportation and Midwest Transportation Consortium, MoDOT report RDT 04-006, 57 pp.

Lowell, S. and Morin, P., (2000), “Unstable Slope Management – Washington State”. Transportation Research News Number 207. March-April. National Research Council, Washington D.C., pp 11-15.

Munfakh, George, Samtani, Naresh C., Castelli, Raymond J., and Wang, Jaw-Nan (1999), Earth Retaining Structures, Federal Highway Administration, National Highway Institute, Arlington, Virginia. Publication No. FHWA NHI-99-025.

Oregon Department of Transportation. 2001. *ODOT Landslide and Rockfall Pilot Study Final Report*. ODOT Geo-Hydro Section, HQ Geo-Hydro Unit.

Rodriguez, A., Del Castillo, H., Sowers, G.F., (1988). “Soil Mechanics in Highway Engineering” Trans Tech Publications, Federal Republic of Germany.

Tanyu, B.F., Sabatini, P.J., and Berg, R.R. (2005), Earth Retaining Structures, Federal Highway Administration, National Highway Institute, Arlington, Virginia. Publication No. FHWA-NHI-05-046

Terzaghi, Karl, Ralph B. Peck, & Gholamreza Mesri (1996). *Soil Mechanics in Engineering*, 3rd edition, John Wiley & Sons, 549 p.

Terzaghi, K. (1950). Mechanism of Landslides. In *Application of Geology to Engineering Practice* (S. Paige, ed.), Geological Society of America, New York, pp. 83-123.

Transportation Research Board. (TRB) (1996) Landslides Investigation and Mitigation Special Report 247, National Academy Press, Washington, D.C.

Transportation Research Board (1996), "Landslides: Investigation and Mitigation," A.K. Turner and R.L. Schuster, editors, National Research Council Special Report 247, 673 pp.

Samtani, Naresh C., and Nowatzi, Edward A. (2006), Soil and Foundations Reference Manual – Volume I, Federal Highway Administration, National Highway Institute, Washington, DC. Publication No. FHWA-NHI-06-088.

Skempton, A. W. (1957). Discussion on "The planning and design of new Hong Kong airport." *Proceedings, Institution of Civil Engineers*, Vol. 7 (3), London, 305-307.

Varnes, D.J. (1978). Slope Movement Types and Processes. In *Special Report 176: Landslides: Analysis and Control* (R.L. Schuster and R.J. Krizek, eds.), TRB, National Research Council, Washington, D.C., pp. 11-33.

Walkinshaw, J., (1992), "Landslide Correction Costs on U.S. State Highway System". Transportation Research Record 1343, National Research Council, Washington D.C.,pp.36-41.

Wittke, W. (1990). *Rock Mechanics: Theory and Applications with Case Histories*, Springer-Verlag, New York.

Wyllie, Duncan and. Mah, Chris W (1998), Rock Slopes Reference Manual, Federal Highway Administration, National Highway Institute, Arlington, Virginia. Publication No. FHWA NHI-99-007.

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