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Soil Slope and Embankment Design

Reference Manual

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16. Abstract

This reference manual is an update of 2002 Reference Manual (FHWA NHI-01-026) for the 2 ½ day NHI Course 132033 "Soil Slope and Embankment Design". This manual describes the basic principles of soil slope stability and state-of-the-practice analysis and design procedures for soil slopes and embankments with particular application to transportation facilities. The main topics covered in this manual include:

- geotechnical and geological factors affecting the performance of soil slopes and embankments;
- fundamental concepts of soil mechanics with respect to slope stability and settlement;
- limit equilibrium methods to analyze soil slopes and available computer programs;
- design, construction and performance of highway embankments;
- investigation and mitigation of landslides;
- common alternatives for soil slope stabilization; and
- construction inspection and long-term maintenance.

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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(°F)	5/9(°F- 32)	Celsius temperature(°C)	Celsius temperature(°C)	9/5(°C) + 32	Fahrenheit temperature(°F)

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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PREFACE

This reference manual is an update of the 2002 Reference Manual (FHWA-NHI-01-026) for NHI Course 132033 Module 3, Soil Slope and Embankment Design, prepared by Parsons Brinckerhoff Quade & Douglas, Inc, and authored by J.G. Collin, C.J. Hung, T. Lee, and W. Lee dated Jan. 2002. It was initially intended to be a part (the third) of a series of eleven modules that constitute a comprehensive training course NHI 132016 "Geotechnical and Foundation Engineering." Therefore, the 2002 manual did not discuss topics and issues which were already presented in other Modules such as Geotechnical Investigations (132031), Ground Improvement Method (132034), and Geotechnical Instrumentation (132041).

In order to provide and/or update practicing transportation professionals with a practical and comprehensive understanding of a full spectrum of geotechnical and foundation engineering considerations involved in planning, design and construction of soil slope and embankment features, Parsons Brinckerhoff Quade & Douglas, Inc. was retained to update the original 2002 Soil Slope and Embankment Design course (NHI Course No. 132033) to be a stand-alone 2½-day stand-alone training course. It is intended to provide transportation engineering professionals with knowledge on which to recognize potential soil slope/embankment stability and deformation problems in transportation projects, and to develop the necessary skills to design and evaluate soil slopes and embankments, and consider the construction and inspection implications.

In addition to this updated manual, the visual aids and Participant Workbook (FHWA-NHI-01-027) were also revised. The students will be provided with an updated Participant Workbook (FHWA-NHI-05-124) that contains copies of the visual aids and student exercises.

This reference manual focuses on the scope of geotechnical investigation including desk study, field exploration, and laboratory testing; slope stability and embankment deformation/settlement analysis, design, and construction; landslide investigation and recognition; embankment design; stabilization methods; and construction, maintenance, and monitoring of soil slopes and embankments for highway and related transportation facilities.

This manual is developed to be used as a desktop reference document. After attending the training session, it is intended that the participant will use it as a manual of practice in everyday work. Throughout the manual, attention is given to ensure the compatibility of its content with those of the participant manuals prepared for the other training modules. **Special efforts are made to ensure that the included material is practical in nature and represents the latest developments in the field.**

ACKNOWLEDGMENTS

Permission by the FHWA to adapt the original manuscript and graphics prepared by Parsons Brinckerhoff Quade and Douglas, Inc. (PB) for the original 2002 version (FHWA-01-026) by James G. Collin, C. Jeremy Hung, T. Lee, and W. Lee dated January 2002.

The Authors would like to acknowledge that the original 2002 version of the manual adopted various information from the 1994 publication "Advanced Technology for Soil Slope Stability - Volume 1: Slope Stability Manual" (FHWA-SA-94-005) prepared by Parsons Brinckerhoff Quade and Douglas, Inc. and authored by Abramson, L., Boyce, G., Lee, T., and Sharma, S. (1994)

Furthermore, the authors greatly appreciate Jerry DiMaggio and Peter Osborn of FHWA for their thorough reviews and recommendations, and would like to acknowledge the assistance of a number of professionals from Parsons Brinckerhoff Quade and Douglas, Inc. (PB) in the preparation of this manual. Especially, we are indebted to Amy K. Pavlakovich, Lauren M. Chu, and Theresa T. Kong for their editorial, graphical and computer supports and to Alejandra Morales for her assistance in overseeing the preparation and coordinating the production.

SOIL SLOPE AND EMBANKMENT DESIGN

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Symbols

Chapter 3

σ	Total Stress
ϕ	Angle of Internal Friction
γ	Unit Weight of Material
ϕ'	Effective Angle of Friction
σ'	Effective Stress
γ'_{wz}	Hydrostatic Pressure
γ_b	Submerged Unit Weight of Soil
ϕ_b	The Slope of the Plot of Matrix Suction when σ - u_a is held Constant
γ_t	Total Unit Weight of the Soil
σ_v	Vertical Subsurface Soil Stress at Depth Z
γ_w	Unit Weight of Water
A	Area of Catchment in Unit Length Square
c	Surface Runoff Coefficient
C	Total Cohesion of the Soil
c	Cohesion
c'	Effective Cohesion
C_v	Coefficient of Consolidation
F	Infiltration Rate in Unit Length Per Time
h	Depth of Wetting Front (Thickness of the Wetting Band of the Time t)
h_1	Pore Water Pressure Head
h_2	True Head
i	Total Hydraulic Gradient
I	Design Mean Intensity of Rainfall in Unit Length Per Time
i_c	Critical Hydraulic Gradient for Water Flowing Opposite to Gravity
K	Stress Ratios
k	Coefficient of Permeability
n	Porosity
NLIC	National Landslide Information Center
Q	Maximum Runoff in Cubic Length Per Time
R_u	Pore Pressure Ratio
SCS	Soil Conservation Service
S_f	Final Degree of Saturation
S_o	Initial Degree of Saturation
SPT	Standard Penetration Test
t	Duration of Rainfall
u	Overburden Pressure
u_a	Atmospheric Pressure
USC & GS	United States Coast and Geodetic survey
USDA	US Department of Agricultural
USGS	US Geological Survey
u_w	Pore Water Pressure
Z	Soil Depth
z	Depth of the Point of Interest from the Slope Surface
Z_c	Depth of Tension Crack

Chapter 4

$\overline{U}_h$	Average Degree of Consolidation
ν	Poisson's Ratio
ξ	A Dimensionless Factor which can be Obtained from Figure 4-25
δ'	Differential Settlement
$\Delta\sigma'_v$	Change in Effective Vertical Stress at Mid-height of Layer
$\Delta\sigma_1$	Change in Major Principal Stress
$\Delta\sigma_3$	Change in Minor Principal Stress
σ'_p	Preconsolidation Pressure
$\Delta\sigma_v$	Total Vertical Stress
σ'_{vf}	Final Effective Vertical Stress at the Center of Layer n
σ'_{vo}	Initial Effective Vertical Stress at the Center of Layer n
Δu	Excess Pore Pressure
ε_v	Vertical Strain
Δz	Layer Thickness
A	Pore Pressure Parameter A
B	Pore Pressure Parameter B
B	Width of the Embankment
B_f	Width of Foundation
C	Bearing Capacity Index
C_α	Coefficient of Consolidation
C_c	Compression Index
CD	Consolidated Drained Strength
C_d	A Factor to Account for the Shape of the Loaded Area and the Position of the Point for Which the Settlement is being Calculated
c_h	Coefficient of Consolidation for Horizontal Drainage
C_r	Recompression Index
CU	Consolidated Undrained
C_v	Coefficient of Consolidation (cm/sec)
D	Diameter of the Cylinder of Influence of the Drain
d	Diameter of an Equivalent Circular Drain
e_0	Initial Void Ratio
E_s	Young Modulus for the Soil
ESA	Effective Stress Analysis
F(n)	Drain Spacing Factor
H_0	Layer Thickness
H_d	The Distance to the Drainage Boundary (cm)
I	An Influence factor Which Depends on m and n
I_p	Influence Factor
K	Coefficient Lateral Earth Pressure
Q	Load Applied at the Ground Surface
q_0	The Vertical Stress at the Base of the Loaded Area
q_c	Penetration Resistance
r	Horizontal Distance between the Point of Load Application (a) and the Point where the Vertical Pressure is being Determined (N)
S_i	Immediate Compression
S_s	Secondary Compression
S_s	Secondary Consolidation

S_u	Shear Strength
t	Time (sec)
TSA	Total Stress Analysis
USA	Undrained Stress Analysis
UU	Unconsolidated Undrained
Y_m	Horizontal Displacement

Chapter 5

β	Angle of the Slope
γ	Total Unit Weight of Soil for Slopes Above Water
θ	Inter-slice Force Angle
μ'_w	Seepage Correction Factor
τ_b	Shear at the Base of Each Slice
$\gamma_{buoyant}$	Buoyant Unit Weight for Submerged Slopes
μ_q	Surcharge Correction Factor
θ_r	Right Hand Side Inter-slice Force Angle
τ_{req}	Required Shear Strength
μ_t	Tension Crack Correction Factor
σ_v	Total Vertical Subsurface
μ_w	Submergence Correction Factor
c_d	Developed Cohesion
c_u	Undrained Shear Strength
FDM	Finite Difference Method
FEM	Finite Element Method
GLE	General Limit Equilibrium
H'_w	Height of Water Within the Slope
h_w	Phreatic Surface
H_w	Depth of Water Outside the Slope
k	Seismic Coefficient
K_a	Active Earth Pressure Coefficients
k_{hc}	Critical Coefficient
K_p	Passive Earth Pressure Coefficients
L	Arc Length of Failure Surface
L	Length of Central Block
L_{arc}	Length of Circular Arc
L_{chord}	Length of Chord
nH	Distance from the Toe to the Failure Circle as Indicated by the Short Dashed Curve
O	Center of the Sliding Mass
OMS	Ordinary Method of Slices
P_a	Active Force (Driving)
P_p	Passive Force (Resisting)
q	Surcharge Load
R	Radius of Circular Surface
R_c	Perpendicular Distance from the Circle Center to Force S_m
r_u	Pore Pressure Ratios
W	Weight of Sliding Mass Per Unit Length of Slope
x	Horizontal Distance between Circular Center
x_i	Coordinate of the Right Side of Slice I

Chapter 6

pwp	Pore Water Pressure

Chapter 7

D_d	Maximum Depth of the Displaced Mass, Measured Perpendicular to the Plane Containing W_d and L_d
D_r	Maximum depth of the Rupture Surface below the Original Ground Surface Measured Perpendicular to the Plane Containing W_r and L_r
IAEG	International Association of Engineering Geology
L	Minimum Distance from the Tip of the Landslide to its Crown
L_{cl}	Distance from Crow to tip of Landslide Through Points on Original Ground Surface Equidistant fro Lateral Margins of Surface of Rupture and Displaced Material
L_d	Length of Displaced Mass
L_r	Length of the Rupture
W_d	Width of the Displace Mass
W_r	Width of the Rupture Surface

Chapter 8

MSE	Mechanically Stabilized Earth
RSS	Reinforced Soil Slope

Chapter 9

EPS	Expanded Polystyrene
MPBXs	Multiple-point Borehole Extensometers
QA	Quality Assurance
QC	Quality Control
RSS	Reinforced Soil Slopes
XPS	Extruded Polystyrene

CHAPTER 1
INTRODUCTION

1.1 PURPOSE AND SCOPE

Soil slopes and embankments are a common and complex part of highway engineering in so far as soil mechanic is concerned. A slope is defined as any surface which is at an angle to the permanent horizontal surface of an earth mass. An embankment is a man-made change in elevation created by the use of back to back slopes. Often referred as cut and fill, slope and embankments are routinely utilized to maintain grades and right-of-ways of highways when space is adequate.

All soil slopes (man-made or natural) and embankments are subjected to stresses (i.e., gravity, seepage and earthquake forces) acting on the soil mass that tends to cause downward and outward movements. When the forces induce shear stresses which exceed the resisting shear strength or stiffness of the soil mass, movement occurs and is typically manifested as slope instability or excessive settlement. The study of a slope or embankment must consider a variety of factors, including topography, geology, and material properties. Often, some idealization of problems is required for clarity and for the effective application of basic concepts. Since nature does impose a number of limitations on the application of the principles of mechanics and mathematics to slope stability and settlement problems. In most highway engineering applications, the primary purpose of slope stability analysis is to contribute to the safe and economic design of excavations, embankments, earth dams, and spoil heaps. The term slope stability can be defined as the safety of an earth mass against failure or movement. Preliminary slope stability analyses can identify critical geological, material, environmental, and economic parameters. Due to the severe consequences that slope failures might bring to a highway project, soil slopes must be kept stable both during construction and operation. Therefore, it is essential for this Reference Manual to discuss significant factors that relate to slope stability and settlement analysis.

In practice, the angle is usually designed as the steepest one at which the slope will remain stable as long as necessary. Therefore, the construction costs for slopes and embankments generally are more economical when comparing to other earth retaining and/or structural solutions used when the space is limited. However, this is also the essential point and the reason why the slope must be studied, designed and constructed carefully. Otherwise, the cost savings from design and construction stages might be offset by the long-term maintenance and/or repairs.

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 huge economic losses and even numbers of casualties. The economic losses incurred by slope failures (landslides) include both direct and indirect costs that affect public and private properties. The direct costs are the repair, replacement, or maintenance resulting from damage to highways structures and public/private properties, or from damages caused by flood-induced landslides. 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; and
- Loss of human or animal productivity.

In the spring of 1983, a massive landslide as shown in Figure 1-1 at Thistle, Utah began moving in response to the groundwater buildup from heavy rains in the previous September and the melting of deep snowpack during the winter of 1982-83. Within a few weeks the landslide dammed the Spanish Fork River, obliterating U.S. Highway 6 and the main line of the Denver and Rio Grande Western Railroad. The town of Thistle was inundated under floodwaters that rose behind the landslide dam. Total costs (direct and indirect) incurred by this landslide exceeded $400 million, the most costly single landslide event in U.S. history.

Figure 1-1: Thistle Utah Landslide (U.S. Geological Survey)

Settlement may occur in the embankment or man made slope both during and after construction (Figure 1-2). Although not as dramatic as large-scale landslides, excessive slope and embankment settlement and deformation problems can affect the performance of highway and increase the costs for maintenance and repairs.

Therefore, geotechnical design of soil slopes and embankments must be governed by both the stability and settlement considerations. A successful soil slope or embankment must be able to meet the safety and performance criteria and not to fail under excessive shear stresses, horizontal displacement, and/or vertical settlement.

Figure 1-2: Side Hill Fill Settlement

1.2 TYPES OF SOIL SLOPES AND EMBANKMENTS

Soil slopes along a surface transportation system can be classified into two main categories, i.e. natural and man-made slopes and embankments. General descriptions for natural soil slope and man-made soil slopes and embankments is given below. Detailed discussion and typical concerns and problems associated with each type of structure are presented in Chapter 2.

1.2.1 Natural Slope

Most highways intersect numerous ridges and valleys that can be prone to slope stability problems (Figure 1-3). A natural slope which is stable and without any human interference in its recent past can be at a long-term condition of equilibrium. However, natural slopes that have been stable for many years may suddenly fail because of changes in topography, seismicity, groundwater flows, loss of strength, stress changes, and weathering process. Natural slopes share many of the same failure modes as man-made slopes. When failures do occur in the natural slopes, they are generally on a larger scale than those of man-made slopes.

Figure 1-3: H-3 Highway in Hawaii curves through Mountains (PB Slide Library)

1.2.2 Man-made Slopes/Embankments

In general, man-made slopes in transportation engineering applications include both embankments and cut slopes. Occasionally, fill (reinforced and unreinforced) slopes and side slopes are also encountered. Embankments include highway and railway embankments, highway and bridge approach embankments, and levees (Figure 1-4). The engineering properties of materials used in the embankments are typically controlled by borrow source grain size distribution, Atterberg Limits, the methods and quality of construction, and the degree of compaction.

Cut slopes are the result of excavations made in a natural formation. The stability of an excavation depends on the properties of the soil forming the sides and base of the excavation, the slope inclination and the slope height. The process of excavation represents unloading and the manner in which excavation is carried out may have considerable influence on stability. The engineering issues of cut slopes that require earth retention or reinforcement are discussed in NHI 132036 "Earth Retaining Structures."

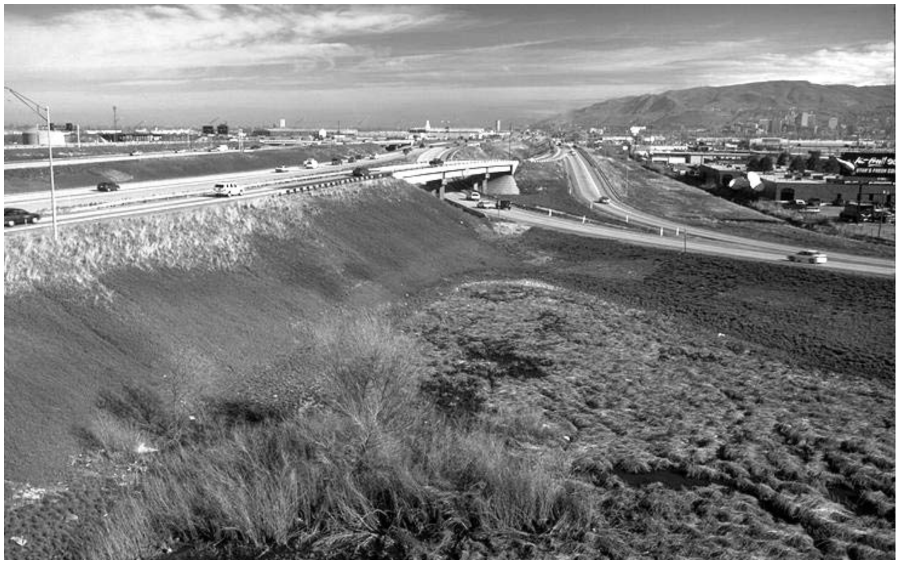

Figure 1-4: Typical highway embankments for improvements and rehabilitation of I-15 Project in Utah. (PB Slide Library)

1.3 ORGANIZATION OF THE MANUAL

This Reference Manual is prepared in conjunction with the NHI training course 132033A – "Soil Slope and Embankment Design" which addresses the design and construction considerations for soil slope and embankment, especially in regard to geotechnical engineering aspects. It is focused on the investigation, design, construction, and maintenance of all types of soil slopes and embankments in highway engineering practice. Discussions of rock slope engineering are included in NHI 132035 "Rock Slopes."

This manual focuses on the scope of subsurface investigation, analysis, design, construction, maintenance, and mitigation of soil slopes and embankments for highway and related transportation facilities. It is intended to be a stand-alone document and is geared towards providing the practicing engineer with a thorough understanding of the design, construction, and maintenance issues for man-made slopes and embankments and natural soil slopes for highway engineering. However, considering the broad scope and fundamental importance of the scope of this manual, many topics already covered in the other training curriculum materials for the following NHI training courses in geotechnical and foundation engineering are heavily referred in addition to the primary references listed in Section 1.4:

- NHI 132031 – Subsurface Investigation (major reference in Chapter 3)
- NHI 132034 – Ground Improvement Methods (major reference in Chapter 9)
- NHI 132035 – Rock Slopes
- NHI 132036 – Earth Retaining Structures (major reference in Chapters 3 and 9)
- NHI 132037 – Shallow Foundations (major reference in Chapter 4)
- NHI 132039 – Geotechnical Earthquake Engineering (major reference in Chapter 5)
- NHI 132041 – Geotechnical Instrumentation (major reference in Chapters 8 and 10)

The organization of the manual is presented below.

- Chapter 2 introduces the types of slope and embankment movements including creep, fall, topple, slide, spread, flow and settlement, and 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 affecting the performance of soil slopes and embankments.

- Chapter 4 discusses the fundamental concepts of total and effective stress, stress distribution, stress-strength relationship, and consolidation concepts with respect to slope stability and settlement analysis and discusses settlement analysis for cohesive and cohesionless soils.

- Chapter 5 discusses slope stability concepts and analysis, and presents and several limit equilibrium methods to analyze soil slopes (i.e., Bishop's Simplified Method, Janbu's Simplified Method, Spencer's Method, etc.).

- Chapter 6 introduces available computer software for slope stability analysis.

- Chapter 7 discusses landslide recognition, identification, and investigation.

- Chapter 8 describes issues related to the design, performance and construction for highway embankments.

- Chapter 9 presents common alternatives for slope stabilization and remediation.

- Chapter 10 discusses construction inspection and long-term monitoring and maintenance.

1.4 PRIMARY REFERENCES

This Reference manual is the update of the 2002 Reference Manual (FHWA-NHI-01-026) for NHI Course 132033 Module 3, Soil Slope and Embankment Design, prepared by Parsons Brinckerhoff Quade & Douglas, Inc. and authored by J.G. Collin, C.J. Hung, T. Lee, and W. Lee dated Jan. 2002. However, significant changes and additions have been made in order to focus the manual specifically on soil slope and embankment issues. Several FHWA and AASHTO references were utilized as primary references in the preparation of this manual. The following is a listing of these primary references. A detailed list of references is provided in Chapter 11.

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CHAPTER 2

COMMON STABILITY AND DEFORMATION PROBLEMS FOR SOIL SLOPES AND EMBANKMENTS OF HIGHWAY ENGINEERING

2.1 INTRODUCTION

Cut slopes and embankment fills are commonly constructed for highway projects. The aim in a slope or embankment design is to determine a height and inclination(s) that are economical and that will remain stable for a reasonable life span. Although design and construction of soil slopes and embankments are common, they are the most complex part of highway engineering insofar as soil and rock mechanics are concerned, since the most complicated soil and rock mechanics problems are associated with slope stability. Sowers (1979) stated that slope stability and deformation problems present major challenges of geotechnical engineering for two reasons: (1) naturally occurring movements interfere with human activities and well-being, and (2) activities of humans aggravate, accelerate, or precipitate instability and subsequent motion.

Landslides, slips, slumps, mudflows, falls are simply various names given for slope failures and instability. Stability problems can occur when a highway alignment is located adjacent to and within areas of potential landslides, sidehill cuts and fills, pre-historic landslides areas, fault zones, abandoned mining operations, soft foundations, buried stream channels, and landfills. For embankments/fills, satisfactory performance depends not only on their own stability but also the foundations that they are resting on. The poor performance of an embankment is more likely to be caused by poor foundation conditions than the embankment itself. It is not difficult to construct an embankment that is competent, however, the “bump” at the end of a bridge is often caused by poor compaction of the embankment fill or improper embankment material selection.

This chapter focuses on typical slope movements including creep, fall, topple, slide, flow and spread, (Section 2.2) and discusses typical slope and deformation problems in highway engineering. Section 2.3 discusses typical slope problems in man-made soil slopes. Section 2.4 discusses typical slope problems in natural soil slopes. Section 2.5 introduces the subject of seismically induced slope movement. Detailed discussions of seismic stability of soil slopes are included in NHI 132039 – Geotechnical Earthquake Engineering.

This chapter sets the foundation for the remaining chapters in this manual. An accurate evaluation of the stability of slopes and embankments is predicated on an understanding of the fundamental modes of movement and deformation. As we will see in chapter 5, we must know the most probable shape(s) of the potential failure surface for a slope or embankment in order to select the appropriate analysis method (i.e., circular failure surface, three part wedge, etc.).

2.2 TYPES OF SOIL SLOPE MOVEMENT AND INSTABILITY

Except for slope creep and subsidence (vertical deformation), soil slopes movements mostly involve a large body (mass) of soil moving along a more or less definite slip surface. Once the induced shear stresses exceed the peak shear strengths of the slope and the underlying soils, a slope becomes unstable and a slope movement/failure could occur along the developed slip surface. Some movements occur suddenly with little or no warning, while others develop slowly over many years. In the case of mass slope movement, it can be kinematically classified into five classes: fall, topple, slide, flow and spread (Varnes, 1978). Fall and topple movements generally occur in steep faces of soil and rock where the fall materials are separated from the parent materials as only minimal contact remains between the parent and the falling materials. In a slide, the downslope movement of the material occurs on a distinct boundary and the moving material is still in direct contact with the underlying material. It is the most common type of soil slope movement. Flow occurs where the movement does not follow a distinct boundary and the moving material does not necessarily remain in contact with the surface of the ground it travels over. A spread represents a sudden movement on water-bearing (liquefied) seams of sand/silt or sensitive clay/silt squeezed laterally by the weight of overlying cohesive soil or rock mass. Small slope failures are usually slides, falls or flows. However, for larger slope movements the failure type can change from one form to the other. For example, the toppling/fall of a steep soil slope can change to a flow and finally becomes a slide after the fallen material is brought to rest. It is important to be able to identify the type of movement or instability of a slope or embankment. The method of stability analysis used to analyze the problem will be depended on the type of failure.

This chapter focuses on common soil slope movements concerning highway engineering such as creep, slide, flow, subsidence (deformation) and the associated problems. Chapter 5 will discuss the different methods of analysis that are appropriate for the different types of movement. Chapter 7 focuses on the landslide definition and investigation.

2.2.1 Creep

Almost all slopes and embankments move, it is just a matter of magnitude. Creep is a very slow downward movement of soil slopes. Terzaghi in 1950 defined creep as a continuous slope movement, which proceeds at an average rate of less than a foot per decade, which is about 30 mm per year. Because the process is slow, most slopes that experience creep may remain stable for a very long period of time. However, slope creep may eventually develop into a mass slope movement and fail rapidly. Broms et al (1991) reported that, as failure is imminent, the creep rate may be as high as a few centimeters per day.

Terzaghi (1950) also identified two general types of creep. One is the seasonal creep which affects mostly the upper crust of a slope due to seasonal freezing and thawing, or wetting and drying. The depth of the surface layer that is affected by seasonal creep is often equal to or less than the depth of seasonal temperature and moisture variation (usually less than 2 meters (6.6 feet) thick) (Broms et al, 1991). This seasonal creep movement can vary from season to season but is always present. Typically the rate of surface creep movement is relatively faster at the surface and decreases with depth (Figure 2-1).

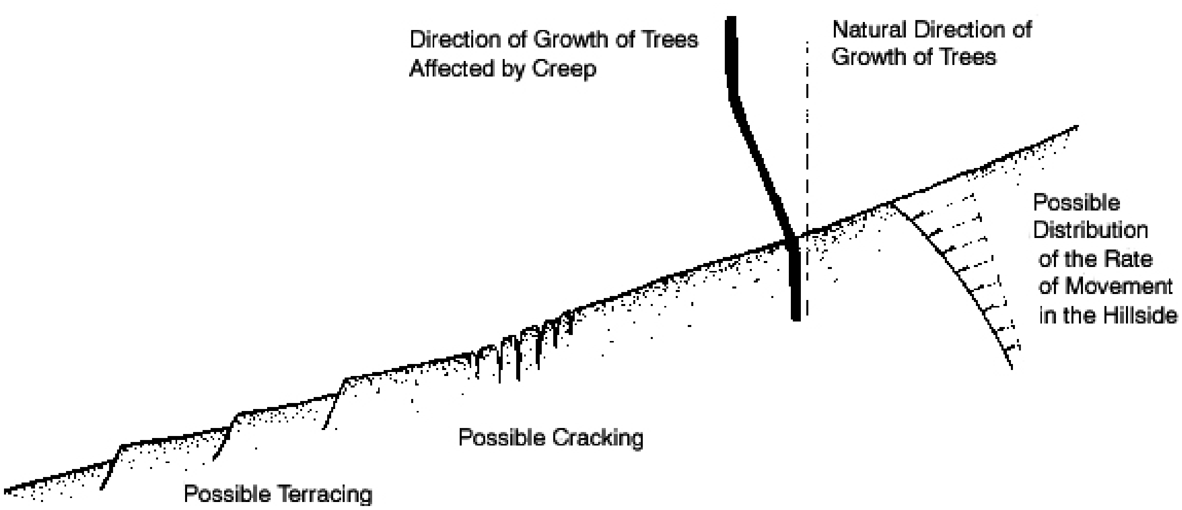


Figure 2-1: Signs of a Surficial Slide or Creep (After Rodriguez et al., 1988)

The second is massive creep which produces almost constant movements in the upper crust ranging up to several meters thick. It occurs more commonly in clayey soils than granular soils. The process of massive creep movements is not very well understood (Rodriguez et al, 1988). However, it has been theorized that an intrinsic creep limited strength may exists which may only be a fraction of the peak shear strength. Below this threshold, no creep movements are expected and the slope surface should remain stable. When shear stresses are above this limit but below the peak shear strength, creep movements will occur. Ultimately enough creep displacements will accumulate which will manifest itself in a slope failure. If the stresses associated with the creep strain are above the peak shear strength, a rapid movement will occur and a landslide will result.

Once creep has started it may progress over a large area and is very difficult to stop. However, its rate of movement can be decreased or controlled by means such as drainage (Chapter 9). It is, therefore, prudent to investigate and identify slope creep before commencing a highway project. The best indication of creep is the gentle curving of tree, with the convex side point downhill in the direction of movement, as shown in Figure 2-1 and Figure 2-1A.

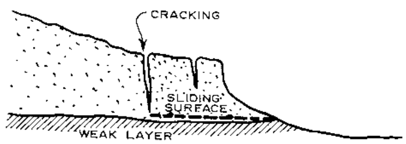
Figure 2-1A: Evidence of Surficial Creep – Curved Tree Trunks

2.2.2 Slide

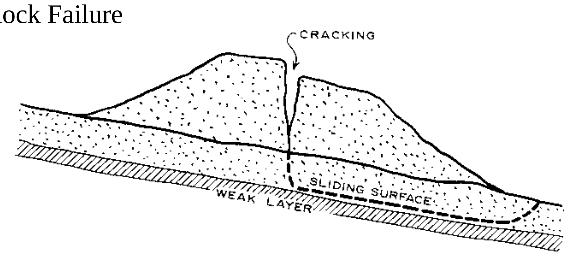
A slide is a downslope movement of a soil or rock mass occurring dominantly on a more or less definite slip (sliding) surface. A slide can be rotational, translational, or composite. It may also be made of a singular or multiple complex slides.

Translational

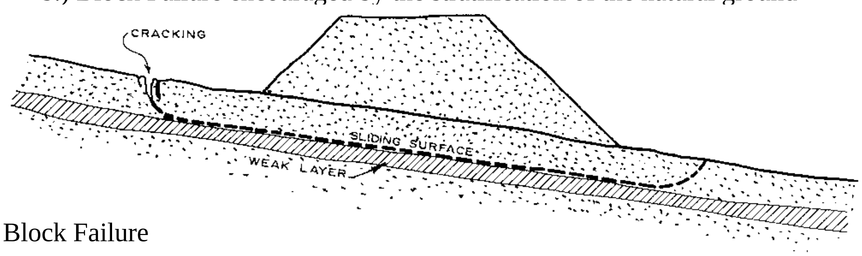
Most slides are translational in nature. This linear movement is especially true when the slide is shallow. The sliding surface is usually shallow and along a plane of weakness (a thin weak layer or stratum boundary (Figure 2-2)). Therefore, translational slides are most common in thinly bedded soil slopes with soft clays, fine sands or loose non-plastic silts. A lateral thrust 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.) Block Failure



b.) Block Failure encouraged by the stratification of the natural ground



c.) Block Failure

Figure 2-2: Typical Translational Slides (Rodriguez et al., 1988)

Figure 2-2A: Translational Slide

Rotational

In weaker soils, the shearing surface can occur through the soil mass and the layer boundaries in the form of an arc. Figure 2-3 shows typical circular rotational slides and some basic terminologies. In soft relatively homogeneous cohesive materials, a deep-seated rotational slide into the foundation (a base slide) is most 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 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 be observed at the head of the failing slide mass.

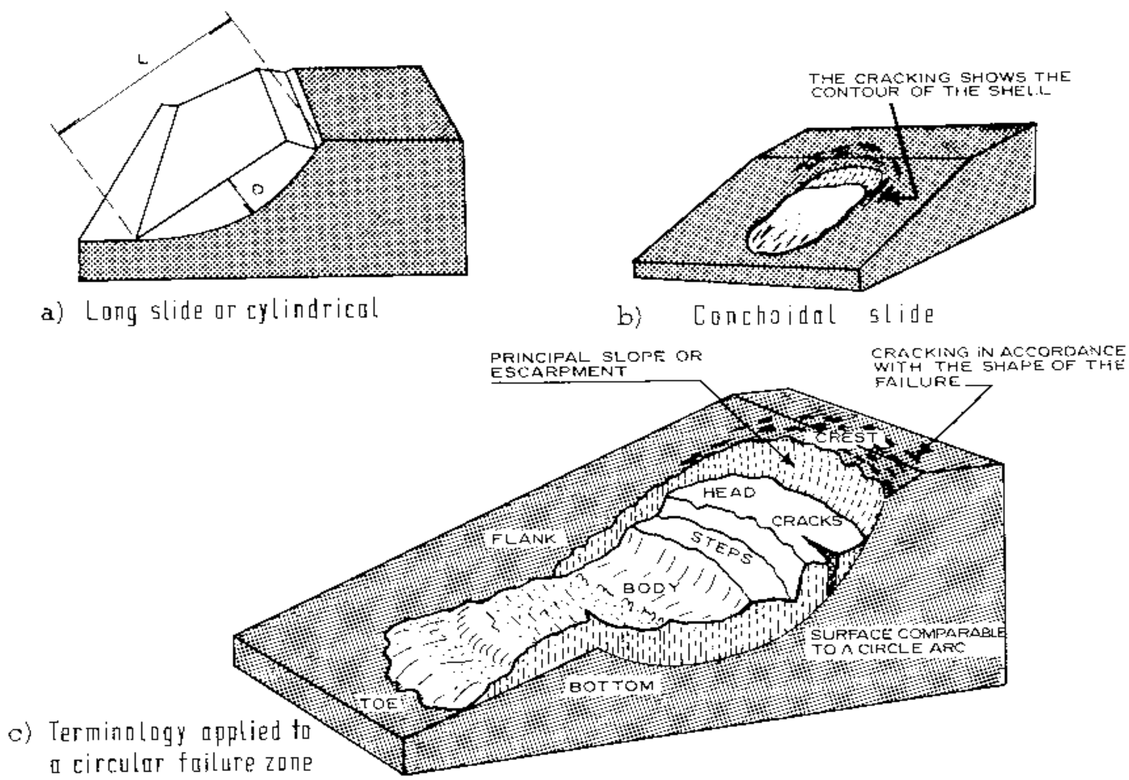


Figure 2-3: Types of Rotational Slides (Rodriques, et al., 1988)

For slopes with layers of materials with varying strengths, only a portion of the slide may rotate. In fact, it is quite common that only the top part of the slide is rotational in stratified soil deposits. The combination of a flat and curved sliding surface can cause large stress on the rear part of the slide. A counterscarp may form on the slide and in most severe cases the rear part may be sheared off and fall to form a feature called a graben. Figure 2-4 illustrates eight common types of circular sliding failures (Varnes, 1978).

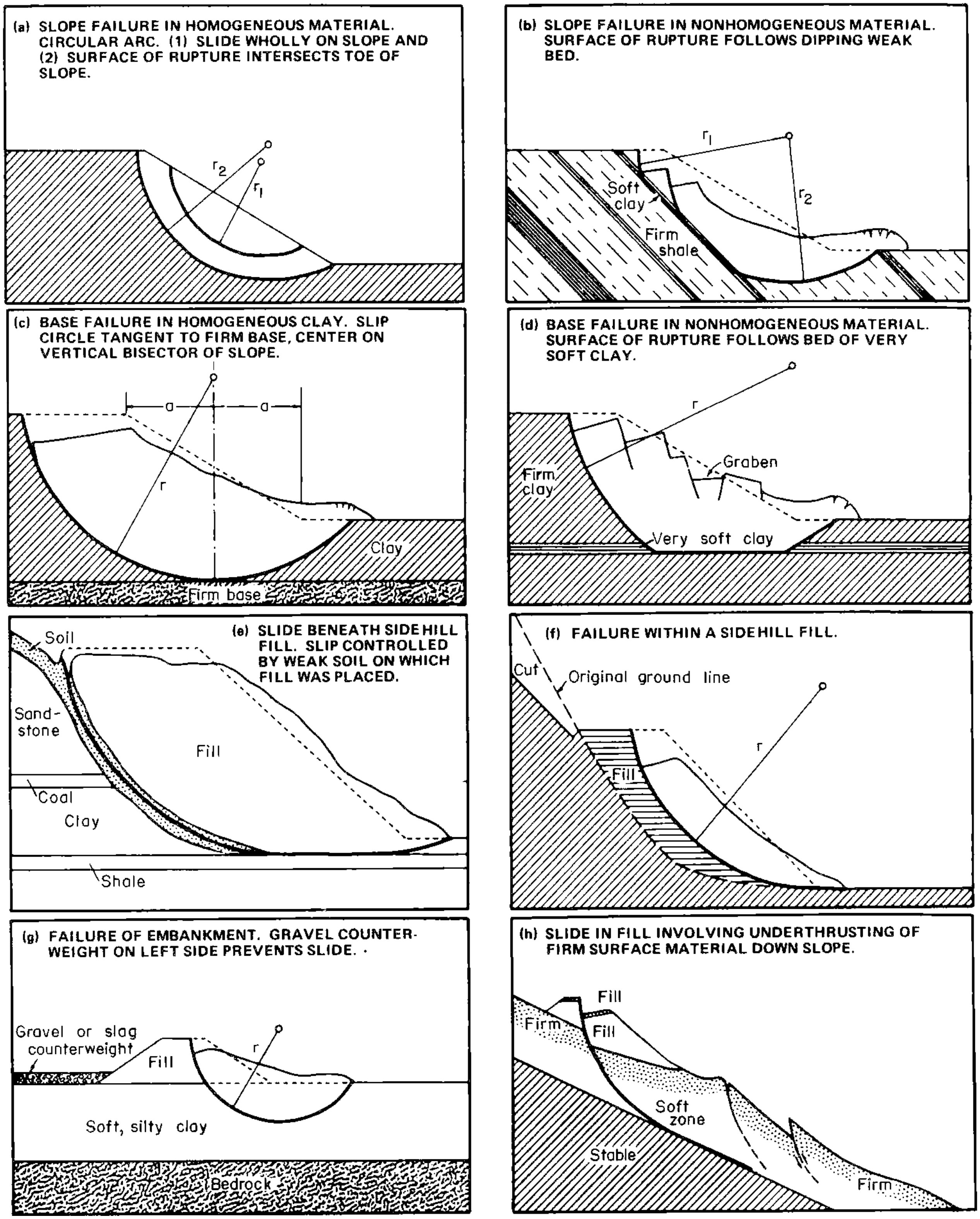


Figure 2-4: Rotational Slides (Varnes, 1978)

Multiple/Compound Slide

In some cases, a slide can occur in a combination of different forms due to the complex structure of the slope (Figure 2-5). For example, because of the multi-benching of the underlying competent materials (e.g. bedrock), the slide can be “multi-storied”. Such slides can sometimes be difficult to predict if the depth to the weak layer is very great.

Disturbed soils are very susceptible to rainwater. In particular, a backward tilting element such as a graben of a slide can lead to the forming of a pond. Water pressures resulting from the ponded water can cause secondary slide movements. Also, during an intense storm the disturbed materials can absorb large amounts of water triggering a secondary slide. These slides exhibit high mobility (yet not quite like a fluid) but can not be classified as flows because they occur along a distinct boundary surface.

Occasionally a compound slide can be ***retrogressive***, meaning that it can keep cutting back uphill into the slope (Figure 2-5a & b). After a single failure, the vertical head 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. Retrogressive rotational slides commonly occur in areas where the topography changes abruptly due to various erosion features. This often occurs when thick layers of overconsolidated fissured clay or shales, with thick overlying layers of rock or firm soils. Retrogressive translational slides are often associated with surface layers of fissured clays and shales. It appears that the more cohesive the material, the fewer independent sliding units are likely to form.

Sometimes a large slide can trigger additional movement in the disturbed material in its toe, or by overriding a lower slope and cause more slides. These are ***progressive*** slides which occur below the original slide surface (Figure 2-5c). The falling material from the newly steepened scarp could be the source of such progressive movements. Very small step-like failures are sometimes formed.

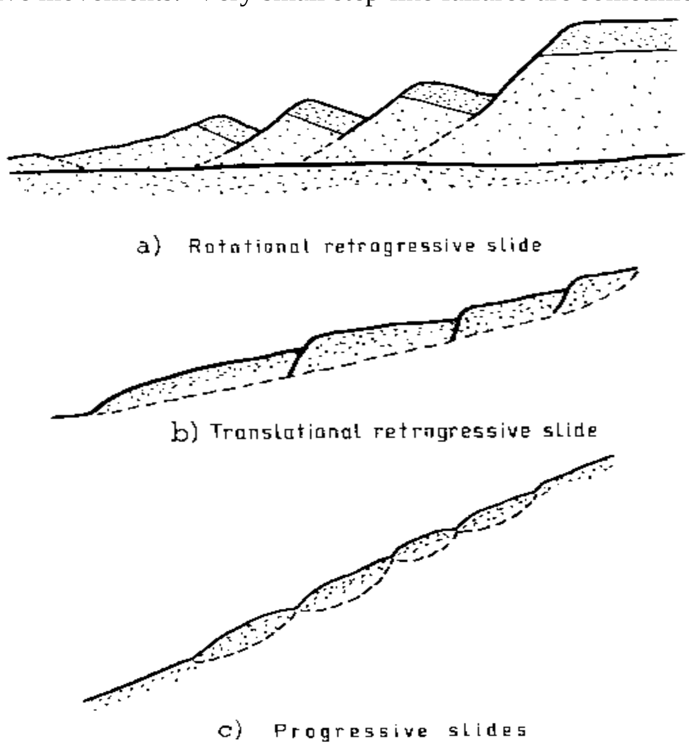


Figure 2-5: Multiple Slides (Rodriguez et al., 1988)

2.2.3 Flow

A flow is defined as a mass movement with much greater internal deformation than a slide. While a slide moves along a distinct boundary, the movements of a flow take place over a large number of discrete shear surfaces, or by the water content of the moving mass being so high that it behaves like a fluid. 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 moisture content is much higher than the liquid limit. In fine-grain non-cohesive soils, flows can take place in loose, dry, silts and sands. These movements are 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 very clear. Detritus flow is sometimes used to describe a flow containing at least 50 percent of gravel, boulders or rock fragments. Figures 2-6, 2-6A, and 2-7 present the common types of flows.

If enough energy is generated, high pore pressure can develop within the water or air trapped inside the (sometimes large size) debris of a fall or a slide. This high pressure will lower the shearing resistance of the debris and a flow can result. However, for large size debris to flow, a sufficiently large amount of water is required. Other flows may result in cuts in sensitive clay areas. Such material is stable in its initial condition, but once it is disturbed, either by physical forces (such as a seismic event or increase in pore pressure) or a change in chemistry of the soil minerals, its shear resistance will decrease dramatically and it becomes quite unstable.

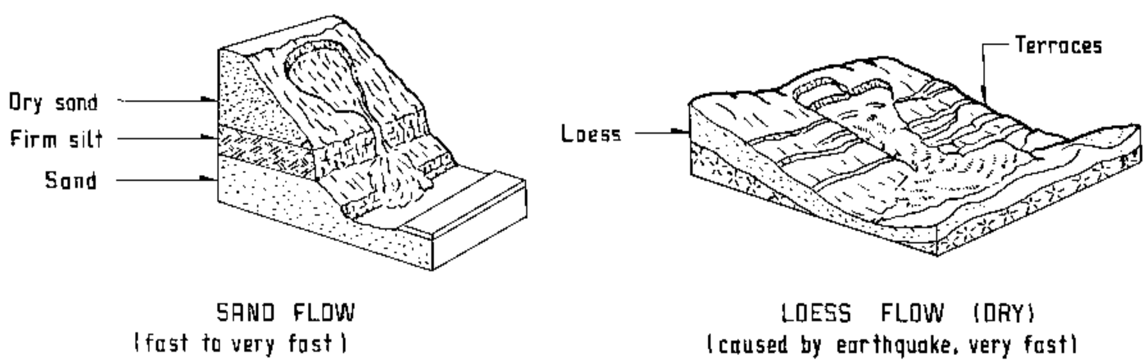


Figure 2-6: Flows in Dry Soils (Rodriguez et al., 1988)

Figure 2-6: Mud Flow

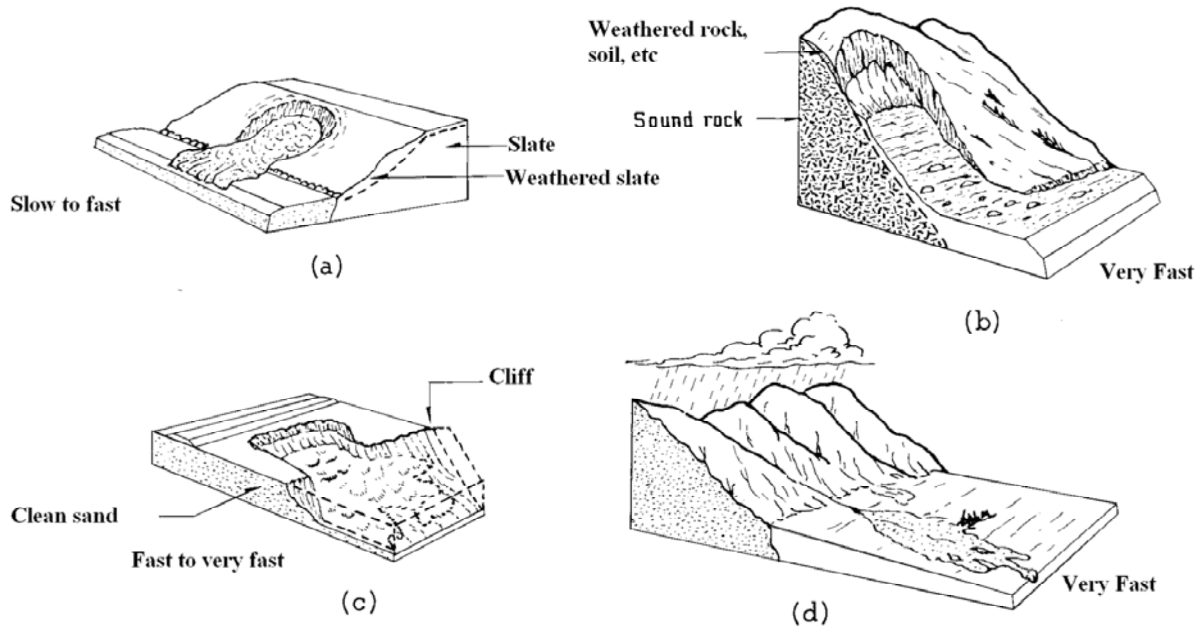


Figure 2-7: Flows in Moist Materials (Rodriguez et al., 1988)

2.2.4 Spread

Terzaghi and Peck in 1948 introduced the term "spread" to geotechnical engineering to describe sudden movements on water-bearing seams of sand or silt overlain by homogeneous clays or loaded by fills (Figure 2-8). Varnes (1978) defined spread as an extension of a cohesive soil or rock mass combined with a general subsidence of the fracture mass of cohesive material into softer underlying material. Spreads typical involve rock, however, soil spreads could occur in condition where overconsolidated soils overlying liquefied material or sensitive soils (quick clays) as illustrated in Figure 2-8.

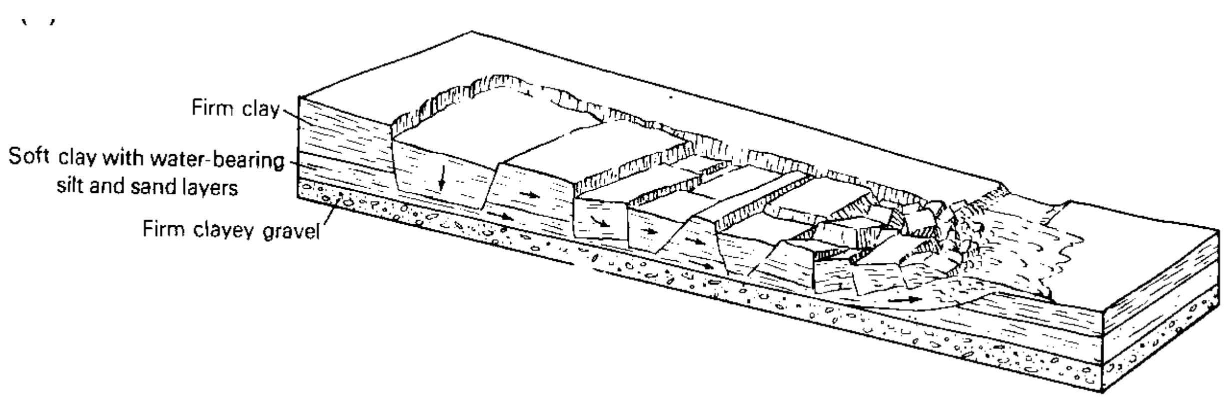


Figure 2-8: Soil Spread (Varnes, 1978)

2.2.5 Fall and Topple

A fall is commonly associated with rock and highly overconsolidated cohesive materials and normally only occurs in steep slopes (Figure 2-9a). 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. Once shear surfaces have developed due to gravity and other stresses, the soil mass above these surfaces moves and is projected out from the face of the slope. As the soil mass continues to project away from the face of the slope, a fall follows. In 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 propagate the depth of the cracks and ultimately forces the material to separate and fall. Ice wedging in these cracks and seismic forces can lead to similar falls.

Falls are a relatively rare mode of failure in soil slopes because they require a steep slope as a staging ground. As the shear strengths 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. Readers are referred to NHI 132035 "Rock Slopes" for detailed discussion for rock falls.

A topple consists of a forward rotation of a mass of soil, debris or rock (Varnes, 1992). The toppling may develop into an abrupt falling or sliding, but the form of movement involves the overturning of interacting columns as shown in Figure 2-9b. Toppling failure mostly occur in rock and debris. Soil toppling usually occurs in clay-rich column-jointed soils. Readers are also referred to NHI 132035 "Rock Slopes" for detailed discussion for rock topples.

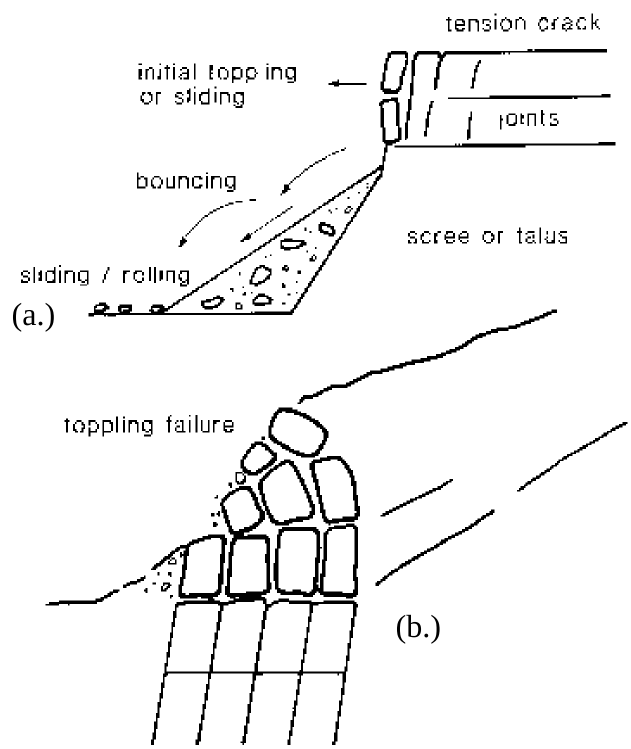


Figure 2-9: Fall and Topple Failures

2.2.6 Embankment Deformation and Settlement

For embankments founded on firm foundations the typical deformation mode is settlement. This settlement may be in the embankment material or in the foundation. Embankment settlement for embankments constructed using granular soils will typically occur very rapidly (i.e. during construction) while embankments constructed with cohesive soils will continue to settle after construction. The settlement of the foundation soil must also be considered.

When constructing embankments over soft compressible soils the global stability of the embankment may control deformation. Settlement of the embankment fill for foundation over soft soils may be relatively small compared to settlement of the foundation. Chapter 4 presents methods for estimating the settlement of the embankment fill and foundation soil.

2.3 COMMON SLOPE STABILITY AND MOVEMENT PROBLEMS IN HIGHWAY ENGINEERING

Shallow and deep cuts and embankment fills are commonly constructed for highway projects. In addition, often roads and highways are routed along the toes of natural slopes which have been in existence for thousands and millions of years. The aim in a slope or embankment design is to determine a height and inclination that is economical and that will remain stable for a reasonable life span. The design is influenced by geological conditions, in situ and fill material properties, seepage pressures, the possibility of flooding and erosion, and the construction methods.

Steep cuts and fills often are necessary because of right-of-way constraints. The initial design must include preventive and protective measures. In some situations, stability at the end of construction may be critical. Many slopes, although stable in the short-term, will fail several to many years later without much warning. Settlement of cohesive soils will occur with time and may not be evident for several years after construction, while cohesionless soils will settle rapidly and will typically be evident during or shortly after construction is complete.

Flat slopes, which may be stable for an indefinite period, are often uneconomical and impractical. However, slopes that are too steep may remain stable for only a short time and pose a real danger to life and property. Frequent failures would also result in tremendous inconvenience and the expense for maintenance, and stabilization measures.

Table 2-1 and 2-2 summarize the important aspects of the stability of cut and fill (man-made) slopes, respectively.

Natural slopes share many of the same failure modes as man-made slopes. When failures occur in natural slopes they are often of a larger scale than those of man-made slopes – since the earlier signs of movements are undetected. In addition, it is uncommon to find homogeneous materials on natural slopes.

The soils are, therefore not typically represented by a simple set of strength parameters. Hillsides consisting of soft cohesive soils (clays) are probably the only major exception to this generalization.

TABLE 2-1
IMPORTANT ASPECTS OF THE STABILITY OF EXCAVATION SLOPES (Duncan et al., 1987)

	Soil Type	
	Cohesionless	Cohesive
Factors that control stability	ϕ' of fill Slope angle Pore pressures External water	Strength of soil γ of soil Slope angle Pore Pressures External water
Failure mechanism	Surface raveling	Deep sliding, possibly extending below toe of slope
Special problems	Surface erosion Liquefaction during earthquake	Strength loss in stiff-fissured clays surface erosion
Critical stages for stability	Long-term or earthquake	End-of-construction, Long-term, or Rapid drawdown
Analysis procedures	Effective stress, or Dynamic	Total stress, Effective stress, or Combination

TABLE 2-2
IMPORTANT ASPECTS OF THE STABILITY OF COMPACTED FILLS (Duncan et al., 1987)

	Type of Fill and Foundation		
	Cohesionless fill on firm foundation	Cohesive fill on firm foundation	Any type of fill on weak foundation
Factors that control stability	ϕ' of fill Slope angle Pore pressures External water	Strength of soil γ of soil Slope angle Pore pressures External water	Strength of foundation Depth of weak foundation layer Strength of fill γ of fill Height of fill Slope angle Pore Pressures External water
Failure mechanism	Surface raveling	Sliding tangent to top of foundation	Deep sliding extending into foundation
Special problems	Surface erosion Liquefaction during earthquake	Surface erosion Weathering and weakening of compacted shales	Embankment cracking Progressive failure Surface erosion
Critical stages for stability	Long-term or earthquake	End-of-construction, Long-term, or Rapid drawdown	End of construction, Long-term, or Rapid drawdown
Analysis procedures	Effective stress, or Dynamic	Total stress, Effective stress or, Combination	Total stress, Effective stress, or Combination

It is plausible that a long standing natural slope is actually at a stage of marginally stable condition. When a large soil mass, under the influence of gravity, deforms over a long period of time, the accumulated deformation typically reduces the shear resistance of the soil (as shear strength normally decreases with increased deformation) and subsequently the soil fails at locations with the highest concentration of shear stresses. Once the soil fails, a failure surface starts to form and there is a redistribution of the shear stresses. If the redistributed shear stresses across the entire soil mass are higher than the available shear resistance, a continuous failure surface/zone will form and a rapid slide will result. Otherwise, the failure surfaces may only be local and no sliding will occur. However, any external events (such as a road cut or higher than usual rainfall) and/or internal events (such as a rise in groundwater levels) can tip this balance and cause the slope to fail.

2.3.1 Slope Stability - Cohesionless Soil

A slope in clean dry sand (Cohesionless soil) will be stable regardless of its height providing the slope inclination β is less than the internal friction angle ϕ of the sand. The stability of a cohesionless soil slope can be indicated by the calculated factor of safety (FS) that is simply equal to:

$$FS = \frac{\tan \phi}{\tan \beta} \tag{2-1}$$

As shown in equation 2-1, the most critical area of the slope is at the surface since any sand grains below the surface will have a flatter inclination than the surface, hence a higher FS. Therefore, more critical sliding surfaces tend to be near the surface of the slope.

As summarized in Table 2-1, in addition to the slope and strength, water plays an important role in deciding the degree of stability of a coheshionless slope. For determining the stability of a partially submerged slope in sand, Equation 2-1 is no longer valid. The FS against failure will also depend on the depth of water table, the groundwater exiting points on the slope and the density of the sand. Because of the lack of cohesion, sands are very susceptible to surface erosion and piping. This may eventually cause the slope face to be steepened to the point that a failure occurs. However, if a coheshionless slope is underwater and totally submerged, equation 2-1 is again valid for a submerged slope in sand providing the ϕ angle is the effective friction angle.

When fine sand gets wet, capillary forces may increase the apparent friction angle of the sand and therefore enable a slightly steeper slope to be built. However, this increase in strength will disappear once the moisture has evaporated and cannot be relied on for long term stability purposes.

Man-made slopes/embankments in cohesionless soils are also susceptible to settlement problems during and immediately after construction. Control of fill placement and specifications that address compaction requirements of the fill are required for the successful construction of fill slopes in cohesionless soils. The potential for settlement of the foundation soil from the weight of the slope or embankment must also be considered during the design.

Natural slopes are more complex since the soils are more heterogeneous thus the stability may be provided by some apparent cohesions (bindings). Furthermore, vegetation may provide some added resistance.

2.3.2 Slope Stability - Cohesive Soil

Relatively deep circular rotational slides are often associated with slopes or embankments constructed on homogeneous cohesive (clayey) materials. Usually the steeper the slope, the deeper the depth of the slide. However, since homogeneous clays are mostly found only in embankments, circular slides in their pure form are typically confined to embankments. For overconsolidated clays, pre-existing fissures, cracks and planes of weakness may distort the circular slip surface.

Cut Slopes

Figure 2-10 shows the general variations of factor of safety, strength, excess pore pressure, load, and shear stresses over time for a cohesive soil cut slope. The initial shear strength is equal to the undrained shear strength. This is based on the assumption that no drainage occurs during construction. In contrast to an embankment slope, the pore pressure within the cut increases over time. This increase is accompanied by a swelling of the clay, which results in reduced shear strength. Thus, the factor of safety decreases over time until an unstable condition is reached, which explains why cut slopes may fail over time after excavation.

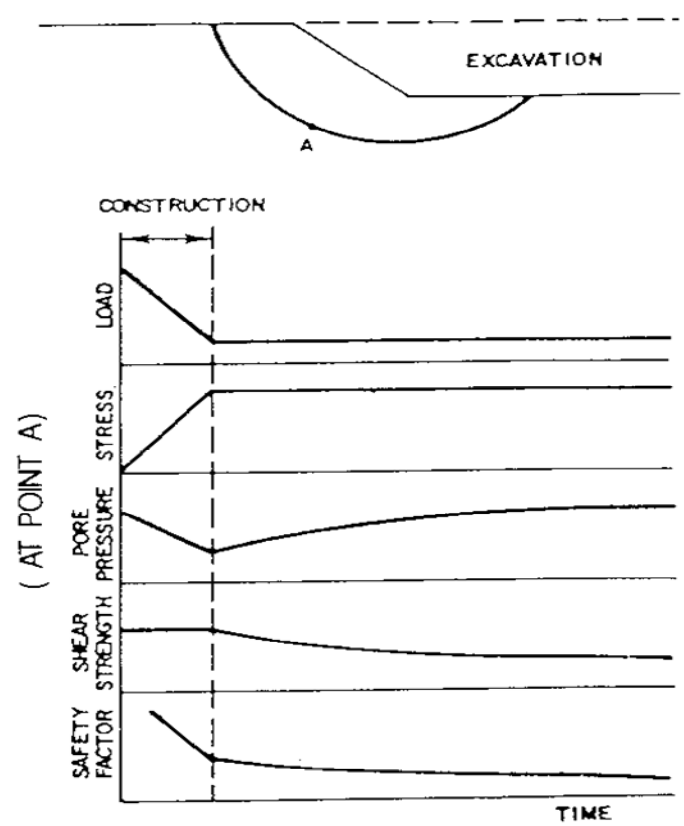


Figure 2-10: Stability Conditions for a Cut Slope (Bishop and Bjerrum, 1960)

For cuts in overconsolidated clays, the in situ shear strength is a direct function of the maximum past overburden pressure; that is, the higher the maximum past overburden pressure, the greater the shear strength. However, if the clay is subjected to long-term unloading conditions (permanent cuts), the strength of the clay no longer depends on the prior loading. The strength of a cut slope will decrease with time. The loss in strength is generally attributed to the reduced negative pore pressure after the excavation. This loss in strength has been observed to be a time-dependent function and appears to be related to the rate of dissipation of negative pore pressure.

In practice, the loss in strength or the rate of negative pore pressure dissipation after a cut is made is not easily determined. According to McGuffey (1982), the time dependency of a cut clay slope failure can be hypothesized to be a function of the Terzaghi hydrodynamic lag model. The estimated time to failure can be expressed as (McGuffey, 1982):

$$t = \frac{h^2 T_{90}}{C_v} \tag{2-2}$$

Where
- t = time to failure
- h = average distance from the slope face to the depth of the maximum negative pore pressure
- T_{90} = time factor for 90% consolidation = 0.848, (Taylor, 1948)
- C_v = coefficient of consolidation.

This model was used with some success by McGuffey (1982) to determine and back-analyze the time for stress release leading to slope failures in clay cuts in New York.

For cuts in the mountainous area, slope stability problems may occur within the residual soils, colluvium, and or saprolite (Section 3.3). Slope movements may occur quickly during or soon after the excavation, or gradually develop over the years. Figure 2-13 shows four common slope movements found in residual soil slopes (Deere and Patton, 1971).

Long-term cut slope stability also is dependent on seepage forces and, therefore, the ultimate ground water level in the soil. After excavation, the free-water surface will usually drop slowly to a stable zone at a variable depth below the new cut surface. This drawdown occurs rapidly for cohesionless slopes but is very slow for cohesive slopes. Although the rate and shape of groundwater drawdown curves for cut slopes have been proposed, none has proved useful for correctly predicting the time or rate of drawdown of preconsolidated clays. The main obstacle to such prediction comes from the difficulty in correctly modeling the recharge of the area in the vicinity of the cut slope.

Embankment Slopes

The engineering properties of materials used in the construction of embankments are controlled by the borrow source grain size distribution, Atterberg Limits, the methods of construction, and the degree of compaction. Since these materials are specified, their properties are better known than the foundations soils. When embankments are built on weak foundations materials, sinking (settlement), spreading, and piping failures may occur irrespective of the engineering properties of the new overlying embankment material.

Figure 2-11 shows the variations of safety factor, strength, pore pressures, load, and shear stresses with time for an embankment constructed over a clay deposit. Over time, the excess pore pressure in the clay foundation soil diminishes, the shear strength of the clay increases, and the factor of safety against stability failure increases.

Embankment fills over soft clay foundations are frequently stronger and stiffer than their foundations. This leads to the possibility that the embankment will crack as the foundation deforms and settles under its own weight, and the possibility of progressive failure because of stress-strain incompatibility between the embankment and its foundation. Settlement of the embankment material must also be considered during the design and is a function of the material used to construct the embankment, the density that the material is placed at, the moisture content of the fill, and the loads imposed on the embankment.

Peak strengths of the embankment and the foundation soils cannot be mobilized simultaneously because of stress-strain incompatibility. Hence, a stability analysis performed using peak strengths of all soils would overestimate the factor of safety (FS). Many engineers perform stability analyses using soil strengths that are smaller than the peak values to allow for possible progressive failure. Otherwise, a failure may occur even though an erroneously adequate FS would be indicated based on the peak strength analysis.

Chapter 7 of this manual discusses the design issues of embankments of soft ground in detail.

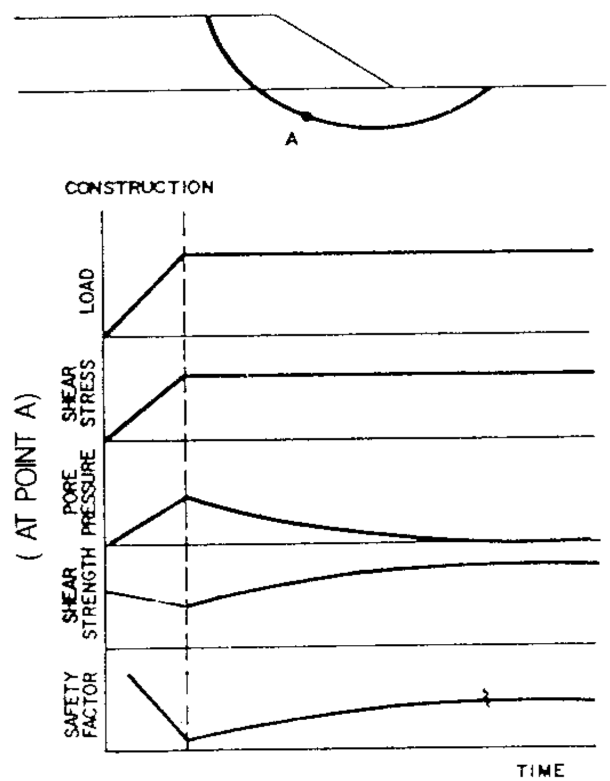


Figure 2-11: Stability Conditions for an Embankment Slope over a Clay Foundation (Bishop and Bjerrum, 1960)

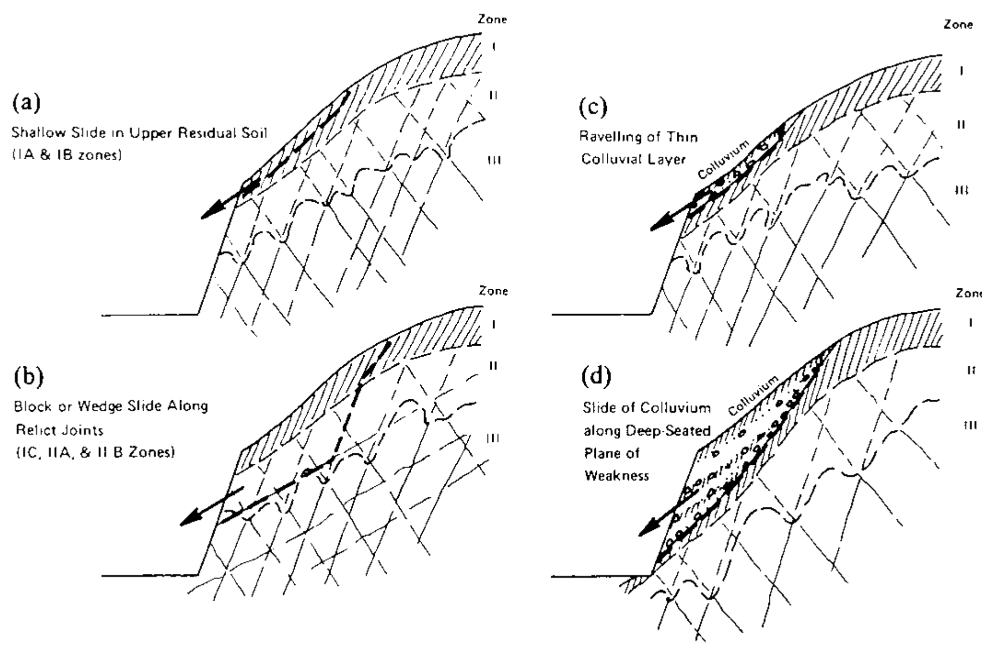
2.3.3 Embankment Settlement

The most common settlement problems are associated with embankments and the "bump at the end of the bridge." Settlement of both the foundation soil and embankment material must be considered when designing embankments. A detailed discussion of settlement analysis and the design of embankments to reduce or eliminate the bump are discussed in Chapters 4, 7 and 9.

2.4 SLOPE STABILITY PROBLEMS FOR SLOPES IN RESIDUAL SOILS

The stability of slopes in residual soils is primarily controlled by three factors. They are the weathering profile, groundwater, and relic structure inherited from the parent rock.

The weathering profile of residual soils is typically a series of soil layers in varying degrees of weathering and, therefore, having varied strength/deformation properties (Section 3.3). The degree of weathering can vary significantly due to rock type, erosion conditions, ground/surface water, topography and temperature. Generally, the strength of residual soils decreases as the degree of weathering becomes more advanced. The residual soils created from clay shale are often thin and the most typical failure mode is a shallow slide. This type of failure is usually preceded by abnormally high groundwater in the underlying fissured shale. In residual soils of metamorphic and igneous rock, most failures occur in the upper layer and are usually associated with high pore pressure due to rainfall. For residual soils originating from limestone and other carbonated rock, stability problems are usually related to sink holes, intense fracturing and frequent interbedding of soft clay. High groundwater pressures commonly exist in rock and partially weathered rock. Such high pressures sometimes are confined by an impervious residual soil cover. If the cover soil is removed or reduced in thickness during a cut, a stability problem will likely develop. Figure 2-12 shows four common slope movements found in residual soil slopes (Deere and Patton, 1971).



*Note: See Figure 3-6 for zone definition

Figure 2-12: Four Common Slope Movements in Residual Soils (Deere and Patton, 1971)

In many instances, residual soils will inherit the exfoliations, joints, cracks, shear zones and other structural defects from the parent rocks. This means that the strength of an intact sample of the residual soil will not necessarily represent the soil mass strength. Slides are more likely to occur along these pre-existing weak planes, similar to that of a rock slide along its joints. Therefore, it is important to establish not only the strength of the intact soil but also of the entire soil mass. This is a complicated affair and may require extensive tests. Laboratory tests conducted on samples containing defects are sometimes performed but direct field tests are likely to be more appropriate. Back calculations from existing slides are also a source for estimating the soil mass strengths.

2.5 SEISMICALLY-INDUCED SLOPE MOVEMENTS/FAILURES

There are numerous documented cases of landslides generated by earthquake ground motions. Liquefaction of even thin seams can induce an overall stability failure of a slope or embankment. These slope failures can occur during or after an earthquake. The slumping of the Los Angeles (Lower San

Fernando) Dam in the 1971 San Fernando earthquake is perhaps the best known example of a liquefaction-induced slope failure.

Lateral spreading is the lateral displacement of large surficial blocks of soil as a result of liquefaction in a subsurface layer. Liquefaction of a layer or seam of soil in even gently sloping ground can often result in lateral spreading. Movements may be triggered by the inertial forces generated by the earthquakes and continue in response to gravitational loads. Lateral spreading has been observed on slopes as gentle as 5 degrees. Figure 2-13 through 2-15 shows schematic examples of liquefaction-induced instability and displacements (Seed et al, 2001). For a detailed discussion on seismically induced slope and embankment movements and their analysis see NHI 132039 - Geotechnical Earthquake Engineering.

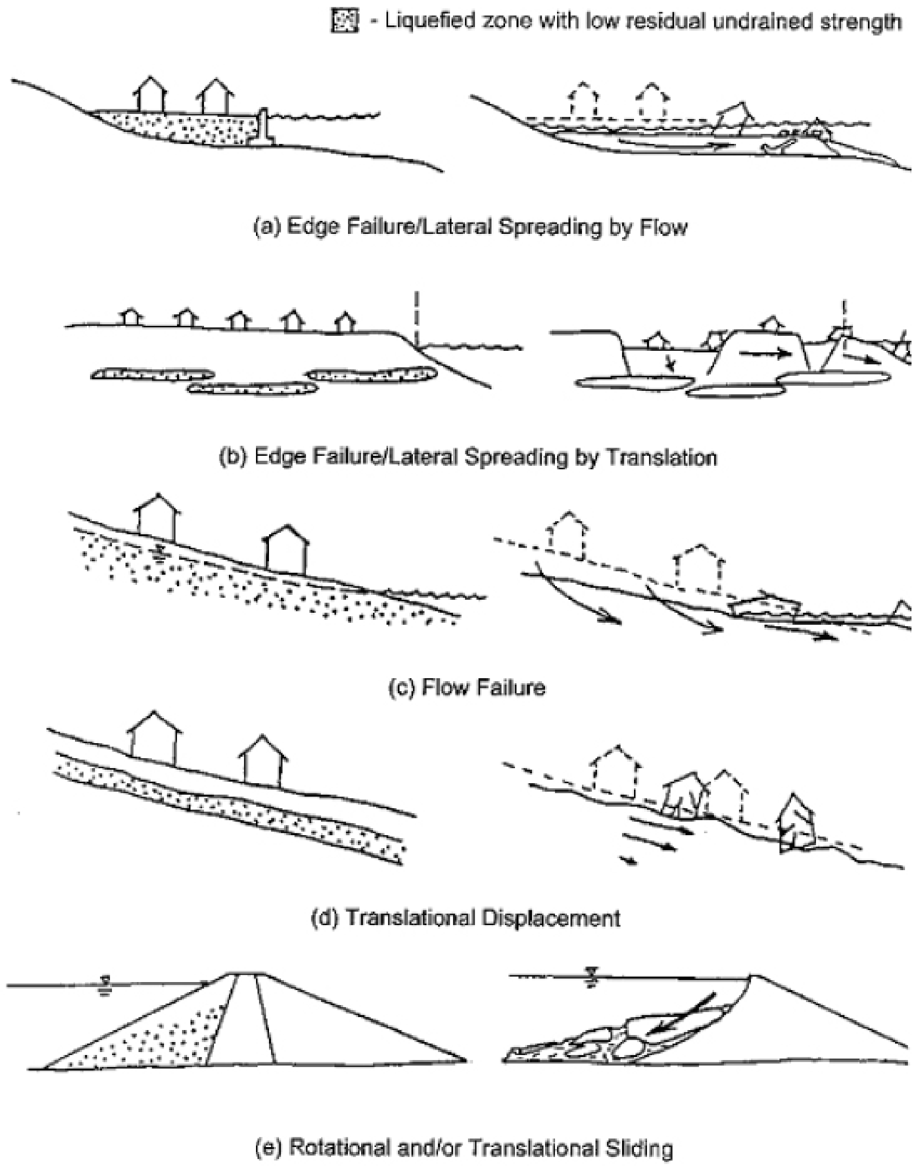


Figure 2-13: Schematic Examples of Liquefaction-Induced Global Site Instability and/or "Large" Displacement Lateral Spreading (Seed et al., 2001)

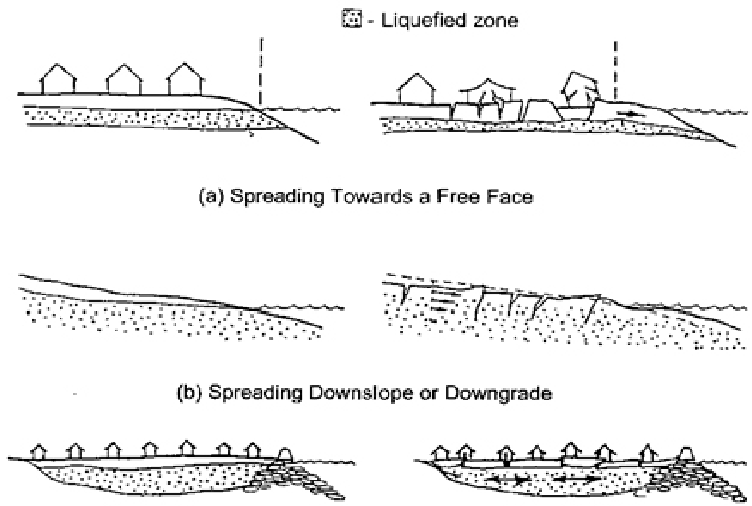


Figure 2-14: Schematic Examples of Modes of “Limited” Liquefaction-Induced Lateral Translation (Seed et al., 2001)

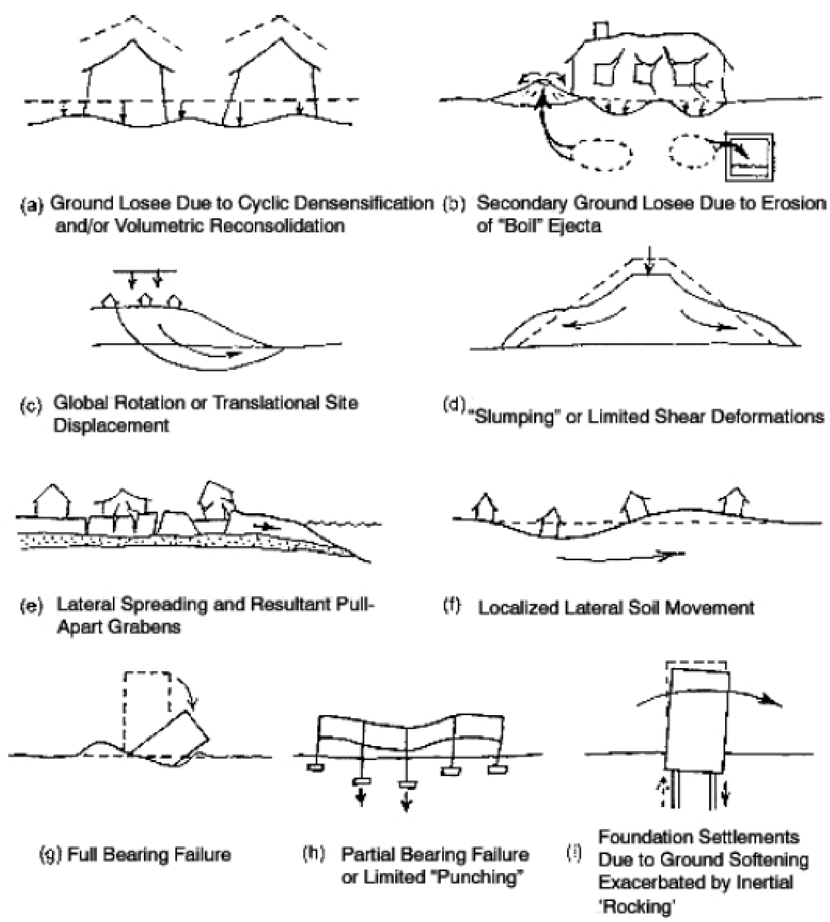


Figure 2-15: Schematic Illustration of Selected Modes of Liquefaction-Induced Vertical Displacements (Seed et al., 2001)

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CHAPTER 3

FACTORS AFFECTING THE BEHAVIOR OF SOIL SLOPES AND EMBANKMENTS

3.1 INTRODUCTION

This chapter discusses numerous factors that can affect the behavior of soil slopes and embankments adversely, and presents information on the evaluation of major ground characteristics and parameters required for design, construction and serviceability of soil slopes and embankments. These required parameters include subsurface stratigraphy and index and performance properties of in-situ soils (e.g., foundation soils below an embankment or retained ground behind a cut slope), and embankment fill soils.

Slope stability is affected by processes that either increase shear stresses or decrease shear strengths of the soil mass. Factors such as the geologic, groundwater and seismic conditions will affect these processes. Discussions of these conditions and other pertinent factors are presented in this chapter.

Settlement of embankments on soft soil is caused by increasing the stresses on the foundation soil. The weight of the embankment will increase the stresses on the foundation soil, as well as cause changes to the groundwater table. The compressibility of the foundation soil affects the amount of settlement that will occur and the permeability of the foundation soil will affect the rate of settlement. The embankment may also settle due to its own weight. The amount of settlement is a function of the compressibility of the embankment material, the moisture content of the fill as it is being compacted, and the compaction energy applied. This chapter discusses the geological, geotechnical and geohydraulic conditions affecting the behavior of soil slopes and embankments. Shear strengths and stiffness parameters are discussed in Chapter 4.

A more detailed coverage of the evaluation of soil parameters for the design of geotechnical features is provided in the following sources:

- NHI Course No. 132031, Subsurface Investigations and accompanying reference manual, FHWA NHI-01-031 (Mayne et al., 2001); and

- Geotechnical Engineering Circular (GEC) No. 5, Evaluation of soil and rock properties, FHWA IF-02-034 (Sabatini et al., 2002).

3.1.1 Ground Characteristics for Design of Soil Slopes and Embankments

Key factors that affect the behavior, analysis, design and construction of slopes and embankments are the ground characteristics and parameters required for analysis, design and construction. They are:

- Geotechnical Parameters (Section 3.3)

- Geological Condition (Section 3.4)

- Shear Strength (Chapter 4)

- Stress-Strain Characteristics (Chapter 4)

- Groundwater Condition (Section 3.5)

- Regional and Site Seismicity (Section 3.6)

This chapter discusses in details of the above key parameters with the exception of shear strength and stress-strain characteristics which are discussed in Chapter 4. In addition, other factors that may affect the behavior of slopes and embankment during and after construction are discussed with the processes that increase shear stresses and/or decrease shear strengths hereafter. These processes are discussed in Sections 3.1.2 and 3.1.3.

3.1.2 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.

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.

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.

Table 3-1 presents the factors that commonly lead to an increase in shear stresses in natural or man-made slopes and embankments. These factors will be discussed further in this Module as they relate to modeling, design, construction, and maintenance of highway soil slopes and embankments.

Table 3-1
FACTORS THAT MOST COMMONLY CAUSE AN INCREASE IN THE 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
     1. Cuts and excavations
     2. Removal of retaining walls or sheet piles
     3. 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
     1. Construction of fill
     2. Buildings and other overloads at the crest
     3. 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)
   - E. 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.

3.1.3 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 3-2 lists the factors that often cause a reduction in the shear strength of the materials of natural and man-made slopes.

TABLE 3-2
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
	D.	Stratification
II.	Changes caused by weathering and physiochemical activity	
	A.	Wetting and drying processes
	B.	Hydration
	C.	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.		

3.1.4 Identify Data Needs for Planning Geotechnical Investigation

A successful design of a soil slope or embankment requires knowledge of the geotechnical parameters required and his/her understanding of other data needed

The ultimate goal of this phase of a site investigation is to identify geotechnical data needed for the project and potential methods available to obtain this information. During this phase it is necessary to:

- Identify design and constructability requirements
- Identify performance criteria and schedule constraints
- Identify areas of concern on site and potential variability of local geology
- Develop likely sequence and phases of construction
- Identify engineering analyses to be performed
- Identify engineering properties and parameters required for these analyses
- Evaluate methods to obtain parameters and assess the validity of such methods for the material type and construction methods
- Evaluate the number of tests/samples needed and appropriate locations for them

As an aid to assist in the planning of the site investigation and laboratory testing, Table 3-3 provides a summary of the information needed and testing considerations for cut and fill wall applications. Detailed descriptions of the field and laboratory tests listed in Table 3-3 are provided in Chapter 4 of Geotechnical Engineering Circular (GEC) No. 5 (Sabatini et al., 2002).

This Chapter discusses the above characteristics except for the shear strength and stiffness (consolidation) which will be discussed in the following Chapter.

3.2 PLANNING OF GEOTECHNICAL INVESTIGATION FOR SOIL SLOPE AND EMBANKMENT DESIGN

3.1.1 General

Planning of a geotechnical investigation including field investigation and laboratory testing program for a soil slope or embankment project requires that the engineer be aware of the parameters needed for design and construction, as well as having an understanding the geologic, surface, and subsurface conditions. Specific steps of planning a subsurface investigation and laboratory testing program include:

1. Identify data needs and review available information
2. Develop and conduct a site investigation program including a site visit/reconnaissance and collection of disturbed and undisturbed soil sampling
3. Develop and conduct a laboratory-testing program (Chapter 4). Specific planning steps are addressed in the following sections

TABLE 3-3
SUMMARY OF INFORMATION NEEDS AND TESTING CONSIDERATIONS FOR SOIL SLOPE AND EMBANKMENT APPLICATIONS

Geotechnical Issues	Engineering Evaluations	Required Information for Analyses	Field Testing	Laboratory Testing
Excavations and Cut Slopes	• slope stability • bottom heave • liquefaction • dewatering • lateral pressure • soil softening/progressive failure • pore pressures	• subsurface profile (soil, ground water, rock) • shrink/swell properties • unit weights • hydraulic conductivity • time-rate consolidation parameters • shear strength of soil and rock (including discontinuities) • geologic mapping including orientation and characteristics of rock discontinuities	• test cut to evaluate stand-up time • piezometers • CPT • SPT (granular soils) • vane shear • dilatometer • geophysical testing	• hydraulic conductivity • grain size distribution • Atterberg Limits • triaxial tests • direct shear tests • moisture content • slake durability
Embankments and Embankment Foundations	• settlement (magnitude & rate) • bearing capacity • slope stability • lateral pressure • internal stability • borrow source evaluation (available quantity and quality of borrow soil) • required reinforcement	• subsurface profile (soil, ground water, rock) • compressibility parameters • shear strength parameters • unit weights • time-rate consolidation parameters • horizontal earth pressure coefficients • interface friction parameters • pullout resistance • geologic mapping including orientation and characteristics of rock discontinuities • shrink/swell/degradation of soil and rock fill	• nuclear density • plate load test • test fill • CPT • SPT (granular soils) • Dilatometer • vane shear • geophysical testing	• 1-D Oedometer • triaxial tests • direct shear tests • grain size distribution • Atterberg Limits • organic content • moisture-density relationship • hydraulic conductivity • geosynthetic/soil testing • shrink/swell • slake durability • unit weight

The site conditions obviously play a very important role in the design and performance of soil slopes and embankments. Adequate knowledge of the surficial and subsurface conditions is essential for a successful design. The two principal components of site exploration associated with slopes and embankments are desk studies and field investigations. Useful information can be gathered from desk studies and from an examination of the construction records and performance of existing structures in the vicinity of the site. The desk study should form the first phase of a geologic site exploration, and the subsurface work should be planned only after assessing the results of the desk study. The engineer should also visit the site during the initial phase of the investigation to get familiar with the site conditions. The planning of the geotechnical investigation should be based on observations of the site conditions and findings of the desk studies with emphasis focused on the potential problem areas.

A flow chart is presented in Figure 3-1 that shows a series of operations used for planning a geologic site exploration for the design of slopes. This flow chart is intended for general guideline purposes, as the sequence of the operations could be altered one way or the other according to the nature of the project.

3.2.1 Review of Available Data – Desk Study

The first step of an investigation and testing program requires that the engineer review all available data and understand the project requirements and the site conditions and/or restrictions. The extent of the investigation should be consistent with the project scope (i.e., location, size, risk, and budget), the project objectives (i.e., purpose of the wall system), and the project constraints (i.e., geometry, constructability, performance, aesthetics, and environmental impact). Before any equipment is mobilized to the site, existing data for the site (both regionally and locally) should be evaluated and the geotechnical and design engineers should conduct a site reconnaissance as logical initial steps in the investigation. Existing data and the site visit will provide information which can reduce the scope of the subsurface investigation, help guide the location of sampling and testing points, and reduce the amount of time in the field due to unexpected problems. Currently, many state DOTs are developing geotechnical management systems (GMS) to store historical drilling, sampling, and laboratory test data for locations in their states. Such data, if available, should be used to facilitate development of a testing program that is correctly focused and not redundant. A list of information sources along with the type of information available is presented in Table 3-4.

In addition to the available data listed in Table 3-4, many have found the following data useful for slope and embankment projects.

<u>Landslide Records</u>

Many State transportation departments, geologic surveys, and university geology and civil engineering departments have gathered records of landslides in their States. Each landslide record may consist of: (1) location of the landslide, (2) date and time of occurrence, (3) geometry of the slope before and after the landslide (which is accompanied by a photograph), (4) material of the slope, (5) possible causes that may have triggered the landslide, and (6) rainfall data. Locations of the landslides are usually summarized in a State or county map for future reference. These records are essential for the engineers planning exploration programs, as well as for making decisions regarding slope stability at the site. Details of particular landslides sometimes can be obtained from local residents. The qualitative description of such incidents may be reasonably accurate, however, the details of timing are often less reliable.

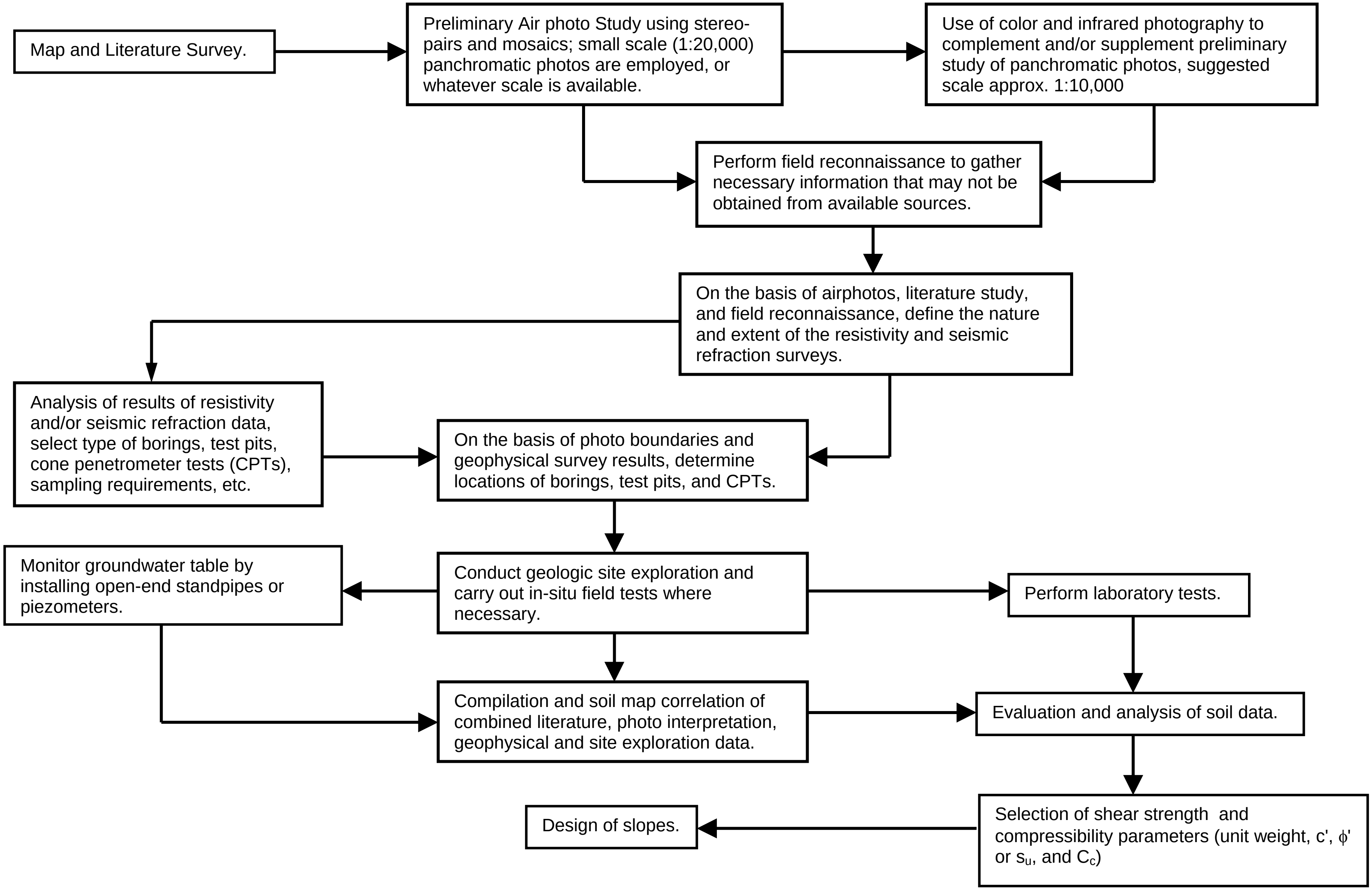


Figure 3-1: Flowchart of the Planning of Site Geologic Exploration for the Design of Slope (Modified from Mintzer, 1962)

TABLE 3-4
SOURCES OF HISTORICAL SITE DATA

Source	Functional Use	Location	Examples
Aerial Photographs	• Identifies manmade structures • Identifies potential borrow source areas • Provides geologic and hydrological information which can be used as a basis for site reconnaissance • Track site changes over time	Local Soil Conservation Office, United States Geological Survey (USGS), Local Library, Local and National aerial survey companies	Evaluating a series of aerial photographs may show an area on site which was filled during the time period reviewed
Topographic Maps	• Provides good index map of the site • Allows for estimation of site topography • Identifies physical features in the site area • Can be used to assess access restrictions	USGS, State Geological Survey	Engineer identifies access areas/restrictions, identifies areas of potential slope instability; and can estimate cut/fill capacity before visiting the site
Prior Subsurface Investigation Reports	• May provide information on nearby soil/rock type; strength parameters; hydrogeological issues; foundation types previously used; environmental concerns	State DOTs, USGS, United States Environmental Protection Agency (USEPA)	A five year old report for a nearby roadway widening project provides geologic, hydrogeologic, and geotechnical information for the area, reducing the scope of the investigation
Geologic Reports and Maps	• Provides information on nearby soil/rock type and characteristics; hydrogeological issues, environmental concerns	USGS and State Geological Survey www.usgs.gov	A twenty year old report on regional geology identifies rock types, fracture and orientation and groundwater flow patterns
Water/Brine Well Logs	• Provide stratigraphy of the site and/or regional areas • Varied quality from state to state • Groundwater levels	State Geological Survey/National Resources	A boring log of a water supply well two miles from the site area shows site stratigraphy facilitating evaluations of required depth of exploration
Flood Insurance Maps	• Identifies 100 to 500 yr. Floodplains near water bodies • May prevent construction in a floodplain • Provide information for evaluation of scour potential	Federal Emergency Management Agency (FEMA), USGS, State/Local Agencies www.fema.gov	Prior to investigation, the flood map shows that the site is in a 100 yr floodplain and the proposed structure is moved to a new location
Soil Survey	• Identifies site soil types • Permeability of site soils • Climatic and geologic information	Local Soil Conservation Service	The local soil survey provides information on near-surface soils to facilitate preliminary borrow source evaluation
Sanborn Fire Insurance Maps	• Useful in urban areas • Maps for many cities are continuous for over 100 yrs. • Identifies building locations and type • Identifies business type at a location (e.g., chemical plant) • May highlight potential environmental problems at an urban site	State Library/Sanborn Company (www. Sanborncompany.com)	A 1929 Sanborn map of St. Louis shows that a lead smelter was on site for 10 years. This information prevents an investigation in a contaminated area.

The National Landslide Information Center (NLIC) of USGS distributes information about landslides to the lay public, researchers, planners, and local, state, and federal agencies. The NLIC provides a toll-free number (1-800-654-4966) and a webpage, http://landslides.usgs.gov whereby anyone can make inquiries about landslides. USGS also complies a landslide overview map indicating the landslide susceptibility which was defined as the probable degree of response of (the aerial) rocks and soils to natural or artificial cutting or loading of slopes, or to anomalously high precipitation.

Seismic Records

Historical seismic records are used to assess earthquake hazards. For example, records exist from the three great earthquakes that occurred in 1811-1812 near New Madrid, Missouri to the Northridge Earthquake, which occurred in Northridge, California, a suburb of Los Angeles, in 1994. During the past 60 years, a second source of data has emerged that can pinpoint the location of earthquake epicenters and determine their magnitudes by means of sensitive seismographs located around the U.S. This more complete set of data gives a much better picture of seismic activity. Although large earthquakes occur less frequently in the Midwest and East than in the West, the seismic hazard in these regions should not be overlooked. AASHTO provides a contour map of mean horizontal acceleration on rock (expressed as a percentage of gravity) with 90% probability of not being exceeded in 50 years. USGS produced a series of similar maps with different degrees of probability in exceedance in 50 years as shown in Figure 3-2.

Records of slope failures caused by earthquakes are documented by the USGS and some State Transportation Departments. With these records, USGS geologists and geotechnical engineers and the State Transportation Departments have developed techniques for mapping areas most likely to be hazardous because of earthquake-triggered earth movements (Figure 3-3). Such maps are a valuable tool for regional planning and are available on USGS web site, http://geohazards.cr.usgs.gov/eq/html/intermaps.html. NHI 132039 "Geotechnical Earthquake Engineering" also contains information about how to obtain relevant seismic records.

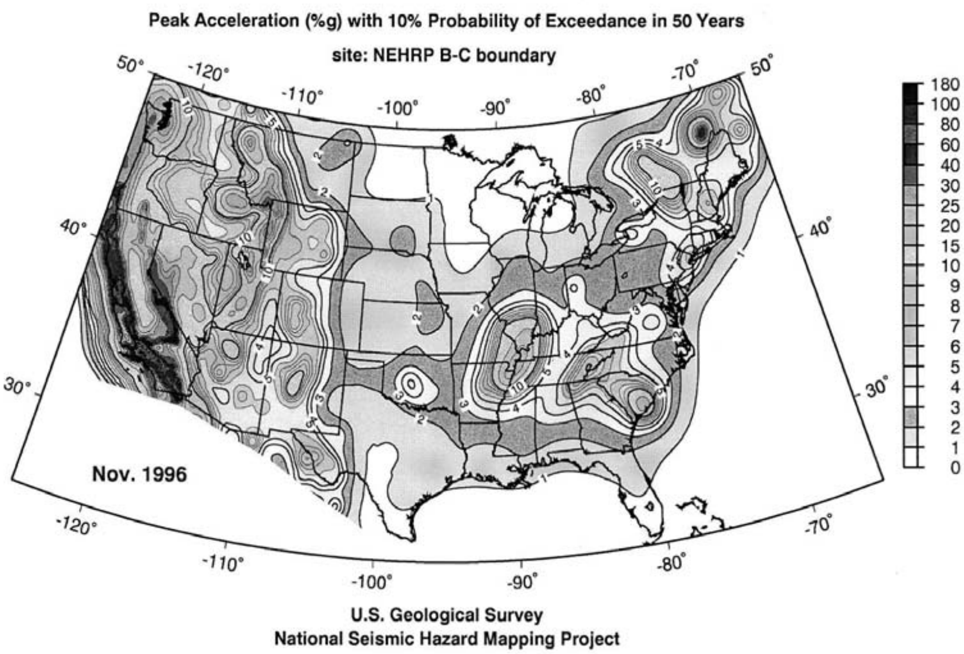


Figure 3-2: Contour Map of United States Showing Peak Acceleration (%g) with 10% Probability of Exceedance in 50 years (USGS, 1996)

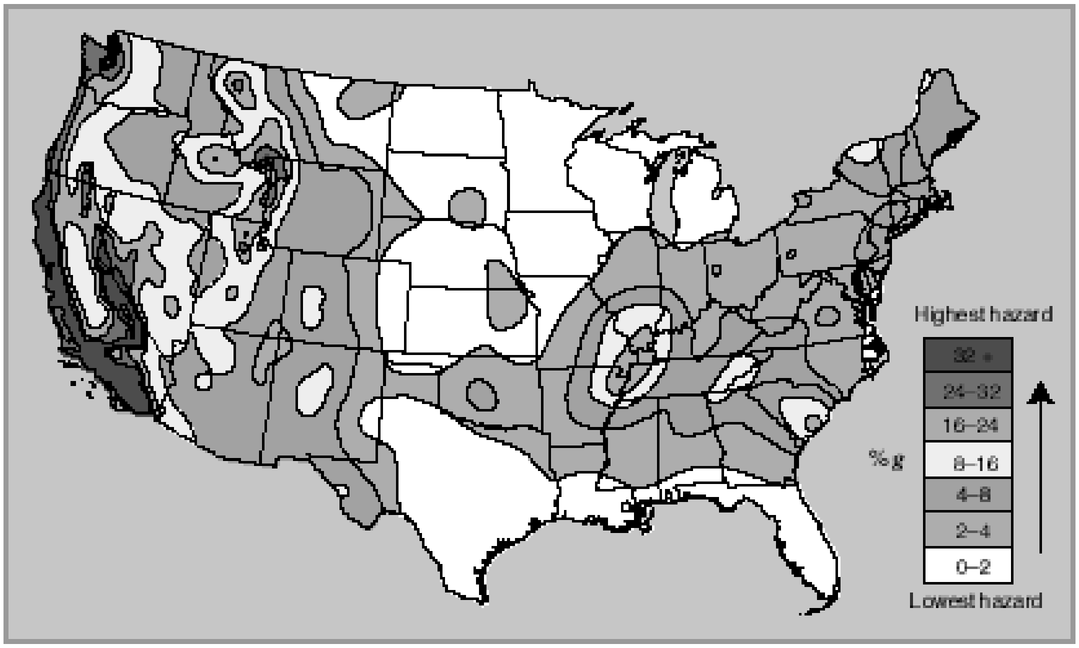


Figure 3-3: Seismic Hazard Map of United States (USGS, 1996)

Remote Sensing Data

Remote sensing data can effectively be used to identify terrain conditions, geologic formations, escarpments and surface reflection of faults, buried stream beds, site access conditions and general soil and rock formations. Remote sensing data from satellites (i.e. LANDSAT images from NASA), aerial photographs from the USGS or state geologists, U.S. Corps of Engineers, commercial aerial mapping service organizations can be easily obtained, State DOTs use aerial photographs for right-of-way surveys and road and bridge alignments, and they can make them available for use by the geotechnical engineers.

3.2.2 Developing a Field Investigation Program

The planning phase involves decisions regarding sampling/investigation methods, boring locations, number of samples, number and types of laboratory tests, and the number of confirmatory samples. At this stage, the types of potential sampling/investigation methods should have been identified and assessed (see 132031A Manual on Subsurface Investigation (2001) for types and features of field subsurface investigation methods). For major slope/embankment projects, the investigation might consist of a preliminary investigation phase (which may be performed as part of a larger phased geotechnical investigation during the preliminary project phase) followed by a final design phase investigation. The preliminary phase includes a limited number of fairly widely-spaced borings to define overall subsurface conditions and to identify problem areas.

The type, number, location, and depth of investigation points are dictated, to a large extent, by the project stage (i.e., feasibility study, preliminary, or final design), availability of existing geotechnical data, variability of subsurface conditions, cut or fill, and other project constraints.

For conceptual design or route selection studies, very wide boring spacing (up to 300 m or 1000 feet) may be acceptable particularly in areas of generally uniform or simple subsurface conditions. For preliminary design purposes a closer spacing is generally necessary, but the number of borings would be limited to that necessary for making basic design decisions. For final design, however, relatively close spacing of borings may be required (30 – 60 m (100 – 200 ft)). AASHTO recommended that a minimum of one boring should be performed for each cut slope or embankment section. For a feature more than 60 m (about 200 feet) in length, the spacing between borings along the length of the cut should generally be between 60 and 120 m (200 and 400 feet). At critical locations and high cuts, provide a minimum of three borings in the transverse direction to define the existing geological conditions for stability analyses. For an active slide, place at least one boring upslope of the sliding area.

The guidelines for depth interval selection should also be developed in recognition of specific site/project conditions and design property/parameter requirements. Basically, investigation depth must be deep enough to fully penetrate soft highly compressible soils (e.g., peat, organic silt, soft fine grained soils) into competent material of suitable bearing capacity (e.g., stiff to hard cohesive soil, compact dense cohesionless soil, or bedrock). AASHTO suggested that for a cut slope (excavation) project, borings should extend a minimum of 5 m (about 17 feet) below the anticipated depth of the cut at the ditch line. Borings depths should be increased in locations where base stability is a concern due to the presence of soft soils, or in locations where the base of the cut is below groundwater level to determine the depth of the underlying pervious strata. As for an embankment project, AASHTO requires borings extended to a minimum depth equal to twice the embankment height unless a hard stratum is encountered above this depth. Where soft strata are encountered which may present stability or settlement concerns the borings should extend to hard material.

The results of the subsurface investigation program should consider the following:

1. Special groundwater conditions such as the presence of artesian pressure or perched water can have a critical effect. Groundwater considerations are discussed in Section 3.5. Experience with soil running up into a borehole during withdrawal of drilling tools, loss of drilling fluid, or sloughing of soil from walls of the borehole should be recorded.

2. In boring operations, any evidence of extremely compact materials, cobbles, boulders, cemented layers or lenses, "hardpan", rubble fill, debris or other material that would impede excavation should be recorded. Such information should be recorded on boring logs to be included in the contract information for bidders.

3. At a site where rock or rock-like material are present, the borings must define the character of the materials where there is a transition between residual soil and bedrock, and should locate the surface of materials of rock-like hardness.

3.3 SITE EXPLORATION TECHNIQUES

Field exploration methods usually consist of borings, sampling, and in situ testing. Borings are usually employed to identify the subsurface stratigraphy, and to obtain disturbed and undisturbed samples while in situ tests are normally used to predict the engineering and index properties of the subsurface material. Some in-situ tests such as cone penetrometer test, however, can also be used for stratigraphy identification purposes. Common boring techniques include augers and rotary wash borings in soils. In rock, coring is usually performed. Common sampling techniques included split spoon samples (disturbed) and Shelby tube samples (undisturbed). Common in-situ tests include standard penetration test (SPT), cone penetrometer test, field vane shear, pressuremeter test, plate-load test, dilatometer test and various geophysics tests. NHI 132031 "Subsurface Investigation" provides detailed discussions (including applicability and planning) on the commonly used boring, sampling and in situ testing techniques. A summary of available in-situ testing devices is provided in Table 3-5. Subsurface investigative recommendations specifically related to landslides are covered in Chapter 7, although the general procedures addressed in this chapter are routinely used.

3.4 GEOLOGIC CONDITIONS

The geology of a region has a paramount effect on the performance of slopes and embankments. Thus, the performance of slopes and embankments is an interdisciplinary endeavor requiring concepts and knowledge from engineering geology and soil/rock mechanics. Awareness of geology is necessary for appropriate idealization of ground conditions and the subsequent development of realistic geotechnical models.

TABLE 3-5
IN-SITU TESTING METHODS USED IN SOIL

Method	Procedure	Applicable Soil Types	Applicable Soil Properties	Limitations / Remarks
Electric Cone Penetrometer (CPT)	A cylindrical probe is hydraulically pushed vertically through the soil measuring the resistance at the conical tip of the probe and along the steel shaft; measurements typically recorded at 2 to 5 cm intervals	Silts, sands, clays, and peat	Estimation of soil type and detailed stratigraphy Sand: ϕ', D_r, σ_{ho}' Clay: s_u, σ_p'	No soil sample is obtained; The probe may become damaged if testing in gravelly soils is attempted; Test results not particularly good for estimating deformation characteristics
Piezocone Penetrometer (CPTu)	Same as CPT; additionally, penetration porewater pressures are measured using a transducer and porous filter element	Silts, sands, clays, and peat	Same as CPT, with additionally: Sand: u_o / water table elevation Clay: σ_p', c_h, k_h OCR	If the filter element and ports are not completely saturated, the pore pressure response may be misleading; Compression and wear of a mid-face (u_1) element will effect readings; Test results not particularly good for estimating deformation characteristics
Seismic CPTu (SCPTu)	Same as CPTu; additionally, shear waves generated at the surface are recorded by a geophone at 1-m intervals throughout the profile for calculation of shear wave velocity	Silts, sands, clays, and peat	Same as CPTu, with additionally: V_s, G_{max}, E_{max}, ρ_{tot}, e_o	First arrival times should be used for calculation of shear wave velocity; If first crossover times are used, the error in shear wave velocity will increase with depth
Flat Plate Dilatometer (DMT)	A flat plate is hydraulically pushed or driven through the soil to a desired depth; at approximately 20 to 30 cm intervals, the pressure required to expand a thin membrane is recorded; Two to three measurements are typically recorded at each depth.	Silts, sands, clays, and peat	Estimation of soil type and stratigraphy Total unit weight Sand: ϕ', E, D_r, m_v Clays: σ_p', K_o, s_u, m_v, E, c_h, k_h	Membranes may become deformed if overinflated; Deformed membranes will not provide accurate readings; Leaks in tubing or connections will lead to high readings; Good test for estimating deformation characteristics at small strains
Continued on next page				

Method	Procedure	Applicable Soil Types	Applicable Soil Properties	Limitations / Remarks
Pre-bored Pressuremeter (PMT)	A borehole is drilled and the bottom is carefully prepared for insertion of the equipment; The pressure required to expand the cylindrical membrane to a certain volume or radial strain is recorded	Clays, silts, and peat; marginal response in some sands and gravels	E, G, m_v, s_u	Preparation of the borehole most important step to obtain good results; Good test for calculation of lateral deformation characteristics
Full Displacement Pressuremeter (PMT)	A cylindrical probe with a pressuremeter attached behind a conical tip is hydraulically pushed through the soil and paused at select intervals for testing; The pressure required to expand the cylindrical membrane to a certain volume or radial strain is recorded	Clays, silts, and peat in sands	E, G, m_v, s_u	Disturbance during advancement of the probe will lead to stiffer initial modulus and mask liftoff pressure (p_o); Good test for calculation of lateral deformation characteristics
Vane Shear Test (VST)	A 4 blade vane is slowly rotated while the torque required to rotate the vane is recorded for calculation of peak undrained shear strength; The vane is rapidly rotated for 10 turns, and the torque required to fail the soil is recorded for calculation of remolded undrained shear strength	Clays, Some silts and peats if undrained conditions can be assumed; not for use in granular soils	s_u, S_t, σ_p'	Disturbance may occur in soft sensitive clays, reducing measured shear strength; Partial drainage may occur in fissured clays and silty materials, leading to errors in calculated strength; Rod friction needs to be accounted for in calculation of strength; Vane diameter and torque wrench capacity need to be properly sized for adequate measurements in various clay deposits

Symbols used in Table 3-5.

ϕ':	effective stress friction angle	G_{max}:	small-strain shear modulus
D_r:	relative density	G:	shear modulus
σ_{ho}':	in-situ horizontal effective stress	E_{max}:	small-strain Young's modulus
s_u:	undrained shear strength	E:	Young's modulus
σ_p':	preconsolidation stress	ρ_{tot}:	total density
c_h:	horizontal coefficient of consolidation	e_o:	in-situ void ratio
k_h:	horizontal hydraulic conductivity	m_v:	volumetric compressibility coefficient
OCR:	overconsolidation ratio	K_o:	coefficient of at-rest earth pressure
V_s:	shear wave velocity	S_t:	sensitivity

3.4.1 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), (6) marine deposits, and other problem soils such as organic soils, sensitive clays, landfills, etc. Each of these soil types has unique engineering characteristics. Some data from a number of U.S. cities about the strength and sensitivity of varved clays are listed in Table 3-6.

TABLE 3-6
STRENGTH AND SENSITIVITY OF COHESIVE SOILS (Hunt, 1984)

Material	Type	Location	γ_d, g/cm^3	w, %	LL, %	PI, %	s_u, kg/cm^2	$\bar{c}$, kg/cm^2	$\bar{\phi}$	Remarks
CLAY SHALES (WEATHERED)										
Carlisle (Cret.)	CH	Nebraska	1.48	18				0.5	45	ϕ extremely variable
Bearpaw (Cret.)	CH	Montana	1.44	32	130	90		0.35	15	
Pierre (Cret.)		South Dakota	1.47	28				0.9	12	
Cucaracha (Cret.)	CH	Panama Canal		12	80	45			17	$\phi_r = 10°$
Pepper (Cret.)	CH	Waco, Texas		17	80	58		0.4	20	$\phi_r = 7°$
Bear Paw (Cret.)	CH	Saskatchewan		32	115	92		0.4	22	$\phi_r = 8°$
Modelo (Tert.)	CH	Los Angeles	1.44	29	66	31		1.6	27	Intact specimen
Modelo (Tert.)	CH	Los Angeles	1.44	29	66	31		0.32	26	Shear Zone
Martinez (Tert.)	CH	Los Angeles	1.66	22	62	38		0.25		Shear Zone
(Eocene)	CH	Menlo Park, Calif.	1.65	30	60	50				
								Free swell	100%; P=10kg/cm^2	
RESIDUAL SOILS										
Gneiss	CL	Brazil; buried	1.29	38	40	16		0	40	$e_0 = 1.23$
Gneiss	ML	Brazil; slopes	1.34	22	40	8		0.39	19	c, ϕ, unsoaked
Gneiss	ML	Brazil; slopes	1.34		40	8		0.28	21	
COLLUVIUM										
From shales	CL	West Virginia		28	48	25		0.28	28	$\phi_r = 16°$
From gneiss	CL	Brazil	1.10	26	40	16		0.2	31	$\phi_r = 12°$
ALLUVIUM										
Back swamp	OH	Louisiana	0.57	140	120	85	0.15			
Back swamp	OH	Louisiana	1.0	60	85	50	0.1			
Back swamp	MH	Georgia	0.96	54	61	22	0.3			$e_0 = 1.7$
Lacustrine	CL	Great Salt Lake	0.78	50	45	20	0.34			
Lacustrine	CL	Canada	1.11	62	33	15	0.25			
Lacustrine (volcanic)	CH	Mexico City	0.29	300	410	260	0.4			$e_0 = 7$, S_t=13
Estuarine	CH	Thames River	0.78	90	115	85	0.15			
Estuarine	CH	Lake Maricaibo		65	73	50	0.25			
Estuarine	CH	Bangkok		130	118	75	0.05			
Estuarine	MH	Maine		80	60	30	0.2			
MARINE SOILS (OTHER THAN ESTUARINE)										
Offshore	MH	Santa Barbara, CA	0.83	80	83	44	0.15			$e_0 = 2.28$
Offshore	CH	New Jersey		65	95	60	0.65			
Offshore	CH	San Diego	0.58	125	111	64	0.1			Depth = 2m
Offshore	CH	Gulf of Maine	0.58	163	124	78	0.05			
Coastal Plain	CH	Texas (Beaumont)	1.39	29	81	55	1.0	0.2	16	ϕ_r=14, e_0=0.8
Coastal Plain	CH	London	1.60	25	80	55	2.0			
LOESS										
Silty	ML	Nebraska-Kansas	1.23	9	30	8		0.6	32	Natural w%
Silty	ML	Nebraska-Kansas	1.23	(35)	30	8		0	23	Prewetted
Clayey	CL	Nebraska-Kansas	1.25	9	37	17		2.0	30	Natural w%
GLACIAL SOILS										
Till	CL	Chicago	2.12	23	37	21	3.5			
Lacustrine (varved)	CL	Chicago	1.69	22	30	15	1.0			$e_0 = 0.6$ (OC)
Lacustrine (varved)	CL	Chicago		24	30	13	0.1			$e_0 = 1.2$ (NC)
Lacustrine (varved)	CH	Chicago	1.18	50	54	30	0.1			
Lacustrine (varved)	CH	Ohio	0.96	46	58	31	0.6			$S_t = 4$
Lacustrine (varved)	CH	Detroit	1.20	46	55	30	0.8			$e_0 = 1.3$ (clay)
Lacustrine (varved)	CH	New York City		46	62	34	1.0			e_0 =1.25(clay)
Lacustrine (varved)	CL	Boston	1.35	38	50	26	0.8			$S_t = 3$
Lacustrine (varved)	CH	Seattle		30	55	22			30	$\phi_r = 13°$
Marine	CH	Canada-Leda clay	0.89	80	60	32	0.5			$S_t = 128$
Marine	CL	Norway	1.34	40	38	15	0.13			$S_t = 7$
Marine	CL	Norway	1.29	43	28	15	0.05			$S_t = 75$

kg/cm^2 = 2,048 lb/sf   kg/cm^3 = 62,428 lb/cf

Alluvial Deposits

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.

Fluvial deposit may range in size from boulders to colloidal clay, although at a particular location, the particle size range is usually very narrow. Fluvial deposits typically are stratified and extremely variable with frequent interbedding. Permeability in the horizontal direction often is greater than in the vertical. Unless subject to fill placement, removal of overburden, or desiccation, the deposits are normally consolidated. Clays tend to be soft and sands tend to be loose to medium dense. Because of their variability in composition and engineering properties, fluvial deposits have a high susceptibility to slides, and may experience large deformations when loaded.

Lake or lacustrine deposits are the fine-grained materials deposited on lake bottoms. They may contain appreciable amounts of organic matter and also fragments of shells and skeletons from aquatic animals. If exposed in valley walls or cuts, land areas that were former lake beds can be expected to present slope problems. Slides of considerable magnitude have occurred in lake clays for two distinct circumstances: (a) where lake clays are interbedded with or are overlain by granular deposits, and (b) where lake clays overlie bedrock at shallow depth and the base level of erosion of the area is lowered (TRB, 1978).

Glacial Deposits

Glacial deposits were transported by glaciers, whose action resembles a giant bulldozer. Glacial deposits may vary in grain size composition from boulders to clays, and can be commonly found in the Midwest, Pacific Northwest, northeastern and eastern United States, and Alaska. Glacial deposits vary greatly in terms of composition and engineering properties.

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, 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 overconsolidated, thus dense to very dense and is of high strength and low compressibility. However, 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. Dense hard glacial till can be of a concern for excavation.

Glaciofluvial deposits are materials moved by glaciers and subsequently sorted and deposited by streams flowing from the melted ice. 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 compact. ***Glacio-lacustrine*** deposits consist of lacustrine soils derived from outwash that are deposited in glacial lakes formed during previous periods of glaciation. The typical enfolding of glacial lakes consists of varved clays (or alternating thin layers of clay and silt) with occasional sand seams or paintings. The varves can range from several millimeters to several centimeters in thickness, as are found in Connecticut and New Jersey. Varved clays are either normally or overly consolidated, depending on the process of deposition. Soft and normally consolidated varved clay deposits often bring the most difficult geotechnical engineering changes to slope and embankment projects where both slope stability and settlement concerns can be critical.

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. Typical engineering properties for silt and clay loess soils are shown in Table 3-6.

Dune sand deposits are 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 consist 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. Seepage below the groundwater table can cause slope failures in loess (for example, bank erosion) when the binding material is removed as shown in Figure 3-4. 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.

Eolian deposits should be avoided for foundation support if practicable. Information on the in-place density of eolian soils is of vital importance in planning their usefulness for foundations (Bureau of Reclamation, 1960).

Residual Soils

Residual soils are formed in place by mechanical and chemical weathering of their parental bedrock. Figure 3-5 depicts typical weathering profiles for metamorphic and igneous rocks (Deer and Patton, 1971). Some residual soil engineering properties are shown in Tables 3-6 and 3-7. 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.

Saprolites and ***laterites*** are two types of residual soils. Saprolites are zones consisting of completely weathered or highly weathered bedrock that contain soil-like materials but retain the original relic rock structure. 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 type of parent bedrock has a pronounced influence on the character of the residual soils. Always determine the rock type when assembling data for appraisal of a residual soil deposit.

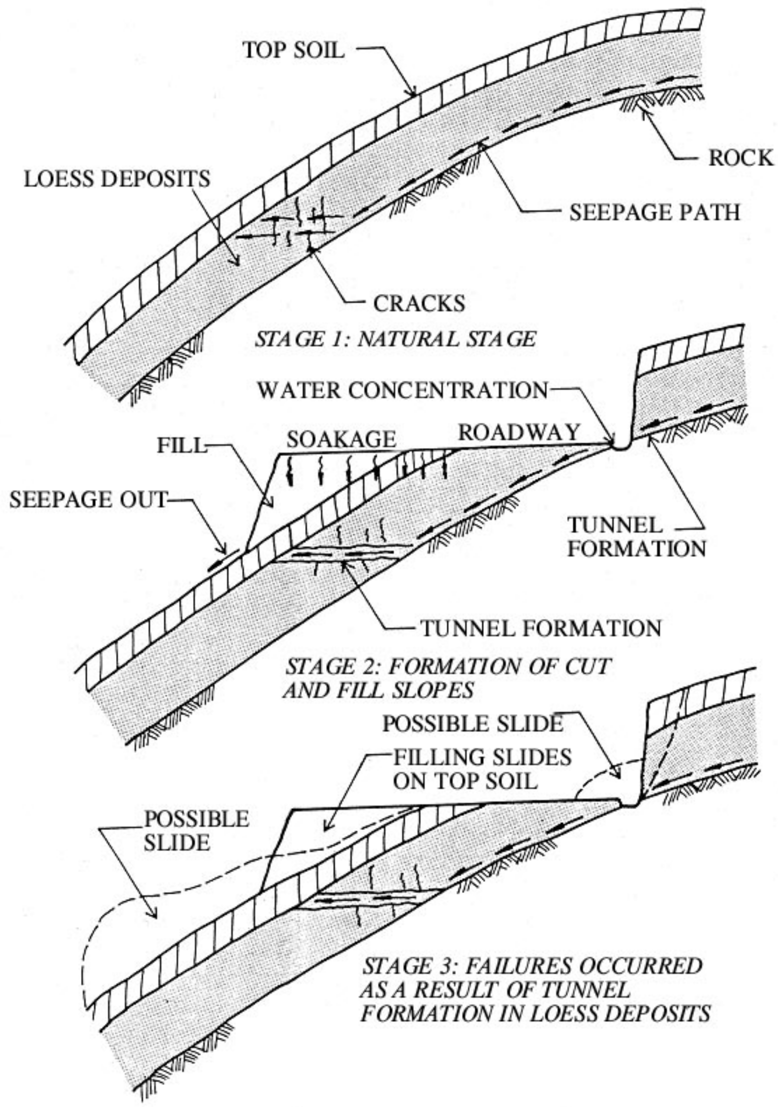


Figure 3-4**:** Schematic Diagrams Showing Landslides as a Result of Tunnel Formation in Loess Deposits (Evans, 1977)

Landslides and slope instability are common in deep residual soils, particularly during periods of intense rainfall. The rainfall-induced landslides are usually shallow. 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. Table 3-7 lists the strength parameters for various residual soils established by past researches.

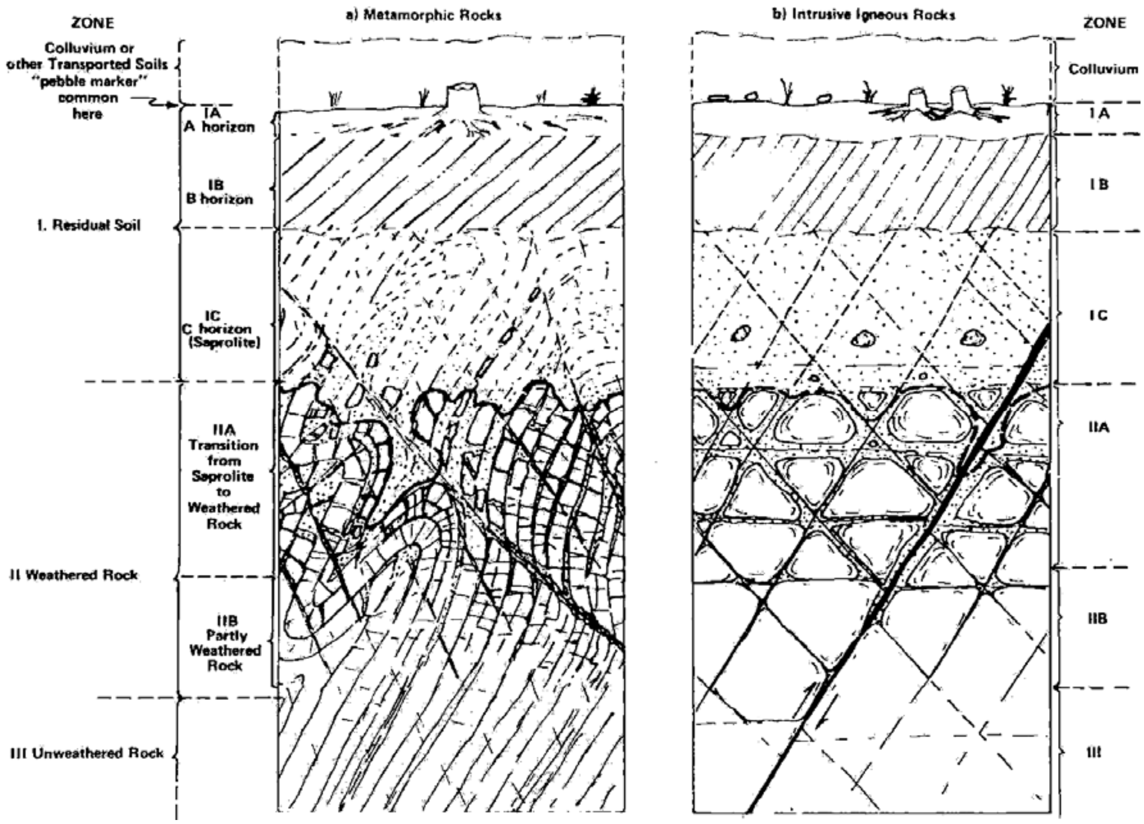
Note: These values should be used as preliminary guidelines only. Project specifications based on considerations of weathering profile, groundwater and inherited structure should still be specified.

<u>Organic Soils</u>

Organic soils (i.e., organic clays and organic silts) present similar engineering challenges as soft silts and clays, including low undrained shear strengths and high compressibility. In addition, organic silts and clays undergo significant secondary (or creep) deformations. Such long-term, continuous deformation can present significant maintenance issues for embankments and other structures that may be founded over such materials. Like other organic soils, peats also undergo significant secondary deformations. Readers are referred to GEC 5 "Evaluation of Soil and Rock Properties" (Sabatini, et. al, 2002) for methods for the identification and classification of organic soils and peats, and information on evaluating shear strength and compression properties for these materials.

Sensitive Clays

Sensitive clays are generally confined to specific geographic regions of the world where there was extensive glaciation followed by isostatic uplift. The strain softening response in undrained shear loading for both loose hydraulically placed sands and sensitive clays are attributed to their unique metastable structure. The metastable structure of sensitive clays is established when relatively low plasticity clays are deposited in brackish (i.e., salty) waters in a flocculated particle orientation. The resulting high void ratio soil structure, due to the edge-to-face alignment of the clay plates, is stable although the soils in the deposit have high natural moisture content and a moderately high liquidity index. In this state, the clay is weak, but not likely sensitive. The sensitive characteristics are introduced when fresh water is leached through the uplifted deposit, replacing the brackish water with the fresh water. As a result of the replacement with fresh water, the electro-chemical interaction between clay plates is altered to the point where the Atterberg limits reduce and the liquidity index increases. In this state, the disturbed clay behaves as a dispersed soil structure, thus a significant reduction in the post-peak shear strength due to the large amount of unbound pore water associated with the dispersed soil structure. Readers are referred to GEC 5 "Evaluation of Soil and Rock Properties" (Sabatini, et. al, 2002) for methods for the identification and classification of organic soils and peats, and information on evaluating shear strength and compression properties for these materials.



ZONE	DESCRIPTION	ROCK QUALITY DESIGNATION (NX CORE %)	PERCENT CORE RECOVERY (NX CORE)	RELATIVE PERMEABILITY	RELATIVE STRENGTH
I. Residual					
IA. A Horizon	Topsoil; roots; organic material; zone of leaching; and eluviations; may be porous	Not applicable	0	Medium to High	Low to Medium
IB. B Horizon	Characteristically clay enriched; also accumulation of Fe, Al and Si; hence may be cemented; no relict structures present	Not applicable	0	Low	Common Low (high if cemented)
IC. C Horizon (Saprolite)	Relict rock structure retained; silty grading to sandy material; less than 10% core stones often micaceous	0 or Not applicable	Generally 0 – 10	Medium	Low to Medium (relict structures very significant)
II. Weather Rock					
IIA. Transition	Highly variable, soil like to rocklike; fines commonly fine to coarse sand (grass); 10 to 90% core stones; spheroidal weathering common	Variable generally 0 – 50	Variable generally 10 – 90	High	Medium to low where weak and relict structures present
IIB. Partly weathered rock (PWR)	Rocklike soft to hard rock; joints stained to altered; some alteration of feldspars and micas	Generally 50 – 75	Generally 90	Medium to High	Medium to High
III. Unweathered rock	No iron stains to trace along joints; no weathering of feldspars or micas	Over 75 (generally 90)	Generally 100	Low to Medium	Very High

Figure 3-5: Schematic Diagram of Typical Residual Soil Profile (Deere and Patton, 1971)

TABLE 3-7
TYPICAL SHEAR STRENGTH PARAMETERS OF RESIDUAL SOILS AND PARTIALLY WEATHERED ROCKS

Type of Residual Soils	Degree of Weathering	Strength Parameters				Type of Shear Test
		Total Stress		Effective Stress	Residual	
		C_u (kg/cm^2)	ϕ_u	ϕ	ϕ_r	
METAMORPHIC ROCKS						
Gneiss	Moderately decomposed	8	35°			Direct shear tests with rock-concrete contacts
	Very decomposed	4	29°			
	Very decomposed (Failure zone)	1.5	27°			
	Decomposed	-	18.5°			Consolidated-undrained triaxial tests
Schists	Partially weathered	0.7	35°			Sheared normal to schistosity
	Weathered	-	24.5°			
	Moderately weathered	-	15			Consolidated undrained triaxial test with degree of saturation at 50 and 100%
		-	21			
	Weathered	-	26-30°			Direct shear test on compacted rockfill
Phyllites	Residual soil	0	24°			Shear normal to schistosity
		0	18°			Shear parallel to schistosity
IGNEOUS ROCKS						
Granite	Partially weathered	-			27-31°	Direct shear tests in the laboratory
	Weathered	-			26-33°	
	Very decomposed	0	25-34°	35°		
	Residual soil	-		28°		
Diorite	Decomposed	0.1	30°			Consolidated-undrained triaxial tests
	Partially weathered	0.3	22°			
Rhyolite	Decomposed	-		30°		
SEDIMENTARY ROCKS						
Marl	Moderately weathered	-		32-42°	22-29°	Drained and consolidated-undrained triaxial tests
	Highly weathered	-		25-32°	18-24°	
London Clay	Weathered	-		19-22°	14°	
	Unweathered	-		23-30°	15°	
Black Clay	Fissured	-			10.5°	Consolidated-undrained triaxial tests
	Unfissured	-			14.5°	
SOIL MINERALS						
Quartz sand		-			30-35	
Kaolinite		-			12°	
Illite		-			6.5°	
Montmorillonite		-			4-11°	
Muscovite		-			17-24°	
Hydrated Mica		-			16-26°	

Note: This table should be used only as a guideline to check the reasonableness of laboratory and field shear strength tests or for preliminary designs.

Colluvial/Talus Deposits

Colluvial/talus deposits are soil deposits that have been moved down slope by gravity on steep slopes (Figure 3-6). Colluvial/talus deposits often are loose and unconsolidated. Typically, colluvium is a poorly sorted mixture of angular rock fragments and fine-grained materials overlying the residual soil/rock slope, and is generally thinnest near the crest and thickest near the toe of the slopes (Turner, 1996). Talus deposits are mostly accumulated at the base of steep hills containing large numbers of rock fragments ranging in size from small to very large. These materials can be identified in air photos as bare slopes in mountainous areas, but they are not obvious on vegetated lower slopes. Figure 3-6 shows a typical example of colluvium developed by residual soils (formed by weathering of parent soil and rock materials) that have traveled down slope.

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 movements of creep to the more discernible movements of several centimeters (few inches) per week (Hunt, 1984). 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. Table 3-6 shows the engineering properties of some of the colluvial deposits found in West Virginia and Brazil. Due to the presence of rock fragments, field investigations may sometime misleading and result in overestimating the shear strengths of the colluvial/talus deposits. Furthermore, the shear strengths may also vary horizontally and vertically within the deposit randomly due to the heterogeneous natures. Readers are referred to GEC 5 "Evaluation of Soil and Rock Properties" (Sabatini, et. al, 2002) for information on issues related to subsurface exploration, and evaluating shear strength and compression properties for these materials.

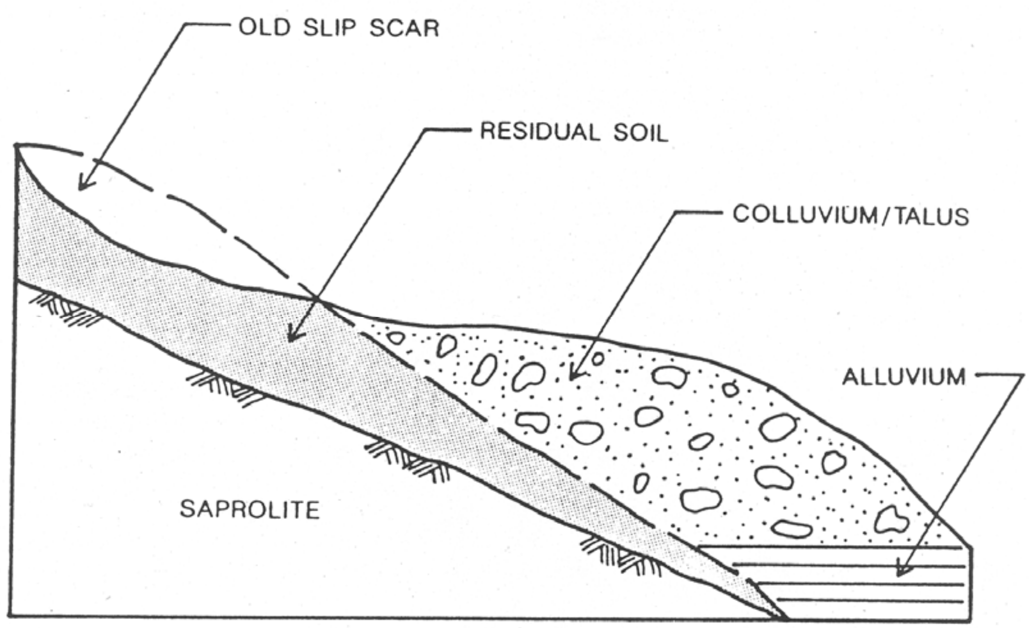


Figure 3-6: Colluvial/Talus Deposits Moving Downslope (FHWA, 1994)

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.

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, but at depths below the sea floor of about 90 meters or more, they are often stiff to very stiff in consistency. Table 3-6 shows the engineering properties of some marine deposits in the United States and elsewhere. Marine clays tend to flocculate, settle relatively quickly to the bottom, and are devoid of laminations and stratifications. Highway embankments often are constructed over marine deposits. Care must be exercised to avoid formation of mud waves during construction. Both short- and long-term stability analyses should be performed to evaluate the stability of embankments constructed on marine clays as well as settlement analysis to determine the time rate and magnitude of consolidation.

Volcanic Clays

***Volcanic clays* are deposited when volcanic ash and dust are released into the atmosphere during volcanic eruptions. The weathering processes often alter the ash and dust into montmorillonite clay, which is found in most states west of the Mississippi, as well as Tennessee, Kentucky, and Alabama.**

Ash and dust produced by the volcanic activities often modify the landscape. Potentially unstable hill slopes are marked by terracettes, by mudflows, and by landslides. Any construction activity on these slopes will tend to lead to renewed movements. Areas with so-called reactive soils are areas occupied by soils which are expansive and unsaturated in arid and semiarid areas. The soils tend to expand when wet and shrink when dry, thus causing process-induced stresses within the soil mass. These stresses may produce damage to or even failure of the slopes.

Duricrusts

Duricrusts are highly indurated zones within a soil formation, often of rocklike consistency, and include either laterites, ironstone, or ferrocrete (iron-rich); bauxite (alumina-rich); and calcite or calico (lime-rich). **It is usually variable in form, hard and rocklike in many areas and soft in other areas. With respect to slope stability, it is a highly questionable material.**

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.

3.5 GROUNDWATER CONDITIONS

Water is a major factor in most slope and embankment stability and settlement problems. Therefore, knowledge of groundwater conditions is essential for the analyses and design of slopes and embankments. This section describes the groundwater impact on the behavior of slope and embankment. Also, it discusses other water related topics such as rainfall, runoff, and rapid drawdown.

3.5.1 Groundwater Basics

Groundwater is derived from many sources but primarily originates from rainfall and melting snow. Some water infiltrates into the ground and percolates downwards to the saturated zone at depth, while some water moves as surface runoff into a stream or river. Groundwater in the saturated zone moves toward rivers, lakes, and seas where it evaporates and returns to the land as clouds of water vapor, which precipitates as rain and snow, and completes the hydrologic cycle.

As shown in Figure 3-7, the water zone in the soil mass may be divided into a saturated zone below the groundwater table (*phreatic surface*) and a unsaturated (capillary) zone above it. In the unsaturated zone, the voids fill partially with water that decreases (as air increases) as the distance from the water table increases. The water in the unsaturated zones is held in place by capillary attraction (due to water surface tension) and exerts relatively large stabilizing forces on the structure of soil (which is called negative pore pressure or soil suction). The capillary zones may reach a considerable height above the water table in fine-grained soils. Conversely, in soils like gravel, the capillary height is negligible.

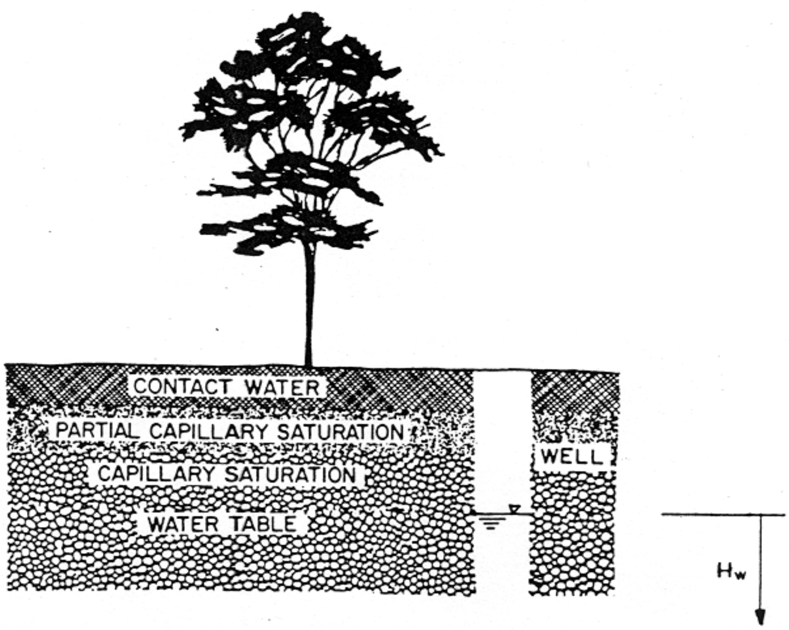


Figure 3-7: Capillary Water System (Dunn, et al., 1980)

The mass groundwater zones that transmit water with ease through their pores and fractures, respectively, are called aquifers. Typical aquifers are composed of gravel, sand, sandstone, limestone, and fractured igneous and metamorphic rocks. Basically, aquifers can be unconfined or confined by aquicludes which consist of strata or discontinuities that are sufficiently less pervious than the adjoining strata and are barriers to groundwater. Typical aquicludes are clay, shale, and unfractured igneous and metamorphic rocks. Figures 3-8 shows basic concepts of different types of aquifers.

Within a confined aquifer if the groundwater pressure (piezometric head) is higher than the upper surface of the aquifer, an artesian condition is derived. In other words, if piezometers were installed in the artesian aquifer, the water in the piezometer tubes would rise to greater elevations from the deeper artesian strata than from strata nearer the ground surface. Artesian condition is critical for fill and embankment applications due to the high water pressure can reduce the frictional resistance significantly and can cause slope instability.

Figure 3-8 also shows a perched water table which is sustained above either an aquiclude or an impermeable stratum, such as a clay layer. Perched water may be transient, rapidly developing in response to heavy rainfall and dissipate quickly, or it may be permanent in response to seasonal variations in rainfall levels. Like the groundwater table, perched water can be monitored by means of piezometers. They are often first recognized during exploratory borings when water is encountered above the permanent groundwater table. Shallow failures of slopes are sometimes associated with the rise of perched water over an impermeable layer. Perched groundwater can create saturated zones within unsaturated zones. Different modes of groundwater flow develop in the unsaturated and saturated zones and affect the stability of slopes as discussed in later sections.

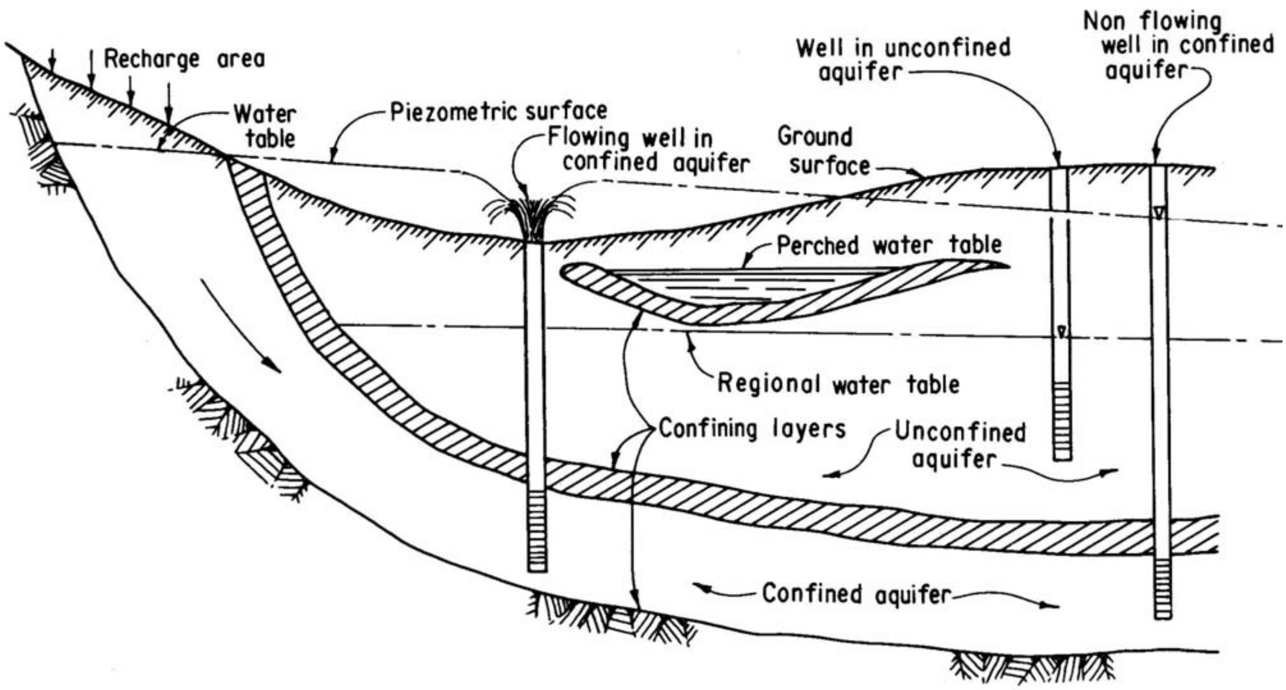


Figure 3-8: Types of Aquifer (US Department of the Interior, 1981)

3.5.2 Runoff and Infiltration

Runoff is that proportion of rainfall or snowmelt that flows from a catchment area into streams, lakes, or seas. It consists of surface runoff and groundwater runoff, where groundwater runoff is derived from rainfall or snowmelt that infiltrates into soil down to the water table.

Surface runoff from a catchment area depends on the following factors:

- Rainfall intensity
- Area and shape of the catchment area
- Steepness and length of the slopes being drained
- Nature and extent of vegetation or cultivation
- Condition of the surface and nature of the subsurface soils.

Readers are referred to other National Highway Institute Hydraulic training courses for methods of estimating rainfall and runoff for a catchment area.

Runoff-induced earth movements are governed by the relief surface. One example is that earthflows generally head toward large basins in the upper part of the slope where slope debris and weathering material accumulate. Heavy rainfall may trigger the movement of this loose soil mass that, as a narrow flow follows an erosion gully or the channel of a brook, forms a loaf-shaped bulge at the foot of the slope. Another example is that concave slopes or valley-like landforms are more vulnerable to gully erosions where heavy rainfalls concentrate flows of water. Shallow sloughing is common during intense rainstorms, especially in slopes composed of residual soils. For shallow sloughing, positive flow regimes with seepage are more or less parallel to the slope face behind an advancing wetting front. Chapter 5 explains surficial stability in more details. Sloughing in residual soils may be attributed to the day lighting of relict joints or structures that loose strength when saturated by rainwater infiltration. Chapter 7 discusses landslide issues in residual soils.

Infiltration is the portion of rainfall that infiltrates through earth to reach the groundwater table. Infiltration through an unsaturated zone is vertical and causes no positive pore pressures. If the infiltrating rainfall or snow melt, encounters a material of lower permeability, during its descent, flow will be impeded if the permeability of this lower zone is less than the rate of infiltration. Under this situation, a perched water table will form on the surface of the impermeable zone and a lateral flow will take place along the upper surface of the impermeable zone (Figure 3-9). Below the impermeable zone, the infiltration rate will be reduced to the value of the permeability of the zone.

When the infiltrating rainfall meets the groundwater table (phreatic surface), most of the vertical flow component will be destroyed and the lateral flow takes place in the general direction of groundwater flow. Under these circumstances, the groundwater table rises by an amount equal to the depth of saturation caused by the descending zone of infiltration and will be less than the depth of this zone. Within this zone, lateral flow takes place and positive pore pressures exist.

These two modes of groundwater flow affect slope stability differently. Above the phreatic surface, the infiltrating rainfall raises the degree of saturation of the soil, which reduces the negative pore pressure and thus the shear strength. As lateral flow develops, pore pressures increase, and as a result, effective stresses and shear strength are reduced. The increase of positive pore pressure occurs when the infiltrating rainfall forms a perched water table or has caused a rise in the groundwater table.

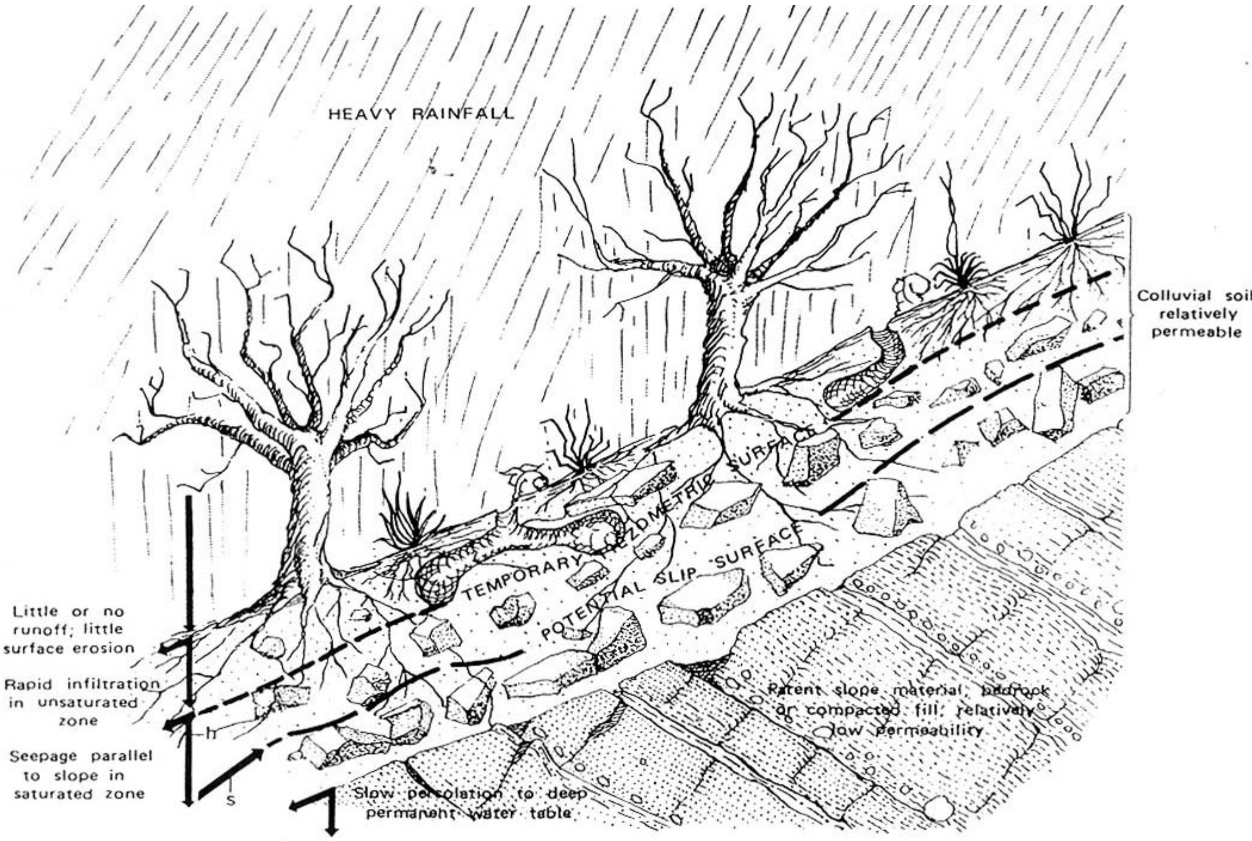


Figure 3-9: Infiltration and Build up of Perched Water Table (Campbell, 1975)

Deposits of gravel and sand permit water infiltrate without difficulty whereas clay-rich mantles retard the ingress of water and characteristically remain wet after periods of rainfall. Vegetation protects the delicate porous structure of many superficial deposits, especially the crumb-structure of topsoil. Ground covered by vegetation has more uniform infiltration than bare ground. Infiltration ceases once the voids within the ground are filled with water. If the rate at which water is supplied to the surface exceeds the rate at which it can percolate, the infiltration capacity will diminish and the excess water will be transported by runoff.

3.5.3 Hydrostatic and Pore Water Pressure

At the groundwater level (phreatic surface), the pore water is at atmospheric pressure, while below the phreatic surface, the pore pressure will be above atmospheric (note atmospheric pressure is usually taken as the zero pressure datum and so a positive pore pressure zone exists below the water table). If there is no flow, the pore pressures are hydrostatic and the water level measured in a piezometer within the saturation zone will coincide with the water table. Pore pressures will no longer coincide with water table hydrostatic if there is any flow (Figure 3-10). The pressure in a static body of water (Figure 3-10a) has a triangular distribution with a magnitude which is called hydrostatic pressure. This hydrostatic pressure distribution also exists in the pore water surrounding the soil particles in Figure 3-10b.

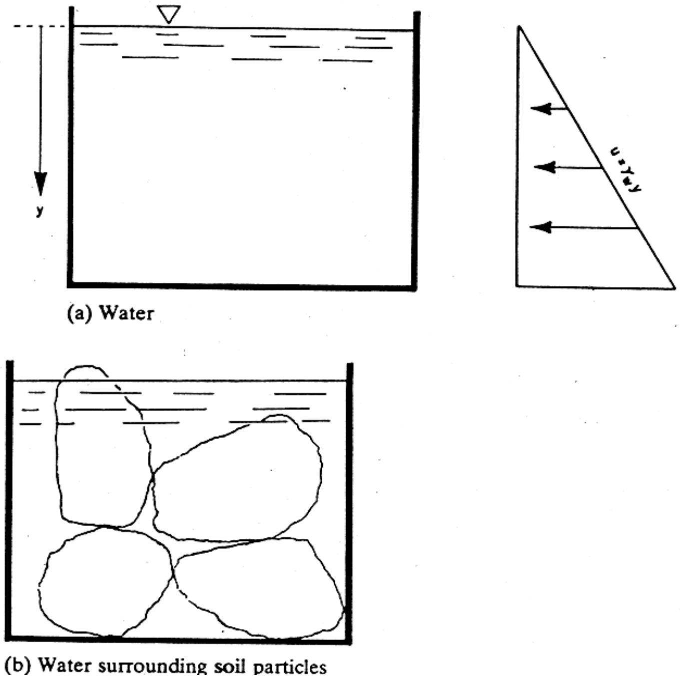


Figure 3-10: Hydrostatic Pressure (Dunn, et al., 1980)

A soil mass, when fully saturated, is composed of two distinct phases: the soil skeleton and the water-filled pores between the soil particles. Table 3-8 lists the void size that may be expected in these materials and their likely permeability.

TABLE 3-8
TYPICAL VOID SIZES FOR SOIL AND ASSOCIATED PERMEABILITIES

Material	Void Size (cm^3)	in^3	Permeability (cm/sec)
Clay	$<10^{-4}$ to 10^{-3}	$<6.1^{-6}$ to 6.1^{-5}	$<10^{-6}$
Silt	10^{-3} to 10^{-2}	6.1^{-5} to 6.1^{-4}	10^{-6} to 10^{-4}
Sand	10^{-2} to 10^{-1}	6.1^{-4} to 6.1^{-3}	10^{-4} to 10
Gravel	$> 10^{-1}$	$> 6.1^{-3}$	10 to 10^{2}

1 cm/sec = .4 in/sec

As shown in Figure 3-11, any stresses (the total stress, σ) imposed on such a soil mass will be sustained by the soil skeleton (the effective stress, σ') and the pore water pressure (u). The total stress is equal to the total force per unit area acting perpendicular to the plane. Further discussion of the effective and total stress principles is presented in Chapter 4. The pore pressure can be determined from the groundwater conditions, which will be discussed in subsequent sections.

Seepage flows in soil slopes are often encountered, and slope stability analyses must include these flowing conditions. To illustrate the application of the effective stress principle in these cases, it is essential to understand the basic principle of water forces that develops in the soil mass. Figure 3-12 shows a setup in which water is flowing vertically upward through a soil mass under a total hydraulic gradient, *i*, which is equal to h/Z. The seepage force per unit volume is equal to the total hydraulic gradient *i* times the unit weight of the water, that is, $i\ \gamma_w$. In an isotropic soil, the seepage force always acts in the direction of flow.

Since the seepage force acts in the direction of flow, it tends to move the soil grains in that direction (piping). Piping will occur at a hydraulic gradient greater than the critical hydraulic gradient, i_c, a condition where the effective stress becomes zero:

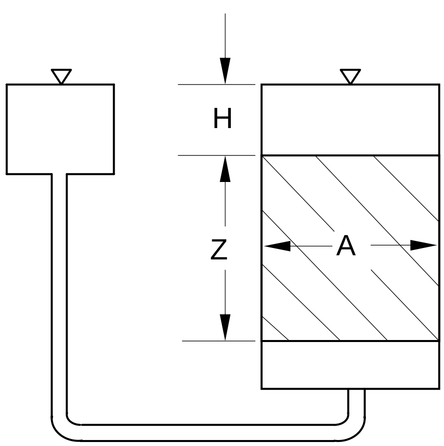
$$\sigma' = \gamma_b\ Z - i_c\ \gamma_w\ Z = 0 \tag{3-1}$$

$$i_c\ \gamma_w\ Z = \gamma_b\ Z \tag{3-2}$$

$$i_c = \frac{\gamma_b}{\gamma_w} \tag{3-3}$$

where:

σ'	=	effective stress
γ_b	=	submerged unit weight of soil
γ_w	=	unit weight of water
Z	=	soil depth
i_c	=	critical hydraulic gradient for water flowing opposite to gravity



$\sigma = \sigma' + u$

where:

$u = (H + Z)\ \gamma_w$

$\sigma = H\ \gamma_w + Z\ \gamma$

$\sigma' = \sigma - u$

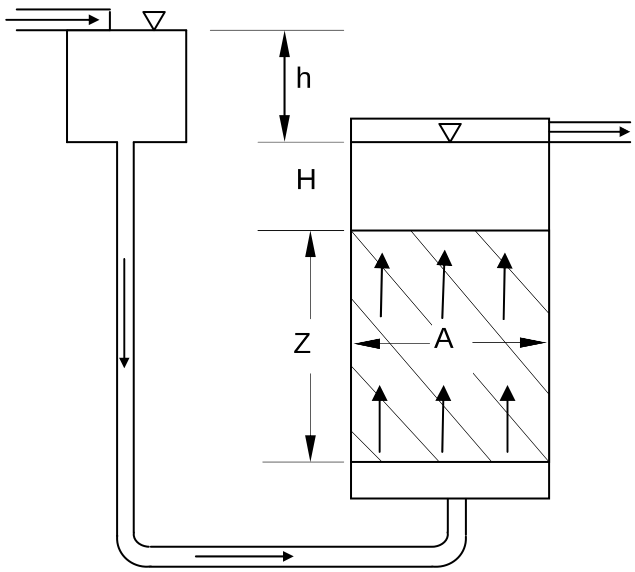
$= H\ \gamma_w + Z\ \gamma - (H + Z)\ \gamma_w$

$\sigma = (\gamma - \gamma_w)\ Z = \gamma_b\ Z$

γ_b = submerged unit weight of soil

Figure 3-11: Effective Stress Principle

Seepage force results from a change in the neutral stress due to groundwater flow:



Effective stress without flow:
$\sigma' = \sigma - u = \gamma_b Z$ (Effective Stress)

Effective stress with upward flow:
$\sigma' = \gamma_b Z - \Delta u = \gamma_b Z - \gamma_w h$

Hydraulic gradient $I = h/Z$
Where h = differential head

Therefore, $\sigma' = \gamma_b Z - I \gamma_w Z$

Seepage pressure: $I \gamma_w Z$
Seepage force: $I \gamma_w Z A$
Where A = Area vertical to waterflow

Figure 3-12**:** Seepage Force

In other words, the critical hydraulic gradient is equal to the ratio of the submerged unit weight of soil and the unit weight of water. For practical purposes, i_c is close to 1.0.

Total Stress = Effective Stress + Neutral Stress
Total Pressure = Soil Skeleton Pressure + Water Pressure

3.5.4 Effects of Pore Pressures

At the phreatic surface, the pore water is at atmospheric pressure, while below the phreatic surface, the pore pressure will be above atmospheric (note atmospheric pressure is usually taken as the zero pressure datum and so a positive pore pressure zone exists below the water table). The phreatic surface may be delineated in the field by using open standpipes or piezometers (Figures 3-13 and 3-14).If there is no flow, the pore pressure is hydrostatic and the water level measured in a piezometer within the saturation zone will coincide with the water table. Pore pressures will no longer coincide with water table hydrostatic pressure if there is any flow (Figure 3-15). In this instance, the pore pressure from any point within the soil mass is computed from the difference in head between that point and the free water surface. Calculations to determine the pore pressure in slopes is discussed above.

By lowering effective stress, positive pore pressure reduces the available shear strength within the soil mass, thereby decreasing the stability. Rapid increases in positive pore pressure can occur after a period of heavy rainfall. That is the main reason why the frequency of slope failure increases after heavy rainfall. Many disastrous slope failures occurred during and/or after a major storm. 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.

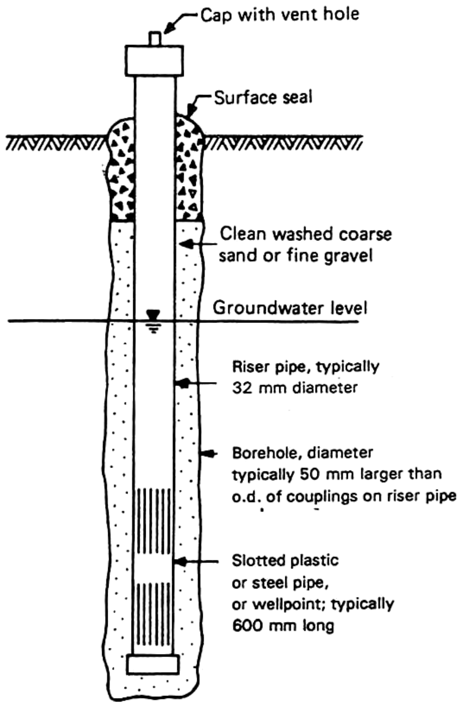


Figure 3-13: Schematic of Observation Well (Dunicliff, 1999)

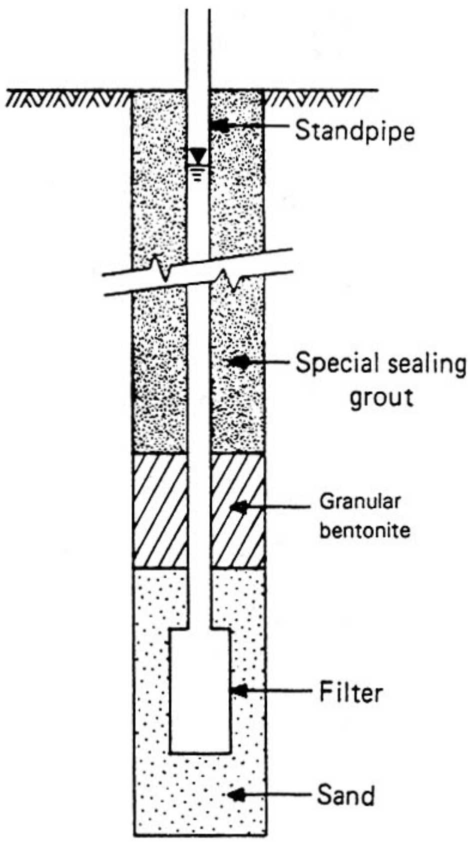


Figure 3-14: Schematic of Open Standpipe Piezometer Installed in a Borehole (Dunicliff, 1999)

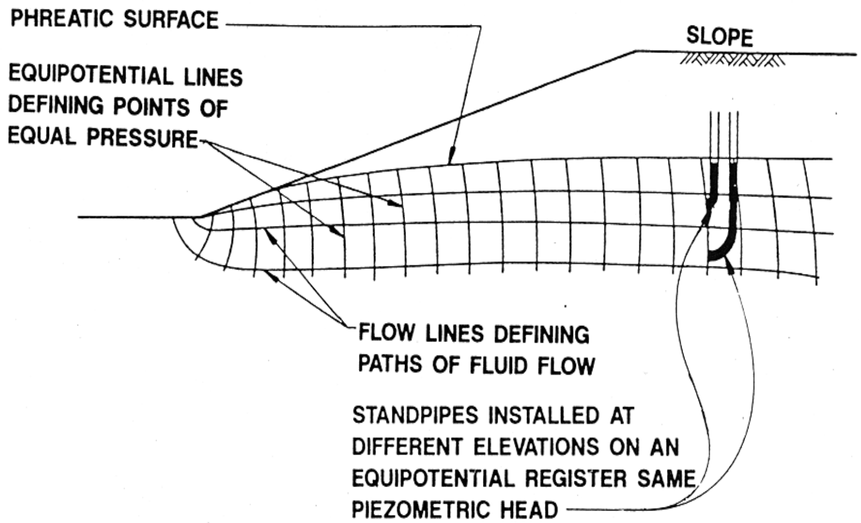


Figure 3-15: Pore Pressures in an Equipotential Line (FHWA, 1994)

In the unsaturated zone located above the phreatic surface, the magnitude of the negative pore pressure (sometimes called soil suction) is controlled by surface tension at the air-water boundaries within the pores and is governed by grain size. In general, the finer the soil particles, the larger the saturation capillary head and the higher the negative pore pressure. Negative pore pressures increase the effective stresses within a soil mass and improve the stability of a slope. Measurement of negative pore pressure is not as common as the measurement of positive pore pressure.

Positive pore pressures can be represented by several methods for analysis as discussed in Chapter 5.

3.5.5 Groundwater Levels for Design

Design of slopes commonly is based on the groundwater conditions that would result from a return period of either 10, 20, or 50 years, which is determined by local authorities. The water level of a specified-year return period can be determined in two ways (GCO, 1984). The first involves the analysis of piezometric data taken before, during and after rainfall. Various methods are available for determination of water levels from piezometric records, including the statistical correlation of groundwater response with rainfall, groundwater modeling of the aquifer system, and the extrapolation of observed piezometric responses. The second involves the solution of an equation that describes the formation of a wetting band zone of 100 percent saturation (see following paragraphs) which is dependent on the porosity, permeability, initial and final degrees of saturation of the soil forming the slope, and the percentage of the anticipated return period rainfall that will infiltrate into the ground directly above or behind the slope.

Different aquifer systems have different responses to rainfall, depending on their storage characteristics. Some aquifers may display rapid response to intense rainfall, while others may react with a gradual rise in water level during the wet season. Because of this variability in aquifer response, it is recommended that the groundwater conditions to be used in stability analysis be based on water levels measured in the field by piezometers or open standpipes. For sites where there is no piezometer data available, the wetting band approach may be used to give a rough estimate of groundwater levels.

Wetting Band Approach

Wetting band approach (Lamb, 1962 and 1975) assumes that the wetting band descends vertically under the influence of gravity, even after the cessation of rain, until either the main water table or a zone of lower permeability (for example, a clay stratum) is reached. Under the latter condition, a perched water table will form above the zone of lower permeability, and pore pressure will become positive.

When the descending wetting band reaches the main water table, the surface of the main water table will rise with an increase in pore pressure. The rise in the main water or the thickness of the perched water table will equal approximately the thickness of the wetting band. The wetting band thickness is inversely related to the difference between the initial and final degree of saturation of the soil mass. Therefore, thicker wetting bands are more likely to occur after a series of heavy rainfall events than after dry spells.

The relationship between rainfall on unprotected slopes, infiltration, and the depth of the wetting front can be approximated by the following equation (Lamb, 1975):

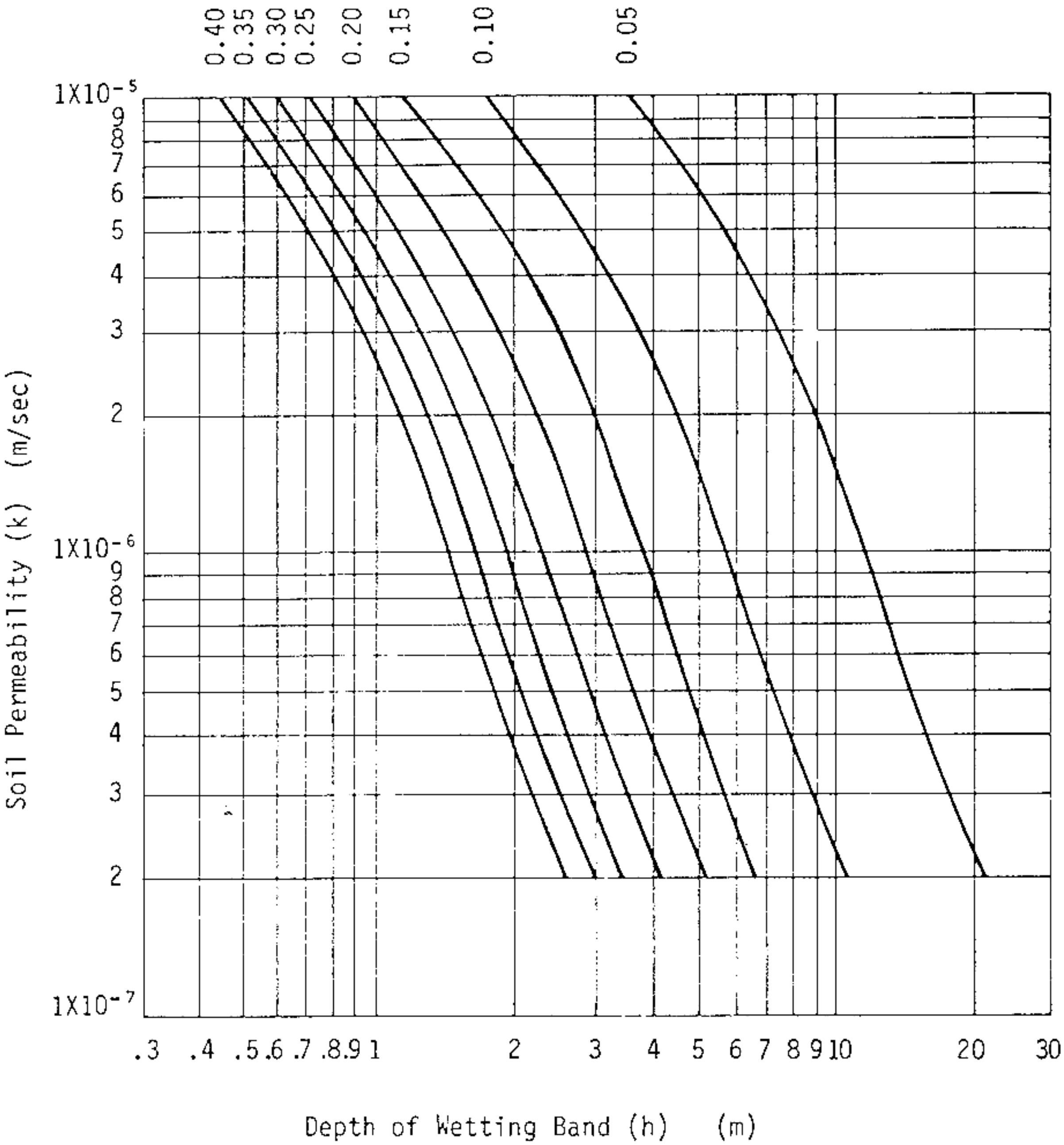
$$h = \frac{kt}{n\,(S_f - S_o)} \qquad (3\text{-}7)$$

where
- h = depth of wetting front (thickness of the wetting band after time t) (cm or inch)
- k = coefficient of permeability (cm/sec or inch/sec)
- t = duration of rainfall (sec)
- n = porosity
- S_f = final degree of saturation
- S_o = initial degree of saturation

Figure 3-16 gives an example of variation in the value of (S_f - S_o) and the degree of saturation. The value *k* for the surface may be determined by conducting infiltration tests within a sample tube driven into the soil surface (Figure 3-17). During the test, water is fed into the exposed surface at a carefully controlled rate so that minimum ponding occurs on the surface. The total volume of water infiltrating is noted at various time intervals and is plotted against time. Figure 3-17 shows a field infiltrometer while Figure 3-18 presents typical results from the infiltration test.

The assumptions made in equation 3-7 result in an extremely simplified model for infiltration. Throttling on infiltration often arises when a very thin band of impermeable layer exists in the soil profile, or the surface permeability is lower than that of underlying material. Similar analytical difficulties are introduced as a result of the degree of saturation changing throughout the soil profile (GCO, 1979).

The wetting band approach is only applicable to situations where the rise in groundwater level is due to rainfall infiltration. Sloping ground, down slope flow, and the differences in aquifer responses are not accounted for in this simplified method. If the intensity of rainfall is at least sufficient to cause infiltration at the limiting rate, the thickness of the wetting band will depend upon the duration of the rainstorm, that is, the longer the rainstorm, the thicker is the wetting band.



Note:

1. This chart was prepared assuming runoff is 50% and porosity is 40%
2. Curves were plotted for the various degrees of saturation (S_f-S_o) shown.
3. 1 m/sec = 39.3 inch/sec

Figure 3-16: Effect of Permeability and Degree of Saturation on Wetting Band Thickness (GCO, 1984)

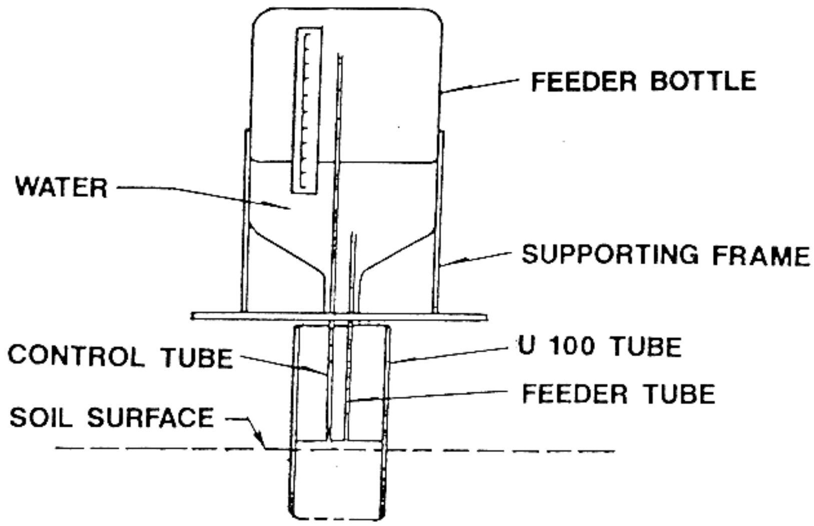


Figure 3-17: Diagram of Field Infiltrometer (Lam, 1974)

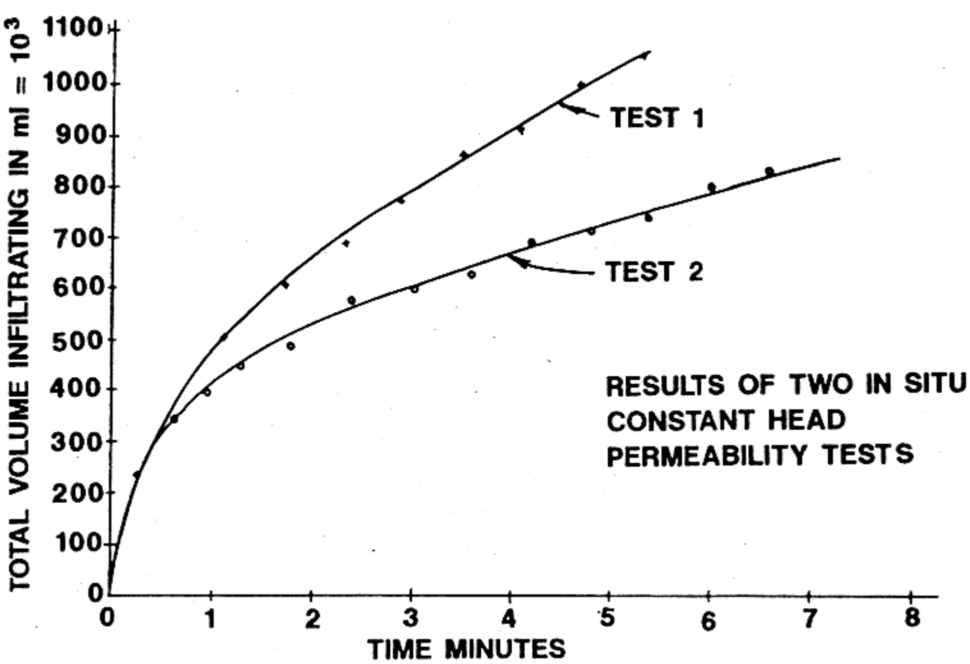


Figure 3-18: Typical Results from Field Infiltration (GCO, 1979)

3.5.6 Field Identification and Interpretation of Groundwater Conditions

It is essential to learn as much as possible about the groundwater conditions during the field investigation. Of primary importance is to determine (Prellwitz, 1990):

- If groundwater exists, is it hydrostatic or flowing?
- If it is flowing, is it in a confined or unconfined aquifer?
- What are the upper and lower limits and slope of the aquifer?
- What are the aquifer characteristics (soil type and permeability, rock discontinuities)?
- What is the proximity of the aquifer to the existing or potential failure surface for stability analysis?

Most important, what is the highest phreatic surface for an unconfined aquifer and/or piezometric surface for a confined aquifer to use in the stability analysis? If it is an existing failure, how high was the groundwater at the time of failure?

All of the aforementioned aspects are related to the engineering geology and the hydro-geomorphology of the slopes under study. In practice, subsurface conditions are very complex and so are the groundwater flows. All groundwater data should be interpreted by an experienced geotechnical engineer or an engineering geologist. To develop a groundwater model for slope stability analysis, the investigator has to form hypotheses early in the investigation and gather the data to verify the hypotheses or be ready to form a new one as additional data is obtained. The investigation should not be concluded until the investigator is confident that a workable understanding of the groundwater flows can be developed.

Other data pertinent to groundwater movement may be noted and documented during investigation. Evidence such as a rust-stained zone might indicate a zone having been subjected to repeated drying and wetting which results in oxidation in an aerobic environment. Another indicator is the presence of a fully saturated gray slick zone, which results from weathering in a reducing and anaerobic environment. This zone is frequently present at a confining layer and can help to determine the lower limits of an unconfined aquifer if a distinct change in soil type is not noted.

As noted earlier, groundwater can be monitored with open standpipes and/or piezometers. The drill holes should be cased with at least 2.5 cm (1 inch) PVC plastic pipe and slotted to allow groundwater to enter only in the anticipated aquifer zone.

Once the piezometers are in place, questions often arise as to how to determine whether the aquifer is confined or unconfined and how to find the limits and slope of the aquifer. As shown in Figure 3-19, if an unconfined aquifer is encountered during drilling, free water should appear in the casing at all depths below the phreatic surface as long as drilling continues within the aquifer. Free water in the casing should be referenced to the depth of highest confinement. If this is the bottom of the casing and the water is flowing, the elevation to which the free water rises should be less as the casing is advanced. This is because equipotential lines from further down slope are lower as depth increases. The water in the casing will rise to the vertical height of the equipotential line intercepted at the bottom. The bottom of the unconfined aquifer should be recognized as the confining surface of lower permeability if encountered.

If a confined aquifer is encountered, it should be below a confining layer of lower permeability and groundwater will probably enter the casing only after this confining layer is penetrated and rise to some height above the bottom of the casing. Confined aquifers in clayey soils also may exist without a change in soil type but may only be a zone of open fissures because of prior slope failure.

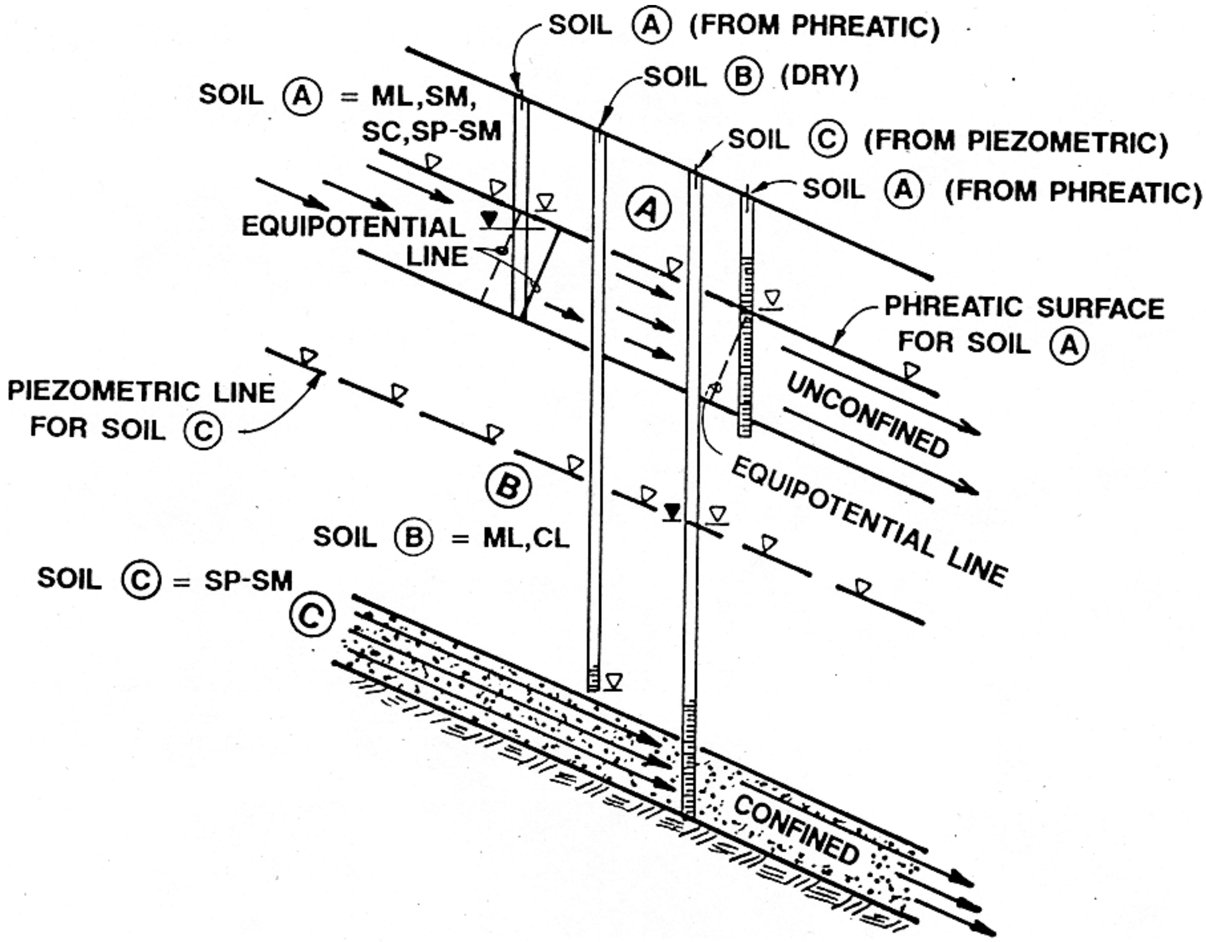


Figure 3-19**:** Groundwater Flows in Confined and Unconfined Aquifers (Prellwitz, 1990)

3.5.7 Sudden Drawdown

Sudden drawdown is a rapid lowering in the level of water standing against a slope. This particular situation is commonly encountered in the design of bridge approach embankments at river crossings. In such a case, rapid drawdown occurs when the river level falls following a flood. Other examples of sudden drawdown include the lowering of a reservoir adjacent to the upstream slope of an earth dam, lowering of the water level next to a natural slope, and a drop in the sea level next to a slope. Most of the highway slope failures caused by sudden drawdown occur at stream crossings.

Failures because of sudden drawdown often occur in embankment slopes composed of clayey materials in which the excess pore water pressures do not dissipate as the water recedes, thereby keeping low the overall shear strength of the clay materials. Undrained shear strength is lower than drained shear strength. Also, the water acts as a stabilizing pressure against the slope face when in place. If the water level against the slope face is suddenly drawn down, this stabilizing pressure is suddenly removed, while the driving forces within the slope are relieved much more slowly, which creates an unbalanced condition.

Figure 3-20 illustrates the three stages necessary for the development of a sudden drawdown condition. The pore pressure changes during sudden drawdown may be estimated by charts and empirical formulae proposed by Bishop (1954), Janbu (1973), and Duncan (1987), see Chapter 5.

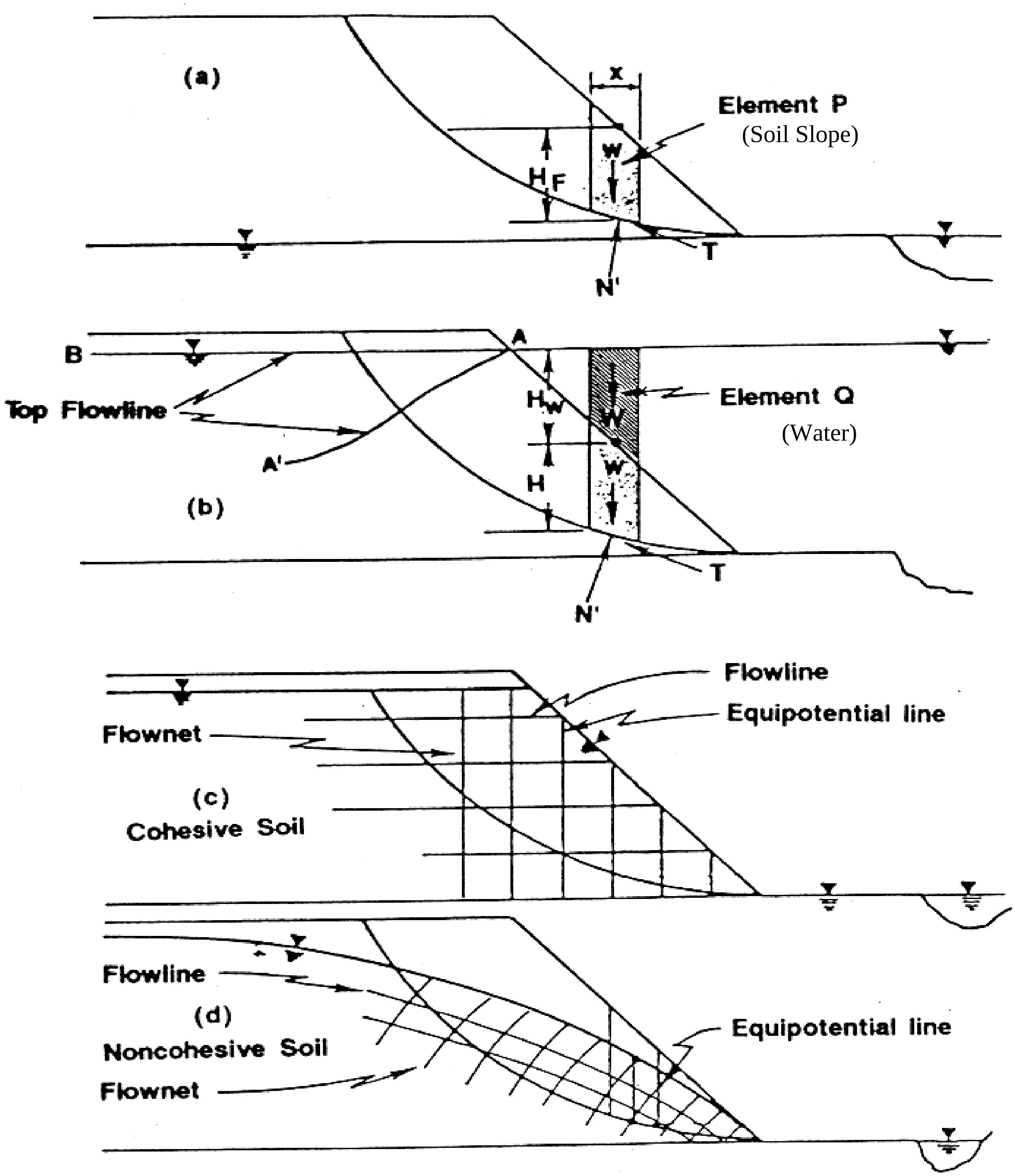


Figure 3-20: Development of a Rapid Drawdown Condition: (a) Before a Rise in the Water Level, (b) High Water Level, (c) Rapid Drawdown in Cohesive Soils, and (d) Rapid Drawdown in Noncohesive Soils (Hopkins, et al., 1975).

3.6 SEISMICITY

The release of energy from earthquakes sends seismic acceleration waves traveling through the ground. Such transient dynamic loading instantaneously increases the shear stresses in a slope and decreases the volume of voids within the materials of the slope which leads to an increase in the pressure of fluids (water) in pores and fractures. Thus, shear forces increase and the frictional forces that resist them decrease. Other factors also can affect the response of a slope during earthquakes.

- Distance of the seismic source from the project site.
- Magnitude of the seismic accelerations.
- Earthquake duration.
- Subsurface profile.
- Dynamic characteristics and strengths of the materials affected.
- Dimensions of the slope.

In addition to the distance of the seismic source to the project site, and the design (anticipated) time history, duration, and magnitude of the bedrock earthquake, the subsurface soil profile can have a profound effect on earthquake ground motions including the intensity, frequency content, and duration of earthquake shaking. Amplification of peak bedrock acceleration by a factor of four or more have been attributed to the response of the local soil profile to the bedrock ground motions FHWA NHI 132039 "Geotechnical Earthquake Engineering" (Karazanjdan et.al, 1999).

The ground accelerations associated with seismic events can induce significant inertial forces that may lead to instability and permanent deformations (both vertically and laterally) of natural and man-made slopes and embankment. In addition, during strong earthquake shaking, saturated cohesionless soils may experience a sudden loss of strength and stiffness, sometime resulting in loss of bearing capacity, large permanent lateral displacements, landslides, and/or seismic settlement of the ground. As shown in Section 2.5, liquefaction beneath and in the vicinity of a soil slope or embankment can have severe consequences since global instability in forms of excessive lateral displacement or lateral spreading failure may occur as a result.

Anyhow, local seismicity is a major factor that may affect the performance of a soil slopes and embankments if the seismic source is capable of producing strong ground motions at the project site. Readers are referred to FHWA NHI 132039 "Geotechnical Earthquake Engineering" (Karazanjdan et.al, 1999) for discussions in detail.

While the main geological features of a slope are always of interest, it has been established during recent years that minor geological details also have a significant impact on the behavior of the slope under seismic loads. For example, an almost instantaneous liquefaction can occur in very sensitive saturated clays and loose saturated or near saturated fine sands.

3.7 OTHER CONSIDERATIONS

Besides the shear strength parameters, the geological and the groundwater conditions, other factors that would affect the stability of a slope include tension cracks, foundation loads from nearby structures, presence of leaking water or sewer lines, and presence of trees and shrubs on slopes. The following sections discuss the influence of these factors.

3.7.1 Effects of Tension Cracks on Stability Analysis

Tension cracks increase the tendency of a soil to fail. First the length of failure surface along which shear strength can be mobilized is reduced, and second, a crack may be filled with water (for example, due to rainfall). An additional force caused by water pressure in a crack increases the tendency for a slip to occur.

Cracks open in cohesive soils because of their low tensile strengths, the depth of which can be found theoretically by

$$z_c = \frac{2c}{\gamma} \tan\left(45 + \frac{1}{2}\phi\right) \tag{3-8}$$

where:

z_c	=	depth of tension crack (m or feet)
c	=	cohesion (kn/m^2 or lb/sf)
ϕ	=	angle of internal friction
γ	=	unit weight of material. (kn/m^3 or lb/f^3)

The question often arises as to whether effective or undrained strength parameters should be applied to the above equation. The depth of a tension crack based on c' and ϕ' is likely to be much less than that based on *c* and ϕ because c' is often considerably less than c. For embankments and undisturbed natural slopes, it would be logical to use the effective stress parameters *c*' and ϕ', to calculate $z_{c,}$ since undrained conditions caused by sudden removal of lateral support probably have not occurred. However, undrained strength parameters for cut slopes may have to be used because tension cracks would be formed in the short term after a cut is made and their depth would not likely decrease thereafter.

The importance of tension cracks and their effect on stability is not always emphasized in soil mechanics. This may be due in part to conclusions reached by some that the effect of tension cracks on factor of safety for embankment slopes is negligible (for example, Spencer, 1967 and 1973). Such conclusions must be understood in terms of the small depth of tension cracks predicted by the equation for z_c. In cut slopes, tension cracks could extend to considerable depth and exert a significant influence on the value of factor of safety, as in the case of the slips at Bradwell (Skempton and La Rochelle, 1965).

Quite apart from the effect of tension cracks on a conventional limit equilibrium analysis, there is the importance of progressive failure. Tension cracks may in some cases be a consequence of the initiation of progressive shear failure. As such, the existence of tension cracks deserves careful attention. See Chapter 7 for signs of slope movement (landslides).

3.7.2 Effects of Vegetation

Vegetation-trees/shrubs or grass-on cut and fill slopes aids erosion control and adds considerable amenity to the landscape. The effect of vegetation on a slope is often to enhance slope stability. However, this effect is a complex interaction of mechanical and hydrological factors that are difficult to quantify. As yet, there are no firm design rules, but in recent years, there has been rapid growth in local experience with a variety of vegetation types. This knowledge can be of valuable assistance to the designer.

As a general rule, do not plant a single species of vegetation in isolation. On typical slopes, vegetation cover should include grass, trees, and shrubs. Consider appropriate vegetation management techniques to assist the natural succession process.

It is widely appreciated that slope vegetation is beneficial for erosion control and its contribution to landscape quality. However, the overall effect of vegetation as a slope stabilizing mechanism cannot be easily categorized as beneficial or adverse as illustrated in Table 3-9. The results of detailed research studies carried out elsewhere provide support for the use of vegetation as a net beneficial mechanism, primarily through the effect of root reinforcement (Prandini, et. al., 1977; Gray, 1978; Gray and Megahan, 1981; and Gray and Leiser, 1982).

Until further data from local studies are available, it is recommended that designs for new highway slopes that include vegetative surface protection should allow for direct infiltration while neglecting the beneficial effects provided by the vegetation. For stability analyses of existing slopes that have an existing tree and shrub cover, it may be appropriate to investigate and quantify the beneficial factors given in Table 3-9. See Gray (1978), Schiechtl (1980), and Gray and Leiser (1982) for guidance on suitable methods for assessing these factors. Where it can be clearly demonstrated that direct surface infiltration is a critical factor for the stability of a slope, and where stability cannot be improved by flattening the slope, it may be worthwhile to consider covering the slope face with rigid surface protection (for example shotcrete) in order to reduce infiltration.

TABLE 3-9
EFFECTS OF VEGETATION ON SLOPE STABILITY

FACTORS	EFFECT TYPES
BENEFICIAL FACTORS	*Type*
Interception of rainfall by foliage, including evaporative losses	H
Depletion of soil moisture and increase of soil suction by root uptake and transpiration	H
Mechanical reinforcement by roots	M
Restraint by butressing and soil arching between tree trunks	M
Surcharging the slope by large (heavy) trees *	M
Arresting the roll of loose boulders by trees	M
ADVERSE FACTORS	M
Surcharging the slope by large (heavy) trees *	H
Maintaining infiltration capacity	M
Root wedging of near-surface rocks and boulders and uprooting in typhoon	

Legend:
H hydrological
M mechanical
* This mechanism may be either beneficial or adverse to stability, depending on particular site factors (see Gray, 1978).

3.7.3 Foundation Loads on Slopes

The stability of a slope can be affected by excavation for construction of foundations on or adjacent to the slope. As a general rule, slope stability affected by foundations should be checked if the slope angle is greater than ½ ϕ' (Vesic, 1975). Where this is so, the foundation can be considered as an equivalent line load or a surcharge imposing horizontal and vertical loads and incorporated into the stability analysis (Figure 3-21).

The stability of a slope can also be impaired by excavation for the construction of shallow foundations on or adjacent to the slope and the demolition of structures supporting the toe of the slope. Consider both these effects during the analysis. In order to minimize the short-term instability of a slope, excavations should be as small as possible and should be properly shored.

Lateral loads because of deep foundations may affect slope stability. However, in a slope that has an acceptable factor of safety against failure, the effects of the lateral loads are usually not considered during design of most foundations (Schmidt, 1977). However, high lateral loading can be transferred to foundations in situations where there is significant ground movement (where the slope above or below the foundation soils or where the slope in front of the foundation is excavated). Under these circumstances, lateral loads on deep foundations should be handled by either stabilizing the slope or designing the foundation to resist the lateral loads.

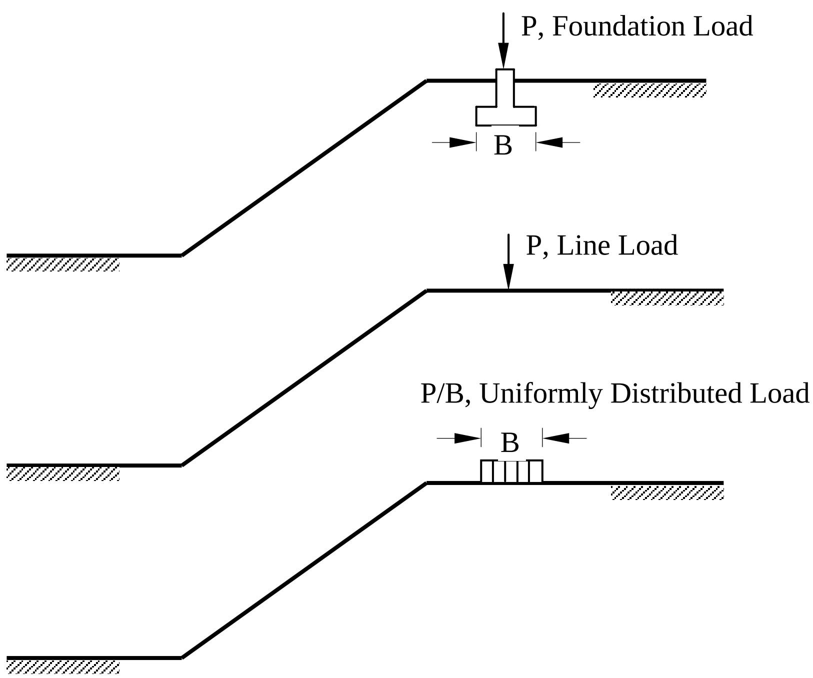


Figure 3-21: Foundation Loads on Crest of a Slope (FHWA, 1994)

3.7.4 Water and Sewer Lines

Where possible, it is imperative to route water or sewer lines away from the crest of a slope. If this is unavoidable, all possible steps must be taken to prevent leakage, which could have a dramatic effect on the stability of the slope. As a general rule, do not place drains, pipes, and services in a slope nearer to the crest of the slope than a distance equal to its vertical height (Figure 3-22). This is a minimum standard, but each case should be considered on its own merits.

In cases where the proposed development cannot be modified to permit the siting of drains, pipes, and services outside of the crest area, the slope should be designed to allow for the effects of possible water leakage. Where drainage pipes are planned to run down slope, they preferably should be left exposed instead of being buried on the slope face. This is because any leakage of the pipes, can be checked through regular maintenance and fixed before there is any detrimental effect on the slope.

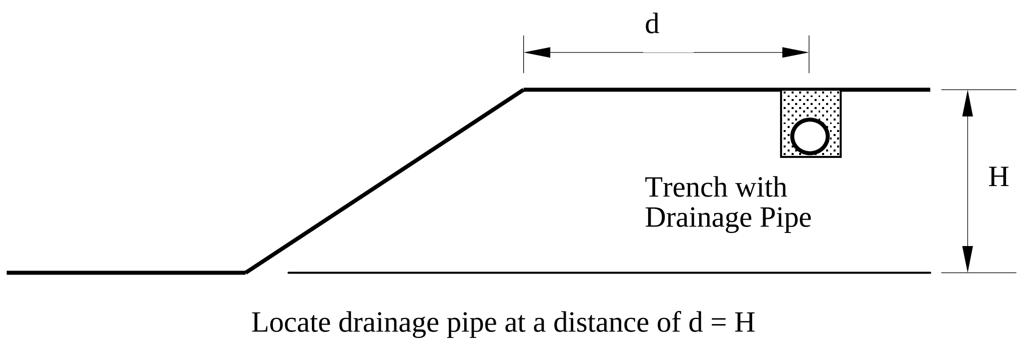


Figure 3-22: Location of Drainage Pipe on Crest of Slope (FHWA, 1994)

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CHAPTER 4
STRESS, STRENGTH, AND CONSOLIDATION CONCEPTS

4.1 INTRODUCTION

The design of soil slopes and embankments requires a full understanding of stress, strength, and consolidation concepts, and a reliable estimate of the geotechnical properties of the slope, embankment, and foundation materials. These parameters are strongly influenced by many complex conditions that include the in situ state of stress, drainage, history of consolidation, loading rates, and soil composition. The objective of this chapter is to provide the reader with sufficient understanding of fundamental soil mechanics concepts so that they may be properly applied to the design of soil slopes and embankments.

After presenting the concept of effective stress and vertical stress distribution, the following sections provide a brief introduction to the use of Mohr's Circle, the Mohr-Coulomb failure criteria, and pore water pressure parameters. They are then followed by a discussion of settlement of cohesive and cohesionless soils.

4.2 IN SITU STRESSES

A saturated soil mass consists of two distinct phases: the soil skeleton and the water-filled pores between the soil particles (Figure 3-11). Normal stresses imposed on such a soil will be sustained by the soil skeleton, and if the soil is fully saturated, the pore water by an increase in pressure providing the water cannot escape from the pores. Typically, the skeleton transmits normal and shear stresses at the inter-particle points of contact, and the pore water will exert a hydrostatic pressure that is equal in all directions (i.e. no shear resistance). The stresses sustained by the soil skeleton are known as *effective stresses,* and the hydrostatic stress from the water in the voids is known as *pore water pressure.*

It is the effective stress that controls the behavior of soil rather than the total stress or pore water pressure. Thus, if the soil particles are to be packed into a denser arrangement, it is the effective stress, rather than the total stress, that must be increased. When the total stress is increased, the increase is initially sustained by an identical increase in the pore water pressure, leaving the effective stresses unchanged. Only as pore water is allowed to escape the soil mass is stress transferred to the soil skeleton.

This correlation of effective stress with soil behavior, especially compressibility and strength, is known as the *principle of effective stress*. The effective stress, σ', acting on any plane within the soil mass is defined by:

$$\sigma' = \sigma - u \tag{4-1}$$

where σ is the *total stress* acting on the plane and *u* is the *pore water pressure*. The total stress is equal to the total force per unit area acting perpendicular to the plane, and the pore water pressure may be determined from the groundwater conditions, as discussed later. It should be noted that the effective stress cannot be calculated directly. It is always calculated indirectly with information about the total stress and the pore water pressure.

Most natural, saturated soils derive their strength from the friction at the inter-particle contacts. Since the shear stresses at the particle contact are frictional, the strength is directly controlled by the effective stresses. This shear strength of a soil is implemented via the Mohr-Coulomb failure envelope, which may be determined using a variety of laboratory and field-tests, as will be discussed in the following sections.

4.3 STRESS DISTRIBUTION IN SOIL DUE TO SURFACE LOADING

4.3.1 Vertical Stress Distribution based on Linear Elasticity

This section discusses methods to determine changes in total vertical stress ($\Delta\sigma_v$) due to foundation loading. The computation of the total vertical stress change induced by the foundation loading will depend on the distribution of the loads. We will present only solutions for total vertical stress change for uniformly loaded surfaces. Other references (Poulos & Davis, 1974) can be consulted for other load distributions. Additionally, the principle of superposition can be used to account for nonuniformly loaded foundations.

Methods based on the theory of linear elasticity are the easiest and most commonly used methods to determine the changes in total stress due to applied pressures. Most closed form solutions based on the theory of elasticity involve the assumptions that the soil is homogeneous, isotropic, and linear elastic.

The following methods may be used to determine the change in vertical stress on a soil particle due to a foundation loading:

i.) The change in vertical stress at any point (N) within a soil mass induced by a point load at the ground surface (Figure 4-1) may be determined by the following equation:

ii.)

$$\Delta\sigma_v = \frac{3\,Q}{2\pi Z^2}\left[\frac{1}{1+(r/Z)^2}\right]^{5/2}, \quad \text{for } Z > 2B_f \tag{4-2}$$

or

$$\Delta\sigma_v = \frac{Q}{Z^2} I_p \tag{4-3}$$

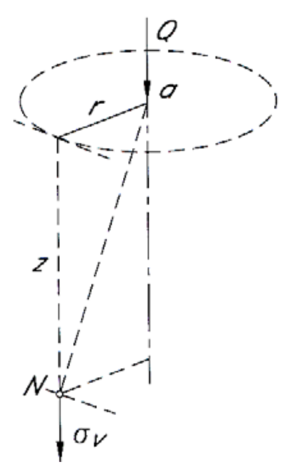


Figure 4-1: Vertical Pressure at the Point N in Interior of Semi-infinite Solid Acted on by Point Load Q (From Terzaghi, Peck, Mesri 1996)

Where:

Q = load applied at the ground surface
Z = vertical depth below point load
r = horizontal distance between the point of load application (a) and the point where the vertical pressure is being determined (N)
B_f = width of foundation.

$$I_p = \text{Influence Factor} = \frac{3}{2\pi}\left[\frac{1}{1+(r/Z)^2}\right]^{5/2} \qquad (4\text{-}4)$$

Note that Equations 4-2 and 4-3 are only valid for an equivalent point load at the depth (Z) to point N greater than 2 times of the width of the embankment. This restriction applies because an equivalent point load induces stress concentration beneath the base of a footing at depths above 2 B_f (Figure 4-2).

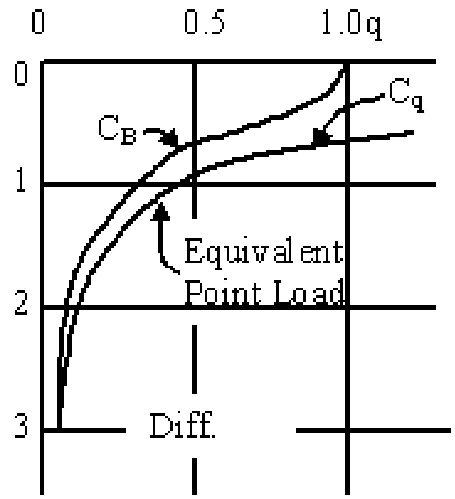


Figure 4-2: Effect on Vertical Pressure of Replacing Uniformly Distributed Load on Square Area by Equivalent Point Load at Center of Square. Curves Represent Stress Along Vertical Line Beneath Center of Square (After Terzaghi, and Peck, 1967)

However, a slope or embankment is not a point load and the restriction that Equations 4-2 and 4-3 applies only when $Z > 2\,B_f$ makes the use of these two equations somewhat impractical.

iii.) Because of limitations of equation 4-2 a more rigorous solution has been developed that considers the total loaded area and not an equivalent point load. Integrating the loaded area, solutions of the form of equation 4-5, may be developed.

iv.)

$$\Delta\sigma_v = \int_0^r \frac{3q_0}{2\pi Z^2}\,\frac{1}{\left[1+(r/Z)^2\right]^{5/2}}\,2\pi r\,dr \qquad (4\text{-}5)$$

Where: q_0 = applied vertical stress

The use of the linear elastic solution (equation 4-5) is particularly convenient as no elastic constants (e.g., Young Modulus and Poisson Ratio) need to be determined. In the process of deriving the closed-form elastic solution for vertical stress increase, the elastic constants cancel out. Equation 4-5 can be incorporated in a computerized code to yield the stress increase for many shapes of loaded area and load intensities. Such a computerized procedure is not as tedious as the chart solution, it allows for a wide range of possible design scenarios to be investigated quickly, and it does not require more input data than that for the charts.

Newmark (1935) performed the integration of the equation (4-5) and derived a equation for the vertical stress under the corner of a uniformly loaded rectangular area, which was simplified as (Holtz et al, 1981):

$$\Delta\sigma_v = I\, q_0 \tag{4-6}$$

Where: I = an influence factor which depends on m and n
m = x / z, n = y / z
x and y are the dimensions of the loaded area on the surface
z is the depth below the loaded surface to the point of interest

Figure 4-3 provides the influence value for vertical stress under corner of a uniformly loaded rectangular area. The stress changes beneath other points can be computed using the principle of superposition, as illustrated in Figure 4-4.

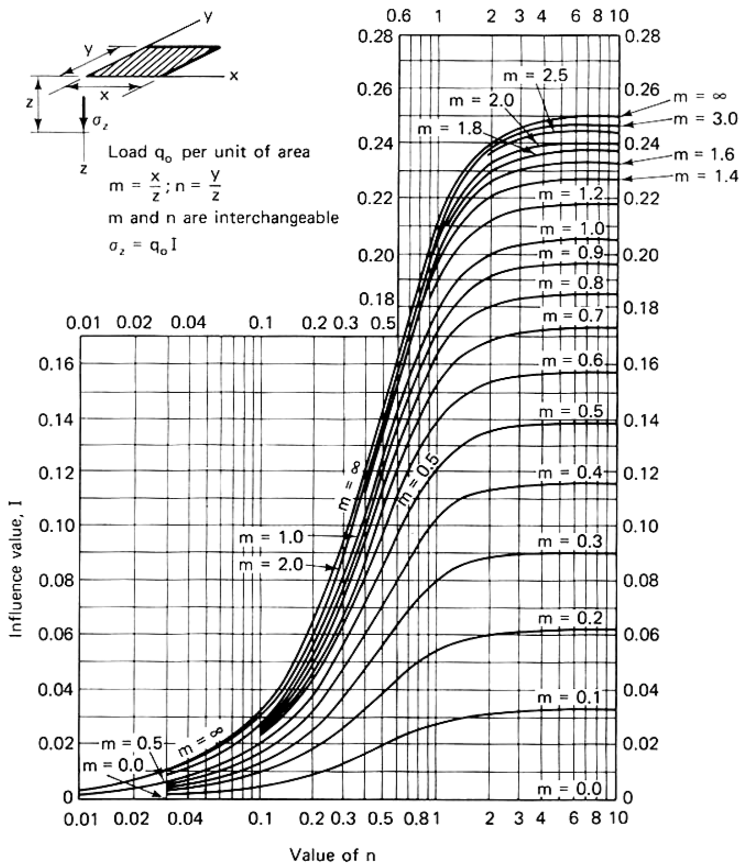


Figure 4-3: Influence Factors for Vertical Stress under an Infinitely Long Embankment for Uniformly Loaded Rectangular Area (After NAVFAC, 1982)

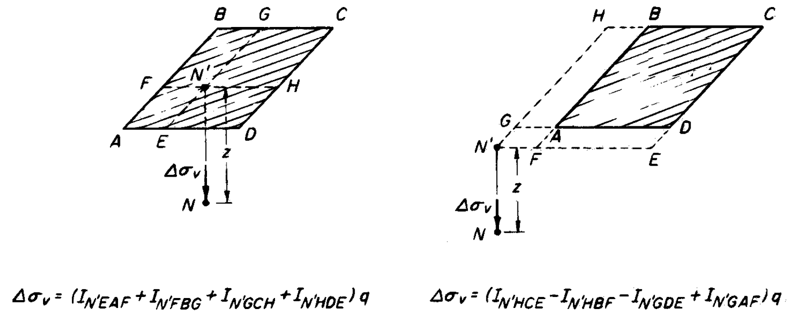


Figure 4-4: Superposition of Solutions to Obtain $\Delta\sigma_v$ at Any Point N In Soil Subjected to Uniform Load Over a Rectangle Area at Ground Surface (from Terzaghi, Peck, Mesri, 1996)

For loading caused by a long embankment, which can be simulated as a trapezoidal loading, Osterberg (1957) integrated the Boussinesq equations and provided influence values in terms of the dimensions and b, as shown in the Figure 4-5. If the embankment is not infinitely long, then use Figure 4-6 together with Figure 4-3 to represent different load configurations (Holtz et al, 1981).

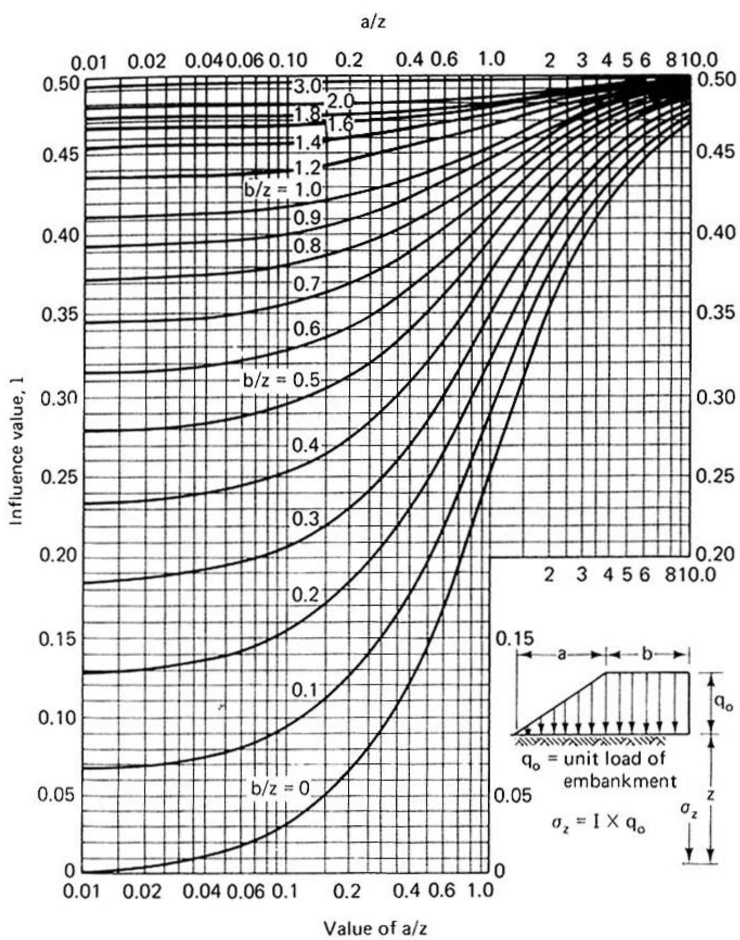


Figure 4-5: Values of Influence Factor I for Vertical Stress Under an Infinitely Long Embankment (NAVFAC, 1982)

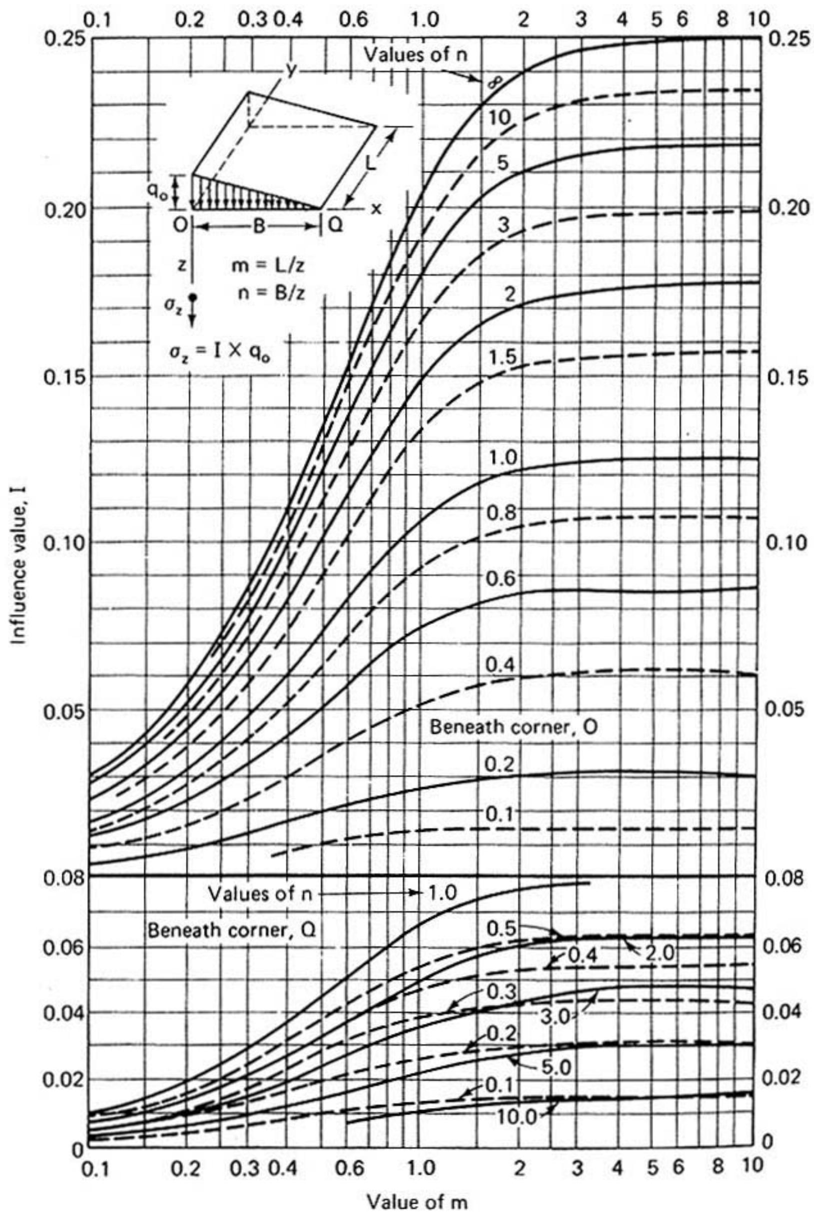


Figure 4-6: Values of Influence Factor I For Stress Under the Corner of a Triangular Loaded of Limited Length (after NAVFAC, 1982)

The methods illustrated above are appropriate for embankment and slope design. However, at the end of an embankment the vertical stress distribution is not easily modeled by the methods presented above. Finite element analysis may be used to determine the change in vertical stress below the end of an embankment. In lieu finite element analysis the following pressure distribution chart may be used. The step by step procedure that should be used with Figure 4-7 is provided below:

Step 1: Determine the distance (b) from the centerline of the approach embankment to the midpoint of sideslope. Multiply the numerical value of "b" by the appropriate values shown on the right vertical axis of the chart (Figure 4-7) to develop the depth at which the distributed pressures will be computed.

Step 2: Select the point (X) on the approach embankment where the vertical stress prediction is desired (normally at the intersection of the centerline of the embankment and the abutment). Measure the distance from this point X to the midpoint of the end slope. Return to the chart and scale that distance on the horizontal axis from the appropriate side of the midpoint of end slope line.

Step 3: Read vertically down from the plotted distance to the various curves corresponding to depth below surface. The "k" value on the left vertical axis should be read and recorded on a computation sheet with the corresponding depth.

Step 4: Multiply each "k" value by the value of total embankment pressure to determine the amount of pressure (ΔP) transmitted to each depth.

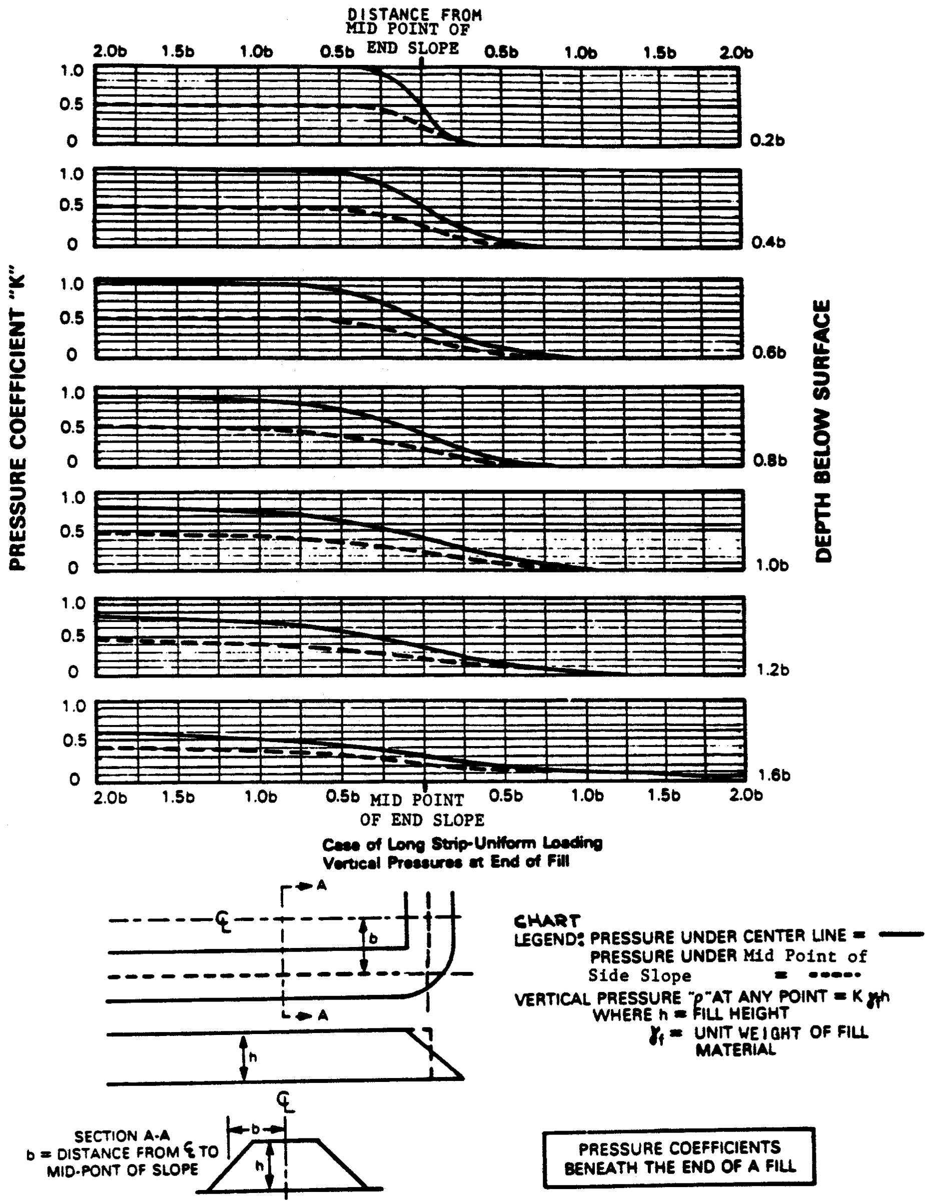


Figure 4-7: Pressure Coefficients Beneath the End of a Fill (Cheney, 2000)

4.3.2 Horizontal Stress Distribution based on Linear Elasticity

While the increase in vertical stress is important in assessing settlements, change in lateral stresses may affect the load acting against, for example, piles supporting a bridge abutment. Furthermore, it will result in lateral displacement within the foundation soil. In the construction of embankments over soft soil, lateral displacements are frequently monitored using inclinometers. Such lateral movements serve to assess whether the embankment is sufficiently stable. In fact, knowledge of the influence zone within which significant lateral stresses are produced is essential in selecting the location and depth of installation for inclinometers. Lateral loads on an adjacent structure or lateral soil movement are a reaction to a change in lateral stress within the foundation due to surface vertical loading.

Similar to equation 4-5, the theory of linear elasticity yields equations for lateral stress distribution in the direction of the longitudinal and transverse axes of the embankment. This solution, however, includes Poisson Ratio as a constant. Hence, use of chart solution in these cases is not as simple as for the vertical stress increase. One can use a computerized solution such as FHWA licensed program FoSSA to generate numbers. For example, Figure 4-8 shows the distribution of lateral stress in the x-direction produced by an embankment ear an abutment; such stress might be induced into piles or cause lateral soil movement. Poisson Ratio is not a constant for soils as assumed in the linear elastic solution. Furthermore, its determination for a 'representative' stress is rather complicated. However, for free-draining soil a reasonable value would be in the range of 0.25 to 0.35; for saturated soil under undrained conditions (e.g., rapidly loaded clay) its value would be 0.5. With the aide of a computerized solution, the user can conduct a parametric study to assess the effects of Poisson Ratio on the resulted lateral stresses. Such a study can assist in selecting design values of lateral stresses.

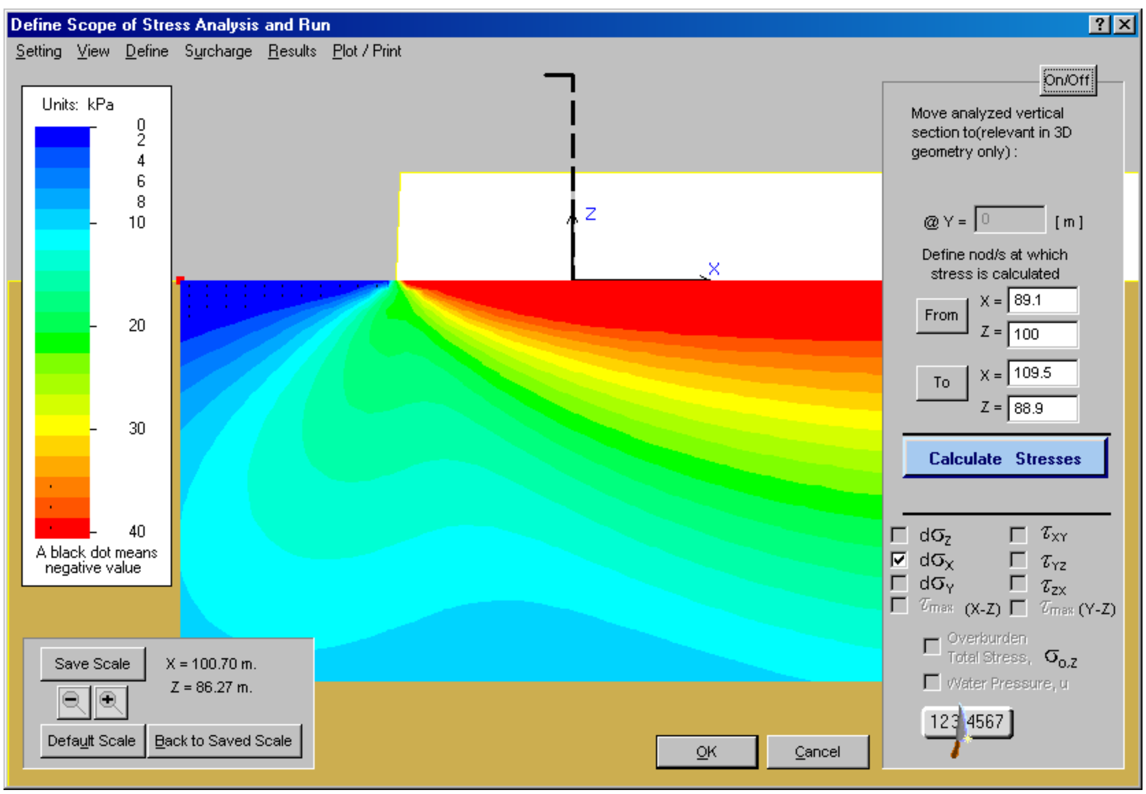


Figure 4-8: Lateral Stress Distribution at a Vertical End of an Abutment Wall

4.4 SOIL SHEAR STRENGTH AND MOHR CIRCLE

As discussed in the previous chapters, if the stress in a soil slope, embankment, or foundation is increased until the deformations (movements) become unacceptably large, the slope or embankment is considered a "failure". In this case, we are basically saying the strength of the soil, which can be defined as the maximum stress the soil can sustain, is insufficient to withstand the applied stress.

Typical strength testing of soils reveals two distinct forms of stress-strain curves, as shown in Figure 4-9. This figure presents a plot of the applied shear force as a function of shear strain for soils exhibiting *brittle* and *nonbrittle* behavior. Typically, dense granular soils and heavily overconsolidated soils exhibit a brittle behavior indicated by a distinct peak strength at low strains followed by a gradual decrease with increasing strains to a noticeably lower residual strength. The change from maximum to minimum strength is attributed to a dilatant behavior initially, which is then followed by contraction as the material approaches the critical state (i.e. shear strains without volume change).

Sensitive soils will also display a brittle stress-strain curve similar to Figure 4-9. Such soils have a metastable, brittle structure that tends to break down with increasing strain, generating positive excess pore water pressures. The large strain structure of these soils is similar to that of a soil that has been remolded completely. For such soils, the strength will have to be selected with great care, as any single localized failure can quickly propagate to a much larger instability due to progressive failure.

Loose granular soils and normally consolidated clays typically display non-brittle behavior, and there is no marked reduction in strength with increasing strains, as shown in Figure 4-9. For slope stability analysis, the shear strength for such soils may be assigned according to a pre-select level of maximum strain at a point along the upper plateau of the stress strain curve.

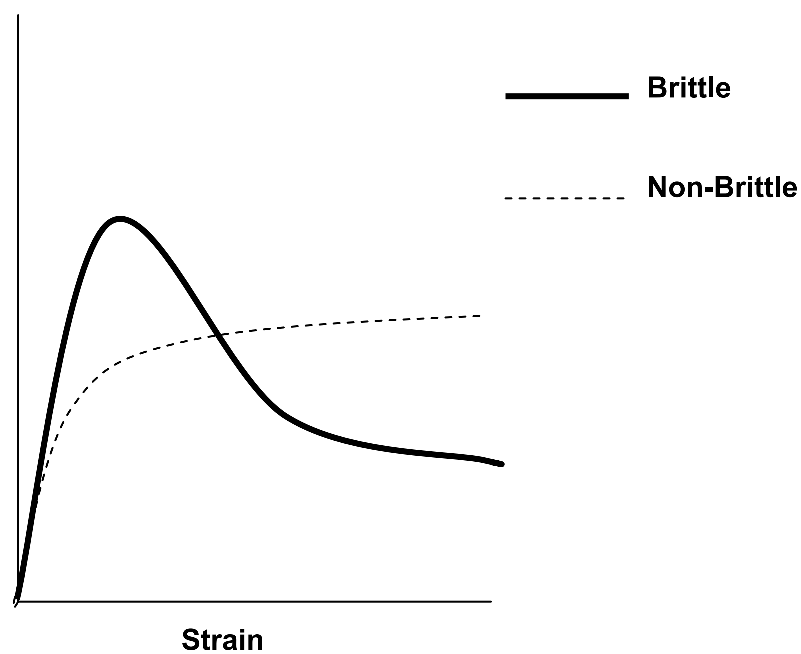


Figure 4-9: Typical Stress-Strain Curves for Soils

In geotechnical engineering, we are generally concerned with the shear strength of soils since, as discussed in Chapter 3, the typical geotechnical problems (failures) of soil slopes and embankments are mostly resulted from excessive applied shear stresses, or insufficient shear strengths. However, before we discuss the shear strength properties of soils, the relationship between the normal and shear stress and the concepts about the shear stress and failure criterion must be discussed first. Therefore, in this section, the concepts of Mohr circle, which can illustrate the relationship between the normal and shear stress, and the Mohr Coulumb failure criterion, are discussed briefly. A more detailed coverage of the evaluation of soil shear strength for the design of slope and embankment projects is provided in the following sources:

- NHI Course No. 132031A, Subsurface investigations and accompanying reference manual, FHWA NHI-01-031 (Mayne et al., 2001); and
- Geotechnical Engineering Circular (GEC) No. 5, Evaluation of soil and rock properties, FHWA IF-02-034 (Sabatini et al., 2002).

4.4.1 Mohr Circle

There exists at any stress point, in a two-dimension soil mass, two orthogonal planes on which there are zero shear stresses. These planes are called the principal stress planes. The normal stresses that act on the two planes are called the principal stresses. The largest of these stresses is called the major principal stress, σ_1 and the smallest is called the minor principal stress σ_3. When the stresses in the ground are geostatic, the horizontal plane through any point and the vertical plane through any point are the principal planes. When the coefficient of lateral earth pressure, K, is less than 1, $\sigma_v = \sigma_1$ and $\sigma_h = \sigma_3$. When K is greater than 1, $\sigma_h=\sigma_1$ and $\sigma_v=\sigma_3$. Given the magnitude and direction of σ_1 and σ_3 it is possible to compute normal and shear stresses in any other direction using the following equations:

$$\sigma_\theta = (\sigma_1 + \sigma_3)/2 + ((\sigma_1 - \sigma_3)/2) \cos 2\theta \tag{4-7}$$

$$\tau_\theta = ((\sigma_1 - \sigma_3)/2) \sin 2\theta \tag{4-8}$$

These equations, which provide a complete description of the state of stress, describe a circle. Any point on the circle, such as point A in Figure 4-10, represents the stress on a plane whose normal is oriented at an angle θ to the direction of the major principal stress. This graphical representation of the state of stress is known as the Mohr circle (Lambe and Whitman, 1969).

In a two-dimensional (e.g. plane strain condition) soil mass, the stresses at a point may be represented by the conceptual, infinitely small soil element (element A) shown in Figure 4-10. The state of stress of this soil element may be represented by shear and normal stresses τ_{xy}, σ_x, σ_y. To construct the Mohr circle for element "A" locate points ($-\tau_{xy}$, σ_x) and (τ_{xy}, σ_y) and draw a circle using these points to define the diameter. The major and minor principal stress planes may be determined by drawing a line through point A parallel to σ_y. The intersection of this line with the circle is point O_p. The line through O_p and σ_1 gives the plane on which the major principal stress acts. The line through O_p and σ_3 gives the plane on which the minor principal stress acts.

The magnitude of the principal stresses and the inclination of the principal planes can be determined graphically or by using equations 4-7 and 4-8. This concept is useful within the framework of shear strength as the state of stress in the ground can be related to the available shear strength via a failure criterion.

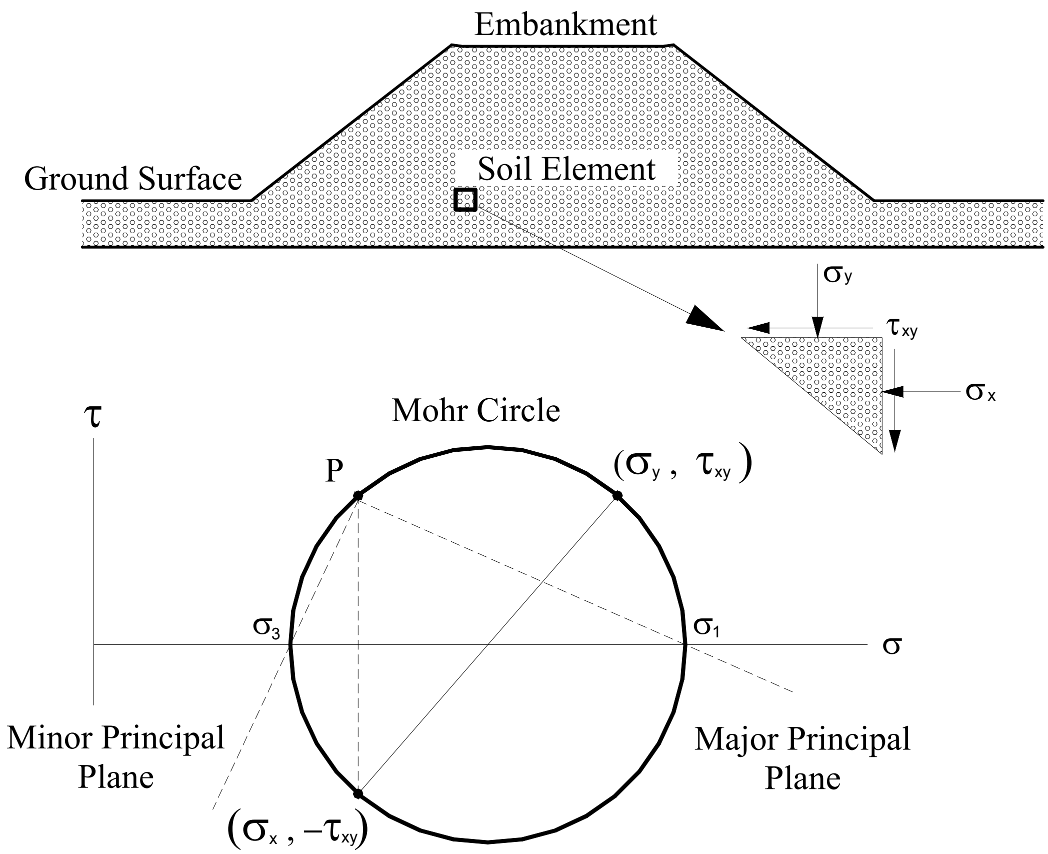


Figure 4-10: Mohr Circle Showing State of Stress at Element under an Embankment

4.4.2 Soil Shear Strength - Mohr Failure Envelope

The shear strength of a soil is usually defined in terms of the maximum shear stresses developed at the peak of the stress-strain curve for "brittle-type" material. However, the stress at "failure" for is sometime selected arbitrarily for "non-brittle" materials. In view of that, the shear strength is defined as the maximum or yield shear stress at some strain at failure.

Using the normal principle stresses at the failure, we can plot a Mohr circle to represent the state of stress at failure. This failure definition, referred as Mohr-Coulomb failure criterion, is illustrated in Figure 4-11. The physical meaning of the Mohr failure envelope may be explained as follows:

- If the Mohr circle for a given state of stress lies entirely below the Mohr envelope for a soil, then the soil will be stable for that state of stress.
- If the Mohr circle is just tangent to the Mohr envelope, then the strength of the soil has been reached on some plane through the soil.
- It is not possible to have a soil with a state-of-stress whose Mohr circle intersects the Mohr envelope for that soil.

The Mohr failure envelope represents that a soil fails when the shear stress on the failure plane of that soil reaches some unique function of the normal stress on that plane.

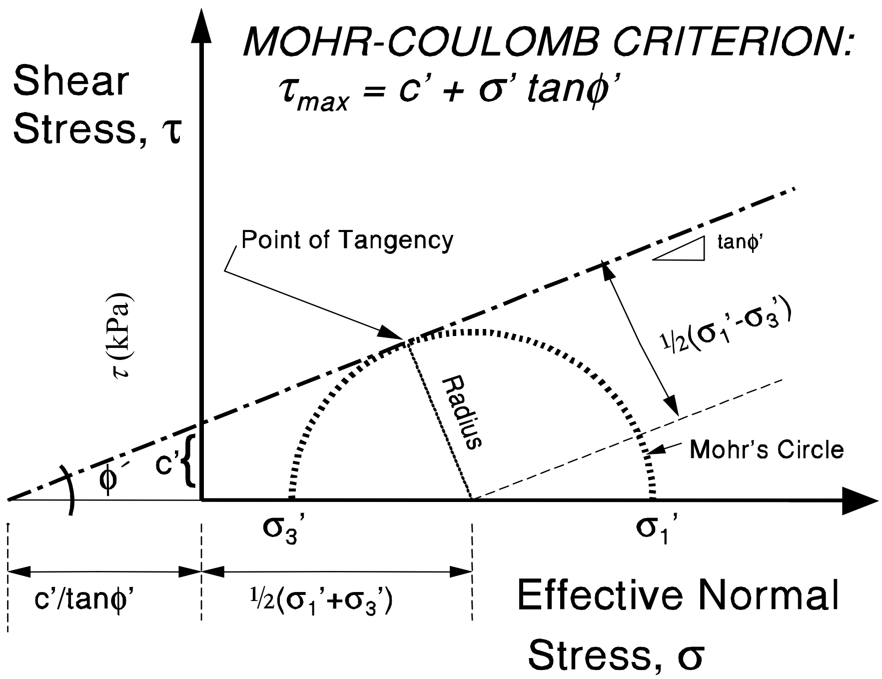


Figure 4-11: Mohr-Coulomb Failure Criteria (Sabatini, et al, 2002)

Mohr-Coulomb failure envelope is regarded as a straight line within the lower normal stress range of interest. However, it is actually a mildly curved line within the entire stress spectrum as shown in Figure 4-12. If one chooses to represent the failure envelope as a straight line as shown in Figure 4-11, the failure envelope is described as a Mohr-Coulomb envelope where:

$$s = c' + (\sigma - u) \tan \phi'$$

or

$$s = c' + \sigma' \tan \phi' \qquad (4\text{-}9)$$

where c' is the intercept on the strength axis (often called *cohesion*) and $\phi'$is the *angle of internal friction* related to the slope of the Mohr-Coulomb line shown in Figure 4-11.

If the design stress range is higer, the graphical representation of a Mohr envelope may also be expressed as a power function of the form:

$$S = A (\sigma')^b \qquad (4\text{-}10)$$

Where A and b are constants determined as part of the curve-fitting procedure following laboratory testing. The power function is popular approach, but there are several other forms of expressing such a nonlinear envelope as well (for example, Hoek and Bray, 1977). This approach is accurate for a very large range of normal stresses. However, when the normal stress is restricted to a range of loads the failure envelop may be approximated by a straight line. This is the approach typically taken the design of embankments and slopes.

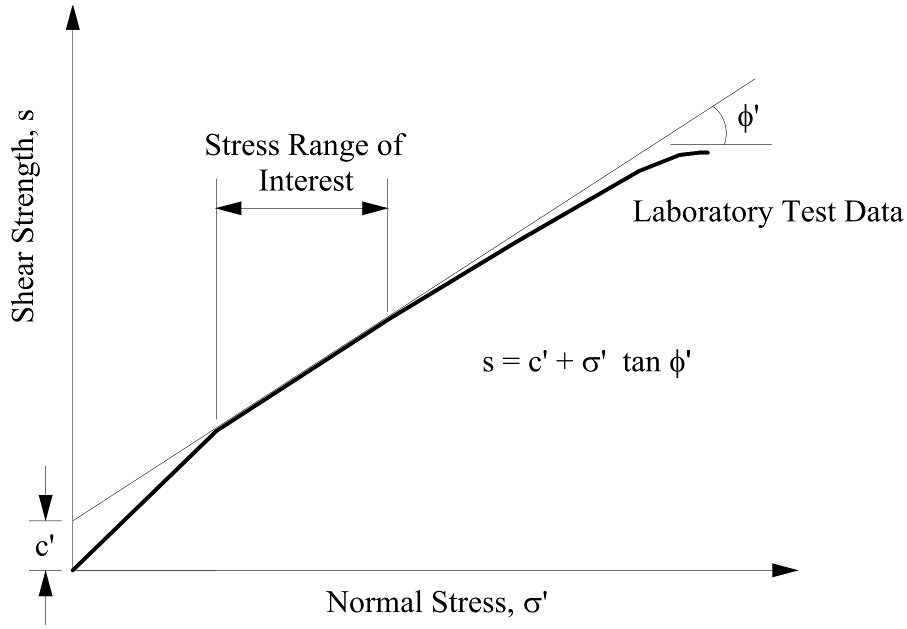


Figure 4-12: Typical Nonlinear Mohr-Coulomb Envelope of Shear Strength (FHWA, 1994)

4.5 SHEAR STRENGTH OF COHESIONLESS SOILS

Cohesionless (granular) soils are classified as materials that are comprised predominantly of sand and gravel and do not display any cohesive behavior under unconfined conditions. The shear strength of saturated cohesionless soils is derived exclusively from inter-particle friction and essentially is a function of the initial void ratio and confining stress. Thus, higher strengths are anticipated for granular soils with lower void ratios (at high unit weights) for typical confining pressures. During typical loading and unloading of saturated granular soils, the higher permeability (compared to cohesive soils) permits the rapid movement of water in the pores, effectively preventing the buildup of excess pore water pressures. Therefore, the shear strength parameters of interest for stability analysis of cohesionless soils are almost always the *drained shear strength*, which is the angle of internal friction.

The drained frictional angle of cohesionless soils can be determined by drained triaxial or direct shear test (Figure 4-13), or by correlations with grain size distribution, relative density, particle shape, and in-situ testing such as Standard Penetration Testing (SPT) (Table 4-1 and Figure 4-14) and Cone Penetration Testing (CPT) (Figure 4-15), etc. However, since undisturbed samples of granular soils are very difficult to obtain and thus, laboratory strength tests are rarely performed to determine their shear strength. Typically, the strength is assigned on the basis of empirical correlations with in situ tests. Figure 4-13 shows a typical Mohr circle plot of a drained triaxial test of cohesionless soils result with the associated Mohr Coulomb failure envelope indicating the drained angle of internal friction, ϕ'. Table 4-2 shows some typical angles of internal friction of cohesionless soils and the effect of void ratio or relative density. Readers are also referred to 132031A- "Subsurface Investigation" for field testing procedures and available empirical correlations.

However, if the rate of loading is very high, for example, during earthquakes, it is possible that the volume changes will not occur quickly enough to prevent the potential buildup of excess pore water pressure and liquefaction phenomenon may occur, as discussed in Chapters 2 and 3. Readers are referred to FHWA NHI 132039 Manual on – Geotechnical Earthquake Engineering for detailed discussions.

If a granular material is unsaturated, a small amount of apparent cohesion, under unconfined conditions, may exist because of negative pore pressures (suction) within the soil structure. However, such cohesion is of a temporary nature and cannot be relied upon for design of slopes.

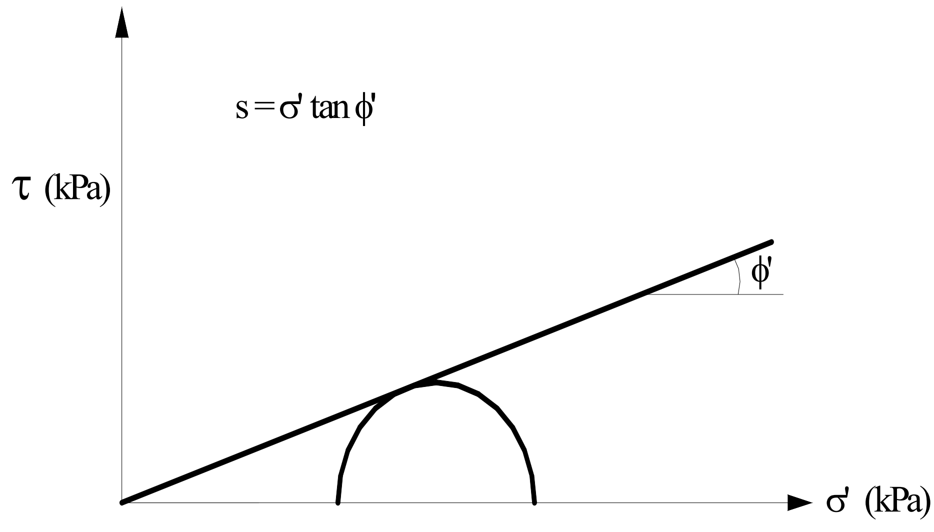


Figure 4-13: Typical Mohr Failure Envelope for Drained Shear Strength of Cohesionless Soil

TABLE 4-1
RELATIONSHIP BETWEEN SPT N VALUES AND INTERNAL FRICTION ANGLE of COHESIONLESS (GRANULAR) SOILS (after AASHTO LRFD, 2004).

State of Packing	Standard Penetration Resistance, $N1_{60}$ (blows/300 mm)	Friction angle, ϕ' (°)
Very loose	<4	25 - 30
Loose	4	27 - 32
Compact	10	30 - 35
Dense	30	35 - 40
Very dense	50	38 - 43

Note: $N1_{60} = N_{60}/(\sigma'_v/Pa)^{0.5}$, where Pa = 100 kPa (1 tsf), σ'_v = vertical effective stress, and N_{60} = ER/60% N (ER = hammer efficiency and N = uncorrected SPT blow count).

TABLE 4-2
TYPICAL ANGLE OF INTERNAL FRICTION OF COHESIONLESS SOILS (Holtz et al., 1981)

No.	General Description	Grain Shape	D_{10} (mm)	C_u	Loose		Dense	
					E	ϕ (deg)	e	ϕ (deg)
1	Ottawa standard sand	Well Rounded	0.56	1.2	0.70	28	0.53	35
2	Sand from St. Peter sandstone	Rounded	0.16	1.7	0.69	31	0.47	37*
3	Beach sand from Plymouth, MA	Rounded	0.18	1.5	0.89	29	-	-
4	Silty sand from Franklin Falls Dam site, NH	Subrounded	0.03	2.1	0.85	33	0.65	37
5	Silty sand from vicinity of John Martin Dam, CO	Subangular to subrounded	0.04	4.1	0.65	36	0.45	40
6	Slightly silty sand from the shoulders of Ft. Peck Dam, MT	Subangular to subrounded	0.13	1.8	0.84	34	0.54	42
7	Screened glacial sand, Manchester, NH	Subangular	0.22	1.4	0.85	33	0.60	43
8**	Sand from beach of hydraulic fill dam, Quabbin Project, MA	Subangular	0.07	2.7	0.81	35	0.54	46
9	Artificial, well-graded mixture of gravel with sands No. 7 and No. 3	Subangular to subrounded	0.16	68	0.41	42	0.12	57
10	Sand for Great Salt Lake fill (dust gritty)	Angular	0.07	4.5	0.82	38	0.53	47
11	Well-graded compacted crushed rock	Angular	-	-	-	-	0.18	60

* The angle of internal friction of the undisturbed St. Peter sandstone is larger than 60° and its cohesion so small that slight finger pressure or rubbing, or even stiff blowing at a specimen by mouth, will destroy it.
** Angle of internal friction measured by direct shear test for No. 8, by triaxial tests for all others.

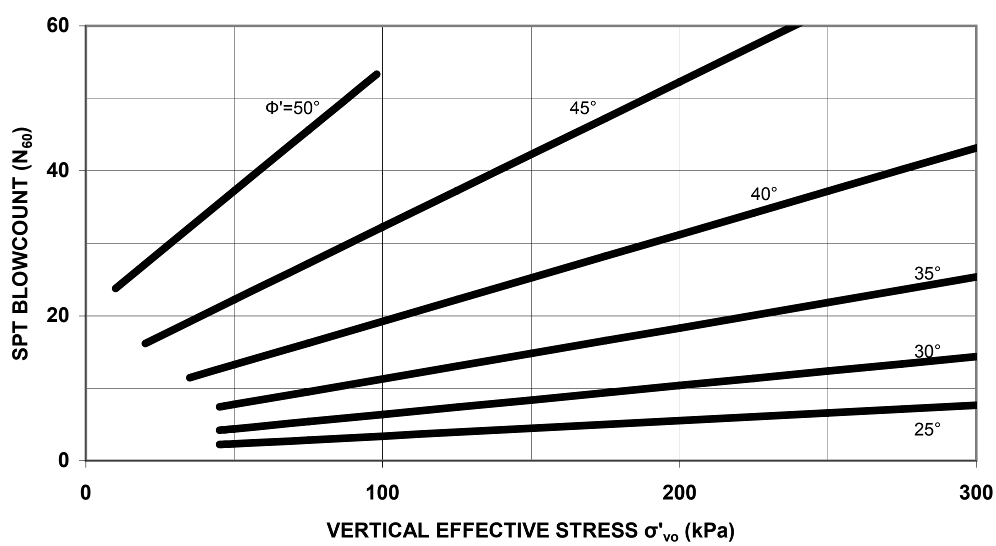


Figure 4-14: Correlation of ϕ' With SPT N_{60} in Clean Sands (after Schmertmann, 1975).

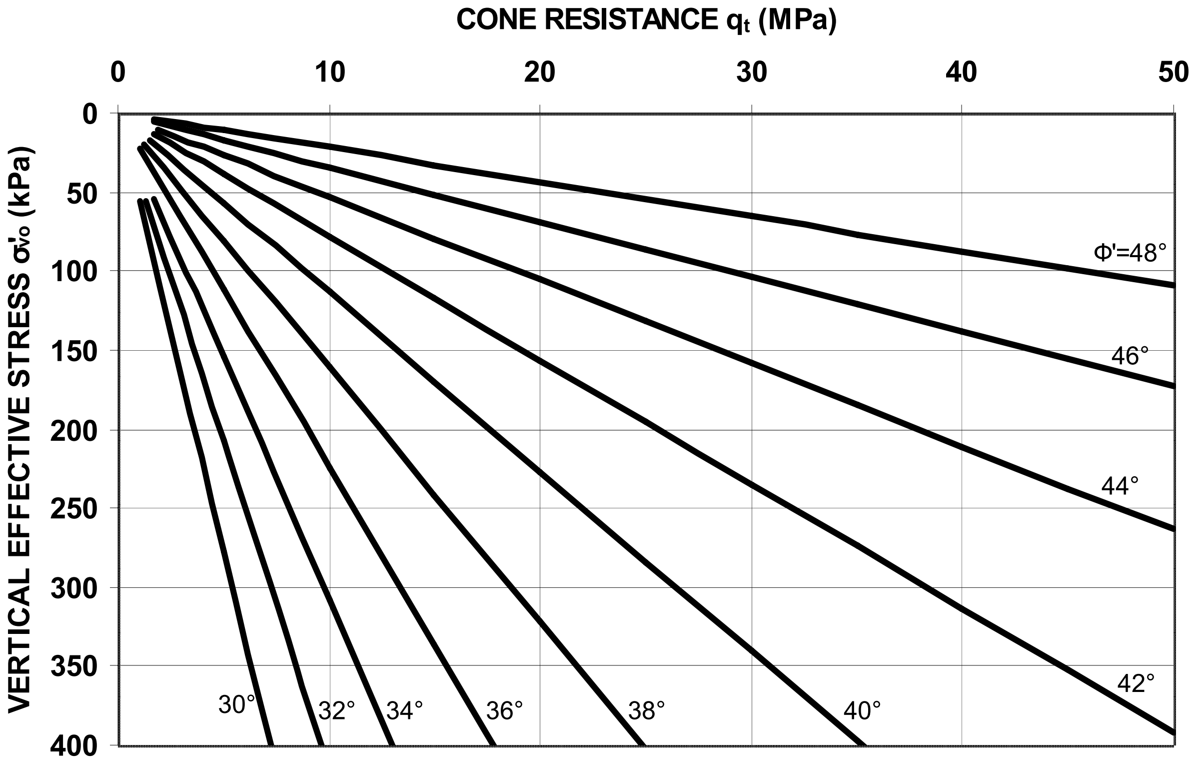


Figure 4-15: Correlation of ϕ' With Normalized CPT q_t Data in Clean Sands (after Robertson and Campanella, 1983)

4.6 SHEAR STRENGTH OF COHESIVE SOILS

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 pressures at the state of failure is known, both in the laboratory and in the field. Excess pore water pressures in cohesionless (granular) soils are expected to dissipate rapidly during construction and, thus, as discussed previously, only drained shear strengths should be considered. On the other hand, typically, loading (i.e. fill and embankment loading) and unloading (i.e. excavation) are applied much faster that the pore water can escape from the pores of a cohesive soil, and consequently excess pore pressure are generated. Soils that consist predominantly of fine-grained clayey particles may have considerable cohesive strength under this type of undrained conditions. This cohesive behavior is usually caused by inherent negative pore pressures within the soil mass that leads to positive effective stresses, which simulates the effect of a confined sample. This is due to as the saturated fine-grained soils have a much lower permeability than granular soils, the tendency for water to move in and out of the pores is severely restricted in the clayey soils, which becomes a consolidation phenomenon as discussed in details in Section 4.8.2.

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)

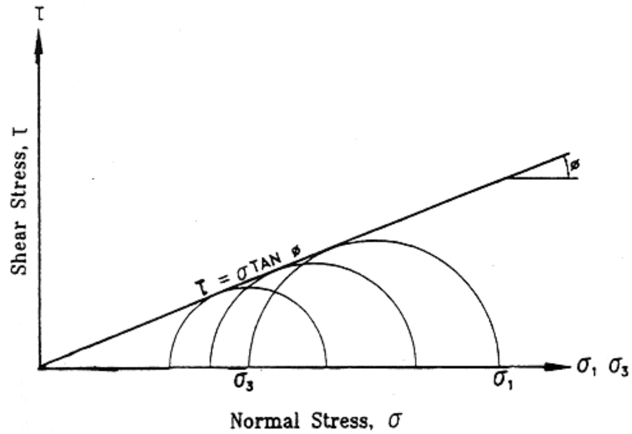
In the laboratory, there are limiting conditions of drainage in triaxial tests which can simulate the above three categories, which include consolidated-drained (CD), unconsolidated-undrained (UU), and consolidated-undrained (CU) conditions.

Another feature unique to clayey soils is the plate-like shape of clay minerals. If randomly oriented, these minerals have a tendency to align themselves in a direction parallel to a shear plane created at large strains. With such a particle arrangement, the shear strength along this realigned zone may be substantially less than the strength of the adjacent undisturbed materials. The strength of this realigned material is known as the *residual strength*.

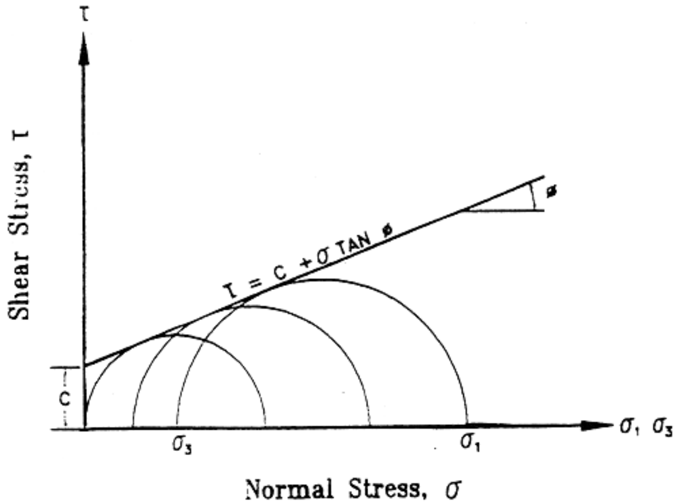
These shear strengths of cohesive soils are discussed hereafter. The use of these different shear strengths in the slope stability analyses are discussed in Chapter 5. For laboratory testing procedures and theories to obtain the UU, CD and CU strengths, the readers are referred to FHWA-NHI-01-031 "Subsurface Investigation", and GEC No 5 (Sabatini, et al, 2002).

4.6.1 Consolidated Drained (CD) Shear Strength of Cohesive Soils

As discussed previously, when the loading rate is very slow with respect to the rate of consolidation, or if the critical condition of a soil slope is in a long-term condition (such as a natural slope), the cohesive soils are likely to behave in a consolidated and drained condition. The available shear strength measured for such loading condition is called the *consolidated drained (CD) shear strength*. For consolidated-drained tests, the procedure is to consolidate the test specimen under some state of stress appropriate to the field or design situation, then the loading is applied very slowly so that essentially no excess pore water pressure develops during the test. Thus, it is sometime referred as the S-test (slow test). Besides CD shear test, drained strength of cohesive soils can also be obtained by drained direct shear tests, or consolidated-undrained tests (Section 4.6.3) with pore pressure measurements on specimens with the same field density and water content, and tested in the range of stresses that will occur in the field. The measured CD strengths are related to the effective stresses by means of the strength parameters c' and ϕ'. Mohr circle and failure envelope plots for CD shear tests are similar to the plots shown on Figures 4-16a.



(a)



(b)

Figure 4-16: Mohr Diagram (a) Consolidated Drained (CD) Triaxial Test, (b) Consolidated Undrained (CU) Triaxial Test

Figure 4-17 illustrates similar effects for an excavation where the loads on the soil in the slope are reduced, with a subsequent generation of *negative* excess pore water pressures (suction). As the pore pressures are negative, the effective stresses are temporarily increased and the soil displays an increase in shear strength. However, the negative pore water pressures will dissipate with time and lead to a subsequent reduction in effective stress, strength, and the factor of safety. Since the minimum factor of safety occurs at the end of the consolidation phase when the negative pore water pressures have dissipated completely, this example reflects the long-term case.

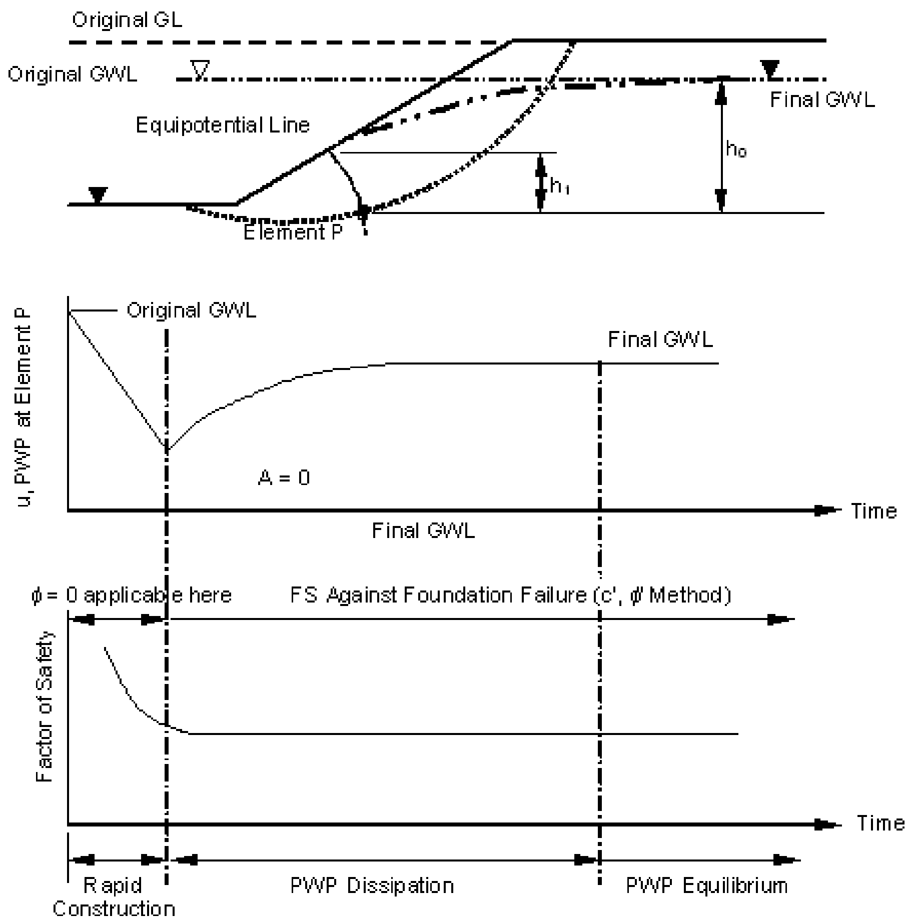


Figure 4-17: Changes in Pore Water Pressure and Factor of Safety of a Cut Slope During and After Construction (After Bishop and Bjerrum, 1960)

4.6.2 Unconsolidated Undrained (UU) Shear Strength of Cohesive Soils and Undrained Strength Ratio

If the construction of an embankment is rapid, the cohesive foundation soil is likely to be loaded in an unconsolidated undrained condition at the end of construction. In this condition, since the volume changes cannot occur, pore pressures increase and the effective stresses (and shear strength) are reduced accordingly. The available shear strength measured for such loading condition is called the *unconsolidated undrained (UU) shear strength*.

In order to obtain the unconsolidated undrained shear strength of a cohesive soil, the soil must be loaded rapidly with no drainage available, i.e. drainage valve closed for UU triaxial test. It is sometime referred as a quick test (Q-test). Figure 4-18 illustrates a typical Mohr failure envelope for unconsolidated undrained shear strength and its associated Mohr circles of total principal stresses.

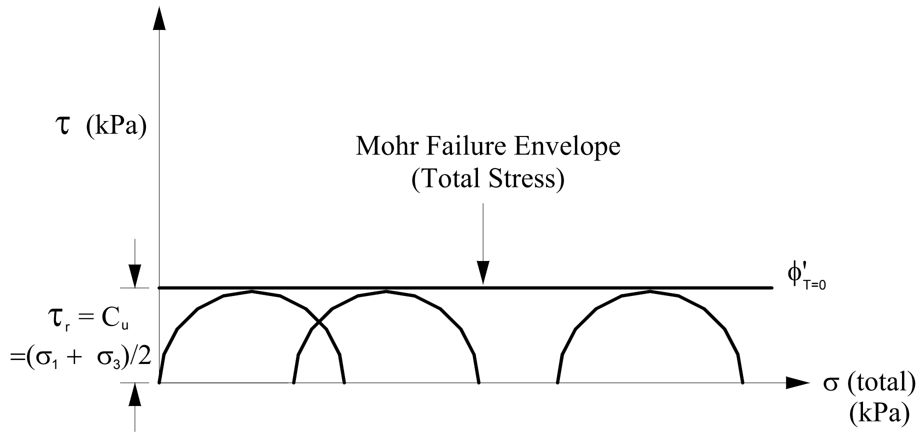


Figure 4-18: Typical Mohr Failure Envelope for Unconsolidated Undrained Shear Strength

Figure 4-19 shows the predicted variation of pore water pressure, at point *P* in a cohesive (fine-grained) soil, with an increasing height of embankment construction. During the construction phase, the pore water pressure increase is difficult to predict at all locations along the failure surface. As these pressures cannot be predicted, a total stress analysis using unconsolidated undrained (UU) shear strength are recommended to assess the stability of the slope during and immediately after construction (one-stage construction). Any excess pore water pressures generated by the construction will eventually dissipate and the embankment loads can be expected to directly affect the soil skeleton. For this case, the factor of safety will be at a critical minimum value at the end of construction when the excess pore pressures are likely to be at their maximum. The continuous reduction of pore water pressure leads to an increase in effective stress, strength, and the factor of safety. This construction example illustrates the short-term stability problem where the factor of safety is a minimum at the end of construction.

In addition to the laboratory testing such as UU triaxial test and direct shear test, the undrained shear strength is routinely estimated using in situ testing such as vane shear or cone penetration tests (CPT), and modified by empirical correction factors. The advantage of in situ testing is that it can economically produce a continuous or near continuous strength profile versus depth. However, empirical correction factors need to be applied to the field data to account for various variables such as soil plasticity index (PI), liquidity index (LI), sensitivity, overconsolidation ration (OCR), stress anisotrophy, loading rate, and allowance for fissuring and cracking in the over-consolidated crust of the soil, cracking in the embankment, the presence of shells, roots, and other obstructions affecting in-situ testing, etc. FHWA-NHI-01-031 "Subsurface Investigation" discusses the in-situ testing procedures and common correction factors. Readers are also referred to Duncan et al, 1987 and GEC 5, Sabatini et al, 2002 for detailed discussions.

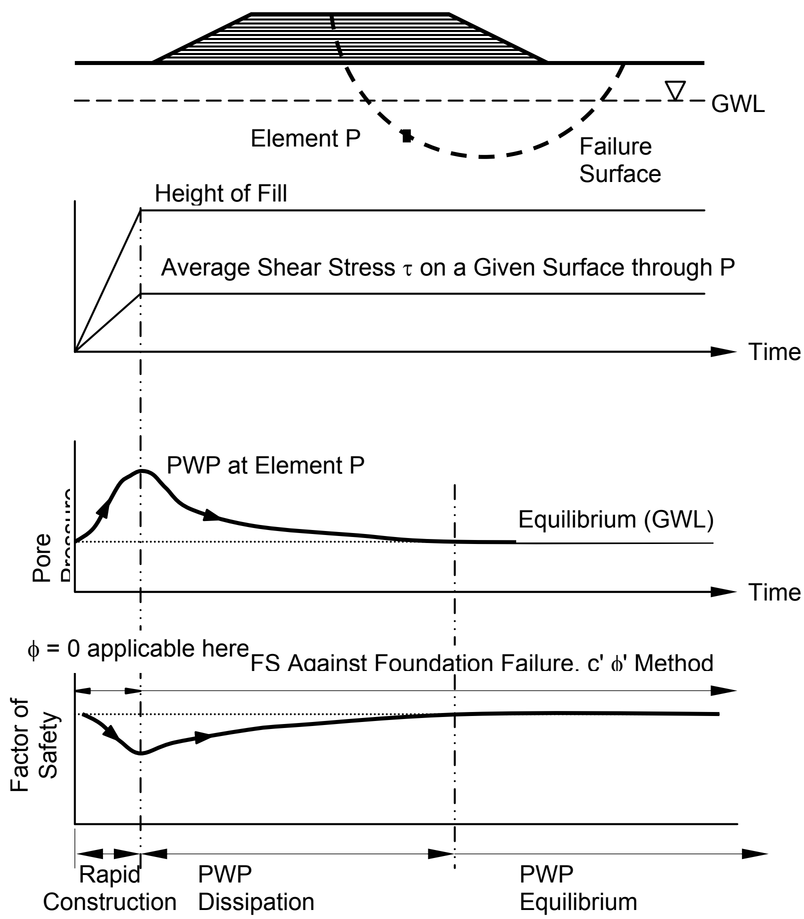


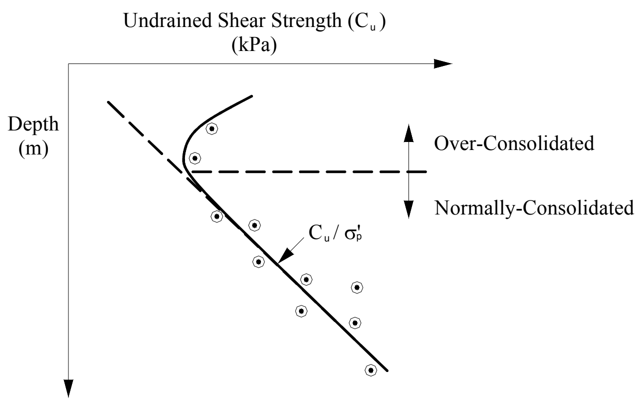
Figure 4-19: Change in Pore Water Pressure and Factor of Safety of an Embankment on Soft Clay during and After Construction (After Bishop and Bjerrum, 1960)

The UU shear strengths are basically considered as the in-situ undrained shear strengths (C_u). Many researchers had found the values of undrained shear strength (C_u) for the soft normally consolidated clays increase with depth. Moreover, this increase is likely to be proportional to the corresponding increase in vertical effective overburden stresses (σ'_p) with depth as shown in Figure 4-20a. This ratio, C_u / σ'_p is sometime termed the undrained strength ratio, or S_u/P ratio. The undrained strength ratio for normally consolidated soft clay typically ranges between 0.1 and 0.25. After developing a undrained shear strength based on all the laboratory and field testing results, a conservative ratio can be chosen as shown on Figure 4-20b.

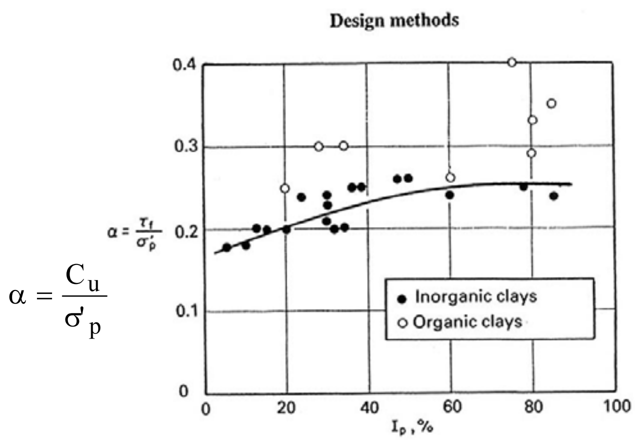
Many empirical correlations have been studied between the undrained strength ratio and other geotechnical properties such as overconsolidation ratio (OCR), plastic index (PI), liquidity index (LI), etc. Leroueil et al. (1990) recommended using Figure 4-20b to obtain the undrained strength ratio. Table 4-3 shows approximate relation between undrained strength ratio and overconsolidation ratio (Duncan et al, 1989). Readers are referred to FHWA-NHI-01-031 “Subsurface Investigation” and Duncan et al. (1989) for various empirical correlations to obtain the undrained shear strength. The undrained shear strength obtained through these empirical correlations can be used for preliminary feasibility studies and to check the reasonableness of initial strengths inferred from in situ and laboratory UU type test programs (Ladd, 1991).

TABLE 4-3
APPROXIMATE RELATION BETWEEN UNDRAINED STRENGTH RATIO AND OVERCONSOLIDATION RATIO

S_u/σ'_{vo}	**Approximately OCR**	
0 – 0.1	Less than 1	Still consolidating
0.10 – 0.25	1	Normally consolidating
0.26 – 0.5	1 to 1.5 (assume 1)	Normally consolidating
0.51 – 1.0	3	Overconsolidated
1 – 4	6	Overconsolidated
Over 4	Greater than 6	Overconsolidated



(a)



(b)

Figure 4-20: Typical Undrained Shear Strength Profile for Normally Consolidated and Overconsolidated Clays and Typical Relation Between Undrained Strength Ratio and Plasticity (after Larsson, 1980)

4.6.3 Consolidated Undrained (CU) Shear Strength

For condition where the slope or embankment is consolidated under one loading condition, and is then subjected to a rapid change in loading, with insufficient time for drainage, the cohesive soils are considered to be in a partially drained condition. Such loading condition includes staged or control-rate construction of an embankment, rapid drawdown, etc. Due to the complexity of predicting pore pressures during the partial drainage, partial consolidation stage, it is proposed that stability of a soil slope under such loading condition be analyzed using total stress analysis (Chapter 5) with *consolidated undrained (CU) shear strength*, which is also called the Undrained Strength Analysis (USA) (Ladd, 1991).

To obtain consolidated-undrained shear strength, the test specimen is first consolidated under some state of stress appropriate to the field or design situation, as in a consolidated-drained test. However, after the initial consolidation is completed, the drainage valves are closed and the specimen is then loaded to failure in an undrained condition. Figure 4-16b illustrate the typical C_u triaxial test results. Furthermore, Ladd and Foott in 1974 proposed the *S*tress *H*istory *a*nd *N*ormalized *S*oil *E*ngineering *P*roperties (SHANSEP) approach attempting to normalize the undrained shear strength (c_u or τ_f) with respect to the in situ *vertical* effective stress, σ'_{vo}. However, regardless how the laboratory testing is performed, the accuracy of the CU strength obtained from the laboratory testing is heavily dependant on good undisturbed sampling so the pre-consolidation pressure of the soil can be accurately determined in the laboratory, and careful testing technique (Leroueil and Jamiolkowski, 1991). Therefore, to avoid the high level of sophistication and expenses of laboratory CU testing, and the potential overestimation of CU shear strength, the CU shear strength of cohesive soil can be obtained based on the following procedures (after Ladd, 1991):

1. Establish the initial in situ stress history of the deposit (profiles of σ'_{vo}) and treat the in situ effective stress as equal to the pre-consolidation stresses (σ'_p).
2. Establish the undrained strength ratio, C_u / σ'_p based on the laboratory and field testing results, and verified with empirical correlations and local experience as discussed in Section 4.6.2
3. Obtain the initial undrained shear strengths for rapid drawdown analysis.
4. Establish changes in the vertical stress history during staged construction.
5. Obtain the consolidated undrained shear strengths based on the undrained strength ratio developed in Step 2, and the increased vertical stress profile developed in Step 4.

4.6.4 Discrepancies Between Field and Laboratory Strengths

There are at least six ways in which the sample strength measured in the laboratory can differ from the field or in situ strength (Skempton and Hutchinson, 1969). These include: (1) sampling, (2) sample orientation, (3) sample size, (4) rate of shearing, (5) softening upon removal of load by excavation, and (6) progressive failure, each of which is discussed in the following sections.

In addition to the factors mentioned above, the shear strength of a given soil is also dependent upon the degree of saturation, which may vary with time in the field. Because of the difficulties encountered in assessing test data from unsaturated samples, it is recommended that laboratory test samples be saturated prior to shearing in order to measure the minimum shear strengths. Unsaturated samples should only be tested when it is possible to simulate in the laboratory the exact field saturation and loading conditions relevant to the design.

<u>Sampling</u>

Stability and deformation predictions based on laboratory shear strengths and compressibility characteristics may have serious limitations for both slightly and heavily overconsolidated clays of high plasticity. This is attributed to the difficulty of obtaining representative samples, measurement of reliable pore pressures, the impact of fissuring, and the gradual degradation of strength with time for overconsolidated clay shales.

Block or large diameter samples usually provide the most reliable laboratory strength results. However, the cost of procuring these types of samples is relatively high and, thus, slope analyses are usually based on samples obtained using thin-walled piston samplers (recommended) or Shelby tubes. The sample disturbance is likely to affect the sample's water content, voids ratio, and structure, which will lead to a poor estimation of in situ shear strength. Table 4-4 provides a summary of the various features that are likely to disturb the soil samples, and, thus, affect the properties measured in laboratory tests. In general, sampling and disturbance will tend to reduce the measured strength of the soil. In soft clays, even the best sampling technique will lead to some reduction in undrained strength because of the changes in total stresses inevitably associated with sampling from the ground. The effect of sample disturbance is most severe in soft sensitive soils and appears to become more significant as the sampling depth increases.

Table 4-4
SOURCES OF SAMPLE DISTURBANCE IN COHESIVE SOILS
(After Jamiolkowski, et al., 1985)

CONDITION	ITEM	REMARKS
Stress Relief	Change in stresses because of drilling hole.	Excessive reduction in σ_v because light drilling mud causes excessive deformations in extension. Overpressure causes excessive deformation in compression.
	Eventual removal of in situ stress	Resultant shear strain should usually be small
	Eventual reduction (removal) of confining stress	Loss of negative u (soil-suction) caused by presence of coarser-grained materials. Expansion of gas (bubbles and/or dissolved gas)
Sampling Technique	Sample geometry: Diameter/Length Area ratio Clearance ratio Accessories, i.e. piston, coring tube, inner foil, etc.	These variables affect: Recovery ratio Adhesion along sample walls Thickness of remolded zone along interior wall
	Method of advancing sampler	Continuous pushing better than hammering
	Method of extraction	To reduce suction effect at bottom of sample, use vacuum breaker
Handling Procedures	Transportation	Avoid shocks, changes in temperature, etc.
	Storage	Best to store at in situ temperature to minimize bacteria growth, etc. Avoid chemical reactions with sampling tube. Opportunity for water migration increases with storage time.
	Extrusion, trimming, etc.	Minimize further straining (i.e., do it carefully)

Sampling Orientation – Anisotropy

Sample orientation can be important where the soil stratum contains discontinuities or fissures, as is often found in residual soils. Where failure in the field could occur along discontinuities or relict joints, this fact must be taken into account when orienting the laboratory samples.

Most natural soil deposits exhibit anisotropic soil behavior. For slope stability analyses, this anisotropy affects the shear strength of the soil and thus the factor of safety. The most significant parameters in a limit equilibrium analysis, with respect to the principal in situ stress are (a) the original consolidation stresses and (b) orientation of the failure plane. The overall concept of this anisotropy is presented in Figure 4-21. In this figure, three soil elements (A, B, and C) are shown along with orientation of the principal stresses at failure and the failure plane itself. In general, the shear strength varies as a function of the angle between the principal planes and the failure plane. For San Francisco Bay Mud (Duncan and Seed, 1966), the strength along the failure plane for element C was about 75 percent of the tested undrained strength for element A. Most triaxial tests are performed on isotropically consolidated samples and the orientation of the failure plane would most likely correspond to element A in Figure 4-21. Thus, the strength for the San Francisco Bay Mud can be expected to range between those at Points A and C, depending on whether the sample was consolidated isotropically or anisotropically. Similar results can be expected for other soils.

The vast majority of conventional strength tests are made (1) on triaxial compression specimens with a vertical axis, (2) on shear box specimens with a horizontal shear plane, or (3) by means of in situ vane tests measuring the undrained shear strength that is controlled essentially by the strength on vertical planes. Because of anisotropy, the strength along a slip surface in the ground may vary considerably from the laboratory strength along a slip surface measured by conventional testing, due solely to differences in orientation.

As pointed out by Lowe (1960, 1967), anisotropically consolidated specimens conform better to probable field conditions and are claimed to yield higher shear strengths. It may be practical on larger projects to perform anisotropically consolidated tests (Johnson, 1974). Possible effects of anisotropic consolidation can be examined using several approximate and preliminary methods based on isotropic consolidation test data. The procedure developed by Taylor, described in detail by Lowe and Karafieth (1960), or the method proposed by Skempton and Bishop (1954), may be used to compute anisotropically consolidated strength from results of isotropically consolidated tests.

For most slope analyses where triaxial testing is employeed, the shear strength values from isotropically consolidated tests are used to model the slope materials. This effectively underestimates the strength of soils consolidated at stress ratios, $K \geq 1.0$, and thus leads to a conservative estimate of the factor of safety.

<u>Sample Size</u>

Ideally, samples should be sufficiently large to contain a representative selection of all the particles and all the discontinuities in the soil. This is particularly true for fissured clays for which the sample size can play an important role. For 38mm by 76mm triaxial samples, a wide scatter is usually found among the results principally because of fissures that may or may not be present in the test specimen. In this case, samples of at least 10cm in diameter should be tested, and an average strength should be selected on the basis of a considerable number of tests.

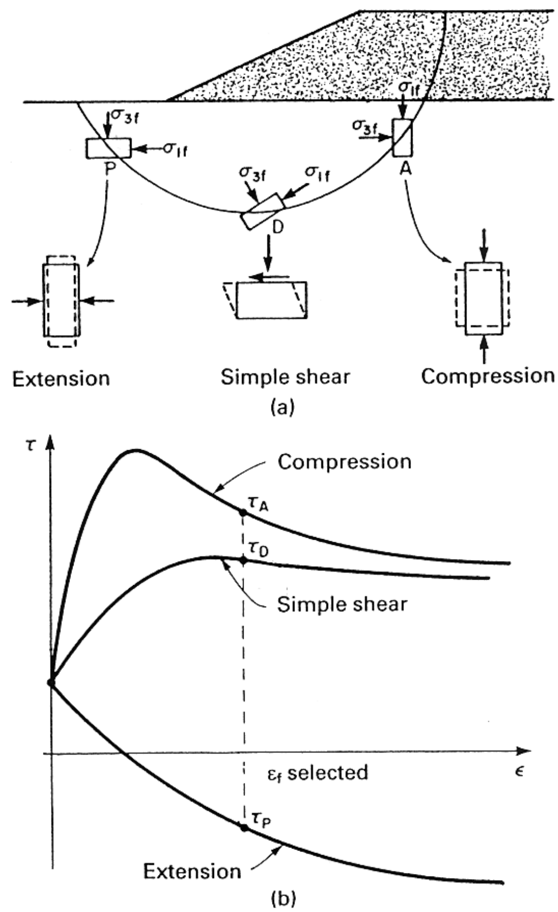


Figure 4-21: Directional Strength Anisotropy (Leroueil, et al., 1990)

Rate of Shearing

Conventional triaxial tests are routinely performed at relatively rapid strain rates as compared to the field loading conditions. Studies by Duncan and Buchignani (1973) and Skempton and Hutchinson (1969) indicated that lower strengths would be obtained for specimens sheared at lower shearing rates in the laboratory. For instance, Duncan and Buchignani (1973) stated for San Francisco Bay Mud, the shearing resistance is only about 70 percent of the values measured in conventional triaxial tests for loads maintained a week or longer. Other data reported by Skempton and Hutchinson (1969) is in similar agreement.

The accelerated rate of shearing in the laboratory thus tends to overestimate the in situ shear strength that may be mobilized during movement of the slope. This overestimation is usually evident if back-calculated strength values from observed failures are compared with high-quality laboratory test data. However, for actual design, this overestimation will be partially offset by an underestimation of strength because of sampling disturbance, as well as the many uncertainties that are accounted for by the final design factor of safety.

Softening

The stress relief associated with removal of load by excavation will initiate a process of softening as water is attracted by the negative excess pore pressures. Because of the low permeability of clays, the final state of equilibrium under the reduced effective stresses (long-term condition) may not be attained until many years after excavation. The lateral expansion followed by load reduction may cause some opening of fissures and an increase in mass permeability and coefficient of consolidation (C_v). Thus, negative excess pore pressures usually dissipate much faster than positive excess pore pressures. Softening may proceed from the face of the open fissures under zero effective stress, thereby causing a reduction in average strength, which in turn, allows for more deformation. Other fissures then open and the process continues until the strength of the clay mass falls to a much lower value and overall failure occurs.

If the softened clay has not been sheared previously (by failure, for example), its effective angle of friction (ϕ') will remain essentially constant, but its effective cohesion (c') will tend to approach zero (Skempton and Hutchinson, 1969). For stiff fissured clays, the effect of internal softening will result in strengths that are lower than those measured in conventional drained tests. If the softened clay is sheared past its peak strength, the effective friction angle will also decrease and the discrepancy may be even larger.

Progressive Failure

As mentioned in an earlier section, the magnitude of the mobilized strength along a failure surface is far from uniform along its entire length. If at some time, the shear stress exceeds the available strength in a small zone along the failure surface, the excess loading will have to be transferred to adjacent zones. However, if the soil exhibits brittle behavior, the stress transfer is likely to lead to failure in adjacent elements as well. Thus, failure having been initiated at a single point generally progresses until the entire mass fails.

Therefore, the peak strength, in a first-time slide, must be reached at some point before others. For brittle clays, the strength will continue to decrease until overall failure occurs. The more brittle the clay, the greater the difference is likely to be between the mobilized strength (say, measured by a field vane test at the initiation point) and the average peak strength (say, determined by laboratory tests on collected "undisturbed" samples) along the slip surface.

Once a progressive failure is initiated, it may proceed slowly or rapidly. There are numerous records of cuts and natural slopes remaining stable or undergoing very slow creep movements for many years before the final period of accelerated movement and failure.

4.7 CONSOLIDATION AND SETTLEMENT OF COHESIVE SOILS

The compression of soils in response to loading can be broadly divided into two types: elastic settlement and time-dependent settlement. Elastic settlements are instantaneous, recoverable, and are commonly calculated from linear elastic theory. Time-dependent settlements occur in both cohesionless and cohesive soils, although the response time for cohesionless soils is usually short. In addition to being time-dependent, the soil’s response to loading is non-linear, and deformations are only partially recoverable. Two types of time dependent settlement are recognized. Primary consolidation results from the squeezing out of water from the soil voids under the influence of excess pore water pressures generated by the applied loading. Secondary compression occurs essentially after all the excess pore water has dissipated. The mechanisms involved, however, are not fully understood.

Total settlement (S_t) of cohesive soil when subjected to a constant normal stress is typically divided into three components:

Immediate Compression (S_i) due to compression of gas in partially saturated cohesive soils and shear deformation of constant volume.

Primary Consolidations (S_c) due to the change in the void ratio of the cohesive soils. The rate of primary consolidation is controlled by the rate of dissipation of excess pore water pressure
Secondary Compression (S_s) due to a time-dependent adjustment of the soil structure and can be considered a creep-type phenomenon (Mitchell, 1976)

Equation 4-15 represents this concept:

$$S_t = S_i + S_c + S_s \tag{4-15}$$

The prediction of settlements of embankments founded on relatively homogeneous cohesive soil involves five steps.

- Computation of the initial vertical effective stress (total vertical stress and pore water pressure) of the layer(s) midpoint
- Determination of overconsolidation pressures
- Computation of changes in vertical effective stress (associated with changes in both total stress and pore water pressures) due to the construction
- Determination of compressibility of the clay, accounting for sampling disturbances
- Computation of layer compressions.

4.7.1 Immediate Settlement

Some settlement due to immediate deformation occurs within a short period of time, generally during construction. Accordingly, most of the immediate settlement may occur prior to the placement of the superstructure, in which case it may be of little or no consequence to the integrity of the superstructure.

Immediate settlement (S_i) is computed for embankments using the Theory of Elasticity (Timoshenko and Goodier, 1951). The immediate settlement of an embankment may be estimated using the following equation (Schleicher, 1926):

$$S_i = q_0 \, (1 - \nu^2) C_d B / E_s \tag{4-16}$$

where:

q_0	=	the vertical stress at the base of the loaded area
ν	=	Poisson's ratio
B	=	width of the embankment
E_s	=	Young's modulus for the soil
C_d	=	a factor to account for the shape of the loaded area and the position of the point for which the settlement is being calculated (Table 4-5)

Laboratory testing (triaxial compression) of relatively undisturbed samples of cohesive soil should be performed to determine E_s (undrained modulus for the immediate settlement case for saturated clays). Typical ranges of the modulus (E_s) and Poisson' ratio (ν) are summarized in Table 4-6. The values of E_s

have also been correlated to standard penetration (N) and cone penetration resistance (q_c) for cohesionless soils, and the undrained shear strength (s_u) for saturated cohesive soils. The factor for the shape of the loaded area and position of the point for which the settlement is being calculated is listed in Table 4-5. These correlation's are also shown in Table 4-6.

TABLE 4-5
SHAPE AND POSITION FACTORS C_d
(Modified from Winterkorn and Fang (1975))

Shape	**C_d Center of Embankment**
Rectangular (length/width)	
1.5	1.36
5	2.10
10	2.53
100	4.00
1000	5.47

The immediate settlement for saturated cohesive soils should be small comparing to the total amount of long-term settlement if the embankment is properly designed. For saturated cohesive soils, primary and secondary consolidation settlements will typically result in a much larger magnitude of settlement than immediate settlement.

The immediate settlement can be the predominant portion of the total settlement for unsaturated and highly over-consolidated soils. For unsaturated soils, the immediate settlements will involve both compression and distortion. Since water is essentially incompressible, the immediate settlements for saturated soils are predominantly distortional (no volume change).

If the applied stress will not exceed the past consolidation pressure the total settlement can be predicted adequately assuming only linear elasticity for many heavily over-consolidated soils. The modulus for this case can be determined in triaxial tests by maintaining the test load under drained conditions, using computer controlled triaxial equipment.

Immediate settlements can be predicted for silts (including loess) provided reasonable values of the elastic moduli are used. However, immediate settlement estimates based on empirical data are commonly utilized instead of the theory of elasticity. For silts, sands and gravels the moduli in Table 4-6 represents the drained modulus.

TABLE 4-6
ELASTIC CONSTANTS OF VARIOUS SOILS
(Modified After U.S. Department of the Navy (1986a) And Bowles (1996))

Typical Range of Values			**Estimating E_s From N**[1]	
Soil Type	**Young's Modulus, E_s (MPa)**	**Poisson's Ratio, *ν***	**Soil Type**	**E_s (MPa)**
Clay:				
Soft sensitive	2-15	0.4-0.5 (undrained)	Silts, sandy silts, slightly cohesive mixtures	0.4 N_{60} or 0.4 N_{corr}[2]
Medium stiff to stiff	15-50		Clean fine to medium sands and slightly silty sands	0.7 N_{60}
Very stiff	50-100		Coarse sands and sands with little gravel	N_{60}
Loess	15-60	0.1-0.3	Sandy gravel and gravels	1.17 N_1
Silt	2-20	0.3-0.35		
Fine Sand:			**Estimating E_s From s_u**[3]	
Loose	8-12		Soft sensitive clay	$400s_u$-$1{,}000s_u$
Medium dense	12-20	0.25	Medium stiff to stiff clay	$1{,}500s_u$-$2{,}400s_u$
Dense	20-30		Very stiff clay	$3{,}000s_u$-$4{,}000s_u$
Sand:				
Loose	10-30	0.2-0.35		
Medium dense	30-50			
Dense	50-80	0.3-0.4		
Gravel:				
Loose	30-80	0.2-0.35	**Estimating E_s From q_c**[4]	
Medium dense	80-100		Sandy soils	$4q_c$
Dense	100-200	0.3-0.4		

Notes
N = Standard Penetration Test (SPT) resistance in blows per 300 mm (12 inches).
N_{60} = SPT corrected to 60% energy. Correct N_1 further for negative pore pressures generated during SPT in very dense fine or silty sands below the water table using N_{corr}=15 + 0.50 (N_{60}-15). N_1 corrected for overburden.
s_u = Undrained shear strength =0.5 (σ_1-σ_3)
q_c = Cone penetration resistance.
mPa = .14 kips per square inch

In summary, immediate settlements for saturated cohesive soils should be small if the foundation is properly designed. For unsaturated and highly overconsolidated soils immediate settlement may be the predominate portion of the total settlement.

4.7.2 Primary Consolidation of Cohesive Soils

Primary and secondary consolidation of clays or clayey deposits may result in substantial settlements when the structure is founded on saturated or nearly saturated (>80%) cohesive soils. Settlement resulting from primary consolidation may take months or years to be completed. Furthermore, because soil properties may vary beneath the foundation, the duration of the primary consolidation and the amount of settlement may also vary with the location of the footing, resulting in differential settlement. If such settlements are not within tolerable limits the structure may be damaged.

When a foundation is wide compared to the compressible layer thickness beneath it, a large portion of the soil will settle vertically (one-dimensionally) with very little lateral displacement because of the constraining forces exerted by the neighboring soil elements. Based on measurements of lateral displacements beneath embankments and tanks, it is usually reasonable to assume (Terzaghi, and Peck, 1967) that the settlements beneath a foundation will be one dimensional even for narrow (compared to the soil thickness) foundations, provided an adequate factor of safety is applied to bearing capacity. A Desiccated crust will tend to minimize the amount of lateral displacements. Experience suggests that reasonably good estimates of settlement can be predicted assuming one-dimensional compression as long as the factor of safety is at least three (3) against bearing capacity failure. For embankments over soft compressible soils where the factor of safety for bearing capacity failure may be considerable less than three, the resulting consolidation will involve horizontal as well as vertical flow and horizontal as well as vertical strains. One-dimensional consolidation is still often used to estimate settlement for these three dimensional cases, however, corrections to the calculated settlement are typically required. One-dimensional consolidation theory will be presented in this section.

The one-dimensional consolidation test (ASTM D-2435) is commonly used to determine the compressibility of clays. Settlement due to consolidation can be estimated from the slope of the one-dimensional consolidation test void ratio (e = volume of void/volume of solids) versus the logarithm of the vertical effective stress (σ'_v) curve (Figure 4-22). This procedure is generally used in practice despite the fact that not all points beneath the embankment undergo one-dimensional compression.

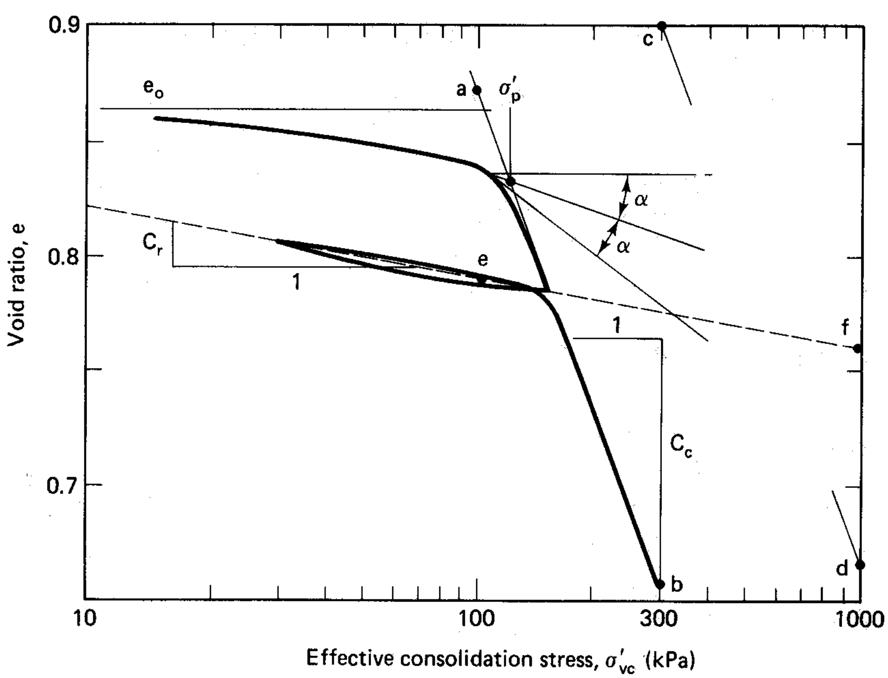


Figure 4-22: Typical Plot of Void Ratio Versus Log Effective Vertical Stress from a Consolidation Test on Clay (Holtz et al, 1981)

The slope of the one-dimensional consolidation test is typically nonlinear and it is convenient to use the logarithmic scale for stress. The slope of a-b line of the e-log σ'_v curve is defined as the compression index (C_c) (Figure 4-22). Soils subjected to stress-void ratio states corresponding to line a-b in Figure 4-22 are called normally consolidated soils. The highest level of stress to which normally consolidated soils are subjected is due to existing overburden loads. Soils in this state are compressible and may experience relatively large settlements when the effective stress is increased.

Numerous correlations have been made between C_c and common index tests for normally consolidated soils and several are included in Table 4-7 and Figure 4-23.

Note that the values of C_c can vary by as much as a factor of 5 (using the average trend line) in these empirical correlations, and they should not be used for final design.

The effective vertical stress at which the soil begins to undergo a substantial compressibility is called the preconsolidation pressure (σ'_p). This stress can be considered as equivalent to the onset of yield, where plastic strains develop. It also represents the loads that the soil had been subjected to in the past which has resulted in consolidation (over consolidation) of the soil stratum. The strains that develop at pressures below the preconsolidation pressure (to the left of σ'_p in Figure 4-22) are normally considered to result from minor slipping at the soil interparticle contacts. The magnitude of σ'_p is influenced by the largest stress the soil has been subjected to, and the strength of the bonding and cementation. The more bonding and cementation in the soil, the more abrupt the change in the slope of the void ratio-effective stress curve once the stress level exceeds σ'_p.

The slope of the e-log σ'_v curve below σ'_p is called the recompression index (C_r). The recompression index of the soil is significantly smaller than the compression index. The ratio of C_r / C_c typically ranges from 0.02 to 0.20 (Terzaghi, and Peck, 1967). The low value is typical of highly structured and bonded soft clay or silt, while the largest ratio corresponds to micaceous silts and fissured stiff clays and shales. In reality, the value of C_r depends on whether loading or unloading is occurring, since some hysteresis does occur when the soil is subjected to cycles of loading and unloading.

Generally, it is sufficiently accurate to assume C_r is constant for unstructured clays. It may not be adequate to rely on a single value of C_r for loading and unloading in the case of highly structured soft clays or stiff clay shales. In the case of highly structured soft clays (Terzaghi, and Peck, 1967) the initial value of C_r is steep as a result of flocculation (edge to face structure of clays) and bonding that allows the soil to be stable at high void ratios until the stress exceeds σ'_p. The subsequent rebound slope can be significantly different than the initial C_r.

TABLE 4-7
CORRELATIONS FOR C_c (After Holtz et al., 1981)

Correlation	Soil
C_c=0.156 e_0 + 0.0107	All Clays
C_c=0.30 (e_0 - 0.27)	Inorganic, silt, silty clay
C_c=0.009 (LL-10) (2)	Clay of Medium to low sensitivity (S<4)(1)
C_c=0.0115 w_n (3)	Organic Soils, Peat
C_c=0.75 ((e_0 - 0.50)	Low plasticity clays

(1) S = sensitivity =Undisturbed undrained shear strength/Remolded undrained shear strength
(2) LL = liquid limit (3) w_n = natural water content

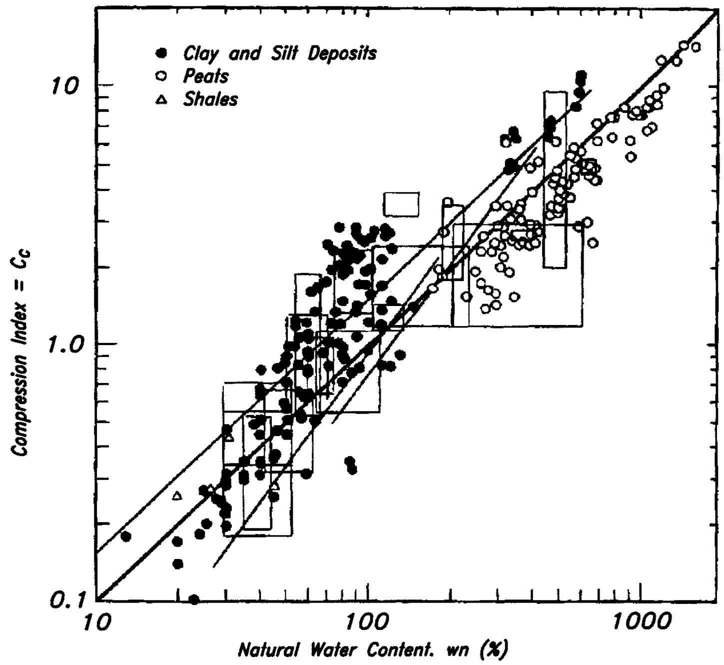


Figure 4-23: Empirical Correlation Between Compression Index and In Situ Water Content for Clay and Silt Deposits, Shales and for Peats (From Terzaghi, Peck, & Mesri 1996)

4.7.3 Correction of Laboratory One-Dimensional Consolidation Curves

The process of sampling soils (FHWA-NHI-01-031 "Subsurface Investigation") will cause some sampling disturbance no matter how carefully the samples are taken. This sampling disturbance will affect virtually all measured physical properties (compressibility, strength, permeability, etc.) of the soil. The sampling disturbance will usually cause the measured laboratory void ratio-effective stress "break" to occur at a lower apparent maximum past vertical pressure (σ'_p) (preconsolidation) than would be measured for an undisturbed specimen. The effect of disturbance from the sampling procedure is illustrated in Figure 4-24.

Figure 4-24 shows three consolidation curves for an insensitive clay from Fond du Lac, Wisconsin. The curve for the remolded sample is the flattest curve without a well defined break between reloading and virgin compression. The curve for a 50mm (2 inches) tube sample shows the more typical break in the curve. The curve for the 75mm (3 inches) tube sample trimmed to a 50mm (2 inches) sample shows a well defined break that is very similar to the estimated field compression curve.

It is still necessary to "correct" the e-log σ'_v curve of good quality samples since no sampling technique is perfect. There are several techniques available to correct the consolidation curve. The laboratory curve can be corrected according to Figures 4-25a and 4-25b for normally consolidated and overconsolidated soils, respectively. Table 4-8 presents the reconstruction procedures.

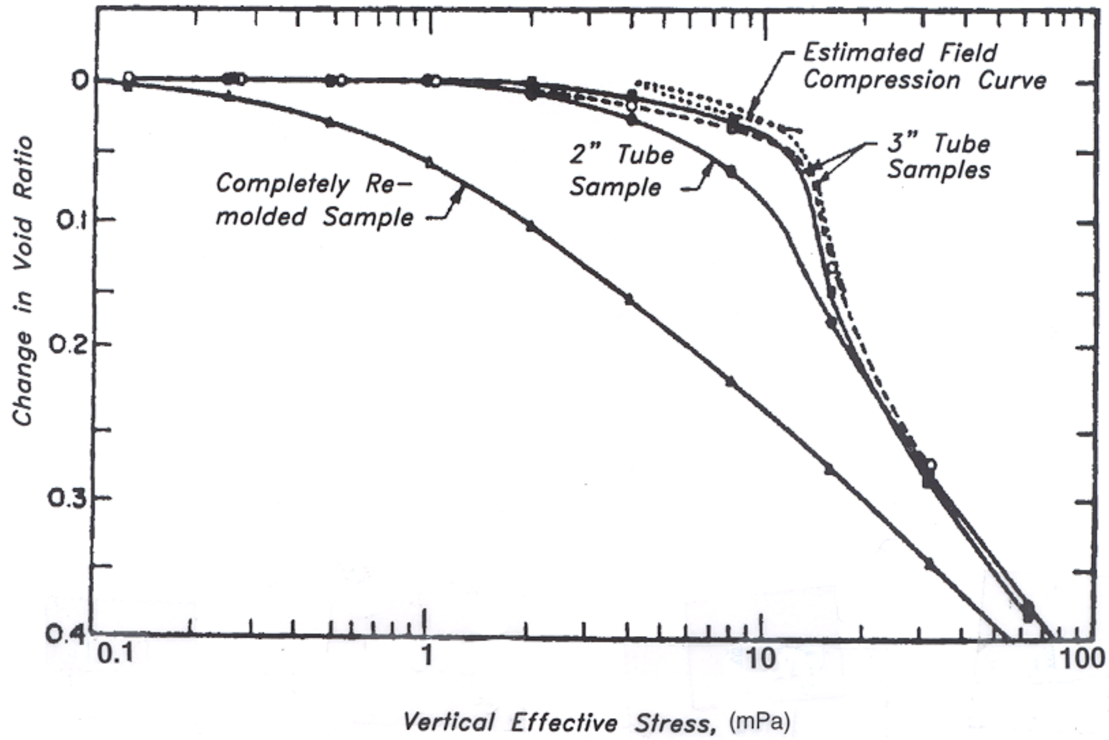
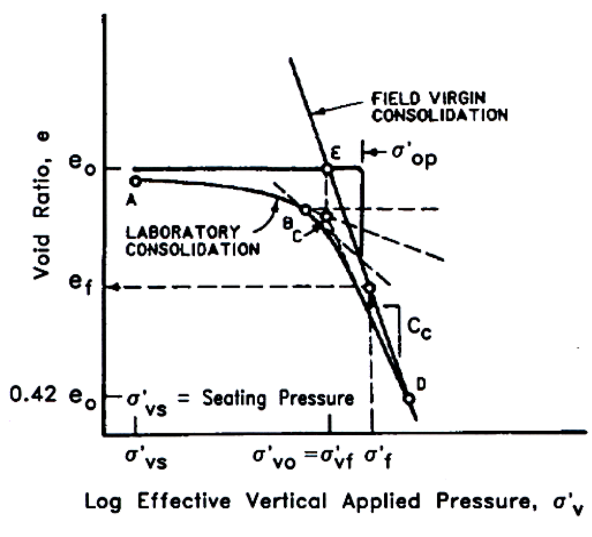


Figure 4-24: Effect of Disturbance on One-Dimensional Consolidation Void Ratio-Effective Stress Curve (Olson, 1995)



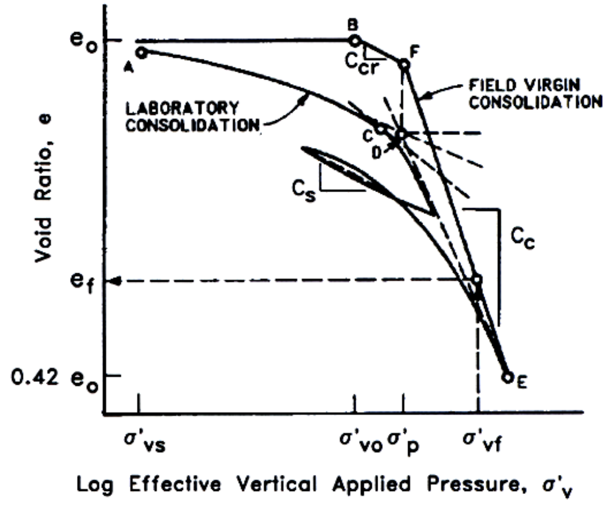


Figure 4-25: Construction of Field Virgin Consolidation Relationships (From USACE, 1994)

TABLE 4-8
RECONSTRUCTION OF VIRGIN FIELD CONSOLIDATION (from USACE, 1994)

Step	Description
	a. Normally Consolidated Soil (Figure 4-25a)
1	Plot point B at the point of maximum radius of curvature of the laboratory consolidation curve.
2	Plot point C by the Casagrande construction procedure: (1) Draw a horizontal line from B; (2) Draw a line tangent to the laboratory consolidation curve through B; and (3) Draw the bisector between horizontal and tangent lines. Point C is the intersection of the straight portion of the laboratory curve with the bisector. Point C indicates the maximum preconsolidation (past) pressure σ'_p.
3	Plot point E at the intersection e_o and σ'_p. e_o is given as the initial void ratio prior to testing in the consolidometer and σ'_p is found from step 2.
4	Plot point D at the intersection of the laboratory virgin consolidation curve with void ratio e = $0.42e_0$.
5	The field virgin consolidation curve is the straight line determined by points E and D.
	b. Overconsolidated Soil (Figure 4-25b)
1	Plot point B at the intersection of the given e_o and the initial estimated in situ effective overburden pressure σ'_{vo}.
2	Draw a line through B parallel to the mean slope C_r, of the rebound laboratory curve.
3	Plot point D using step 2 in Table 4-8a for normally consolidated soil.
4	Plot point F by extending a vertical line through D up through the intersection of the line of slope C_r, extending through B.
5	Plot point E at the intersection of the laboratory virgin consolidation curve with void ratio e = $0.42e_o$.
6	The field virgin consolidation curve is the straight line through points F and E.

4.7.4 Computation of Primary Consolidation Settlements

The settlement of a foundation resting on n layers of normally consolidated soils (σ'_p=σ'_{vo}) can be computed from (Figure 4-26a):

$$S_c = \sum_i^n \frac{C_c}{1+e_o} H_o \log_{10} \frac{\sigma'_{vf}}{\sigma'_{vo}} \tag{4-17}$$

where:

- C_c = compression index of the normally consolidated portion of the void ratio versus log σ'_v curve (Figure 4-25a)
- e_o = initial void ratio
- H_o = layer thickness
- σ'_{vo} = initial effective vertical stress at the center of layer n
- σ'_{vf} = final effective vertical stress at the center of layer n.

The final effective vertical stress is computed by adding the stress change due to the embankment load to the initial vertical effective stress. The total settlement will be the sum of the compression of the n layers of soil.

Normally the slope of the virgin portion of the e-log σ'_v curve is determined from the corrected one-dimensional consolidation curve measured on specimens taken from each relevant soil in the stratigraphic column. Common correlations for estimating C_c were presented in Table 4-7 and can be used to check laboratory results.

Sometimes the consolidation data is presented in terms of vertical strain (ε_v) instead of void ratio (Figure 4-26b). In this case the slope of the virgin portion of the ε_v versus log σ_v curve is denoted as $C_{c\varepsilon}$ and the settlement is computed using Equation 4-18 for normally consolidated soils.

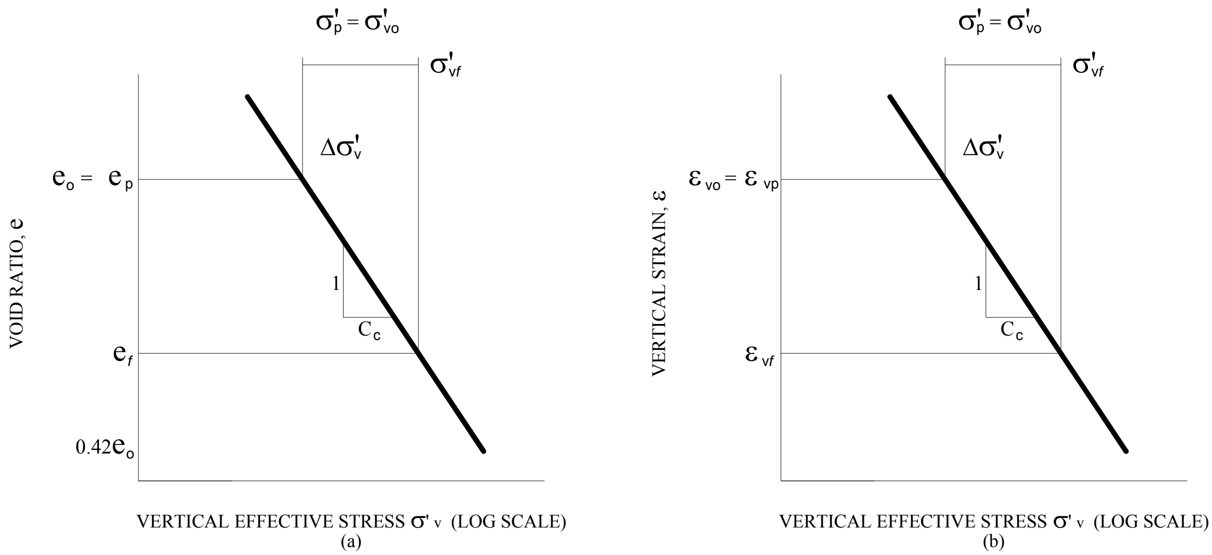


Figure 4-26: Typical Consolidation Compression Curve for Normally Consolidated Soil Void Ratio & Vertical Strain Versus Vertical Effective Stress (FHWA-NHI-01-023 "Shallow Foundations")

$$S_c = \sum_1^n H_o C_{c\varepsilon} \log_{10}(\sigma'_{vf} / \sigma'_{vo}) \tag{4-18}$$

If the water content of a clay layer below the water table is closer to the plastic limit than the liquid limit the soil is almost certainly overconsolidated. As a result, the field state of stress will reside on the initially flat portion of the e versus log σ'_v curve. The settlements for the case of n layers of over-consolidated soils (OCR>1) will be computed from Equation 4-19 (Figure 4-27a) or Equation 4-20 (Figure 4-27b).

$$S = \sum_1^n \frac{H_o}{1+e_o}\left(C_{cr}\log_{10}\frac{\sigma'_p}{\sigma'_{vo}} + C_c \log_{10}\frac{\sigma'_{vf}}{\sigma'_p}\right) \tag{4-19}$$

$$S = \sum_1^n H_o \left(C_{r\varepsilon}\log_{10}\frac{\sigma'_p}{\sigma'_{vo}} + C_{c\varepsilon} \log_{10}\frac{\sigma'_{vf}}{\sigma'_p}\right) \tag{4-20}$$

The total settlement is computed by subdividing each compressible layer within the zone of influence (Z_I) into a sufficient number of sublayers such that the initial and final stress calculated at the center of each layer is representative of the average stress for the layer, and the material properties are reasonably constant within the layer. The layers are typically 1.5 m to 3m (5 to 10 feet) thick in highway bridge applications. In cases where the various stratigraphic layers represent combinations of both normally and overconsolidated soils, the settlement is computed using the appropriate combinations of Equations 4-17 through 4-20.

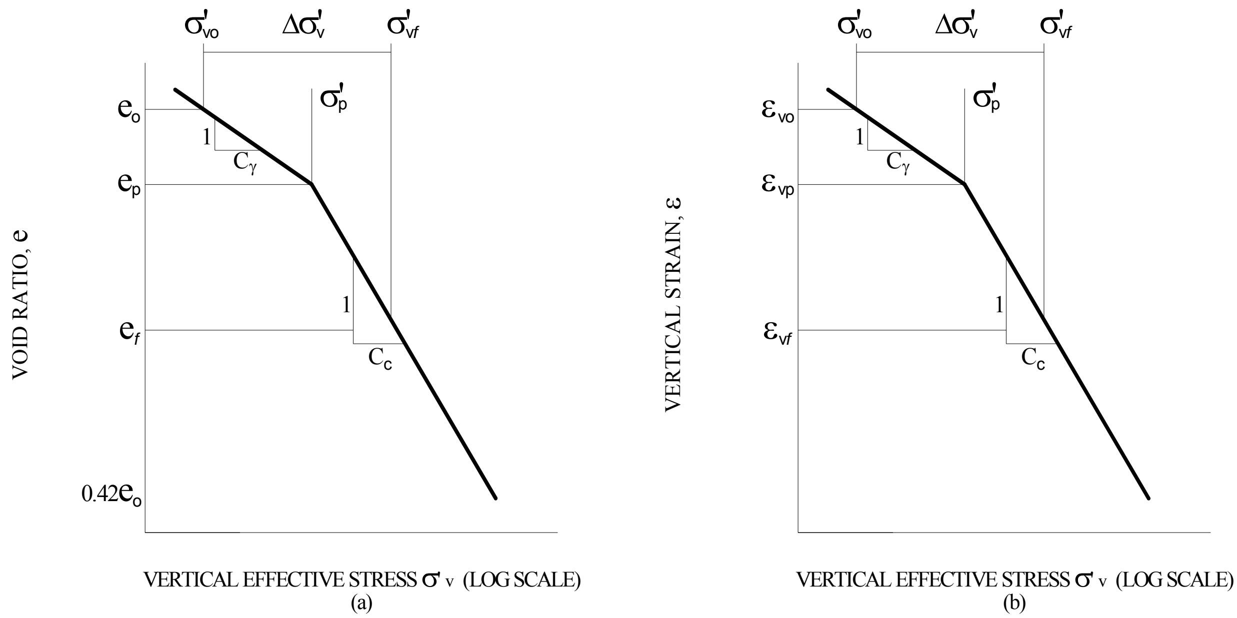


Figure 4-27: Typical Consolidation Curve for Over Consolidated Soil Void Ratio & Vertical Strain versus Vertical Effective Stress (FHWA-NHI-01-023 "Shallow Foundations")

4.7.5 Consolidation Rates

The rate of consolidation should be considered for the design of embankments on soft clay. The embankment will settle relative to a bridge foundation supported on piles, creating an undesirable bump at the end of the bridge. Hence, time rate as well as differential settlement (Section 4.10) between the bridge and embankment are extremely important.

The initial (static) pore water pressure distribution (u_s) is assumed to be linear in a layer of saturated clay as illustrated in Figure 4-28. Experimental measurement of pore pressures in saturated clays subjected to one-dimensional loading indicate that when a load is applied instantaneously the pore water pressure will increase an amount equal to the change in total vertical stress ($\Delta\sigma_v$). The increase in pore water pressure above the static value is called the excess pore water pressure (u'). If the clay layer is confined between two permeable sand layers the excess pore water pressure will immediately drop to zero at the drainage boundaries. As time progresses, the distribution of the excess pore water pressure will decrease and the isochrones (vertical distribution of excess pore water pressure at any constant time) will evolve from the C_1 distribution, to the C_2 distribution, and finally to the initial distribution of the static water pressure.

The average degree of consolidation ($U = S_{(t)}/S_{(ultimate)}$) at any time can be defined as the ratio of the settlement at time t ($S_{(t)}$) to the settlement at the end of primary consolidation ($S_{(ultimate)}$), when excess pore water pressures are zero. The average degree of consolidation (U) can be represented versus a normalized time called the time factor, T_v, (see Table 4-9):

$$T_v = c_v\, t/(H_d^2) \tag{4-21}$$

where: c_v = coefficient of consolidation (cm/sec)
H_d = the distance to the drainage boundary (cm)
t = the time (sec).

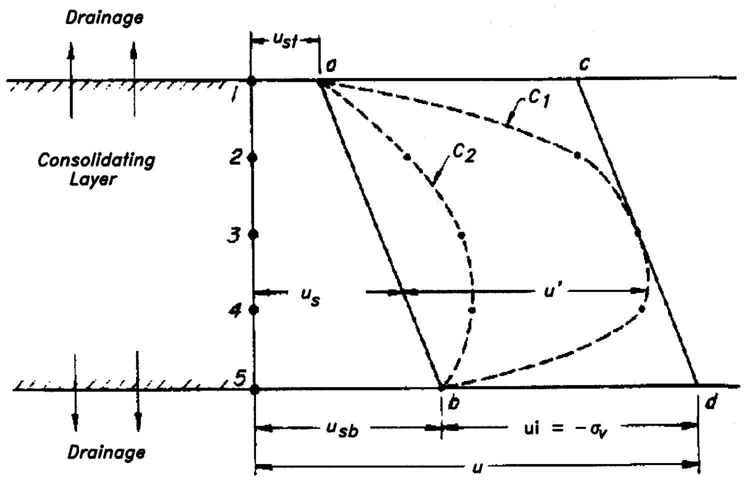


Figure 4-28: Diagram Illustrating Consolidation of a Layer of Clay Between Two Previous Layers (From Terzaghi, Peck and Mesri, 1996)

The drainage distance of a soil layer confined by permeable sand layers on both ends is equal to one-half of the layer thickness. When confined by a sand layer on one side and an impermeable boundary on the other side the drainage distance is equal to the layer thickness. The dimensionless time factor as a function of the degree of consolidation may be determined from Table 4-9.

TABLE 4-9
DEGREE OF CONSOLIDATION VERSUS TIME FACTOR FOR UNIFORM INITIAL INCREASE IN PORE WATER PRESSURE

T_v	U %
0.02	16.0
0.1	35.7
0.2	50.4
0.3	61.3
0.4	69.8
0.5	76.4
0.6	81.6
0.8	88.7
1	93.1
1.5	98.0
2	99.4

The step-by-step process for determining the amount and time for consolidation to occur for a single-stage construction of embankments on soft ground is outlined below:

1. From laboratory consolidation test determine the e log σ curve and estimate the change in void ratio that results from the added weight of the embankment. Create the virgin field consolidation curve using the guidelines presented in Table 4-8.
2. Determine if the foundation soil is normally consolidated or overconsolidated.
3. Compute the primary consolidation settlement using either equation 4-17 or 4-18 for normally or over consolidated foundation
4. Determine c_v from laboratory consolidation test.
5. Calculate the time to achieve 90% - 95% consolidation using equation 4-21.

For a more detailed discussion of consolidation theory the readers are referred to FHWA-NHI-01-023 "Shallow Foundations".

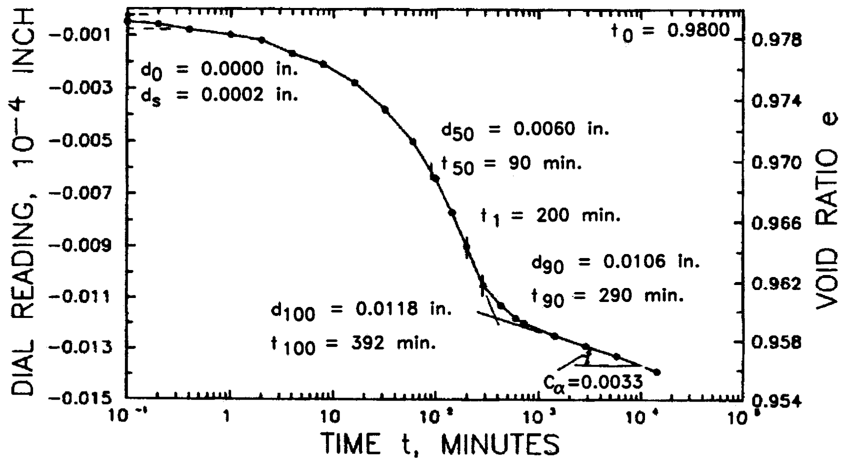
An alternative approach to hand calculations is using a computerized approach to calculates the time rate settlement for various boundary conditions including the effects of stage construction and strip drains. It allows for simulation of multiple layers undergoing simultaneous consolidation. In any event, the step-by-step hand calculations can serve to verify the correctness of benchmark computed cases thus ascertain the correctness of any computerized procedure.

4.7.6 Secondary Compression of Cohesive Soils

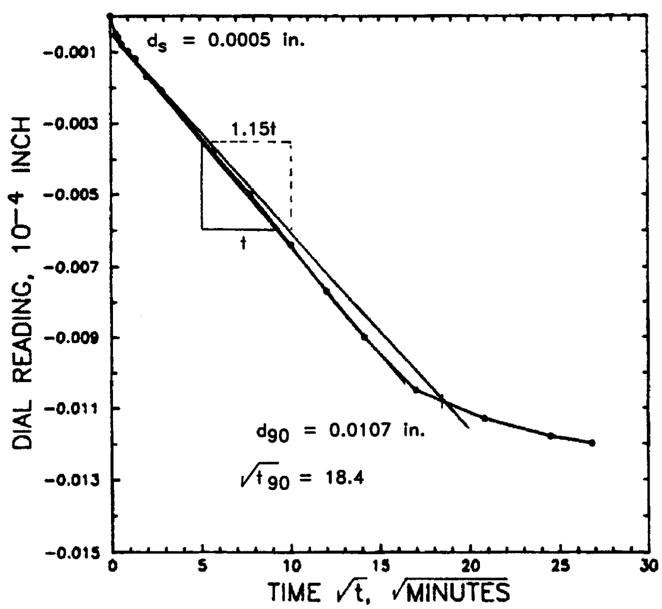
Secondary compression is the process where by the soil continues to vertically displace in spite of the fact that the excess pore pressures are dissipated to a negligible level (primary consolidation is essentially completed). Secondary compression is normally evident in the settlement-log time plot (Figure 4-29) when the specimen continues to consolidate beyond 100 percent of primary consolidation. There are numerous hypotheses (Olson, 1985) as to the reason for the secondary consolidation. The most obvious reason is associated with the simplifications involved in the theory of one-dimensional consolidation derived by Terzaghi. More rigorous numerical solutions accounting for the simplifications can often predict apparent secondary compression effects.

To assess the impact of secondary compression it may be more appropriate to rely on the behavior of similar structures on the same deposits rather than apparent properties developed from small-scale laboratory tests. However, it will typically be conservative to estimate the magnitude of secondary compression from the secondary coefficient of consolidation (C_α) as determined from laboratory tests using Equation 4-22 and Figure 4-29.

$$C_\alpha = \frac{\Delta e}{\log_{10}\left(\frac{t_{2\,lab}}{t_{1\,lab}}\right)} \tag{4-22}$$



a. VOID RATIO-LOGARITHM TIME



b. DISPLACEMENT-SQUARE ROOT TIME

Figure 4-29: Example Time Plots From One-Dimensional Consolidometer Test (1 in. = 25.4 mm) (USACE, 1994)

where:
t_1 = time when secondary consolidation begins and is typically taken as the time when 90 percent of primary consolidation has occurred
t_2 = an arbitrary time or the service life of the structure

The settlement due to secondary consolidation (S_s) is then determined from Equation 4-23.

$$S_s = \frac{C_\alpha}{1+e_0} H_c \log_{10}(t_2/t_1) \tag{4-23}$$

The values of C_α can be determined from the log time plots associated with the one-dimensional consolidation test. Typical ranges of the ratio of C_α/C_c are presented in Table 4-10.

TABLE 4-10
VALUES OF C_α/C_c FOR VARIOUS GEOTECHNICAL MATERIALS (Terzaghi, et al. 1996)

Material	C_α/C_c
Organic clays and silts	0.05 ± 0.01
Peat and muskeg	0.06 ± 0.01

4.8 SETTLEMENT IN COHESIONLESS SOIL

For cohesionless soils (sand, gravel and non-plastic silt), the pore pressure dissipation is relatively rapid thus the void ratio reduces instantly, so the time for settlement to occur in these soils is generally not an issue as the settlement is often complete at the end of construction. As the void ratio decreases, the internal friction angle increases, thus, the design of embankment foundations on cohesionless soils is often controlled by settlements since a distinct bearing capacity failure usually does not develop. Semi-empirical methods are the predominant techniques used to estimate settlements of embankments on cohesionless soils due to the difficulty of obtaining undisturbed samples and the erratic nature of such deposits. There are more than a dozen commonly used methods for estimating the settlement in cohesionless soils. FHWA-NHI-01-023 “Shallow Foundations” discusses the Terzaghi and Peck method, Peck and Bazarra method, Hough method, and Schmertman method based on the acceptance within general foundation engineering practice for highway structures. FHWA-NHI-01-023 “Shallow Foundations” also recommends that shallow foundations in sand should be evaluated in terms of settlement predicted by using all four methods, which in most cases, will bracket the anticipated settlement of the foundation.

Hough’s method is commonly used by the FHWA and may State Highway agencies as described in the FHWA “Soils and Foundations Workshop” Reference Manual (2000). The following section briefly discusses the Hough method. Readers are referred to FHWA-NHI-01-023 “Shallow Foundations” for detailed discussions of other methods.

4.8.1 Hough Method

Hough (1959) developed an empirical method for predicting settlements of shallow foundations on cohesionless soils which follows the same approach as that used for calculating consolidation settlement of clay layers. The soil is divided into layers, and the change in effective vertical stress at the mid-height of the layer as a result of the applied load is estimated using elastic theory. The method is applicable only for normally consolidated sands. The total settlement S_t is calculated as follows:

$$S_t = \sum (1/C') \, \Delta z \log((\sigma'_{vo} + \Delta\sigma'_v)/\sigma'_{vo}) \qquad (4\text{-}24)$$

where:

C' = Bearing Capacity Index = $(1 + e_o)/C_c$ (Figure 4–30)
e_o = initial void ratio
C_c = virgin compression index
Δz = layer thickness
σ'_{vo} = initial effective overburden pressure at mid-height of layer
$\Delta\sigma'_v$ = change in effective vertical stress at mid-height of layer

Note that SPT N blowcounts used in Figure 4-30 to obtain C' was defined as blowcounts corrected only for the overburden pressure effect. The overburden correction factor can be obtained from empirical correlations such as Figure 4-31. For a more detailed presentation of the Hough method and other methods to estimate the settlement of embankment foundations on cohesionless soil the reader is referred to FHWA-NHI-01-023 "Shallow Foundations."

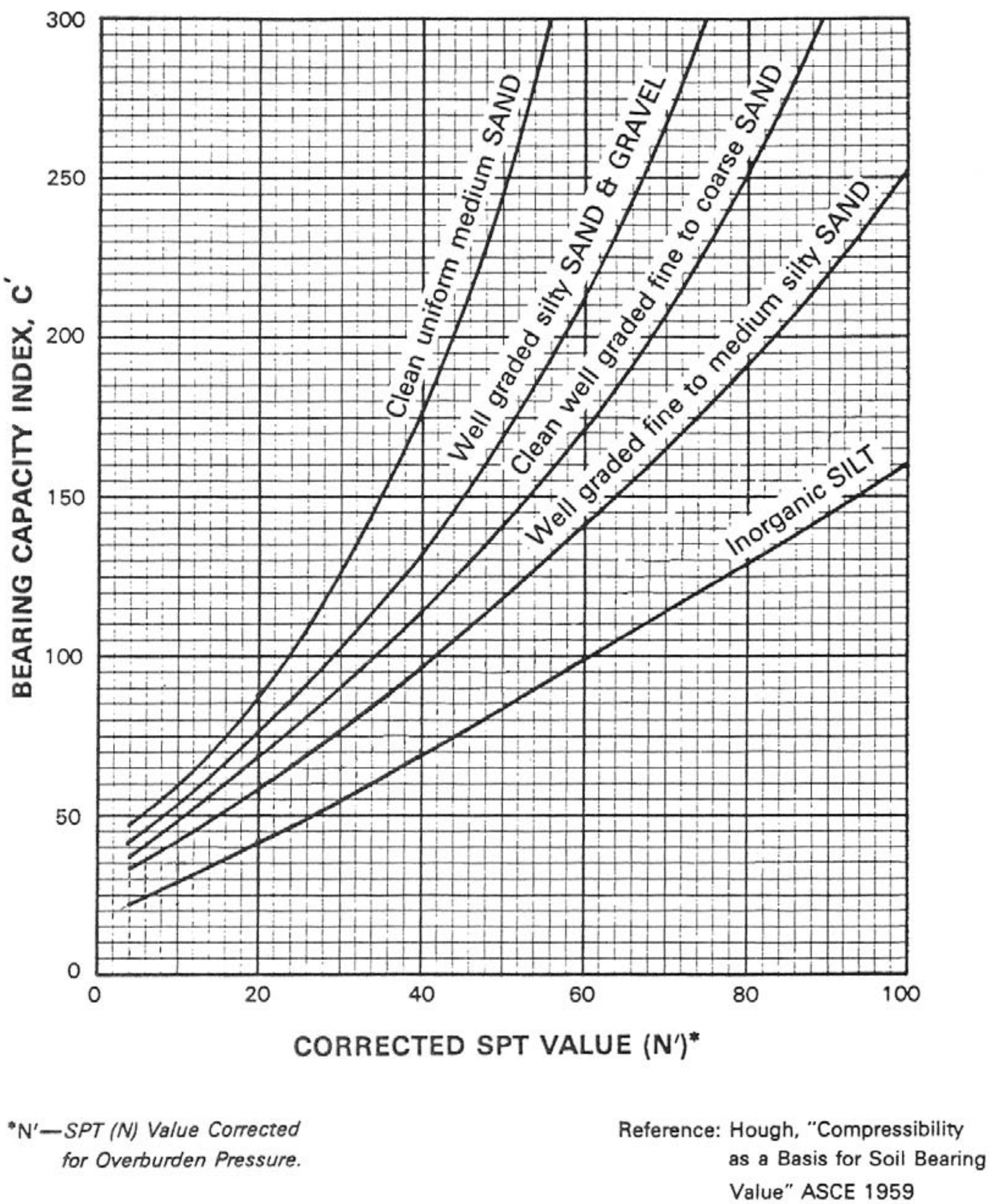


Figure 4-30: Bearing Capacity Index Verses Corrected SPT (Modified after Hough 1959)

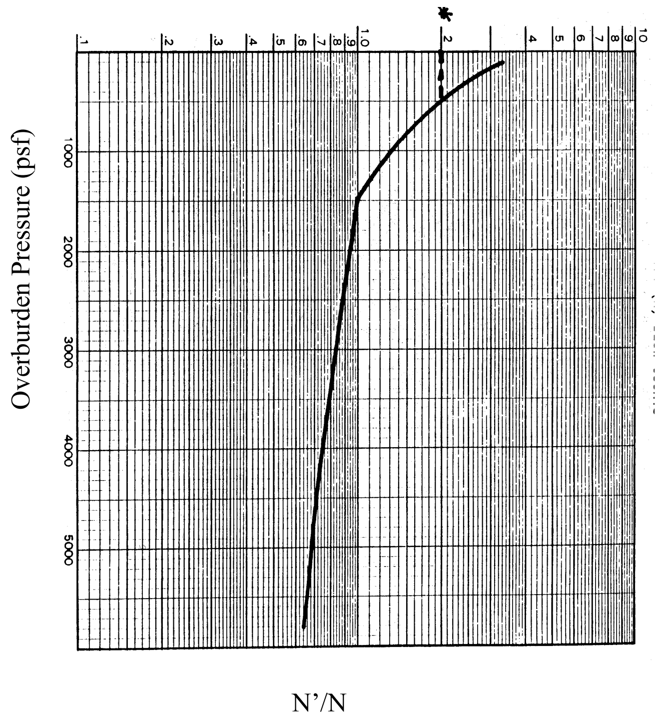


Figure 4-31: Corrected SPT (N') verses Overburden Pressure (Bazaraa (1967)

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CHAPTER 5
SLOPE STABILITY ANALYSIS

5.1 INTRODUCTION

Slope stability analysis must be able to determine whether the given or proposed slope meets the safety and performance criteria at the design stage; review the stability condition of an already constructed slope, and evaluate the influence of a stabilizing procedure that may prove necessary. The methods of analysis range from simpler two-dimensional force/moment equilibrium methods to sophisticated three-dimensional numerical analysis. Once the slope geometry and subsoil conditions have been determined, the stability of a slope may be assessed using either published chart solutions, hand calculations (rarely performed nowadays) or a computer analysis with appropriate drainage conditions and shear strengths. Most of the computer programs used for slope stability analysis are based on limit equilibrium approach for a two-dimensional (2D) model with some also allowing three-dimensional (3D) analysis (see Section 5.14 and Chapter 6 for discussions and comparisons of 2D and 3D analyses).

Other, more complex programs that use the finite element, finite difference or boundary element methods are also available and allow the engineer to perform a refined, two or three-dimensional slope evaluation. However, such analyses require a relatively complete model of the subsoils and their constitutive parameters determined by an extensive program of laboratory tests. Concerns about the laboratory testing, sample disturbance, lack of familiarization with the methodology, and the extensive computing required for each analysis generally have restricted the use of the these methods to only a few special cases for highway slopes. However, most stability analyses methods require information about the shear strength, but do not require information about its stress-strain behavior. They provide no information about the magnitude of movements of the slope. Therefore, in the case that deformation of a slope is of the most interest, numerical analysis is required.

This chapter reviews the definition of factor of safety, the mechanics of the limit equilibrium approach, the classical closed form solutions as well as the popular method of slices. The derivation of several procedures that use the method of slices is presented along with useful design charts for single material slopes.

5.2 MAIN INPUT PARAMETERS FOR SLOPE STABILITY ANALYSIS

There are many methods of analysis available to calculate the factor of safety against local and global slope instability ranging from simpler two-dimensional force/moment equilibrium methods to sophisticated three-dimensional numerical analysis. This Chapter focuses more on the conventional limit equilibrium methods which assume that a soil slope as a free body which likely to move (fail) by sliding along a critical slip surface driven mostly by the force of gravity. The so called critical slip surface is commonly assumed as circular arc, logarithmic spiral arc, curve, single plane or multiple planes to simulated the possible sliding movement as shown in Figures 2-3 through 2-5 and discussed in Chapter 2.

Table 5-1 summarizes the geologic conditions that influence the shape and development of the potential slip failure surface. The planar failure surfaces usually are expected in slopes where a soil layer, or relict jointing with a relatively low strength strongly influences the shape of the failure surface. The translational type of failure occurs in shallow soils overlying relatively stronger materials, and circular failure surfaces usually occur in slopes consisting of homogenous materials. Chapters 2 and 3 also discuss the geological influences on slope stability in detail.

TABLE 5-1
GEOLOGIC FACTORS CONTROLLING SHAPE OF POTENTIAL FAILURE SURFACE

Geologic Conditions	Potential Failure Surface
Cohesionless soils; Residual or colluvial soils over shallow rock; Stiff fissured clays and marine shales within the upper, highly weathered zone	Translational with small depth/length ratio
Sliding block; Interbedded dipping soil; Faulted or slickensided material; Intact stiff to hard cohesive soil on steep slopes	Single planar surface
Weathered interbedded sedimentary rocks; Clay shales and stiff fissured clays; Stratified soils; Side-hill fills over colluvium.	Multiple planar surfaces
Thick residual and colluvial soil layers; Soft marine clays and shales; Soft to firm cohesive soils.	Circular or cylindrical shape

The following sections discuss the main items required to evaluate the stability of a slope including (1) soil profile (2) slope geometry, (3) shear strength of the soils, (4) pore pressures or seepage forces, and (5) loading and environmental conditions.

5.2.1 Soil Profile and Slope Geometry

Regardless what method or computer program is chosen for performing slope stability analysis, the first prerequisite for performing “reasonable“ slope stability analyses is to formulate the right problem, and to formulate it correctly.

To perform slope stability analysis, a representative soil profile must be first formulated after the field reconnaissance and investigation. The slope geometry may be known for existing, natural slopes or may be a design parameter for embankments and cut-slopes.

Figure 5-1 illustrates a design profile for analyzing a cohesive embankment over soft varved clay foundation.

As discussed in Chapter 2, there are many types of slope movements that can result in unsatisfactory performance or even catastrophic failure of a soil slope or embankment. But not all types of failure as discussed in Chapter 2 can be represented by a 2-D mathematical model to be solved by conventional limit equilibrium methods. Especially, as most soils are generally heterogenous, noncircular surfaces, a combination of planar and curved sections are often likely. However, the conventional slope stability analysis can often provide a good indication of instability. For example, retrogressive failures consisting of multiple curved surfaces can occur in layered soils, as shown in Figure 5-2. Such failures are typical where the first slip tends to oversteepen the slope, which then leads to additional failures. Conventional stability analysis must determine the potential for the initial instability of the slope.

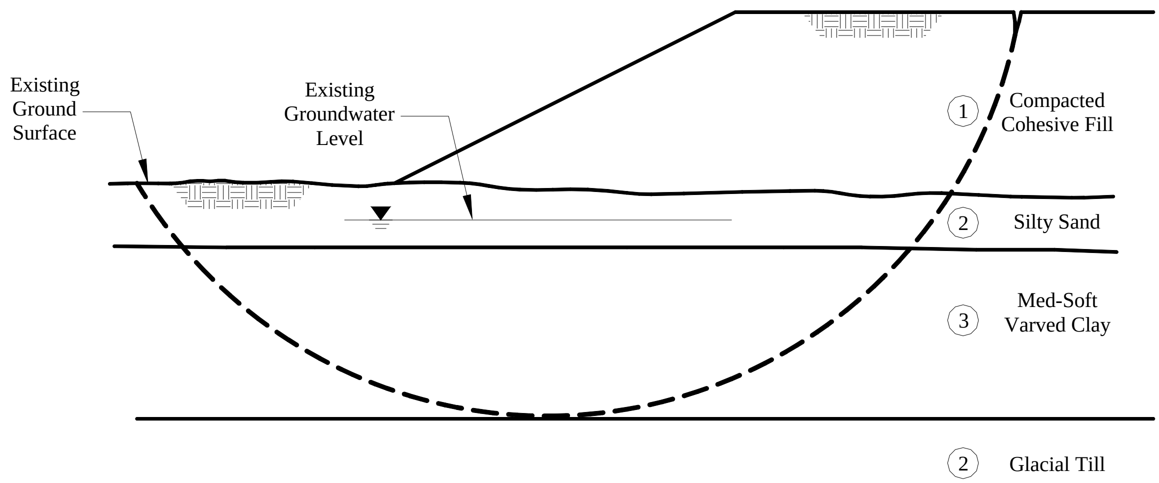


Figure 5-1: Typical Model for Slope Stability Analysis

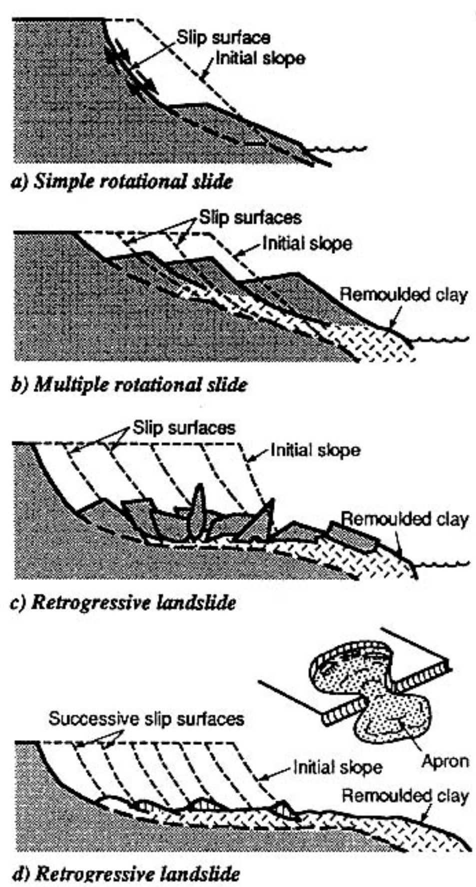


Figure 5-2: Typical Retrogressive Slides (Leroueil et al, 1996)

5.2.2 Shear Strength Selection (Total and Effective Stress Analyses)

As discussed in Chapter 4, the shear strength of the soil along the failure surface can be related to the vertical effective stress. However, the effective stress can only be calculated indirectly if the pore water pressures can be accurately estimated. That is the reason that, in some cases, it is simpler to analyze the stability of a soil slope using total stress analysis, in which internal pore pressure is not needed for analysis.

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
- Intermediate (for staged construction, or rapid drawdown condition)

Excess pore water pressures in cohesionless (free drainage) soils are expected to dissipate rapidly during construction. Thus, both the short-term and long-term stability of cohesionless soils should be in a drained condition and should be analyzed using *effective stress analyses* (ESA) principles with accurately (and conservatively) estimated drained shear strength (ϕ') and pore pressures (μ) (Section 5.2.3).

However, in cohesive (impermeable) soils, the dissipation of excess pore water pressures becomes a consolidation phenomenon, and may take a long time. Therefore, the drained condition is only applicable if the long-term stability of a natural or man-made soil slope is of concern as discussed in Section 4.6 and illustrated in Figure 4-12. In such cases the slope stability analysis may be performed using *effective stress analysis* approach with drained strength parameters (c', ϕ') with pore pressure (μ).

Theoretically, ESA approach is also applicable for undrained and partial drained conditions if pore pressure, especially during rapid loading, can be accurately estimated. However, a *total stress analysis* (TSA) may be more practical for these two types of drainage conditions if the pore water pressures are unknown or difficult to determine.

For analyzing stability of a cohesive soil in short-term and end–of-construction situations, since there is negligible change in water content, we can consider the cohesive soil is in a undrained condition, and its initial in-situ (unconsolidated) undrained shear strengths (Section 4.6.2) controls stability during design and construction since the factor of safety increases with time due to consolidation as illustrated in Figure 4-14. *Total stress analysis* may be used in this case with the strength parameters $\phi_u = 0$ and c_u (unconsolidated undrained (UU) strength). No pore pressure measurement is required for the TSA approach.

For a soil slope or embankment that is consolidated under one loading condition, and then subjected to a rapid change in loading with insufficient time for drainage, the cohesive soils are considered to be in an imtermediate (partially drained) condition. For this multi-stage loading condition, including staged construction for embankments on soft ground (Chapter 7), and rapid drawdown of water retention embankments, Ladd in 1991 suggested using the *total stress analysis* with the effective stress principal, and termed it *Undrained Strength Analysis* (USA). Basically, it is a *total stress analysis* using consolidated undrained (CU) strengths (Section 4.6.3) predicted based on the in-situ undrained strength ratio, and the in-situ and increased effective stresses. Readers are referred to Ladd 1991 for detailed discussions on USA.

Note that external water pressure for water retaining slopes and embankments have a stabilizing effect, and should be taken into account in both total stress and effective stress analyses of all types of slopes.

Duncan (1991) summarized the principals involved in selecting analysis conditions and shear strengths as shown in Table 5-2.

TABLE 5-2
SHEAR STRENGHTS, DRAINAGE CONDITION, PORE PRESSURE, AND UNIT WEIGHTS FOR SLOPE STABILITY ANALYSIS (after Duncan, 1992)

	Condition		
	Undrained/ End of Construction	Intermediate/ Multi-stage Loading	Drained/ Longterm
Analysis procedure and shear strength for free draining soils	Effective stress analysis, using c’ and ϕ’	Effective stress analysis, using c’ and ϕ’	Effective stress analysis, using c’ and ϕ’
Analysis procedure and shear strength for impermeable soils	Total stress analysis, using c and ϕ from in situ, UU, or CU tests	Total stress analysis, using C_u from CU tests and estimate of consolidation pressure	Effective stress analysis, using c’ and ϕ’
Internal pore pressures	No internal pore pressure for total stress analyses, set u equal to zero in computer input	No internal pores pressure for total stress analyses, set u equal to zero in computer input	μ from seepage analyses
	μ from seepage analysis for effective stress analyses	u from seepage analysis for effective stress analyses	
External water pressures	Include	Include	Include
Unit weights	Total	Total	Total

Note: Mulit-stage loading includes stage construction, rapid drawdown, and any other condition where a period of consolidation under one set of loads is followed by a change in load under undrained condition.

5.2.3 Pore Water Pressure

A major contributor to many slope failures is the change in effective stress caused by pore water pressures. These tend to alter the shear strength of the soil along the shear zone. Surface runoff that infiltrates cracks at the head of the failure surface may even cause destabilizing forces to develop. If an effective stress analysis is to be performed, pore water pressures will have to be estimated at relevant locations in the slope. Later sections will discuss the methods used to account for pore water pressure effects within the analytical procedures.

These pore pressures usually are estimated from groundwater conditions that may be specified by one of the following methods:

<u>Phreatic Surface</u>

This surface, or line in two dimensions, is defined by the free groundwater level. This surface may be delineated in the field by using open standpipes as monitoring wells. If a phreatic surface is horizontal, the pore water pressure at a depth can be simply calculated by multiplying the unit weight of ground water by the depth below the phreatic surface. However, if the phreatic surface is inclined, the pore water pressure to a depth for the steady state seepage condition can be calculated according to the sketch shown in Figure 5-3. Thus, if the inclination of the phreatic surface segment is θ, and the vertical distance between the base of the slice and the phreatic surface is h_w, the pore pressure is given by:

$$u = \gamma_w \, (h_w \cos^2\theta) \qquad (5\text{-}1)$$

Where:

γ_w = unit weight of water.

This concept is based on the assumption that *all* equipotential lines are straight and perpendicular to the segment of the phreatic surface passing through a slice-element in the slope. This is the easiest approach that can be readily programmed into a limit equilibrium slope stability program and the phreatic surface may be defined with minimal data.

Piezometric Data

Pore pressures are specified at discrete points within the slope, and an interpolation scheme is used to estimate the required pore water pressures at any location. The piezometric pressures may be determined from:

- Field piezometers
- A manually prepared flow net
- A numerical solution using finite differences or finite elements

This approach in only available in a few slope stability programs (Chapter 6). Discussion about the analytical methods for deriving flow net and pore pressure at discrete points are beyond the scope of the manual.

Although this approach is only available in a few slope stability programs, it is an excellent method for describing the pore water pressure distribution (Chugh, 1981b).

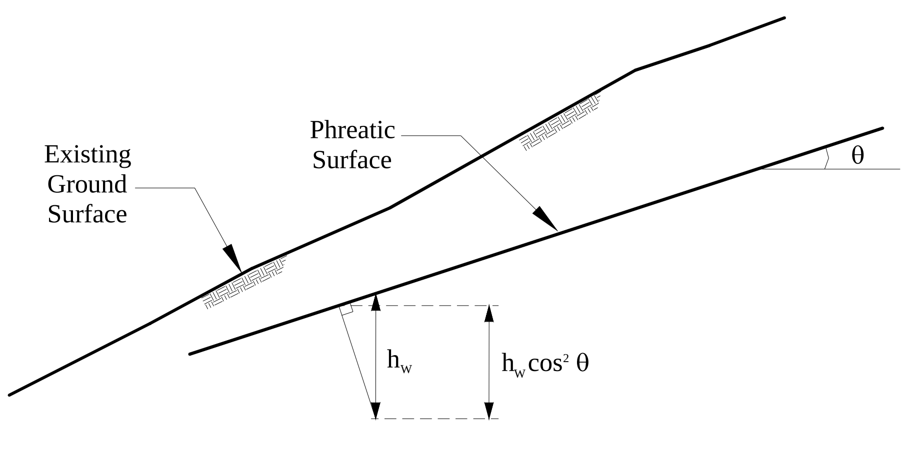


Figure 5-3: Calculation of Pore Water Pressure Head from Phreatic Surface

Pore Water Pressure Ratio

This is a method for normalizing pore water pressures measured in a slope according to the definition of:

$$r_u = \frac{u}{\sigma_v} \qquad (5\text{-}2)$$

Where:

u = pore pressure

σ_v = ***total*** vertical subsurface soil stress at depth z.

Effectively, the r_u value is the ratio between the pore pressure and the total vertical stress at the same depth. This ratio is easily implemented with computer analysis, but the major difficulty is associated with the assignment of the parameter to different parts of the slope. Often, the slope will require an extensive subdivision into many regions with different r_u values. This method, if used correctly, will permit a search for the most critical surface. However, it usually is reserved for estimating the FS of a slope or embankment from slope stability charts or for assessing the stability of a single surface, as presented in an example problem later.

Piezometric Surface

This defines the surface for the analysis of a unique, single failure surface. This approach often is used for the back-analysis of failed slopes. Because the combination of a piezometric and a failure surface is unique, a search for the critical surface is not possible. The reader should note that a piezometric surface is NOT the same as a phreatic surface, as the calculated pore water pressures will be different for the two cases.

Constant Pore Water Pressure

This approach may be used if the engineer wishes to specify a constant pore water pressure in any particular soil layer. This may be used to examine the stability of fills placed on soft-soils during construction where excess pore water pressures are generated according to the consolidation theory.

This is a reasonable assumption for a sloping straight-line phreatic surface, but will provide higher or lower estimates of pore water pressure for curved phreatic surfaces. For the steeply sloping phreatic surface, a convex-shaped phreatic surface (as shown in Figure 5-4) generates an overestimation of the pore water pressures, whereas a concave-shaped phreatic surface may lead to an underestimation. This feature also is shown in Figure 5-4. This overestimation is entirely because of the assumption of straight equipotential lines intersecting the projected phreatic surface line CD.

If the actual phreatic surface (line AB) is steeply curved, the equipotential lines must also curve as shown in Figure 5-4. In this example, a typical solution would use the greater pore water pressure head h_1 rather than h_2. However, this overestimation is small and only will affect a few slices within the sliding mass. Also, this conservatism is not expected to significantly affect the factor of safety.

In cases where the user is concerned about this problem, a pore water pressure grid may be used to simulate the pore water pressure distribution more accurately.

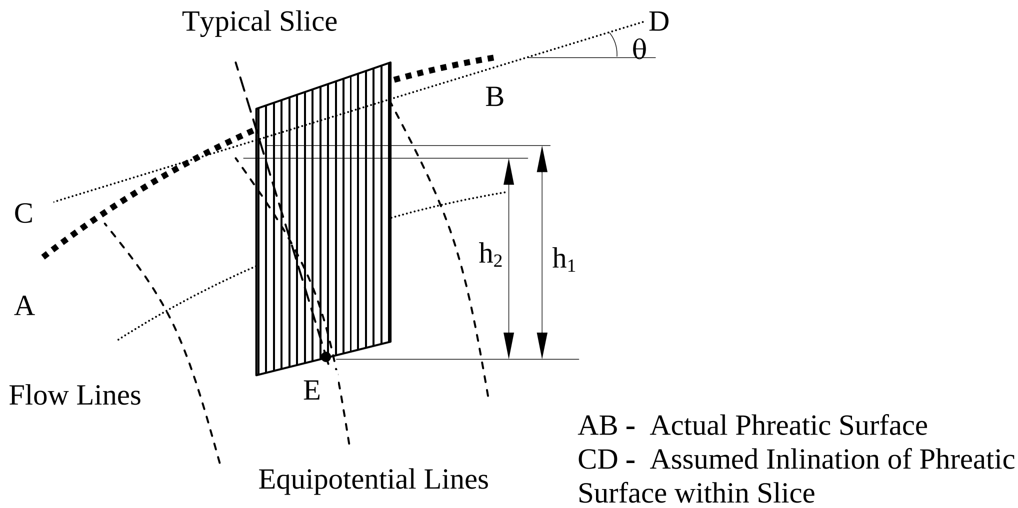


Figure 5-4: Calculation of Pore Water Pressure Head for Specified Piezometric Surface

5.3 FACTOR OF SAFETY CONCEPTS

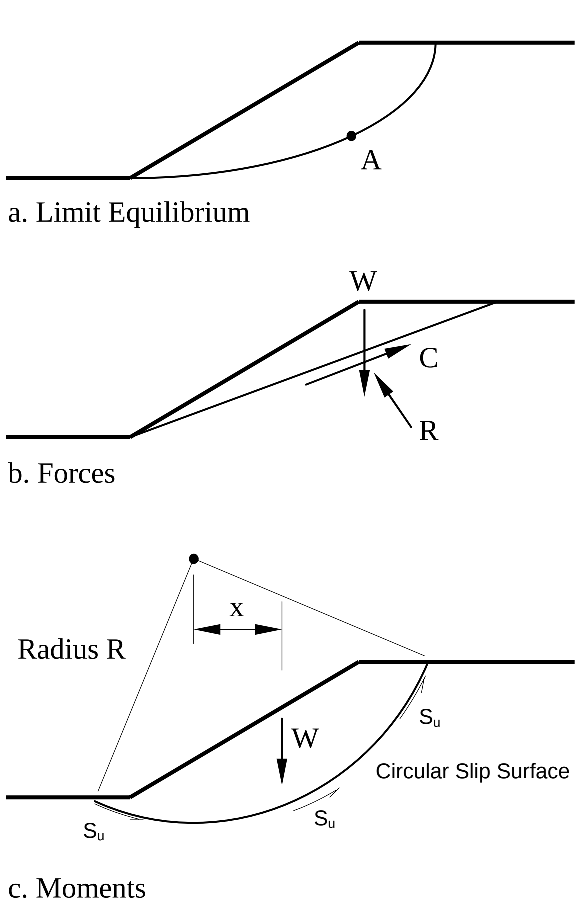
An understanding of the role of the factor of safety (FS) is vital in the rational design of slopes. One well-recognized approach is to account for uncertainty about the reliability of the items that enter into the analysis, (i.e., strength parameters, pore pressure distribution, and stratigraphy) with the factor of safety. In general, the lower the quality of the site investigation, the higher the desired FS should be, particularly if the designer has only limited experience with the materials in question. The FS also constitutes an empirical tool whereby deformation is limited to tolerable amounts, within economic restraints. In this way, the choice of the factor of safety is greatly influenced by the accumulated experience with a particular soil mass. Since the degree of risk that can be taken is also greatly influenced by experience, the actual magnitude of the factor of safety used in design will vary with material type and performance requirements. For example, when analyzing the stability of slope in the "marine clays" in the Washington, DC metropolitan area the residual friction angle (12° -14°) of the "marine clay" is typically used in the stability analysis. The design factor of safety for this case is typically 1.2 to 1.3 which is lower than the factor of safety that would be required if the peak friction angle was used in the analysis.

In most limit equilibrium analyses, the shear strength required along a potential failure surface to just maintain stability is calculated and then compared to the magnitude of available shear strength. In this case, the FS is assumed to be constant for the entire failure surface. For example, at point A in the upper slope shown in Figure 5-5, this average FS will be given by the ratio of available to required shear strength. Thus, a constant proportion of available strength is mobilized at every point on the failure surface to resist potential sliding.

If τ_{req} is the required shear strength, then,

$$\tau_{req} = \frac{s_u}{F} \qquad \text{for total stresses} \qquad (5\text{-}3)$$

$$\tau_{req} = \frac{c'}{F_c} + \frac{\sigma' \tan\phi}{F_\phi} \qquad \text{for effective stresses}$$



$$FS = \frac{s_u}{\tau_{required}} \quad \text{(Total Stress Analysis)}$$

$$FS = \frac{c' + \sigma' \tan\phi'}{\tau_{required}} \quad \text{(Effective Stress Analysis)}$$

$$FS = \frac{C_u}{\tau_{required}} \quad \text{(Undrained Stress Analysis)}$$

$$FS = \frac{\text{Summation of resisting forces}}{\text{Summation of mobilized forces}}$$

$$FS = \frac{\sum \text{Resisting moments}}{\sum \text{Overturning moments}} = \frac{R \int s_u \, ds}{W\,x}$$

c. Moments

Figure 5-5: Various Definitions of Factor of Safety (FS) (FHWA, 1994)

where s_u is the total stress strength, c' and ϕ' are effective stress strength parameters, F is the factor of safety for total stresses and F_c and F_ϕ are the factors of safety for effective stresses. The adoption of F_c and F_ϕ allows different proportions of the cohesive (c') and frictional (ϕ') components of strength to be mobilized along the failure surface. However, most limit equilibrium methods assume $F_c = F_\phi$, implying that the same proportion of the c' and ϕ' components are mobilized at the same time along the shear failure surface.

Another definition of safety factor often considered is the ratio of total resisting forces to total disturbing (or driving) forces for planar failure surfaces or the ratio of total resisting to disturbing moments as in the case for circular slip surfaces. However, one must realize that these different values of the factor of safety obtained using the three methods, that is, mobilized strength, ratio of forces, or ratio of moments, will not give identical values for $c – \phi$ soils.

As discussed above, in general practices, a soil slope is commonly analyzed based on the limit (or force or moment) equilibrium methods that measures its stability by a factor of safety (FS). Although mathematically a factor of safety greater than 1.0 indicates that the slope is stable, a slight overestimation of the available shear strengths, or underestimation of the actual driving stress, or a combination of both, can easily result in an actual factor of safety below the calculated value. Clearly, values of FS computed using equilibrium analyses are as reliable as the data that define the conditions analyzed. As a result, designs of soil slopes and embankments are usually based on an acceptable factor of safety, ranging between 1.3 and 1.5.

While establishing an acceptable factor of safety, a geotechnical engineer should consider:

1. The degree of uncertainty involved in evaluating the conditions and shear strengths for analysis.
2. The possible consequences of failure.

Minimum values of factor of safety used for various conditions need to be based on experience, considering the likely uncertainties involved in defining the conditions analyzed, and the possible consequences of failure. In summary, there are six major considerations for selection of the actual safety factor to be used on a particular project:

1. The degree of uncertainty in the shear strength measurements, slope geometry, and other conditions.
2. Confidence in reliability of subsurface stratification and data
3. The costs of flattening or lowering the slope to make it more stable
4. The costs and consequences of a slope failure
5. Either the slope is temporary or permanent
6. Stability analysis method used (Sections 5.4 through 5.12)

For highway slope designs, the required factors of safety (non-seismic) is usually in the 1.25 to 1.50 range. AASHTO recommends a minimum safety factor of 1.25 for embankment side slopes; 1.3 for embankment end slope; and 1.5 for cut slopes. Higher factors may be required if there is a high risk of loss of life or uncertainty regarding the pertinent design parameters. Typically the required factor of safety for the end of construction (1.3) is lower than the required long term factor of safety of 1.5. For the rapid drawdown case as shown in Figure 3-16 the safety factor is usually between 1.0 and 1.2. The factor of safety during seismic loading (short-term) (Section 5.13) is typically between 1.0 and 1.1.

Although the value of FS affords a useful index of the margin of stability for a slope, it does have several inherent weaknesses (Duncan, 1992). Abramson et al. (1995) summarized the following three weaknesses:

- Incipient failure is assumed at an overall FS equal to one, which is highly influenced by many variables associated with geological details, material parameters, pore water pressures, and so on.
- An assumption of constant FS along the entire slip surface is an oversimplication, especially if different soil materials exist along the failure surface.
- The stress-strain relationship of the soil is neglected, that is, stress-deformation increments and/or decrements within a slope are not simulated by consideration of static equilibrium by itself.

As shown in Figure 5-5, equilibrium analyses of slope stability implicitly assume that the stress-strain behavior of the soil is ductile (non-brittle) as shown as dashed line in Figure 3-8. This limitation results from the fact that the methods provide no information regarding the magnitudes of the strains within the slope, nor any indication about how they may vary along the slip surface. Write, et al. (1973), Tavenas, et. al. (1980) and others have noted that the factor of safety actually varies from place to place along the slip surface, whereas, in most equilibrium analyses the factor of safety is assumed to be constant. However, Duncan (1992) noted that the average value of FS is the same for all practical purposes, even if the factor of safety is assumed to vary from place to place along the slip surface (Chugh, 1986). The average value of FS is thus insensitive to the assumption that FS is the same for every slice.

5.4 INFINITE SLOPE ANALYSIS

A slope that extends for a relatively long distance and has a consistent subsoil profile may be analyzed as an infinite slope. The failure plane for this case is parallel to the surface of the slope and the limit equilibrium method can be applied readily.

5.4.1 Infinite Slopes in Dry Sand

A typical slice for a slope in *dry sand* is shown in Figure 5-6, along with its free body diagram. The weight of the slice (with a unit dimension into the page) is given by:

$$W=\gamma\ b\ h \tag{5-4}$$

The normal (N) and tangential (T) force components of W are determined:

$$N=W\cos\beta$$

and

$$T=W\sin\beta \tag{5-5}$$

The available frictional strength along the failure plane will depend on ϕ and is given by:

$$S=N\tan\phi \tag{5-6}$$

Then, if we consider the FS as the ratio of available strength to strength required to maintain stability (***limit equilibrium***), the FS will be given by:

$$FS=\frac{S}{T}\frac{N\tan\phi}{W\sin\beta}=\frac{\tan\phi}{\tan\beta} \tag{5-7}$$

The FS is independent of the slope height and depth, h, and depends only on the angle of internal friction, ϕ, and the angle of the slope, β. Also, at a FS = 1.0, the maximum slope angle will be limited to the angle of internal friction, ϕ.

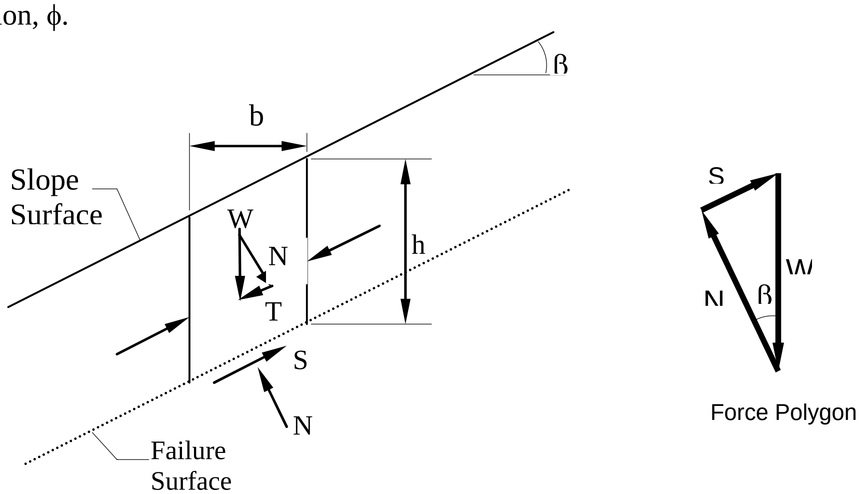


Figure 5-6: Infinite Slope Failure in Dry Sand

5.4.2 Infinite Slope in c-ϕ Soil with Parallel Seepage

If a saturated slope in cohesive (c-ϕ) soil has seepage parallel to the slope surface as shown in Figure 5-7, the same limit equilibrium concepts may be applied to determine the FS, which will now depend on the effective normal force (N'). From Figure 5-7, the pore water force acting on the base of the typical slice will be given by:

$$U = \left(\gamma_w h \cos^2\beta\right)\frac{b}{\cos\beta} = \gamma_w b h \cos\beta \tag{5-8}$$

The available frictional strength along the failure plane will depend on ϕ' and the effective normal force and is given by:

$$S = c'\frac{b}{\cos\beta} + (N - U)\tan\phi' \tag{5-9}$$

So the factor of safety for this case will be:

$$FS = \frac{(c' b/\cos\beta) + (N - U)\tan\phi'}{W \sin\beta} \tag{5-10}$$

By substituting $W = \gamma_{sat}\, b\, h$ into the above expression and rearranging, the FS will be given by:

$$FS = \frac{c' + h(\gamma_{sat} - \gamma_w)\cos^2(\beta)\tan\phi'}{\gamma_{sat}\, h \sin\beta\cos\beta} \tag{5-11}$$

Where:

$\gamma' = (\gamma_{sat} - \gamma_w)$. For a c = 0 soil, the above expression may be simplified to give:

$$FS = \frac{\gamma'}{\gamma_{sat}}\,\frac{\tan\phi'}{\tan\beta} \tag{5-12}$$

From equation 5-12 one can see that for a *granular* material, the FS is still independent of the slope height and depth, *h*, but is reduced by the factor γ'/γ_{sat}. For typical soils, this reduction will be about 50 percent in comparison to dry slopes.

The above analysis can be generalized if the seepage line is assumed to be located at a height of *mh* above the failure surface. In this case, the FS will be given by:

$$FS = \frac{c' + h\cos^2\beta\,[(1 - m)\gamma_m + m\gamma']\tan\phi'}{h\sin\beta\cos\beta\,[(1 - m)\gamma_m + m\gamma_{sat}]} \tag{5-13}$$

and γ_{sat} and γ_m are the saturated and moist unit weights of the soil below and above the seepage line. The above equation may be readily reformulated to determine the critical depth of the failure surface for any seepage condition and a *c*-ϕ soil.

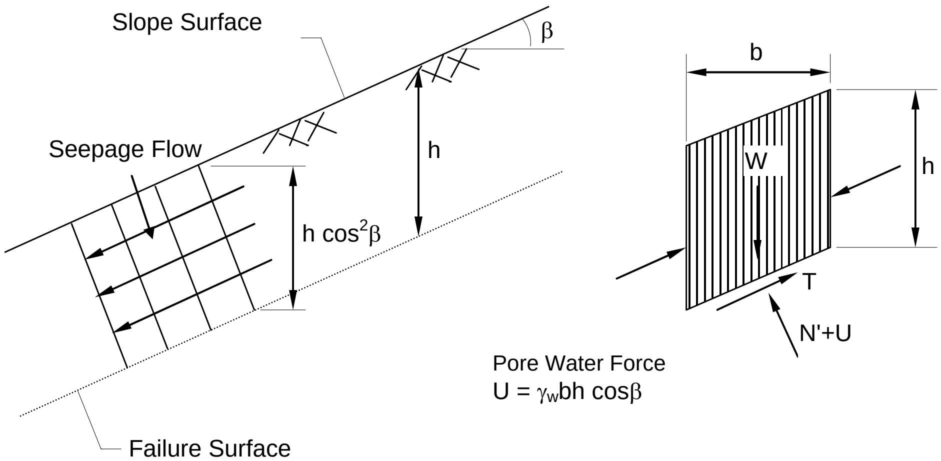


Figure 5-7: Infinite Slope Failure in c-ϕ Soil with Parallel Seepage

5.5 PLANAR SURFACE ANALYSIS (FINITE HEIGHT SLOPE)

Planar failure surfaces usually occur in slopes with a thin layer of soil that has relatively low strength in comparison to the overlying materials. Also, this is the preferred mode of failure for jointed materials that may dip toward proposed excavations.
A planar failure surface produces a statically determinate problem and thus can be readily analyzed with a closed form solution that depends on the slope geometry and the shear strength parameters of the soil along the failure plane. For the slope shown in Figure 5-8, three forces: weight (W), mobilized shear strength (S_m), and the normal reaction (N) need to be determined in order to evaluate the stability.

The weight of the wedge may be determined from the geometry using:

$$L = \frac{h}{\sin\beta} \times \frac{\sin(\beta - \alpha)}{\sin(\theta - \alpha)} \qquad (5\text{-}14)$$

$$W = \frac{1}{2}\gamma H^2 \left[\frac{\sin(\beta - \theta)}{\sin^2\beta} \times \frac{\sin(\beta - \alpha)}{\sin(\theta - \alpha)} \right]$$

The angle, α, in the above equation, is the inclination of the back-slope, with respect to the horizontal. The normal force (N) and the tangential force (T) component of the weight will be given by:

$$N = W\cos\theta \quad \text{and} \quad T = W\sin\theta = S_m \qquad (5\text{-}15)$$

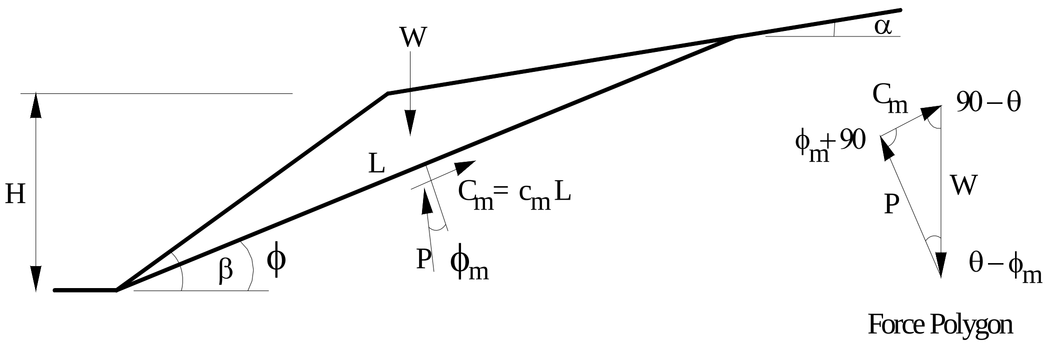


Figure 5-8: Planar (Block) Failure Surface

In addition, if the factors of safety with respect to cohesion (F_c) and friction (F_ϕ) are used such that the mobilized shear strength contributions are given by:

$$c_m = \frac{c}{F_c} \quad \text{and} \quad \tan \phi_m = \frac{\tan\phi}{F_\phi} \tag{5-16}$$

Then by equating the mobilized strength (S_m) calculated using the Mohr-Coulomb criterion and equation 5-17, with the tangential component of the soil weight the following relationship may be developed:

$$\begin{aligned}
W \sin\theta &= c_m L + W \cos\theta \tan\phi_m \\
c_m &= \frac{W}{L}\left[\sin\theta - \cos\theta \tan\phi_m\right] \\
&= \frac{\gamma H^2}{2L}\left[\frac{\sin(\beta - \theta)}{\sin\beta \sin\theta}[\sin\theta - \cos\theta \tan\phi_m]\right] \\
&= \frac{1}{2}\gamma H \left[\frac{\sin(\beta - \theta)[\sin\theta - \cos\theta \tan\phi_m]}{\sin\beta}\right]
\end{aligned} \tag{5-17}$$

Note that the inclination of the back-slope, α, is eliminated from the above equation and thus will not affect the calculations directly. This equation allows for the calculation of the magnitude of the "cohesive" resistance required to satisfy equilibrium. However, this would only provide the solution for a failure surface inclined at an angle θ.

To determine the critical slope, θ_{crit}, equation 5-18 must provide the *maximum* value of the mobilized cohesive resistance. With the assumption that γ, β and *H* are constant in equation 5-18, the first derivative is given by:

$$\frac{\partial}{\partial\theta}\ [\sin\ (\beta\ -\ \theta)\ (\sin\theta\ -\ \cos\theta\ \tan\theta_m)]$$

$$= - \cos (\beta - \theta) [\sin \theta - \cos \theta \tan \phi_m] + \sin (\beta - \theta) [\cos \theta + \sin \alpha \sin \theta] \tag{5-18}$$

$$= [\sin \alpha \cos \theta - \sin \theta\ \cos \alpha] + \tan \phi_m [\cos \alpha \cos \theta + \sin \alpha \sin \theta]$$

$$= \sin (\beta - 2\theta) + \tan \phi_m [\cos (\beta - 2\theta)]= 0$$

From the above equation, one can calculate:

$$\theta_{crit} = \frac{\beta + \phi_m}{2} \tag{5-19}$$

which allows the critical value of c_m to be calculated using:

$$c_m = \frac{1}{4} \gamma H \left[\frac{1 - \cos (\beta - \phi_m)}{\sin \beta \cos \phi_m} \right] \tag{5-20}$$

The above equation may also be manipulated to determine the critical height of a slope by substituting c_m = c and $\phi_m = \phi$ (FS = 1) to give:

$$H_{crit} = \frac{4\,c}{\gamma} \left[\frac{\sin \beta \cos \phi}{1 - \cos (\beta - \phi)} \right] \tag{5-21}$$

For the case of a $\phi = 0$ soil and a vertical slope (for example, $\beta = 90°$), the above equation gives a critical height of $4_c/\gamma$.

For the case of c=0, $\theta_{crit} = \phi_m$ and FS = $\tan \phi / \tan \beta$ (i.e., same as infinite dry cohesionless slope).

For a typical analysis, the procedure requires a trial and error solution for a c-ϕ soil such that the factors of safety with respect to the cohesion and friction are equal. This is typically accomplished using the following steps:

1. Assume a factor of safety against frictional resistance, F_ϕ,
2. Compute the ϕ_m value,
3. Calculate the mobilized cohesive value c_m, using equation 5-18,
4. Calculate the factor of safety, $F_c = c / c_m$
5. Repeat steps 1-4 until $F_\phi = F_c$.

It should be noted that in essence, this formulation was first presented by Culmann (1866).

5.6 LOG SPIRAL AND CIRCULAR SURFACE ANALYSIS

Log spiral or circular failure surfaces are found to be the most critical in slopes of homogenous materials. In fact, the trace of the critical log spiral and the circular arc surfaces are often nearly identical. The log spiral method as well as two methods of analysis, the ϕ = 0 and *friction circle*, are used to calculate the factor of safety for a slope in this section.

5.6.1 Log Spiral Method

The inherent property of the log spiral surface is that the radius vector at any point on its trace is inclined at $\phi_m = \tan^{-1}[\tan(\phi)/FS)$ to the normal at the same point. Hence, the resultant force of each elemental resultant force produced by the normal stress, σ, and its associated shear, $\sigma \tan(\phi_m)$, acts along this vector. Consequently, the moment equilibrium written about the pole of the log spiral is independent of σ thus enabling one to write an expression for FS without making any statical assumption. In a minimization procedure, one assumes the coordinate of the pole (center) and a constant of the log spiral. By trial and error, the respective FS can be calculated by solving the moment equilibrium equation. The absolute minimum FS is determined after repeated calculations for test bodies defined by all combinations of log spirals centers and constants. This minimization process is with respect to three parameters. In a sense, this process is analogous to locating the critical circle as the circular arc geometry is also defined by three parameters – center and radius. However, the log spiral computation is tedious by hand whereas it can be done rapidly in a computerized technique.

Since the log spiral approach does not require statical assumptions to find FS, it is considered accurate provided the kinematics of the failure mechanism is applicable. It should be pointed out that when ϕ = 0, the log spiral degenerate to a circular arc. In that case its FS is identical to most other circular arc methods. Note that for this special case, however, the problem is statically determinate for a circle as the moment equilibrium about the center is independent of the normal stress over it (e.g., see equation 5-22).

5.6.2 Circular Arc (ϕ = 0) Method

The simplest circular analysis is based on the assumption that a rigid, cylindrical block will fail by rotation about its center and that the shear strength along the failure surface is defined by the undrained strength. As the undrained strength is used, the angle of internal friction, ϕ, is assumed to be zero (hence the ϕ=0 method). The factor of safety for such a slope (Figure 5-9) may be analyzed by taking the ratio of the resisting and overturning moments about the center of the circular surface.

If the overturning and resisting moments are given by Wx and c_uLR, respectively, the factor of safety for the slope may be given by:

$$FS = \frac{c_u L R}{W x} \tag{5-22}$$

where c_u = undrained shear strength, L = arc length of failure surface, R = radius of circular surface, W = weight of sliding mass per unit length of slope, and x = horizontal distance between circle center, O, and the center of the sliding mass. If the undrained shear strength varies along the failure surface, the c_uL term must be modified and treated as a variable in the above formulation.

5.6.3 Friction Circle Method

This method is useful for homogeneous soils with $\phi > 0$, such that the shear strength depends on the normal stress. In other words, it may be used when cohesive and frictional components for shear strength have to be considered in the calculations. The method is equally suitable for total or effective stress types of analysis in homogeneous soils.

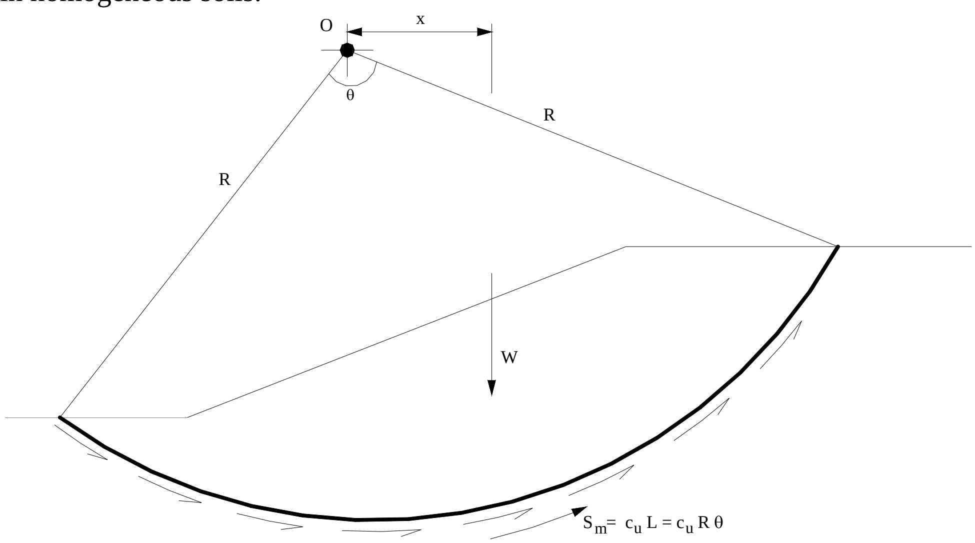


Figure 5-9: Circular Failure Surface in a ϕ= 0 Soil

The method attempts to satisfy complete equilibrium by assuming the direction of the resultant of the normal and frictional component of strength mobilized along the failure surface. This direction corresponds to a line that forms a tangent to the *friction circle* with a radius, $R_f = R \sin \phi_m$. This is equivalent to assuming that the resultant of all normal stresses acting on the failure surface is concentrated at one point. The cohesive shear stresses along the base of the failure surface ("ab" in Figure 5-10) will have a resultant (S_m) that acts parallel to the direction of the chord "ab". Its location may be found by taking moments about the circle center. This line of action of resultant, S_m can be located using:

$$R_c = \frac{L_{arc}}{L_{chord}} \times R \tag{5-23}$$

Where:

R = the radius of failure circle
R_c = the perpendicular distance from the circle center to force, S_m.
L_{arc} and L_{chord} = the lengths of the circular arc and chord defining the failure surface.

The actual point of application, A, is located at the intersection of the *effective* weight force, which is the resultant of the weight and any pore water forces. The resultant of the normal and frictional (shear) force, P, will then be inclined parallel to a line formed by a point of tangency to the friction circle and point A. As the direction of S_m is known, the force polygon can be closed to obtain the value of the mobilized cohesive force. Again, the true factor of safety is achieved for $F_\phi = F_c = FS$.

The solution procedure is usually followed graphically, although there are a few numerical schemes that have been established to generate the answer directly. For general ease of use, the following modified procedure is recommended:

1. Calculate weight of slide, W; per unit length of slope
2. Calculate magnitude and direction of the resultant pore water force, U (may need to discretize slide into slices); per unit length of slope
3. Calculate perpendicular distance (R_c) to the line of action of S_m;
4. Find effective weight resultant, W′, from forces W and U, and its intersection with the line of action of S_m at A;
5. Assume a value of F_ϕ;
6. Calculate the mobilized friction angle, $\phi_m = \tan^{-1}(\tan \phi / F_\phi)$;
7. Draw the friction circle, with radius $R_f = R \sin \phi_m$;
8. Draw the force polygon with W' appropriately inclined, and passing through point *A*;
9. Draw the direction of P, tangential to the friction circle;
10. Draw direction of S_m, according to the inclination of the chord linking the end-points of the circular failure surface;
11. The closed polygon will then provide the value of S_m;
12. Using this value of S_m, calculate $F_c = cL_{chord} / S_m$;
13. Repeat steps 5 through 13 until $F_c \approx F_\phi$.

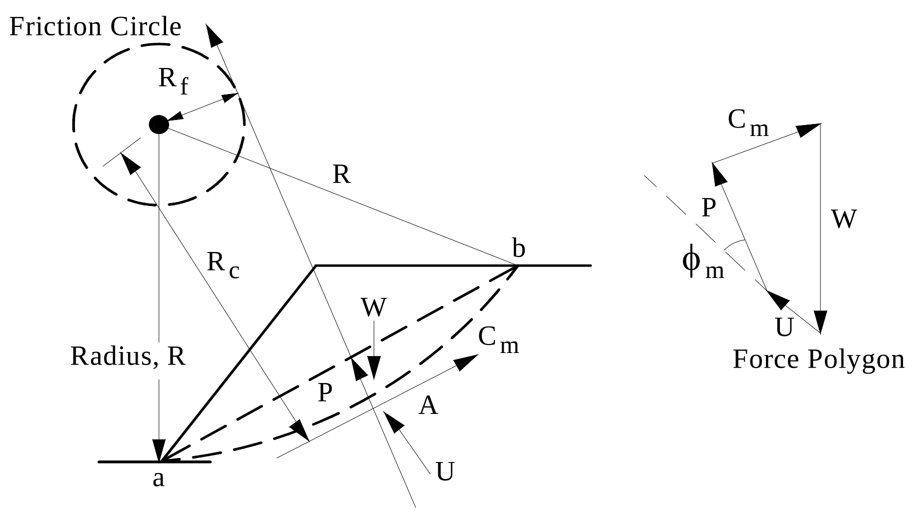


Figure 5-10: Friction Circle Procedure (FHWA, 1994)

5.7 BLOCK ANALYSIS (WEDGE ANALYSIS)

In cases where the soil profile consists of a weak seam or thin layer in the foundation soil a block failure may occur. To analyze the potential, a block analysis may be used to estimate the factor of safety against sliding, as shown in Figure 5-11. In this case, it is necessary to investigate the stability along a surface of failure passing through the foundation of the embankment in addition to the usual studies for possible failure within the embankment itself.

A predominantly planar failure surface develops if the weak foundation layer is relatively thin. Stability may be analyzed by means of a sliding block shearing through the weak foundation layer. The block analysis is fairly simple and straightforward and can be performed quickly by hand calculation. Note that calculations satisfy force equilibrium explicitly but ignore the moment equilibrium for the sliding mass.

For the analysis, the potential sliding block is divided into three parts (Figure 5-11): (1) an *active* wedge at the head of the slide, (2) a *central* block, and (3) a *passive* wedge at the toe.

The factor of safety may be computed by summing forces horizontally to give:

$$FS = \frac{\text{Horizontal Resistance Forces}}{\text{Horizontal Driving Forces}}$$

$$= \frac{P_p + c_m' L + (W - u) \tan \phi_m'}{P_a} \tag{5-24}$$

Where:

- P_a = active force (driving)
- P_p = passive force (resisting)
- L = Length of the Central Block
- c_m', ϕ_m' = strength parameters of the soil at the base of the central block, with effective weight (W - u).

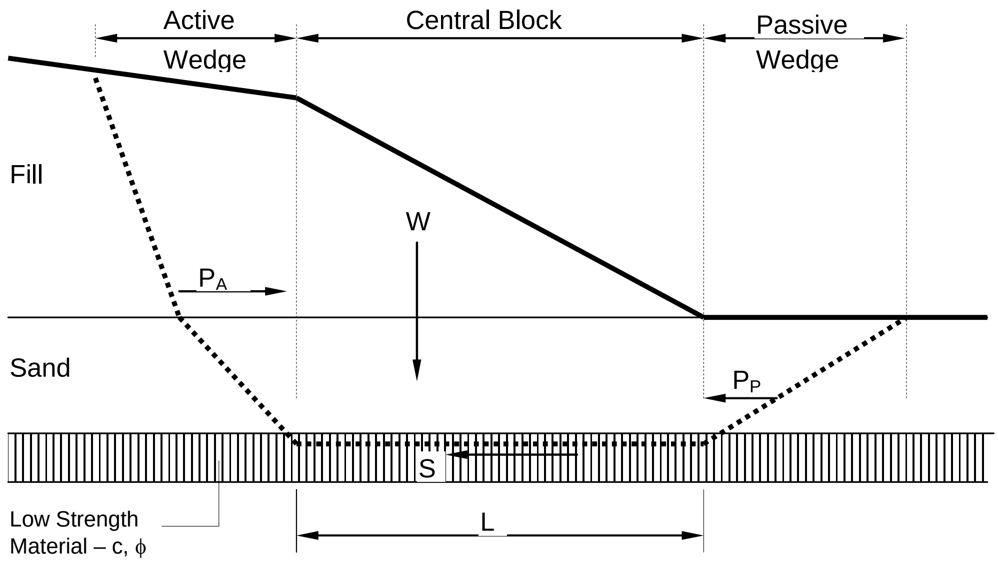


Figure 5-11: Sliding Block Analysis

The active (P_a) and passive (P_p) lateral earth pressures used in the block analysis are calculated using:

$$P_{a/P} = K_{A/P}\, \sigma_v' \mp 2c_m \sqrt{K_{A/P}} \tag{5-25}$$

where K_A and K_P are the active and passive earth pressure coefficients, respectively; σ_v' is the vertical effective stress and c_m is the mobilized cohesion parameter.

The earth pressure coefficients may be estimated using the Rankine expression:

$$K_a = \frac{1 - \sin \phi'_m}{1 + \sin \phi'_m} \quad \text{and} \quad K_p = \frac{1 + \sin \phi'_m}{1 - \sin \phi'_m} \tag{5-26}$$

The above expression is suitable for cases where the backslope is horizontal. For other cases (nonhorizontal backslopes), the readers are referred to FHWA-NHI-05-046 “Earth Retaining Structures”.

When the active or passive wedge passes through more than one soil type with different soil strengths or unit weights, appropriate coefficients, K_a or K_p, should be selected for calculating the forces from each soil layer. The total forces, active and passive, may then be calculated by adding the contribution of each layer.

Several trial locations of the active and passive wedges must be checked to determine the minimum factor of safety. When the weak layer has considerable thickness, the failure plane must be assumed at different depths to find its critical location, which gives the lowest factor of safety or the highest required strength. Note that wedge-type failures are assumed at the head and toe of the slide, similar to what occurs against vertical planes, which are treated as "imaginary" retaining walls. The active and passive forces are computed the same as for retaining wall problems. Assuming conservatively that the resultant interwedge force is horizontal (i.e., frictionless wall)

5.8 DESIGN CHARTS

Slope stability charts are useful for preliminary analysis to compare alternates that may be examined in more detail later. Chart solutions also provide a quick means of checking the results of detailed analyses. Engineers are encouraged to use these charts before a computer program to determine the approximate value of the factor of safety, as it allows some quality control and a check for the subsequent computer-generated solutions.

Slope stability charts are also used to back-calculate strength values for failed slopes (landslides) to aid in planning remedial measures. Assuming a factor of safety of unity for the conditions at failure and solve for the unknown shear strength. Since soil strength often involves both cohesion (c') and friction (ϕ'), there are no unique values that will give a factor of safety equal to one. As such, selection of the most appropriate *c*' and ϕ' depends on local experience and judgement. Since the friction angle is usually within a narrow range for many types of soils and can be obtained by testing with a certain degree of confidence, the cohesion, c', is generally varied in practice, while the friction angle is fixed for the back-calculation of slope failures.

The major shortcoming in using design charts is that most are for ideal, homogeneous soil conditions that are not encountered in practice. They have been devised using the following general assumptions:

1. Two-dimensional limit equilibrium analysis
2. Simple homogeneous slopes
3. Slip surfaces of circular shapes only.

It is imperative that the user understands the underlying assumptions for the charts before using them for the design of slopes.

Regardless of the above shortcomings, many practicing engineers use these charts for non-homogeneous and non-uniform slopes with different geometrical configurations. To do this correctly, one must use an average slope inclination and weighted averages of *c*', ϕ' or c_u values calculated on the basis of the proportional length of slip surface passing through different relatively homogeneous layers. Such a procedure is extremely useful for preliminary analyses and saves time and expense. In most cases, the results are checked by performing detailed analyses using more suitable and accurate methods, for example, the method of slices.

5.8.1 Historical Background

Some of the first slope stability charts were published by Taylor (1937 and 1948). These well-known, classical charts are strictly applicable only for analysis in terms of a *total stress* approach. Pore pressures or pore pressure ratios (r_u) are not considered in these charts. Taylor used the friction circle method to derive the charts. Since then, various charts have been developed by Bishop and Morgenstern (1960), Morgenstern (1963), Spencer (1967), Terzaghi and Peck (1967), Janbu (1968), Hunter and Schuster (1968), and others as summarized in Table 5-3.

5.8.2 Stability Charts

<u>Taylor's Charts</u>

Taylor (1948) developed stability charts, as shown in Figures 5-12 and 5-13, which can be used respectively for the $\phi = 0$ and $\phi > 0$ analysis of a simple slope. As shown in these charts, the slope has an angle β, a height *H*, and base stratum at a depth of DH below the toe, where *D* is a depth ratio. The charts can be used to determine the *developed* cohesion, c_d, as shown by the solid curves; and *nH*, which is the distance from the toe to the failure circle as indicated by the short dashed curve.

TABLE 5-3
SUMMARY OF SLOPE STABILITY CHARTS

Author	Parameters	Slope Inclinations	Analytical Methods	Limitations/Notes
Taylor (1948)	c_u c, ϕ	0 – 90° 0 – 90°	$\phi = 0$ Friction circle	Undrained analysis Dry slopes only
Bishop and Morgenstern (1960)	c, ϕ, r_u	11 - 26.5°	Bishop	One of the first to include effects of water
Gibson and Morgenstern (1962)	c_u	0 – 90°	$\phi = 0$	Undrained analysis with c_u increasing liearly with depth; zero strength at ground level
Spencer (1967)	c, ϕ, r_u	0 – 34°	Spencer	Toe circles only
Janbu (1968)	c_u c, ϕ, r_u	0 – 90°	$\phi = 0$	Extensive series of charts for seepage and tension crack effects
Hunter and Schuster (1968)	c_u	0 – 90°	$\phi = 0$	Undrained analysis with c_u increasing linearly with depth; finite strength at ground level
Chen and Giger (1971)	c, ϕ	20 – 90°	Limit Analysis	
O'Connor and Mitchell (1977)	c, ϕ, r_u	11 – 26°	Bishop	Extended Bishop and Morgenstern (1960) to include $N_c = 0.1$
Hoek and Bray (1977)	c, ϕ c, ϕ	0 – 90° 0 – 90°	Friction Circle Wedge	Includes groundwater and tension cracks 3-D analysis of wedge block
Cousins (1978)	c, ϕ, r_u	0 – 45°	Friction Circle	Extension of Taylor (1948)
Charles and Soares (1984)	ϕ	26 – 63°	Bishop	Nonlinear Mohr-Coulomb failure envelope, ($\tau = A(\sigma')^b$)
Barnes (1991)	c, ϕ, r_u	11 – 63°	Bishop	Extension of Bishop and Morgenstern (1960); wider range of slope angle

If there are loads outside the toe that prevent the circle from passing below the toe, use the long dashed curve to determine the developed cohesion. Note that the solid and the long dashed curves converge as *n* approaches zero. The circle represented by the curves on the left of *n* = 0 does not pass below the toe, so the loading outside the toe has no influence on the developed cohesion.

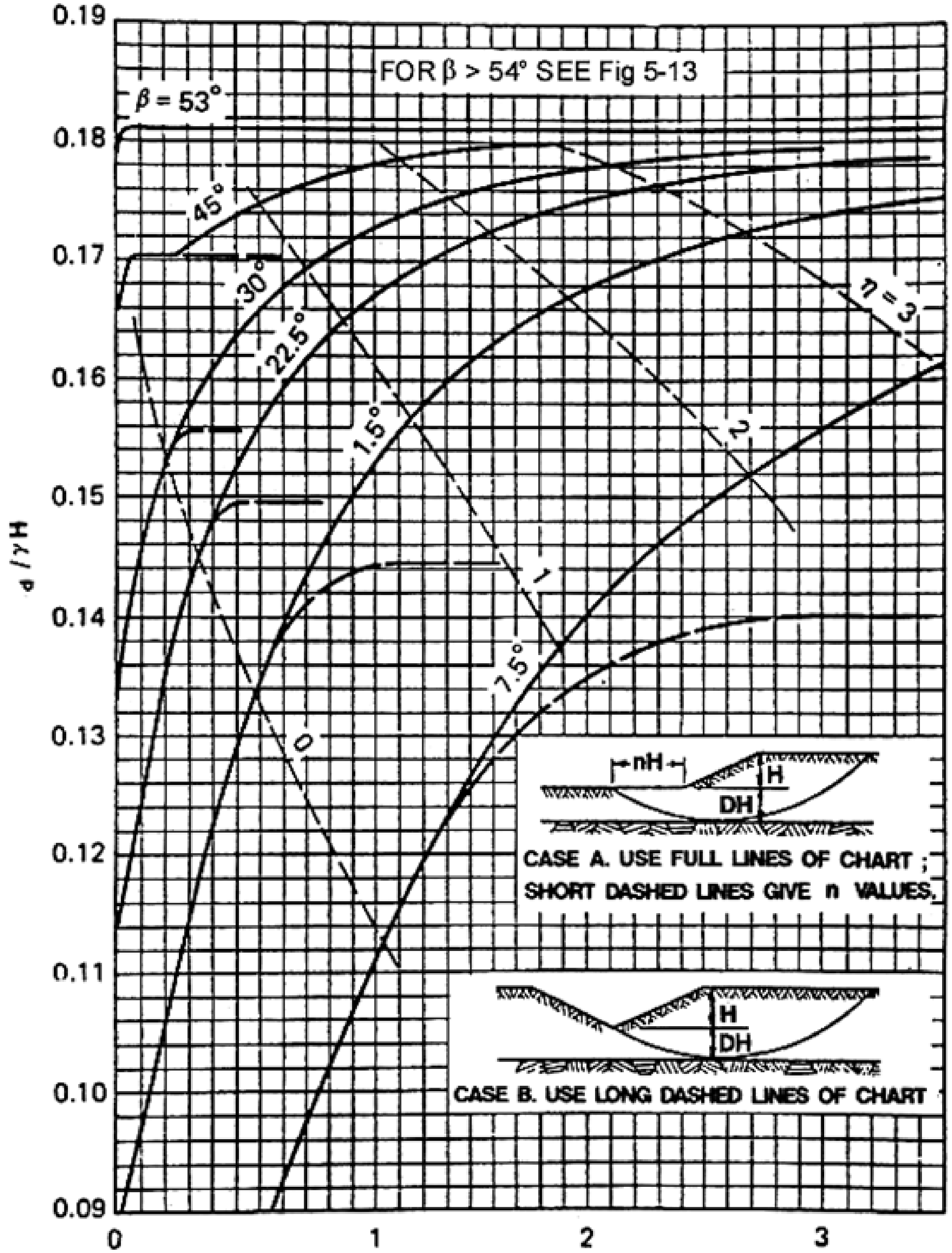


Figure 5-12: Taylor's Chart for ϕ=0 Conditions for Slope Angles (β) less than 54° (After Taylor, 1948)

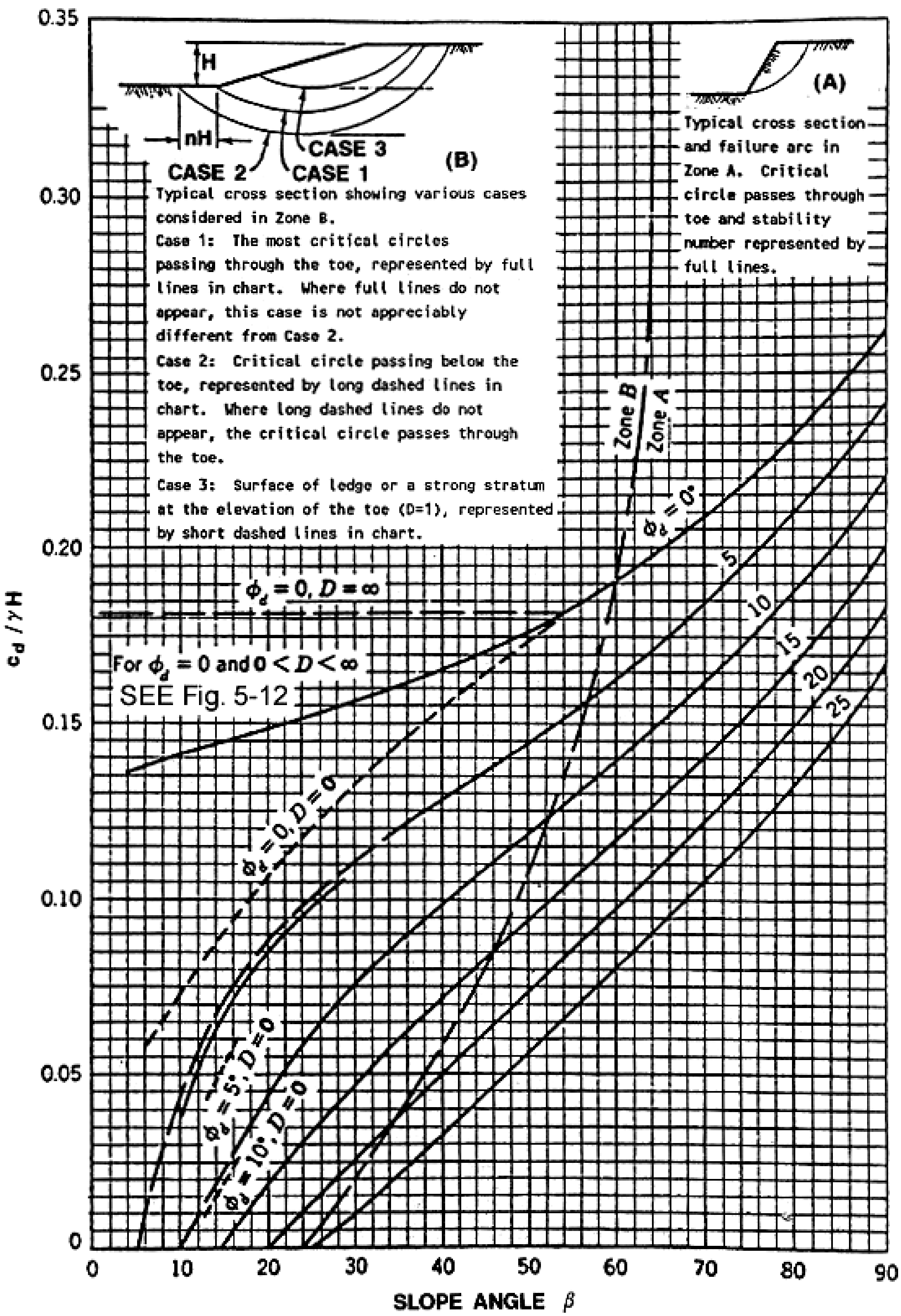


Figure 5-13: Taylor's Chart for Soils with Friction Angle (After Taylor, 1948)

Spencer Charts

The Spencer (1967) charts, shown in Figure 5-14, are based on solutions computed using the rigorous Spencer's Method for circular ship surfaces, which satisfies complete equilibrium. Typically, these charts are used to determine the required slope angle for a specific (preselected) factor of safety. The charts assume that a firm stratum is at a great depth below the slope and present solutions for three different pore pressure ratios, r_u = 0, 0.25, and 0.5. In using the charts, the *developed* friction angle is calculated using:

$$\phi_d = \tan^{-1}\left[\frac{\tan\phi}{FS}\right] \qquad (5\text{-}27)$$

where *FS* is the factor of safety.

Janbu's Charts

In 1968, Janbu published stability charts for slopes in soils with uniform strength (non-uniform soil strengths may also be analyzed using an averaging technique (Example 4)) used throughout the depth of the soil layer assuming $\phi = 0$ and $\phi > 0$ conditions. These charts are presented in Figures 5-15 through 5-18. This series of charts account for several different conditions and provide factors for surcharge loading at the top of the slope, submergence, and tension cracks that can be expected to influence the design of typical highway slopes.

The stability chart for slopes in soils with uniform shear strength throughout the depth of the layer and with $\phi = 0$ is shown in Figure 5-16. Charts for correction factors for the conditions when surcharge loads, submergence and tension cracks are present are shown in Figures 5-16 through 5-17.

Steps for using Janbu's Charts on Figures 5-15 through 5-17, for $\phi = 0$ material.

Step 1: Using the chart at the bottom of Figure 5-15, determine the position of the center of the critical circle, which is located at X_o, Y_o. For slopes steeper than 53°, the critical circle passes through the toe. For slopes flatter than 53°, the critical circle passes tangent to the top of firm soil or rock.

Step 2: Using the estimated critical circle as a guide (step 1), estimate the average value of strength, c. This is done by calculating the weighted average of the strengths along the failure surface, using the number of degrees intersected by each soil layer as the weighted factor.

Step 3: Calculate the depth factor, d:

$$d = D/H$$

Step 4: Calculate P_d using the following equation:

$$P_d = (\gamma H + q - \gamma_w H_w)/(\mu_t\ \mu_q \mu_w)$$

where:

q = surcharge load
γ_w = unit weight of water
H_w = depth of water outside the slope

μ_t = tension crack correction factor (Figure 5-16)
μ_q = surcharge correction factor (Figure 5-17, top)
μ_w = submergence correction factor (Figure 5-17, bottom)

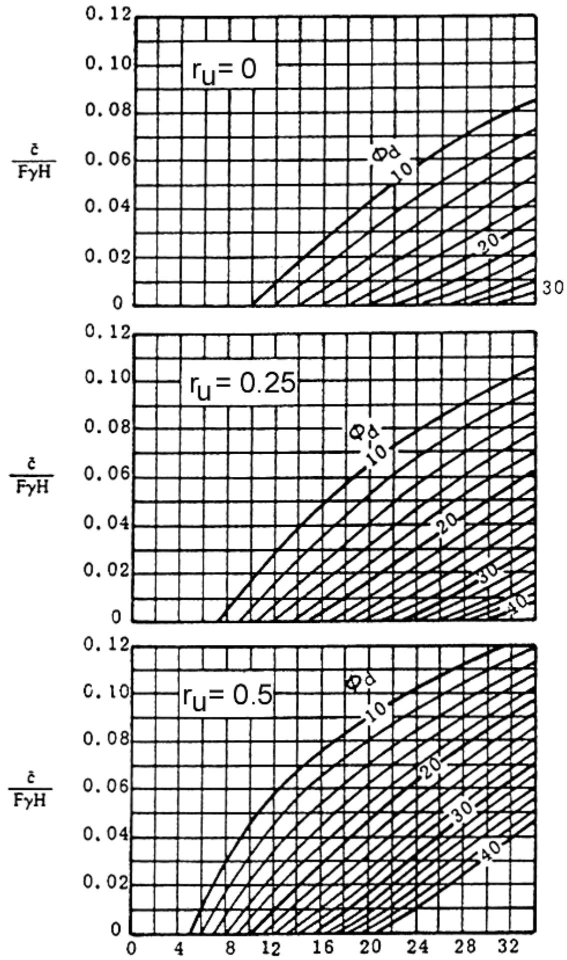


Figure 5-14: Stability Chart For Different Pore Pressure Ratios (Spencer, 1967)

Step 5: Using the chart at the top of Figure 5-15, determine the value of the stability number, N_o, which depends on the slope angle β, and the value of d.

Step 6: Calculate the factor of safety (FS) using the equation:

$$FS = N_o c/P_d$$

Step 7: If a slope contains more than one soil layer, it may be necessary to calculate the factor of safety for circles at more than one depth. If the soil layer is weaker than the layer above, the critical circle will be tangent to the base of the lower layer. If a soil layer is stronger than the layer above, the critical circle may be tangent to the base of either the upper layer or the lower layer, and both possibilities should be examined (Duncan and Buchignani, 1975).

Steps for using Janbu's Charts on Figures 5-16 through 5-18, for $\phi > 0$ materials.

Step 1: Using judgement, estimate the location of the critical circle. For most conditions of simple slopes in uniform soils with $\phi > 0$, the critical circle passes through the toe of the slope. The stability numbers given in Figure 5-18 were developed by analyzing toe circles.

Where conditions are not uniform and there is weak layer beneath the toe of the slope, a circle passing beneath the toe may be more critical than a toe failure. Figure 5-18 may be used to calculate the factor of safety for such cases provided the values of c and ϕ used represent the correct average values for the circle considered.

If there is a weak layer above the toe of the slope, a circle passing above the toe of the slope may be more critical. Similarly, it there is water outside the toe of the slope, a circle passing above the water may be more critical. When these types of circles are analyzed, the value of H should be equal to the height from the base of the weak layer, or the water level, to the top of the slope.

Step 2: Using the circle as a guide, estimate the average values of c and ϕ. This can be done by calculating the weighted average values of c and ϕ using the arc (number of degrees intersected along the arc by each soil layer).

Step 3: Calculate P_d using the following equation:

$$P_d = (\gamma H + q - \gamma_w H_w)/(\mu_t \mu_q \mu_w)$$

where:

q	= surcharge load
γ_w	= unit weight of water
H_w	= depth of water outside the slope
μ_t	= tension crack correction factor (Figure 5-16)
μ_q	= surcharge correction factor (Figure 5-17, top)
μ_w	= submergence correction factor (Figure 5-17, bottom)

Step 4: Calculate P_e using the following equation:

$$P_e = (\gamma H + q - \gamma_w H'_w)/(\mu_q \mu'_w)$$

where:

H'_w	= height of water within the slope
μ_t	= tension crack correction factor (Figure 5-16)
μ_q	= surcharge correction factor (Figure 5-17, top)
μ'_w	= seepage correction factor (Figure 5-17, bottom)

Step 5: Calculate the dimensionless parameter $\lambda_{C\phi}$ using the equation below:

$$\lambda_{C\phi} = P_e \tan\phi / c$$

For c=0, $\lambda_{C\phi}$ is infinite skip to step 6.

Step 6: Using the chart in Figure 5-18 determine the value of the stability number (N_{cf}) which is dependent on slope angle (β) and the value of $\lambda_{C\phi}$.

Step 7: Calculate the factor of safety for the slope.

$$FS = N_{cf}\, c / P_d \quad (c > 0)$$

$$FS = P_e\, b \tan \phi / P_d \quad (c = 0)$$

Step 8: Determine the actual location of the critical circle using the chart on the right side of Figure 5-18. The center of the circle is located at X_o, Y_o, and the circle passes through the toe of the slope, except for the exceptions discussed in Step 1. If the critical circle is much different from the one assumed in Step 1 for the purposed of determining the average strength, Steps 2 through 8 should be repeated.

If a slope contains more than one soil layer, it may be necessary to calculate the factor of safety for circles at more than one depth. If a soil is weaker than the layer above, the critical circle will extend into the lower layer, and either a toe circle or a deep circle within this layer will be critical. If a soil layer is stronger than the layer above, the critical circle may or may not extend into the lower layer, depending on the relative strengths of the two layers. Both possibilities should be examined (Duncan and Buchignani, 1975).

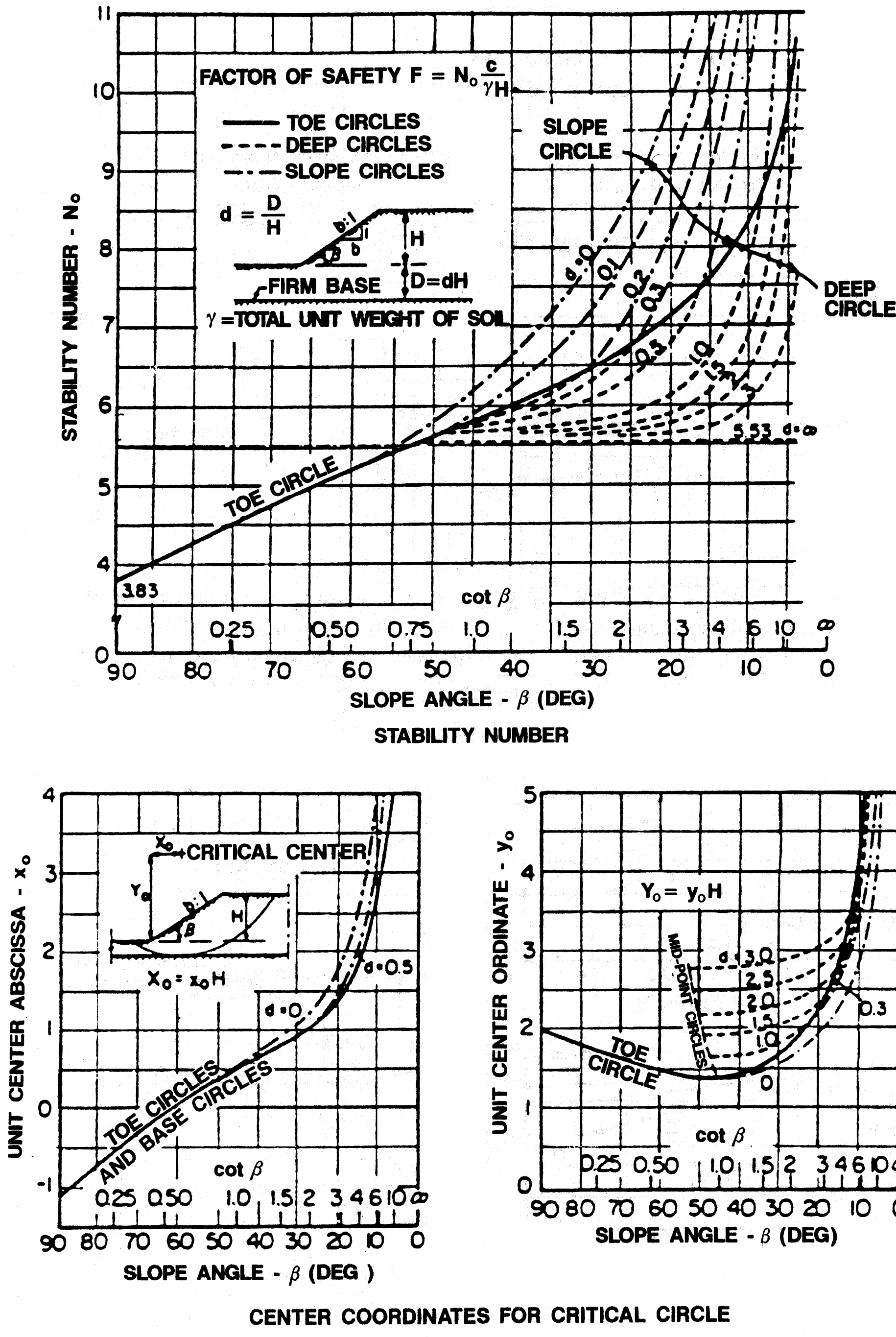


Figure 5-15: Stability Charts for $\phi = 0$ Soils (Janbu, 1968)

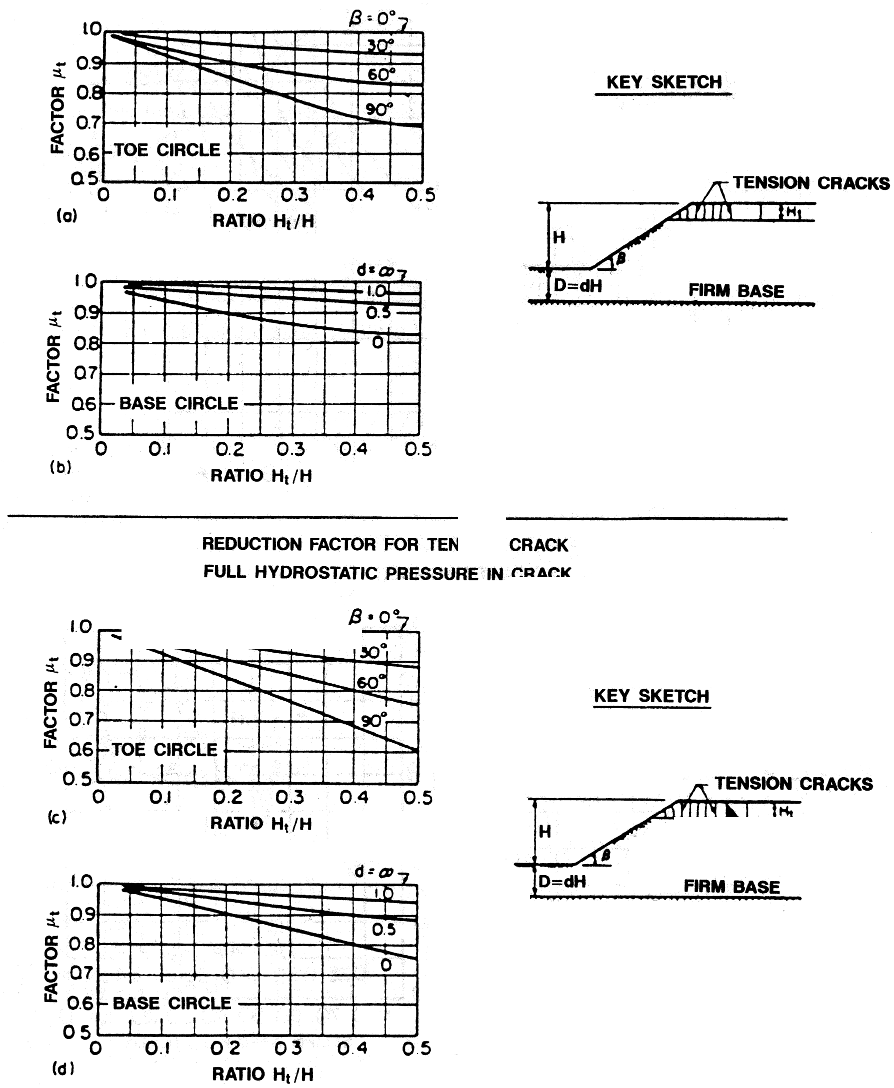


Figure 5-16: Reduction Factors for Stability Charts for ϕ=0 and ϕ > 0 Soils (Janbu, 1968)

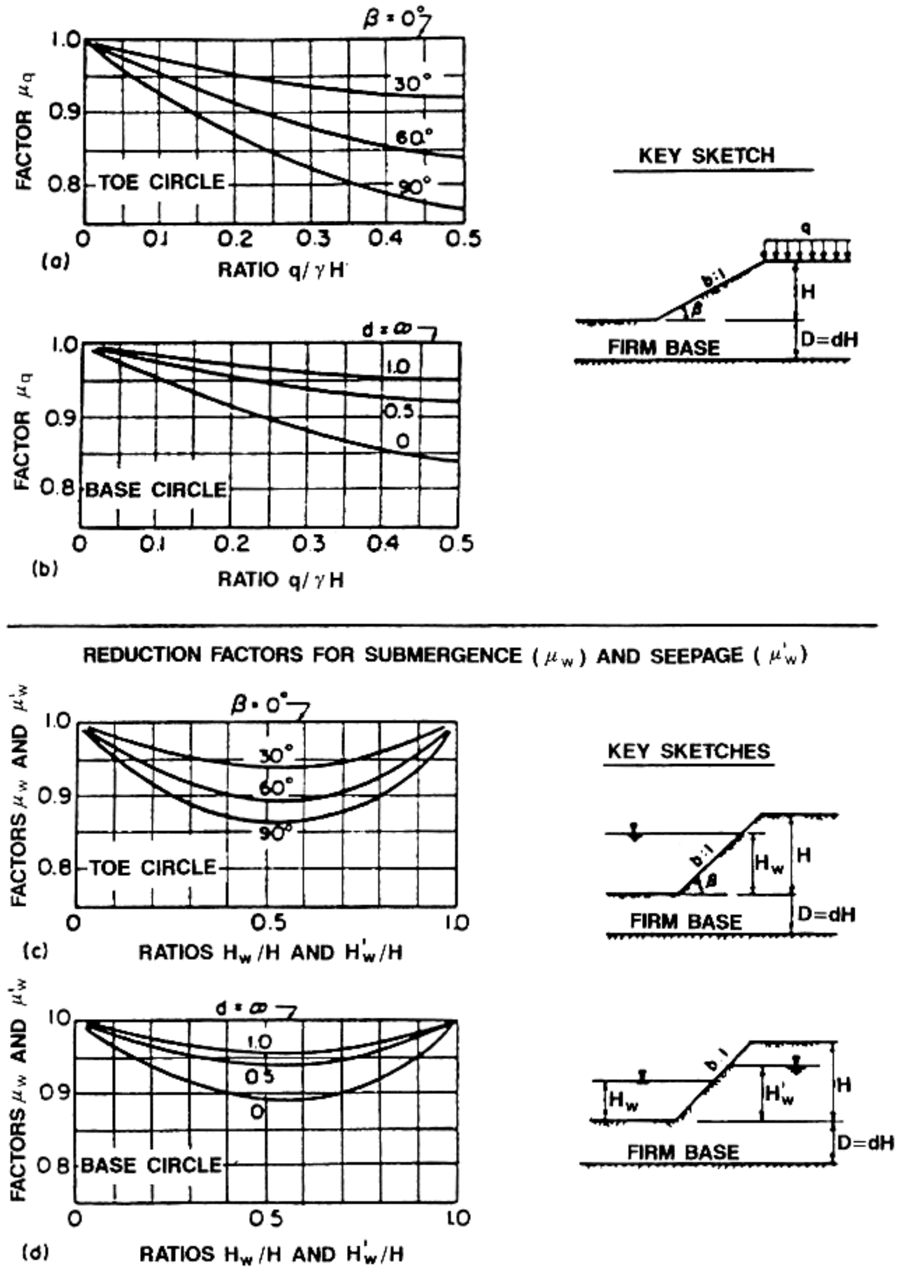


Figure 5-17: Reduction Factors for Stability Charts for ϕ=0 and ϕ > 0 Soils (Janbu, 1968)

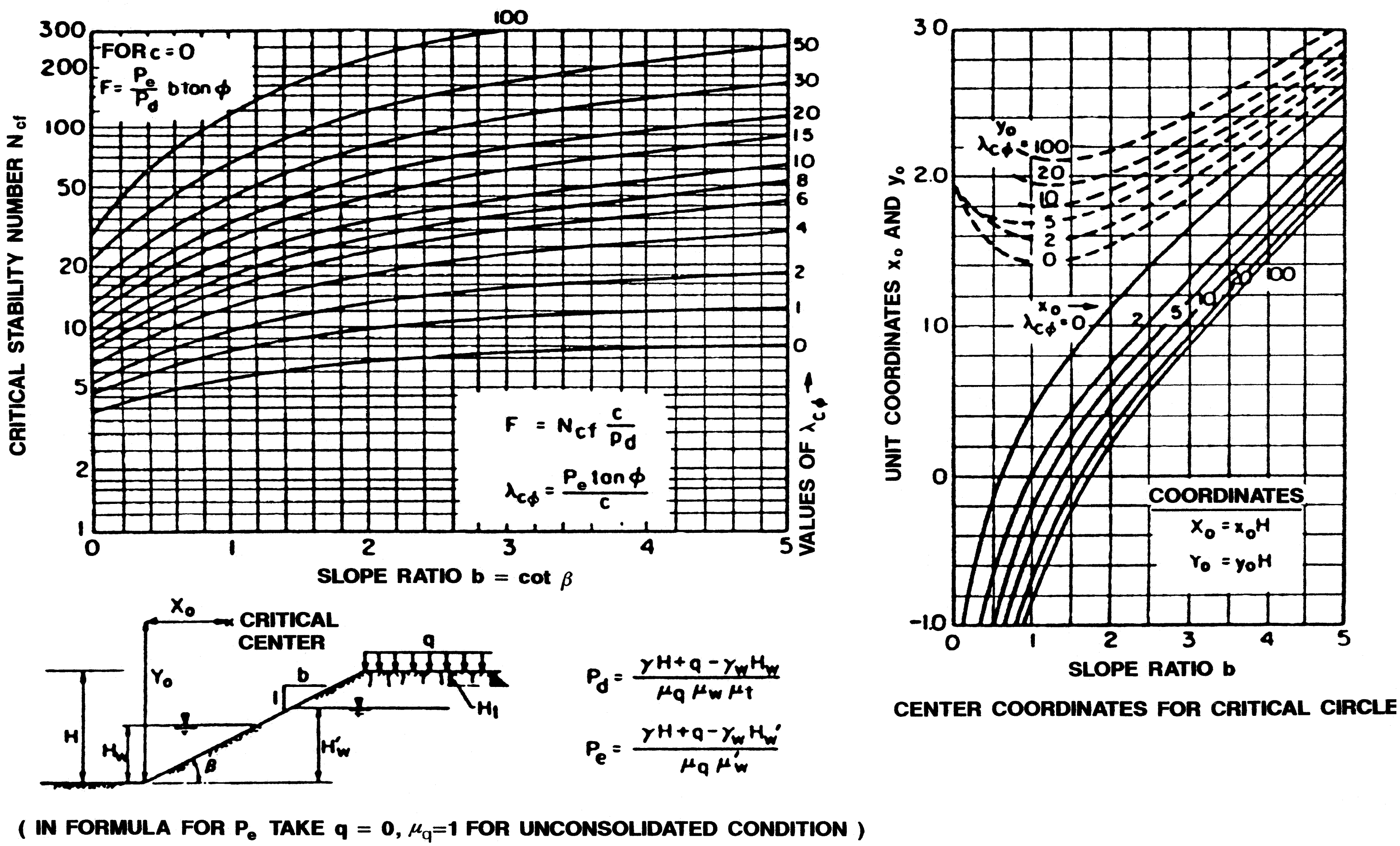


Figure 5-18: Stability Charts for φ > 0 (Janbu, 1968)

Hunter and Schuster Chart

Hunter and Schuster (1968) developed a stability chart (Figure 5-19) that takes into account the case where the shear strength increases linearly with depth, under a ϕ = 0 condition. The steps required to determine the factor of safety are included on Figure 5-19.

Step 1: Select the linear variation of strength with depth which best fits the measured strength data. Extrapolate this linear variation upward to determine Ho, the height at which the strength profile intersects zero, as shown in Figure 5-19.

Step 2: Calculate $M = H_o/H$, where H is the slope height.

Step 3: Determine the dimensionless stability number, N, from the chart in Figure 5-19.

Step 4: Determine the value of strength, c_b, at the elevation of the bottom of the slope.

Step 5: Calculate the factor of safety, FS, using the equation:

$$FS = N\, c_b/\gamma\, (H + H_o)$$

Use: γ = total unit weight of soil for slopes above water
γ = buoyant unit weight ($\gamma_{buoyant}$) for submerged slopes
γ = weighted average for partly submerged slopes

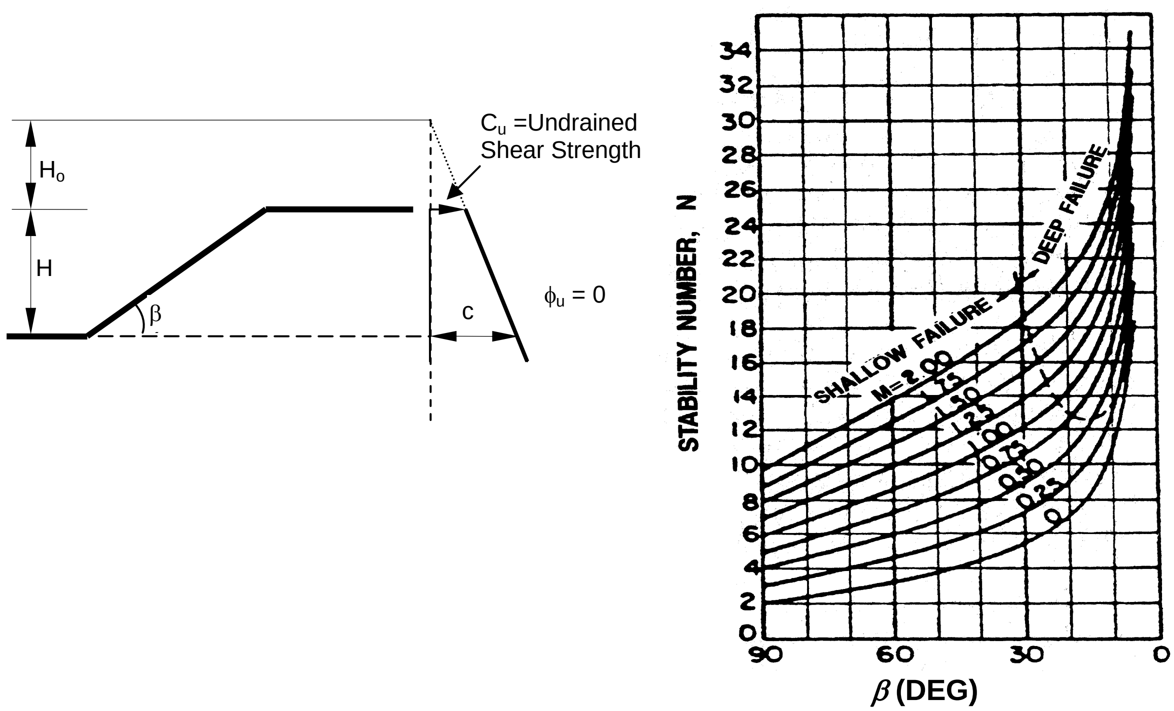


Figure 5-19: Stability Charts for ϕ=0 and Strength Increasing with Depth (Hunter and Schuster, 1968)

5.9 METHOD OF SLICES

The methods discussed earlier do not depend on the distribution of the effective normal stresses along the failure surface. However, if the mobilized strength for a c-ϕ soil is to be calculated, the distribution of the effective normal stresses along the failure surface must be known. This condition is usually analyzed by discretizing the mass of the failure slope into smaller slices and treating each individual slice as a unique sliding block. Such an approach also can readily accommodate complex slope geometry, variable soil conditions, and the influence of external boundary loads. These limit equilibrium methods for slope stability analysis divide a slide-mass into *n* smaller slices as shown in Figure 5-20. Each slice is affected by a general system of forces, as shown in Figure 5-21.

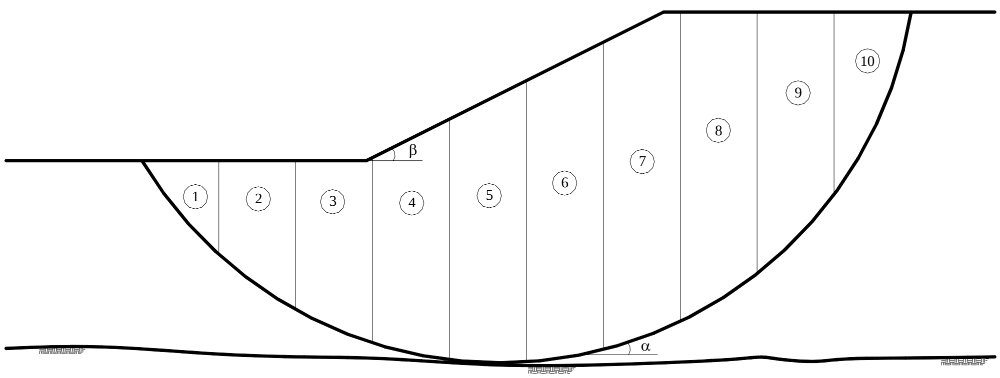
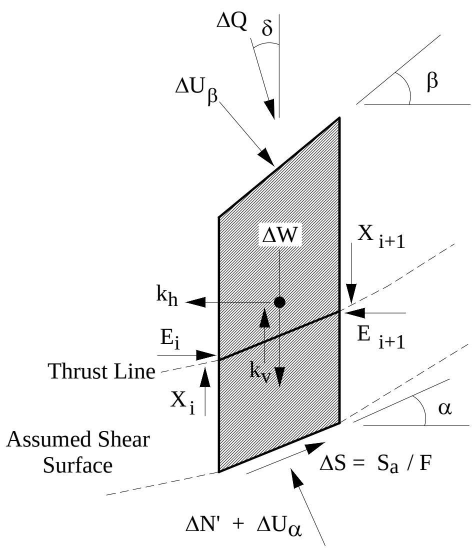


Figure 5-20: Division of Potential Sliding Mass Into Slices



ΔU_β	= surface water force
ΔQ	= external surcharge
ΔW	= weight of slice
$\Delta N'$	= normal force
ΔU_α	= pore water force
Ei	= interslice force
Xi	= interslice shear force
ΔS	= mobilized strength
S_a	= available strength
F	= factor of safety
k_h	= horizontal seismic coeff.
k_v	= vertical seismic coeff.
h	= average height of the slice
h_c	= height from center of base to centroid of slice

Figure 5-21: Forces Acting on a Typical Slice (After FHWA, 1994)

The thrust line indicated in Figure 5-21 connects the points of application of the interslice forces, E_i. The location of this thrust line may be determined using a rigorous method of analysis that satisfies complete equilibrium. The popular simplified methods of analysis neglect the location of the interslice force and therefore complete equilibrium is not satisfied for the failure mass.

For this system, there are (6n - 2) unknowns as listed in Table 5-4. Also, since only 4n equations can be written for the system at limit equilibrium, the solution is statically indeterminate. However, a solution is possible, providing the number of unknowns can be reduced by making some simplifying assumptions. One of the common assumptions is that the normal force on the base of the slice acts at the midpoint, thus reducing the number of unknowns to (5n - 2). This then requires additional (n - 2) assumptions to make the problem determinate. It is these assumptions that generally categorize the available methods of analysis (Sharma and Lovell, 1983).

Table 5-5 lists the commonly available methods of analysis and the conditions of static equilibrium that are satisfied in determining the factor of safety. The assumptions made by each of these methods to render the problem determinate also are summarized below.

TABLE 5-4
EQUATIONS AND UNKNOWNS ASSOCIATED WITH THE METHOD OF SLICES

Equations	Condition
n	Moment equilibrium for each slice
2n	Force equilibrium in two directions (for each slice)
n	Mohr-Coulomb relationship between shear strength and normal effective stress
4n	**Total number of equations**
Unknowns	
1	Factor of Safety
n	Normal force at base of each slice
n	Location of normal force
n	Shear force at base of each slice
n - 1	Horizontal interslice force, E
n - 1	Vertical interslice force, X
n - 1	Location of interslice force (line of thrust)
6n-2	**Total number of unknowns**

TABLE 5-5
STATIC EQUILIBRIUM CONDITIONS SATISFIED BY LIMIT EQUILIBRIUM METHODS

METHOD	FORCE EQUILIBRIUM		MOMENT EQUILIBRIUM
	X	y	
Ordinary Method of Slices	No	No	Yes
Bishop's Simplified	Yes	No	Yes
Janbu's Simplified	Yes	Yes	No
Corps of Engineers	Yes	Yes	No
Lowe and Karafiath	Yes	Yes	No
Bishop's Rigorous	Yes	Yes	Yes
Janbu's Rigorous	Yes	Yes	Yes
Spenser's Method	Yes	Yes	Yes
Sarma's Method	Yes	Yes	Yes
Morgenstern-Price	Yes	Yes	Yes

Ordinary Method of Slices satisfies moment equilibrium but neglects all interslice forces and fails to satisfy horizontal and vertical force equilibrium for the slide mass, as well as for individual slices. However, this is one of the simplest procedures based on the method of slices. It is only applicable to circular slip surfaces.

Bishop's Simplified Method assumes that all interslice shear forces are zero, reducing the number of unknowns by (n - 1). This leaves (4n - 1) unknowns, leaving the solution over-determined as horizontal force equilibrium will not be satisfied for one slice. It can only be used for circular slips.

Janbu's Simplified Method assumes zero interslice shear forces, reducing the number of unknowns to (4n - 1). This leads to an over-determined solution that will not completely satisfy moment equilibrium conditions. However, Janbu presented a correction factor, f_o, to account for this inadequacy. It can be applied on any shape of slip surface.

Lowe and Karafiath's Method (a Force Equilibrium Method) assumes that the interslice forces are inclined at an angle equal to the average of the ground surface and slice base angles, that is, $\theta = ½ (\alpha + \beta)$, where θ is the assumed inclination of the interslice force on the right-hand side of the typical slice shown in Figure 5-21. This simplification satisfies both vertical and horizontal force equilibrium but leaves (4n - 1) unknowns and fails to satisfy moment equilibrium. It can be applied on any shape of slip surfaces.

Corps of Engineers' Method (a Force Equilibrium Method) considers the inclination of the interslice force as either (i) parallel to ground surface ($\theta = \beta$), or (ii) equal to the average slope angle between the left and right end-points of the failure surface. The approach is similar to the one proposed by Lowe and Karafiath and presents an overdetermined system where moment equilibrium is not satisfied for all slices. However, it satisfies both vertical and horizontal force equilibrium and can be applied on any shape of slip surfaces.

Spencer's Method rigorously satisfies static equilibrium by assuming that the resultant interslice force has a constant, but unknown, inclination. These (n - 1) assumptions again reduce the number of unknowns to (4*n* - 1), but the unknown inclination is an additional component that subsequently increases the number of unknowns to match the required 4*n* equations. It can be applied on any shape of slip surfaces.

Bishop's Rigorous Method assumes (*n* - 1) interslice shear forces to calculate a factor of safety. Since this assumption leaves (4*n* - 1) unknowns, moment equilibrium cannot be directly satisfied for all slices. However, Bishop introduced an additional unknown by suggesting there exists a unique distribution of the interslice resultant force, out of a possible infinite number, that will rigorously satisfy the equilibrium equations. Like the simplified method, it can only be applied on circular slip surfaces.

Janbu's Generalized Method assumes a location of the thrust line, thereby reducing the number of unknowns to (4*n* - 1). Sarma (1979) points out that the position of the normal stress on the last (uppermost) slice is not used and hence moment equilibrium is not satisfied explicitly for this last slice. However, similar to the rigorous Bishop method, Janbu also suggested that the actual location of the thrust line is an additional unknown, and thus, equilibrium can be satisfied rigorously if the assumption selects the correct thrust line. It can be applied on any shape of slip surfaces.

Sarma's Method (1973) uses the method of slices to calculate the magnitude of a horizontal *seismic coefficient* needed to bring the failure mass into a state of limiting equilibrium. This allows the procedure to develop a relationship between the seismic coefficient and the *presumed* factor of safety. The static FS will then correspond to the case of a zero seismic coefficient. Sarma uses an interslice force distribution function (similar to Morgenstern-Price), and the value of the seismic coefficient can be calculated directly for the presumed FS. All equilibrium conditions are satisfied by this method. However, it should be noted that the critical surface corresponding to the static FS (for a zero seismic coefficient) often will be different from the surface determined using the more conventional approach where the FS is treated as an unknown. It can be applied on any shape of slip surfaces.

Morgenstern-Price method is similar to the Spencer's Method, except that the inclination of the interslice resultant force is assumed to vary according to a "portion" of an arbitrary function. This additional "portion" of a selected function introduces an additional unknown, leaving 4n unknowns and 4n equations. Also, unlike Spencer's method, the normal force at the base of the slices is obtained by integration. As a result the resultant normal force can be slightly offset from the center of the slice base as assumed by Spencer's method. This is the most complete (and therefore the most complex) of all the established methods. It can be applied on any shape of slip surfaces.

The rigorous methods, such as Morgenstern-Price (M&P) and Spencer, may be used to analyze circular and noncircular failure surfaces. For these methods, moment equilibrium conditions are satisfied either by:

Taking overall moments about a common point (for example, the center of a circle if a circular failure surface is being examined) for *all* slices, or

Satisfying moment equilibrium for each *individual* slice by taking moments about the center of the slice base.

The second approach is used to present the formulation for the Spencer's Method discussed in a later section. In summary, only the Spencer, Janbu and Morgenstern-Price (M&P) methods adequately satisfy all conditions of equilibrium. However, there are an infinite number of arbitrary functions that can be used to satisfy the interslice resultant force angles required for the M&P solution.

Once a factor of safety is computed, Janbu's, Spencer's and the M&P's methods require the user to verify the reliability and "reasonableness" of the calculated factor of safety. This additional complexity prevents use of these methods for automatic search procedures for location of the critical failure surface, reserving them for the analysis of well-defined, single surfaces only. Also, these rigorous methods require a substantially higher level of familiarity if reasonable solutions are to be computed for practical problems or thought line location in Janbu.

The general limit equilibrium (GLE) method (Chugh 1986 and Fredlund et al. 1981) is a relatively recent development which encompasses all of the established limit equilibrium methods with the exception of OMS since it does not satisfy the Newtonian force principles at the inter slices. Each of the various methods mentioned (except OMS) is in fact a special case of the GLE method. With GLE, either moment and force equilibrium or just force equilibrium can be satisfied. Because of its generality, the GLE method is gaining popularity. To achieve complete equilibrium, the GLE procedure involves the incorporation of an appropriate function that models the interslice force angle variations. Like all complex methods such as M&P, Spencer and Janbu, GLE has similar limitations as described in the previous paragraph.

Conversely, Bishop's and Janbu's simplified methods are popular because a factor of safety value can be quickly calculated for most surfaces. However, their assumptions and the factor of safety formulations often lead to the calculation of slightly different results between methods. For circular failure surfaces, the factor of safety computed by Bishop's method is usually greater than the value from Janbu's formulation. Bishop's FS value is also about the same as the FS that may be calculated using a more rigorous approach, such as the M&P, Spencer's or GLE method. So, for the analysis of circular failure surfaces, the simplified Bishop's Method is strongly recommended for analysis. However, Janbu method is more flexible and can be used to analyze circular and noncircular surfaces.

All these methods may lead to a different factor of safety for the same surface, thus, it is important that the user understand details concerning the assumptions and derivation involved in each method. The following sections described the more commonly used slice methods. A description of the GLE method is also included.

5.9.1 Ordinary Method of Slices

The ordinary method of slices (OMS) was one of the earliest analytical methods that used the method of slices to estimate the stability of a slope. The method assumes that the resultant of the interslice forces for all slices is inclined at an angle that is *parallel* to the base of the slice. Note that this simplifying assumption fails to satisfy interslice equilibrium where adjacent slices have different base inclinations. This is the main shortcoming of this method and leads to the calculation of inconsistent effective stresses at the base of the slices.

If the slice forces are resolved in a direction perpendicular to the base of the slice shown in Figure 5-21,

$$\begin{aligned}\sum F_\alpha = {} & \Delta N' + \Delta U_\alpha + k_h\,\Delta W \sin\alpha - \Delta W\,(1 - k_v)\cos\alpha \\ & - \Delta U_\beta \cos(\beta - \alpha) - \Delta Q \cos(\delta - \alpha) = 0\end{aligned} \tag{5-28}$$

The above equation may be arranged for $\Delta N'$ as:

$$\begin{aligned}\Delta N' = {} & -\Delta U_\alpha - k_h\,\Delta W \sin\alpha + \Delta W\,(1 - k_v)\cos\alpha \\ & + \Delta U_\beta \cos(\beta - \alpha) + \Delta Q \cos(\delta - \alpha)\end{aligned} \tag{5-29}$$

If the factor of safety against shear failure is defined as F, and is assumed to be the same for all slices, the Mohr-Coulomb mobilized shear strength (ΔS) along the base of each slice is given by:

$$\Delta S = \frac{\Delta C + \Delta N' \tan\phi}{F} \tag{5-30}$$

where ΔC and $\Delta N' \tan\phi$ are the cohesive and frictional shear strength components of the soil. The overall moment equilibrium of the forces about the center of the circular failure surface for each slice is given by:

$$\begin{aligned}\sum M_o = {} & \sum_{i=1}^{n} [(\Delta W\,(1 - k_v) + \Delta U_\beta \cos\beta + \Delta Q \cos\delta)]\, R \sin\alpha \\ & - \sum_{i=1}^{n} (\Delta U_\beta \sin\beta + \Delta Q \sin\delta)\,(R\cos\alpha - h) \\ & - \sum_{i=1}^{n} \Delta S\, R + \sum_{i=1}^{n} [k_h\,\Delta W (R\cos\alpha - h_c)] \\ = {} & 0\end{aligned} \tag{5-31}$$

where R is the radius of the circular failure surface, h is the average height of the slice and h_c is the vertical height between center of the base slice and the centroid of the slice. The influence of the internal interslice forces has been excluded from this expression as their net resultant moment will be zero. The above equation may be simplified by dividing throughout by the radius to get:

$$\begin{aligned}\frac{\Sigma M_0}{R} = {} & \sum_{1}^{n} [(\Delta W\ (1 - k_v) + \Delta U_\beta \cos\beta + \Delta Q \cos\delta)]\ \sin\alpha \\ & - \sum_{1}^{n} [\Delta S] - \sum_{1}^{n} [\Delta U_\beta \sin\beta + \Delta Q \sin\delta](\cos\alpha - h/R) \\ & + \sum_{1}^{n} [k_h\ \Delta W\ (\cos\alpha - \frac{h_c}{R})]\end{aligned} \tag{5-32}$$

If the FS is assumed to be the same for all slices, substitute equation 5-29 into equation 5-28 to give:

$$F = \frac{\sum_{i=1}^{n} (\Delta C + \Delta N' \tan\phi)}{\sum_{i=1}^{n} A_1 - \sum_{i=1}^{n} A_2 + \sum_{i=1}^{n} A_3} \tag{5-33}$$

where: $A_1 = [\Delta W (1 - k_v) + \Delta U_\beta \cos\beta + \Delta Q \cos\delta] \sin\alpha$

$A_2 = (\Delta U_\beta \sin\beta + \Delta Q \sin\delta)(\cos\alpha - h/R)$ (5-34)

$A_3 = k_h \Delta W (\cos\alpha - h_c/R)$

and ΔN' is given by equation 5-29. This is the formulation that is often used to compute the factor of safety according to the assumptions of the Ordinary Method of Slices.

5.9.2 Simplified Janbu Method

The simplified (or modified) Janbu Method uses the method of slices to determine the stability of the slide mass. It is based on the forces shown in Figure 5-21 for the free-body diagram of a typical slice. The simplified procedure assumes that there are no interslice shear forces. The geometry of each slice is described by its height h, measured along its centerline, its width Δx, and by the inclination of its base and top, α and β, respectively.

Janbu's Method satisfies vertical force equilibrium for each slice, as well as overall horizontal force equilibrium for the entire slide mass (all slices). Vertical force equilibrium for each slice is given by:

$$\sum F_v = (\Delta N' + \Delta U_\alpha)\cos\alpha + \Delta S \sin\alpha - \Delta W (1 - k_v) - \Delta U_\beta \cos\beta - \Delta Q \cos\delta = 0 \tag{5-35}$$

The above equation may be arranged for ΔN' (equation 5-37):

$$\Delta N' = \frac{-\Delta U_\alpha \cos\alpha - \Delta S \sin\alpha + \Delta W (1 - k_v) + \Delta U_\beta \cos\beta + \Delta Q \cos\delta}{\cos\alpha} \tag{5-36}$$

If the factor of safety against shear failure is defined as *F*, and is assumed to be the same for all slices, the Mohr-Coulomb mobilized shear strength (ΔS) along the base of each slice is given by:

$$\Delta S = \frac{\Delta C + \Delta N' \tan\phi}{F} \tag{5-37}$$

where ΔC and ΔN'tan ϕ are the cohesive and frictional shear strength components of the soil. By substituting equation 5-38 into 5-37, the effective normal force acting at the base of the slice can be determined as:

$$\Delta N' = \frac{1}{m_\alpha}\left[\Delta W (1 - k_v) - \frac{\Delta C \sin\alpha}{F} - \Delta U_\alpha \cos\alpha + \Delta U_\beta \cos\beta + \Delta Q \cos\delta\right] \tag{5-38}$$

where

$$m_\alpha = \cos\alpha \left[1 + \frac{\tan\alpha \tan\phi}{F}\right] \tag{5-39}$$

Next, the overall horizontal force equilibrium is evaluated for all slices of the slide mass. In this case, for an individual slice i:

$$[F_H]_i = (\Delta N' + \Delta U_\alpha)\sin\alpha + \Delta W k_h + \Delta U_\beta \sin\beta + \Delta Q \sin\delta - \Delta S \cos\alpha \qquad (5\text{-}40)$$

Then substituting for ΔS from equation 5-37 and rearranging, overall horizontal force equilibrium for the slide mass is given by:

$$\begin{aligned}\sum_{i=1}^{n}[F_H]_i = \sum_{i=1}^{n} & (\Delta N' + \Delta U_\alpha)\sin\alpha + \Delta W k_h + \Delta U_\beta \sin\beta \\ & + \Delta Q \sin\delta - \frac{\Delta C + \Delta N' \tan\phi}{F}\cos\alpha \\ & = 0\end{aligned} \qquad (5\text{-}41)$$

Rearranging the above equation, the following expression may be obtained:

$$\begin{aligned}\sum_{i=1}^{n}\left[(\Delta N' + \Delta U_\alpha)\sin\alpha + \Delta W k_h + \Delta U_\beta \sin\beta + \Delta Q \sin\delta\right] \\ = \sum_{i=1}^{n}\left[\frac{1}{F}(\Delta C + \Delta N' \tan\phi)\cos\alpha\right]\end{aligned} \qquad (5\text{-}42)$$

Then, if each slice has the same factor of safety, *F*,

$$F = \frac{\sum_{i=1}^{n}\left[\Delta C + \Delta N' \tan\phi\right]\cos\alpha}{\sum_{i=1}^{n} A_4 + \sum_{i=1}^{n} \Delta N' \sin\alpha} \qquad (5\text{-}43)$$

where ΔN' is given by equation 5-38. and

$$A_4 = \Delta U_\alpha \sin\alpha + \Delta W kh + \Delta U_\beta \sin\beta + \Delta Q \sin\delta \qquad (5\text{-}44)$$

Equation 5-44 essentially represents a ratio of the *available* shear resistance force and the *driving* shear force, in the horizontal direction, along the failure surface. This format allows the state of the effective stress to be determined and appropriate corrections implemented if ΔN' is calculated to be less than zero.

The reported Janbu FS value is calculated by multiplying the calculated F value by a modification factor, f_o,

$$FS_{Janbu} = f_o \times FS_{calculated} \qquad (5\text{-}45)$$

This modification factor is a function of the slide geometry and the strength parameters of the soil. Figure 5-22 illustrates the variation of the f_o value as a function of the slope geometry (*d* and *L*) and type of soil.

These curves were presented by Janbu in an attempt to compensate for the assumption of negligible interslice shear forces (X_i) in his formulation for the simplified method. Janbu then performed calculations using his simplified and rigorous (that is, satisfying complete equilibrium) methods for the same slopes with homogenous soil conditions. The subsequent comparison between the simplified and rigorous FS values was used to develop the correction curves shown in Figure 5-22.

There is no consensus concerning the selection of the appropriate f_o value for a surface intersecting different soil types consisting of *c*-only, ϕ-only, and *c*-ϕ soils. In cases where such a variety of soils are present, the *c*-ϕ curve is generally used to correct the calculated FS value. For convenience, this modification factor can also be calculated according to the formula:

$$f_o = 1.0 + b_1 \left[\frac{d}{L} - 1.4 \left(\frac{d}{L} \right)^2 \right] \qquad (5\text{-}46)$$

where b_1 varies according to the soil type:

c only soils: $b_1 = 0.69$
ϕ only soils: $b_1 = 0.31$
c and ϕ soils: $b_1 = 0.50$

The appropriate b_1 value is selected for use in equation 5-46, according to the type (that is, c only, ϕ only, or both c and ϕ) of soil encountered *along* the analyzed failure surface. If a mixed soil-type is encountered, use the c and ϕ soil relationship described by the above expression.

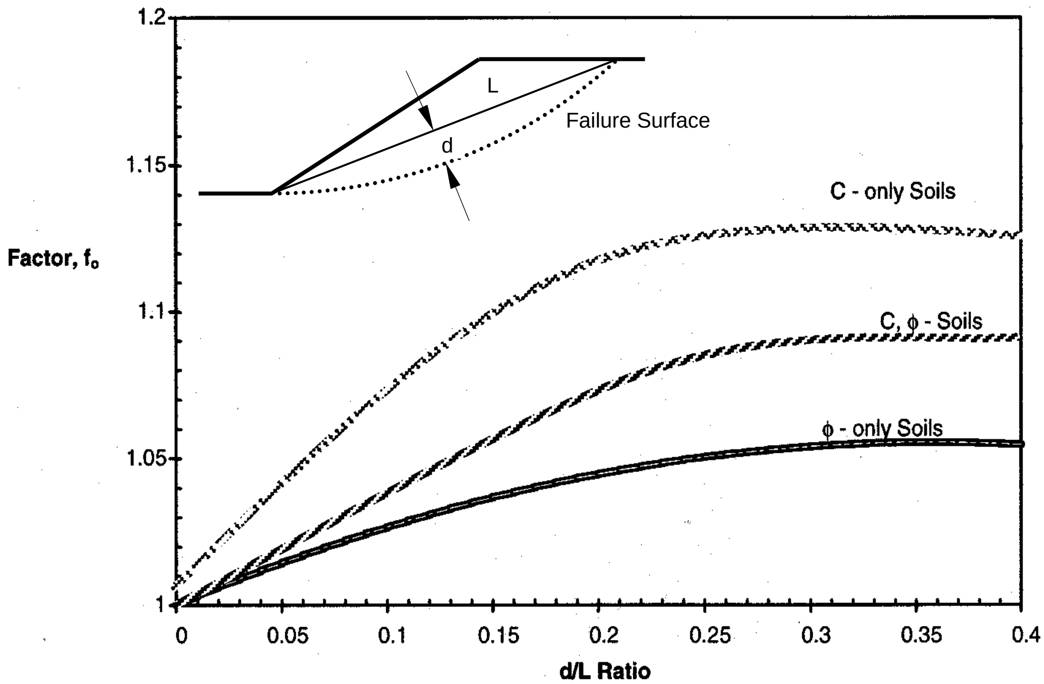


Figure 5-22: Janbu's Correction Factor for the Simplified Method

5.9.3 Simplified Bishop Method

The simplified Bishop Method also uses the method of slices to discretize the soil mass for determining the factor of safety. This method satisfies vertical force equilibrium for each slice and overall moment equilibrium about the center of the *circular* trial surface. The simplified Bishop Method also assumes zero interslice shear forces. Using the notation shown in Figure 5-21, the overall moment equilibrium of the forces acting on each slice is given by:

$$
\begin{aligned}
\sum M_o = & \sum_{i=1}^{n} [\Delta W(1-k_v) + \Delta U_\beta \cos\beta + \Delta Q \cos\delta]\, R \sin\alpha \\
& - \sum_{i=1}^{n} [\Delta U_\beta \sin\beta + \Delta Q \sin\delta]\,(R\cos\alpha - h) \\
& - \sum_{i=1}^{n} [\Delta S]R + \sum_{i=1}^{n} [k_h \Delta W (R\cos\alpha - h_c])\\
= & \ 0
\end{aligned} \tag{5-47}
$$

where R is the radius of the circular failure surface, h is the average height of the slice and h_c is the vertical height between center of the base slice and the centroid of the slice. The above equation may be simplified by dividing throughout by the radius to get:

$$
\begin{aligned}
\frac{\sum M_o}{R} = & \sum_{1}^{n} [\Delta W(1-k_v) + \Delta U_\beta \cos\beta + \Delta Q \cos\delta]\ \sin\alpha \\
& + \sum_{1}^{n} [k_h \Delta W (\cos\alpha - \frac{h_c}{R}) \\
& - \sum_{i=1}^{n} [\Delta S] - \sum_{i=1}^{n} [\Delta U_\beta \sin\beta + \Delta Q \sin\delta](\cos\alpha - \frac{h}{R})
\end{aligned} \tag{5-48}
$$

Please note that the effective normal and pore pressure forces, acting on the base of the slice, do not affect the moment equilibrium expression since they are directed through the center of the circle. Thus, Bishop's Method should not be used to compute a Factor of Safety for noncircular surfaces.

If the FS is assumed to be the same for all slices, substitute the Mohr-Coulomb criterion from equation 5-38 into equation 5-49 to give:

$$
F = \frac{\sum_{i=1}^{n} (\Delta C + \Delta N' \tan\phi)}{\sum_{i=1}^{n} A_5 - \sum_{i=1}^{n} A_6 + \sum_{i=1}^{n} A_7} \tag{5-49}
$$

where:

$$
\begin{aligned}
A_5 &= [\Delta W(1-k_v) + \Delta U_\beta \cos\beta + \Delta Q \cos\delta] \sin\alpha \\
A_6 &= (\Delta U_\beta \sin\beta + \Delta Q \sin\delta)(\cos\alpha - h/R) \\
A_7 &= k_h \Delta W (\cos\alpha\ \frac{h_c}{R})
\end{aligned} \tag{5-50}
$$

Next, forces are summed in the vertical direction for each slice to determine the effective normal force in the same manner as used for Janbu's Method,

$$\Delta N' = \frac{1}{m_\alpha}\left[\Delta W(1 - k_v) - \frac{\Delta C \sin\alpha}{F} - \Delta U_\alpha \cos\alpha + \Delta U_\beta \cos\beta + \Delta Q \cos\delta\right] \qquad (5\text{-}51)$$

where m_α is given by:

$$m_\alpha = \cos\alpha\left[1 + \frac{\tan\alpha \tan\phi}{F}\right] \qquad (5\text{-}52)$$

Equations 5-49 to 5-52 are the expressions that are used to calculate the factor of safety for circular surfaces, according to the simplified Bishop Method.

5.9.4 Spencer's Method

This method was originally presented in 1967 for calculating the FS for circular failure surfaces. Later, Wright (1969 and 1974) and Spencer (1973) extended the method to calculate the FS of any general, arbitrarily shaped failure surface. The method ensures force and moment equilibrium for a common inter-slice force angle, θ, for all slices. The formulation that is used in the interactive program uses an extension of the theoretical development presented by Chugh (1981a).

Force Equilibrium

Rather than considering the normal and shear forces at the vertical slice interfaces separately, as shown in Figure 5-21, Spencer's Method includes their effect via inter-slice resultant forces defined as Z_L and Z_R acting on the left and right side of each slice. These inter-slice forces are inclined at a constant angle $\theta_L = \theta_R$, as shown in Figure 5-23.

The inter slice forces are *total* forces as the hydrostatic component along the inter-slice boundaries is not considered separately. Inter-slice hydrostatic forces can be considered in an analysis but are difficult to implement for layered soils. If force equilibrium is considered in a direction parallel to the base of each slice:

$$\begin{aligned}\Delta S + (Z_L - Z_R)\cos(\alpha - \theta) &= \Delta W(1 - k_v)\sin\alpha + \Delta W k_h \cos\alpha \\ &\quad + \Delta U_\beta \sin(\alpha - \beta) + \Delta Q \sin(\alpha - \delta)\end{aligned} \qquad (5\text{-}53)$$

and if the Mohr-Coulomb strength criterion is adopted,

$$\Delta S = \frac{S_a}{F} = \frac{\Delta C}{F} + N'\frac{\tan\phi}{F} = C_m + \Delta N' \tan\phi_m \qquad (5\text{-}54)$$

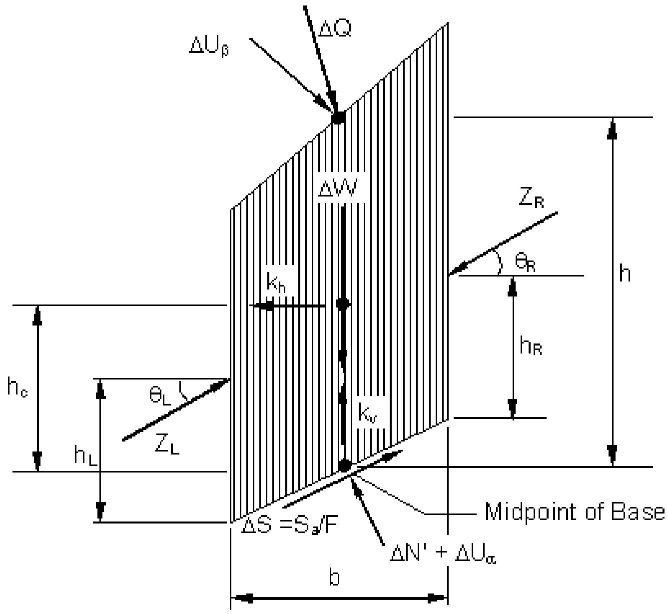


Figure 5-23: Dimensions Used for Moment Equilibrium (after FHWA, 1994)

Then, by substituting equation 5-55 into equation 5-54, the following expression is derived:

$$\begin{aligned}\Delta N' \tan \phi_m &= (Z_L - Z_R) \cos (\alpha - \theta) \\ &+ \Delta W [(1 - k_v) \sin \alpha + k_h \cos \alpha] \\ &- C_m + \Delta U_\beta \sin (\alpha - \beta) + \Delta Q \sin (\alpha - \delta)\end{aligned} \quad (5\text{-}55)$$

Next force equilibrium is formulated in a direction normal to the base of the slice:

$$\begin{aligned}\Delta N' &= (Z_L - Z_R) \sin (\alpha - \theta) - \Delta U_\alpha - \Delta W\, k_h \sin \alpha \\ &+ \Delta W (1 - k_v) \cos \alpha + \Delta U_\beta \cos (\alpha - \beta) + \Delta Q \cos (\alpha - \delta)\end{aligned} \quad (5\text{-}56)$$

By substituting equation 5-56 into equation 5-55, the following force equilibrium equation may be formulated:

$$\begin{aligned}Z_R = Z_L + A_8 \{ & W \cos \alpha (1 - k_v) (\tan \phi_m - \tan \alpha) \\ & + C_m - W \cos \alpha\, k_h (1 + \tan \phi_m \tan \alpha) - U_\alpha \tan \phi_m \\ & + U_\beta [\cos (\alpha - \beta) \tan \phi_m - \sin (\alpha - \beta)] \\ & + Q [\cos (\alpha - \delta) \tan \phi_m - \sin (\alpha - \delta)]\}\end{aligned} \quad (5\text{-}57)$$

where the factor A_8 is given by:

$$A_8 = \frac{1}{\cos (\alpha - \theta) [1 + \tan \phi_m \tan (\alpha - \theta)]} \quad (5\text{-}58)$$

Moment Equilibrium

The conditions for moment equilibrium are satisfied by taking moments of all forces about the midpoint of the base of the slice, as shown in Figure 5-23, which generates the following expression:

$$\begin{aligned} & Z_L \cos\theta \left[h_L - \frac{b}{2}\tan\alpha \right] + Z_L \frac{b}{2}\sin\theta \\ & + Z_R \frac{b}{2}\sin\theta - Z_R \cos\theta \left[h_R + \frac{b}{2}\tan\alpha \right] - \Delta W\, k_h\, h_c \\ & + \Delta U_\beta\, h \sin\beta + \Delta Q\, h \sin\delta = 0 \end{aligned} \tag{5-59}$$

The above expression is simplified to determine the location of the inter-slice force (h_R) on the right-hand-side of each slice using the following:

$$\begin{aligned} h_R = & \frac{Z_L}{Z_R} h_L + \frac{b}{2}\left[\tan\theta - \tan\alpha \right]\left[1 + \frac{Z_L}{Z_R} \right] \\ & + \frac{1}{Z_R \cos\theta}\left[-k_h\, \Delta W\, h_c + h\left(\Delta U_\beta \sin\beta + \Delta Q \sin\delta \right) \right] \end{aligned} \tag{5-60}$$

Solution Procedure

The Factor of Safety (FS) is calculated using an iterative procedure that varies F and θ until force and moment equilibrium conditions are satisfied for all slices. The solution procedure is based on the iterative procedure proposed by Spencer (1973). Alternatively, a numerical/graphical technique may also be used to obtain the final FS value (Chugh, 1981a). Users should ensure that the selected solution procedure iterate to the correct FS or θ without generating misleading answers (Chugh, 1981a).

Similar solution procedure (with a constant inter slice force angle) such as that outline in the following section – Generalized Limit Equilibrium Method can also be followed to obtain FS for Spencer's Method.

5.9.5 Generalized Limit Equilibrium (GLE) Method

In Spencer's procedures, the inter-slice force angle θ is unknown but assumed to be constant. Chugh (1986) generalized this procedure by varying the inter-slice force angle with the function:

$$\theta_r = \lambda\, f(x_i) \tag{5-61}$$

where θ_r is the right hand side inter slice force angle (see Figure 5-23) of slice i and x_i is the coordinate of the right side of slice i. The actual value of the function, $f(x_i)$ for the slide is mostly a choice based on experience and typically ranges from 0 to 1. Figure 5-24 shows the typical function distribution from toe to crest of the slide. λ is a scalar which is treated as an unknown.

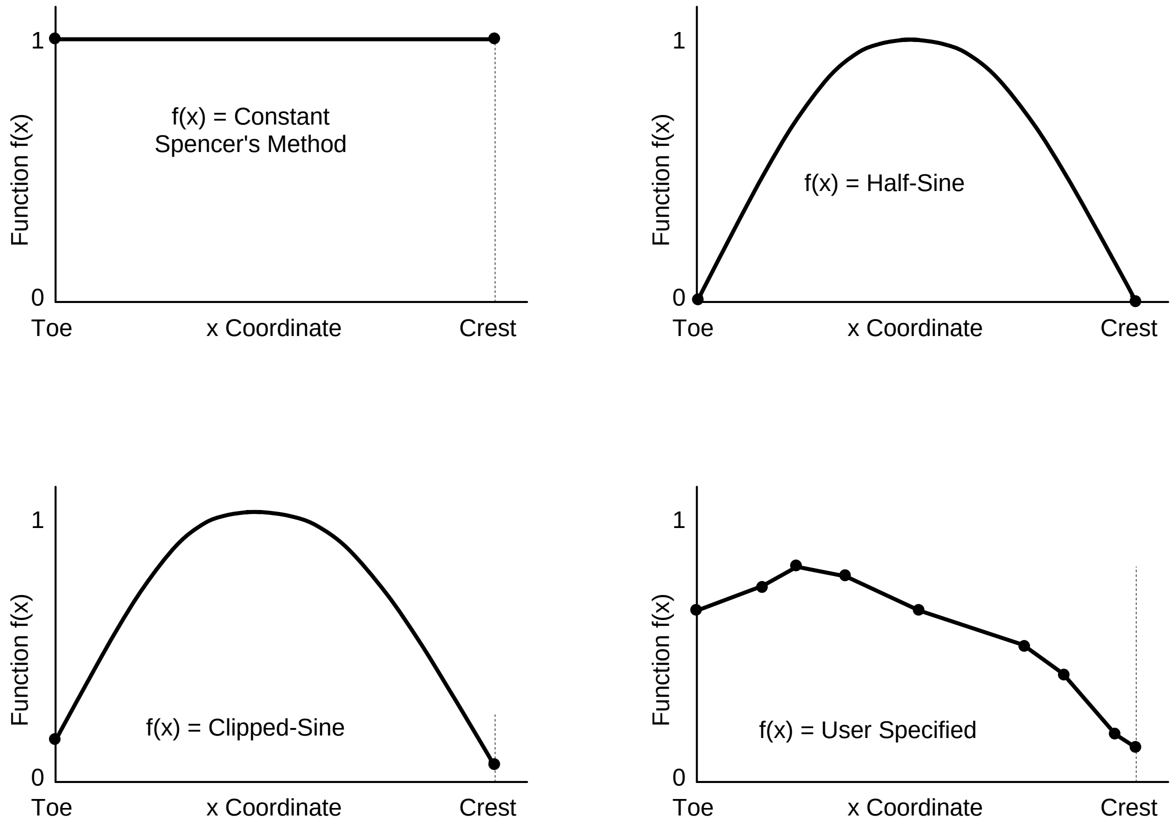


Figure 5-24: Examples of Interslice Force Angle Function $f(x_i)$ (After Fredlund et al. 1981)

<u>Force Equilibrium</u>

The inter-slice forces in the GLE method are total forces which include the hydrostatic components. Assuming force equilibrium is in a direction parallel to the base of each slice and with force components and geometry shown in Figures 5-21 and 5-23, then

$$\begin{aligned} &\Delta S + Z_L \cos(\alpha - \theta_L) - Z_R \cos(\alpha - \theta_R) - \Delta W (1 - k_v) \sin\alpha \\ &- \Delta W k_h \cos\alpha - \Delta U_\beta \sin(\alpha - \beta) - \Delta Q \sin(\alpha - \delta) = 0 \end{aligned} \tag{5-62}$$

Substituting in the Mohr-Coulomb strength criterion (Equation 5-55), then Equation 5-62 becomes:

$$\begin{aligned} \Delta N' \tan\phi_m = {} & Z_R \cos(\alpha - \theta_R) - Z_L \cos(\alpha - \theta_L) \\ & + \Delta W [(1 - k_v)\sin\alpha + k_h \cos\alpha] - C_m \\ & + \Delta U_\beta \sin(\alpha - \beta) + \Delta Q \sin(\alpha - \delta) \end{aligned} \tag{5-63}$$

In a direction normal to the base of the slice, the force equilibrium is then

$$\begin{aligned} &\Delta N' + Z_R \sin(\alpha - \theta_R) - Z_L \sin(\alpha - \theta_L) - \Delta W (1 - k_v) \cos\alpha \\ &+ \Delta W k_h \sin\alpha + \Delta U_\alpha - \Delta U_\beta \cos(\alpha - \beta) - \Delta Q \cos(\alpha - \delta) = 0 \end{aligned} \tag{5-64}$$

By combining Equations 5-63 and 5-64, the total force equilibrium then becomes:

$$
\begin{aligned}
Z_R \;&= A_9 \{Z_L [\cos(\alpha - \theta_L) + \sin(\alpha - \theta_L) \tan \phi_m] \\
&+ \Delta W \cos \alpha \,(1 - k_v)(\tan \phi_m - \tan \alpha\_ + C_m \\
&- \Delta U_\alpha \tan \phi_m - \Delta W k_h (1 + \tan \phi_m \tan \alpha) \cos \alpha \\
&+ \Delta U_\beta [\cos(\alpha - \beta) \tan \phi_m - \sin(\alpha - \beta)) \\
&+ \Delta Q [\cos(\alpha - \delta) \tan \phi_m - \sin(\alpha - \delta)\}
\end{aligned}
\qquad (5\text{-}65)
$$

and

$$
A_9 = \frac{1}{\cos(\alpha - \theta_R)\,[1 + \tan \phi_m \tan(\alpha - \theta_R)]} \qquad (5\text{-}66)
$$

<u>Moment Equilibrium</u>

By taking moment at the base of the slice, with the geometry shown on Figure 5-23, then

$$
\begin{aligned}
&Z_L \cos \theta_L \left(h_L - \frac{b}{2} \tan \alpha \right) + Z_L \frac{b}{2} \sin \theta_L - Z_R \cos \theta_R \left(h_R - \frac{b}{2} \tan \alpha \right) \\
&+ Z_R \frac{b}{2} \sin \theta_R - \Delta W k_h h_c + \Delta U_\beta h \sin \beta + \Delta Q h \sin \delta = 0
\end{aligned}
\qquad (5\text{-}67)
$$

Rearranging Equation 5-67 to solve for h_R:

$$
\begin{aligned}
h_R \;&= \frac{Z_L}{Z_R \cos \theta_R} \left[h_L \cos \theta_L - \frac{b}{2} (\cos \theta_L \tan \alpha + \sin \theta_L) \right] \\
&+ \frac{1}{Z_R \cos \theta_R} [h (\Delta U_\beta \sin \beta + \Delta Q \sin \delta) - h_c k_h \Delta W] \\
&+ \frac{b}{2} (\tan \theta_R - \tan \alpha)
\end{aligned}
\qquad (5\text{-}68)
$$

Complete moment and force equilibrium can then be solved by iterating Equations 5-65 and 5-68. Once the factor of safety has been determined, the total normal (σ_n), vertical (σ_v), and shear (τ_b) at the base of each slice can be estimated by:

$$
\begin{aligned}
\sigma_n \;&= \frac{1}{b \sec \alpha} \{Z_L \sin(\alpha - \theta_L) - Z_R \sin(\alpha - \theta_R) + \Delta U_\beta \cos(\alpha - \beta) \\
&- \Delta U_\alpha + \Delta W[(1 - k_v) \cos \alpha - k_h \sin \alpha] + \Delta Q \cos(\alpha - \beta)
\end{aligned}
\qquad (5\text{-}69)
$$

$$
\sigma_v = \frac{\Delta W + \Delta Q \cos \delta + \Delta U_\beta}{b \sec \alpha} \qquad (5\text{-}70)
$$

$$
\tau_b = C_m + \sigma'_n \tan \delta_m \qquad (5\text{-}71)
$$

Solution Procedure

The GLE factor of safety can be estimated based on the following procedure:

1. Select an initial inter-slice force angle distribution with θ_L for the first slice (at toe) and θ_R for the last slice (at crest) set to zero.
2. Calculate F such that Equations 5-65 and 5-68 are satisfied and that Z_R for the last slice is equal to the boundary force. This force will be equal to the hydrostatic force in a water-filled crack at the crest of the slope. If there is no water-filled crack this boundary force will be zero.
3. Record the calculated inter-slice forces Z_R and Z_L.
4. Use Equation 5-68 to calculate the inter-slice force angles, θ_R. Note that h_R for the last slice is zero or equal to the location of the horizontal hydrostatic force in a water-filled crack. The calculation should be conducted sequentially starting from the first slice with θ_L and h_L equal to zero.
5. Repeat steps 2 to 4, until the successive calculated factors of safety and inter-slice force angles are within an acceptable limit.
6. Evaluate the reasonableness of the final factor of safety by examining the total normal, vertical, and shear stresses at the base of each slice calculated by using Equations 5-69 to 5-71.
7. The entire procedure can be repeated with a different inter-slice force angle distribution function.

In the field, the effective stresses within a slope control the shape and location of the slip surface (Simons and Menzies, 1974). However, for limit equilibrium analysis, effective stresses are calculated at the pre-selected surface and the associated inter-slice boundaries. This may lead to incompatibility with the actual condition. This is particularly true with the rigorous methods (such as GLE, Spencer Janbu's GPS and Morgenstern & Price) that satisfied complete equilibrium with their extensive assumptions. Because of this potential incompatibility with the real world, it is necessary to examine the factors of safety generated from these rigorous methods for their reasonableness. To assist in evaluating the reasonableness of the factors of safety, the associated thrust line, stress and force distribution at the shear surface and at the inter-slice boundaries should be examined to check if any assumptions have been violated.

If an unrealistic factor of safety is generated, it is because an unrealistic surface was used to model the potential failure surface. Often, the insertion of a crack will lead to a more realistic result for all rigorous methods.

5.9.6 Janbu's Generalized Procedure of Slices (GPS)

In the GPS procedure, moment equilibrium is satisfied for all slices. Force equilibrium is also satisfied except for the last slice. Borrowed from the GLE procedure, Equations 5-65 and 5-66 can be used to satisfy the force equilibrium, and Equation 5-67 for the moment equilibrium. In the GPS method, the location of the thrust line is assumed and the equilibrium is satisfied by varying the inter-slice force angles. To maintain moment equilibrium about the center of the slice base, Equation 5-68 can be rearranged:

$$Z_R \cos\theta_R \left[h_R + \frac{b}{2}\tan\alpha\right] - Z_R \frac{b}{2}\sin\theta_R = Z_L \cos\theta_L \left[h_L - \frac{b}{2}\tan\alpha\right]$$

$$+ Z_L \frac{b}{2}\sin\theta_L - \Delta W\, k_h\, h_c + \Delta U_\beta\, h \sin\beta + \Delta Q\, h \sin\delta \qquad (5\text{-}72)$$

Equation 5-72 can be rewritten to:

$$A \sin(\psi - \theta_R) = B \tag{5-73}$$

Expanding the left hand side of Equation 5-73 gives:

$$A \sin\psi \cos\theta_R - A \cos\psi \sin\theta_R = \left[h_R + \frac{b}{2}\tan\alpha\right] \cos\theta_R - \frac{b}{2} \sin\theta_R \tag{5-74}$$

And the right hand side is equal to:

$$\begin{aligned} B &= \frac{Z_L}{Z_R}\cos\theta_L\left[h_L - \frac{b}{2}\tan\alpha\right] + \frac{b}{2}\frac{Z_L}{Z_R}\sin\theta_L \\ &- \frac{1}{Z_R}\,[\Delta W\, k_h\, h_c + U_\beta\, h \sin\beta + \Delta Q\, h \sin\delta \end{aligned} \tag{5-75}$$

θ_R can be solved by the following manipulations:

$$A \sin\psi = h_R + \frac{b}{2} \tan\alpha \tag{5-76}$$

$$A \cos\psi = \frac{b}{2} \tag{5-77}$$

By combining Equations 5-76 and 5-77, then

$$\tan\psi = \frac{2h_R}{b} + \tan\alpha \tag{5-78}$$

$$A^2 = \left(\frac{b}{2}\right)^2 + \left(h_R + \frac{b}{2}\tan\alpha\right)^2 \tag{5-79}$$

and as $$\sin(\psi - \theta_R) = \frac{B}{A} \tag{5-80}$$

Therefore, $$\theta_R = \psi - \sin^{-1}\frac{B}{A}$$

$$= \tan^{-1}\left(\frac{2h_R}{b} + \tan\alpha\right) - \sin^{-1}\frac{B}{A} \tag{5-81}$$

Z_R from Equation 5-65 can be first estimated by assuming θ_R for all the slices. Then θ_R can be recalculated using Equation 5-81 with the selected thrust line. The factor of safety and inter-slice force angle can then be obtained by iterating Equations 5-65 and 5-81. In this procedure, θ_R for the last slice is assumed to be zero, and is not calculated using Equation 5-81. Therefore force equilibrium is not explicitly satisfied for the last slice.

Solution Procedure

Janbu's GPS factors of safety may be estimated based on the following steps:

1 Assume a reasonable inter slice force angle for all inter-slice boundaries.
2 Calculate FS using Equation 5-65 with the same boundary condition requirements as in Steps 1 and 2 of the GPS solution procedure.
3 Estimate the inter-slice forces using Equation 5-65.
4 Specify the location of the thrust line and use Equation 5-81 to calculate the inter-slice force angles required for moment equilibrium.
5 Based on the last calculated inter-slice force angles, repeat steps 2 to 4 until the variation in the FS is less than, say, 0.005.

Janbu (1973) recommended that the thrust line to be located approximately a third of the slice height above the shear surface. However, within the passive zone near the toe, the thrust line height can be higher while within the active zone near the crest, it can be lower. Therefore, in general, the thrust line height should be within 0.2 to 0.4h from the potential shear surface.

5.10 CONTROL OF NEGATIVE EFFECTIVE STRESSES

The limit equilibrium methods that have been developed sometimes converge to solutions where ***negative*** normal effective stresses are calculated along the failure surface:

$$\sigma' = \frac{\Delta N'}{b \sec \alpha} = \frac{\cos \alpha}{b\, m_\alpha}\left[\Delta W(1-k_v) - \frac{\Delta C \sin \alpha}{F} - \Delta U_\alpha \cos \alpha + \Delta U_\beta \cos \beta + \Delta Q \cos \delta\right] \tag{5-82}$$

where

$$m_\alpha = \cos \alpha \left[1 + \frac{\tan \alpha \tan \phi}{F}\right] \tag{5-83}$$

Negative effective stresses are usually encountered for cases that involve: (1) ***high*** pore water pressures, (2) a combination of thin slices with a low self-weight and a high "c-value," and (3) ***steep*** slice-base angles. Most of these problems are associated with the indeterminacy of the limit equilibrium analysis and the failure to adequately satisfy the conditions for complete static equilibrium.

For the GLE method, if a modification of the excessive pore water pressures fails to eliminate the computed negative effective stress, a combination of a small ΔW and a relatively large ΔC component is the problem. In this case, the user may decide to follow one of the following suggestions:

Option-1: Proceed with analysis, but enforce the condition that the Mohr-Coulomb shear strength, $\tau = c' + \sigma' \tan \phi$, is ***always*** greater than or equal to zero.

Option-2: Proceed with analysis without any restrictive conditions concerning the computed shear strength;

Option-3: Conclude that the proposed failure surface is unrealistic because a reasonable FS cannot be computed using the limit equilibrium method.

In selecting option - 1, the assumption that all slices have the same FS, such that the shear strength, ΔS, given by

$$\Delta S = \frac{\Delta C + \Delta N' \tan\phi}{F} \geq 0 \qquad (5\text{-}84)$$

will be violated if ΔS is arbitrarily adjusted to ensure that it is not less than zero. This restriction implies that for the cases where ΔS would have been less than zero, the FS for the slice is implicitly increased to infinity to ensure zero shear strength. Although this approach is offered as an option for practical convenience, it is important that readers recognize the implication of these assumptions. The alternative option (2) does not violate the limit equilibrium assumption of a constant FS for all slices. Thus, it will always generate the lowest factors of safety as the strength mobilized along the failure surface will be a minimum because ΔS may be calculated to be less than zero for some slices.

Note that the FS values resulting from the selection of options (1) or (2) will be identical if values of $\Delta S < 0$ are not computed during the analysis. For other cases where $\Delta S < 0$ is computed, the FS values will be different depending on whether the user selects option (1) or (2). Interestingly, it is also theoretically possible that if the ϕ-angle for the offending layer is increased, the FS may be ***reduced*** because of the mobilization of *negative* frictional strength.

5.11 COMPARISON OF LIMIT EQUILIBRIUM METHODS

The simplified Bishop and Janbu Methods for slope stability analysis have been used extensively since their introduction in the 1950s. Although Bishop's Method fails to satisfy horizontal force equilibrium and Janbu's Method does not satisfy moment equilibrium, a factor of safety can be readily calculated for most slopes. However, these FS values may generally differ as much as ±15 percent upon comparison with results calculated using procedures that satisfy complete force and moment equilibrium, for example, Spencer's Method or the Morgenstern-Price Method.

The limit equilibrium formulation leads to a statically indeterminate solution, and one cannot directly compare these results with a closed-form *correct* solution. Although a direct comparison between the different methods is not always possible, **the FS value determined using Bishop's Simplified Method for circular surfaces can be expected to differ by less than 5 percent with respect to the more rigorous Spencer or Morgenstern-Price solutions.** The Simplified Janbu Method used for noncircular failure surfaces generally *underestimates* the FS by as much as 30 percent with respect to the more rigorous methods. However, for some slopes, the Simplified Janbu Method may also *overestimate* the FS value by as much as 5 percent.

An example comparing the different limit equilibrium methods has been presented by Fredlund and Krahn (1977). The failure surface in the slope shown in Figure 5-25 was analyzed and the results are presented in Figure 5-26 as a function of lambda (λ), which is the ratio of the normal and shear forces acting along the vertical slice boundaries. One can see from this figure that the Spencer and Morgenstern-Price solutions are in good agreement with the simplified Bishop result, whereas the simplified and rigorous Janbu FS values appear to be slightly lower. The two curves labeled F_m and F_f represent the loci of the points corresponding to the FS and λ value in satisfying static moment or force equilibrium, respectively. Generally methods based on equilibrium of moment yield more "accurate" results then methods based on force equilibrium. This also explains why the curve F_m is relatively flat. The intersection of these two curves provides a unique combination of FS and λ that satisfies *complete* static equilibrium within the context of the implied assumptions. For a more complete discussion, and comparison, of the slope analysis methods, the reader should refer to the paper by Fredlund and Krahn (1977).

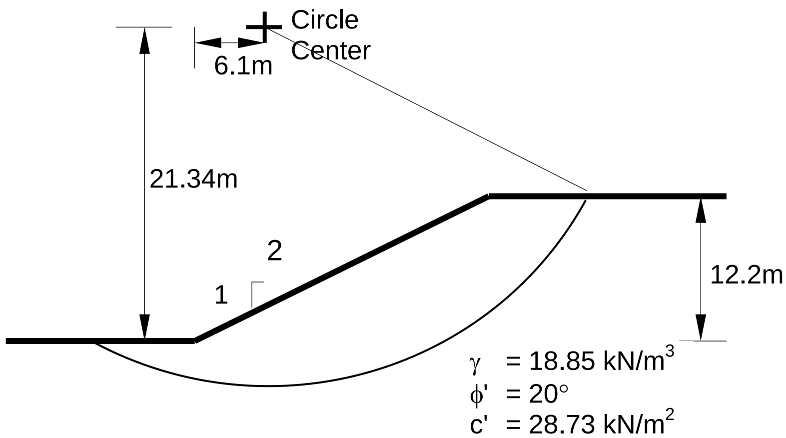


Figure 5-25: Example Slope Used For Comparison Of Limit Equilibrium Methods (Fredlund and Krahn, 1977)

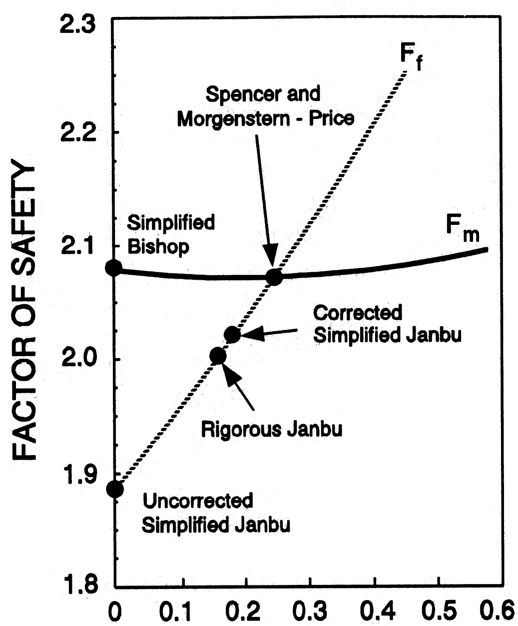


Figure 5-26: Comparison of FS Values Calculated Using Different Limit Equilibrium Methods (Fredlund and Krahn, 1977)

However, it is important to note that a more appropriate comparison of the various methods may be the minimum FS obtained from each method rather than those obtained on an arbitrarily chosen slip surface. This is because the different assumptions associated with each method may lead to different critical surfaces. If factors of safety based on an arbitrary surface are compared, the results will depend on that particular slip surface and can be misleading.

Although it has been proven difficult to find the absolutely correct method, it is possible to select methods with adequate accuracy for all practical purposes. The maximum difference between factors of safety calculated by methods that satisfy all conditions of equilibrium is about 12 percent (say, ±6 percent), usually less, for most problems. These methods can be considered correct for practical purposes since slope geometry, water pressures, unit weights and shear strength can rarely be defined within ±6 percent. Therefore, if such methods were used, the results should be as good as, if not better than the defined slope configuration being analyzed.

However, the methods that satisfy complete equilibrium are more complex and subsequently require a ***much*** greater level of understanding for successfully assessing the stability of slopes. The user should note that the numerical difficulties experienced with the Simplified Bishop (or Janbu) methods usually lead to much more serious problems in the Spencer, GLE, GPS or Morgenstern-Price procedures. These numerical problems often intensify and may lead to unreasonable FS values and an unrealistic thrust line. Such numerical difficulties generally limit the use of these more complex methods for the analysis of single surfaces only, as these methods do not appear suitable for automatic surface generation and analysis.

Duncan (1992) summarized the accuracy of the various methods as follows:

1. The basic accuracy achievable with slope stability charts is as good as the accuracy with which slope geometry, unit weights, shear strengths and pore pressures can be defined in many cases. The principal limitation of these charts is their simplified assumptions which requires approximations when applied to real problems. Nevertheless, if necessary approximations are made judiciously, accurate results can be achieved. A very effective procedure is to perform preliminary analyses using charts, and final analyses using a computer program.

The Ordinary Method Slices (OMS) is highly inaccurate for effective stress analyses of flat slopes with high pore pressures – the computed factor of safety is too low. The method is perfectly accurate for $\phi=0$ analyses, and quite accurate for any type of total stress analysis using circular slip surfaces. The method does not have numerical problems.

Bishop's Modified Method is accurate for all conditions (except when numerical problems are encountered). Its limitations are that it is applicable only to circular slip surfaces, and that it has numerical problems under some conditions. If a factor of safety is calculated using Bishop's Modified Method that is smaller than the factor of safety for the same circle calculated using the OMS, it can be concluded that there are numerical problems with the Bishop's Modified Method analysis. The OMS factor of safety is better is these cases. For this reason it is prudent to use the OMS for each circle when the Bishop's Modified Method is used, for comparison purposes.

Factors of safety calculated using force equilibrium methods are sensitive to the assumed inclinations of side forces between slices. A poor assumption regarding side force inclination can result in a computed factor of safety that is seriously in error. Like all methods that include consideration of side forces on slices, these methods have numerical problems in some cases.

Methods that satisfy all conditions of equilibrium (e.g. Janbu's, Morgenstern and Price's, GLE and Spencer's) are accurate for any conditions (except when numerical problems are encountered). The factor of safety computed using any of these methods differ by no more than about 12 percent from the factor of safety calculated by any other method that satisfies all conditions of equilibrium, and no more than about 6 percent from what can fairly be considered to be the correct answer. All of these methods have numerical problems under some conditions.

Note that as useful as the limit equilibrium methods are in analyzing the stability of slopes, there are limitations inherent to all of them. These include:

1. A constant FS is assumed along the entire failure surface. This is an obvious simplification particularly if various soil types exist along the shear surface.

The force and moment equilibrium does not consider the stress-strain relationship. Therefore, stress induced deformation is not included in the model.

5.12 SELECTION AND USE OF LIMIT EQUILIBRIUM METHODS

Prior to selecting a method for slope stability analysis, it is essential that the following four steps be followed:

1. Attempt to visualize the probable shape of the slip surface or slip surfaces. Give special attention to the existence of major discontinuities, existing slip surfaces, stratification, non-homogeneity, tension cracks, and open joints. In homogeneous soil slopes without discontinuities, assume a slip surface of circular shape, unless local experience dictates otherwise. In embankments, consider method of construction, zones of different materials, and nature of foundations in order to visualize the probable shape of the slip surfaces.

2. Distinguish clearly between first-time slides and possible renewed movements along existing slip surfaces. Rely only on the residual strength along parts of assumed slip surfaces that correspond to existing, or prior shear zones.

3. Make decisions on relative factors of safety with respect to cohesion and friction. Whenever possible, compare strength parameters from back analyses of case records with those from laboratory and field tests. Examine the reliability of data concerning strength parameters and pore water pressures. Consider the possibility of artesian pressures and perched water tables by examining significant geological details. Also consider seepage, submergence, and drawdown conditions, when appropriate.

4. Decide the use of effective or total stress types of analysis. In particular, consider the type of materials, whether analysis is for short-term or long-term conditions, whether reliable estimates of pore pressures can be made in advance, and whether pore pressures are to be monitored in the field.

5.12.1 Selection of Analysis Method

The geometry and relative locations of different soil types within the earth mass and the shapes of the slip surfaces in general, dictate the method selected for determining the stability of a slope. In many instances, especially for a major project, use several methods to assess the stability.

1. For long, uniform slopes, where the failure surface is parallel to the ground surface, simple infinite slope equations given in Section 5.5 are fairly accurate.

2. For shallow, long planar failure surfaces, which are not parallel to the ground surface, the simplified Janbu approach will give fairly accurate results. A planar failure surface can be regarded as a circular one with infinite radius.

3. For planar failure surfaces, a block analysis can be used to determine the factor of safety and critical failure surface location. Higher accuracy may be obtained by means of a generalized procedure of slices or the GLE method.

4. For surfaces that can be approximated by arcs of circles, preliminary studies are facilitated by the use of stability charts. The modified Bishop Method can be used for greater accuracy.

5. For slip surfaces of arbitrary shape, a generalized procedure of slices can be used. Janbu's simplified procedure (without inter-slice forces) and correction factors may be used for preliminary studies. For more accurate studies, Janbu's generalized procedure, Spencer's procedure, Morgenstern and Price Method, GLE or Sarma's Method are available for a more rigorous analysis.

5.12.2 Considerations for All Types of Analyses

All slope-stability analyses should consider the following points:

- Include tension cracks and open joints, if any, in the analysis. Where appropriate, assume water in these cracks.

- Make sensitivity analyses by varying one parameter at a time, for example, cohesion parameter, friction parameter, groundwater table. Plot each parameter against factor of safety.

- Where the strength envelope is curved, exercise great care in selecting appropriate c' and ϕ' values. Selected shear strength values must correspond to stress levels appropriate to the problem analyzed. For shallow slip surfaces, select c' and ϕ' from the portion of the strength envelope in the low-normal stress range; for deep slip surfaces, select them in the high-normal stress range.

- Consider the occurrence of progressive failure and especially the role of slope disturbance, tension cracks, strain-softening, non-uniform stress-strain distribution in initiating or accelerating progressive failure.

- Consider possibilities of delayed failure over time because of decrease in shear strength parameters, increase of pore pressures, and other factors.

5.13 SEISMIC SLOPE STABILITY ANALYSES

As discussed in Chapter 2, there are numerous documented cases of slope failures (landslides) generated by earthquake ground motions. In general, there are two types of seismic slope instabilities. First, the ground accelerations associated with earthquake events can induce significant inertia forces that may lead to instability and permanent deformations of natural and man-made soil slopes and embankments. Second, if soil liquefaction is induced within or underlying the slope and/or embankment during the earthquake event, overall stability failure may occur at or after the earthquake events as illustrated in Figures 2-13 and 14. Therefore, evaluation of seismic slope instabilities should include the above two categories, which are frequently referred as "inertial instabilities", and "post-liquefaction instabilities", respectively.

Similar to the static slope stability analyses, seismic slope stability analyses can be analyzed based on either the conventional limit equilibrium methods (Section 5.9) or complex numerical techniques (Section 5.15). However, to avoid cumbersome numerical modeling and analysis (Section 5.15), seismic stability and deformation of soil slopes are often analyzed using conventional slope stability methods as discussed herein briefly. Readers are referred to Chapter 7 "Seismic Slope Stability" of FHWA HI-99-012 "Geotechnical Earthquake Engineering" for detailed discussions.

5.13.1 Inertial Instability

As discussed previously, strong earthquake motions can induce significant horizontal and vertical dynamic forces in slopes. These inertial dynamic forces can be simulated as a static load (in a pseudo-static way) applied at the center of gravity of the potential failure mass. Subsequently, this static load is assumed to be proportional to the weight of the potential sliding mass times a seismic coefficient, k_s, which is expressed in terms of the acceleration of the underlying earth (g) as illustrated in Figure 5-27.

As shown in Figure 5-27, the seismic force usually is assumed to act in a horizontal and/or vertical directions, inducing an inertial force k_s times W (k_s W) in the slope in which W is the weight of the potential sliding mass. Since the inertial force is considered to be static and not a dynamic force, seismic slope stability can be analyzed using the limit equilibrium methods discussed in Section 5.9, which is frequently referred as pseudo-static limit equilibrium analysis. Therefore, this simplified approach is often termed pseudo-static analysis or seismic coefficient-factor of safety analyses. FHWA HI-99-012 "Geotechnical Earthquake Engineering" discusses procedures to obtain the appropriate seismic coefficient for a site through ground motion, site characterizations and site response analysis.

Most of the limit equilibrium methods discussed in section 5.9, in some form, suitable for performing pseudo-static seismic stability analysis. In principle, pseudo-static limit equilibrium analysis can be performed using either a total or an effective stress analysis, however, problems of estimating pore pressures induced by cyclic shearing are avoided by using a total stress analysis. Pseudo-static limit equilibrium analyses required for both seismic coefficient and permanent seismic deformation analyses are generally carried out using the same model of the slope used in the static stability analysis. The cross sections are often reinterpreted using appropriate dynamic shear strength parameters. However, even if the cross section doesn't change, the search for the critical surface, i.e., the surface with the lowest factor of safety or yield acceleration may have to be repeated because the critical surface from the static analysis is not necessarily the same as the critical surface for the dynamic analysis.

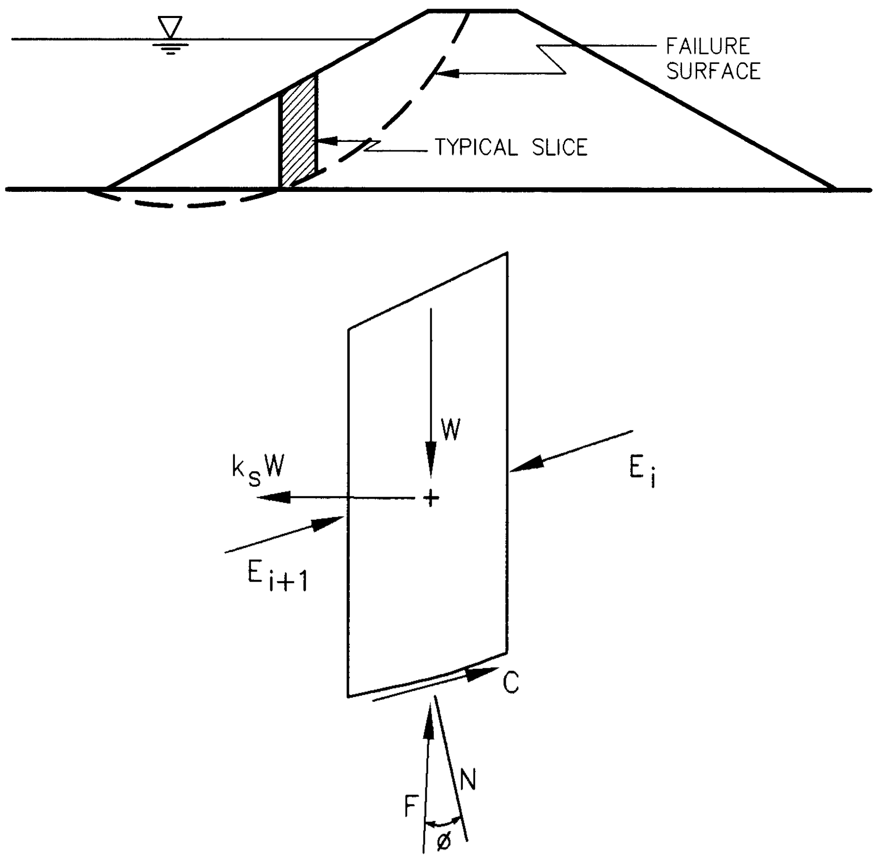


Figure 5-27: Pseudo-static Limit Equilibrium Analysis for Seismic Loads (Kavazanjian, et. al., 1999).

The seismic slope stability analysis based on this pseudo-static concept must be able to determine whether the given or proposed slope meets the safety and performance criteria, which means the seismic factor of safety must be satisfactory, and/or the horizontal and vertical deformation must be tolerable. However, as discussed previously, the limit equilibrium method only calculates the factor of safety of a slope, but offer no deformation information associated with slope failure. Permanent seismic deformation analyses for slopes and embankment are generally conducted using the Newmark method (1965) in which the failure mass is modeled as a block on a plane as shown in Figure 5-28. The calculation of permanent seismic deformations using the Newmark approach is depicted in Figure 5-28.

The shearing resistance between the potential sliding mass and the underlying soil, or between the block and the plane, is evaluated in terms of the yield acceleration, k_y, the acceleration that will reduce the factor of safety obtained in a pseudo-static analysis to 1.0 (shown as the critical coefficient on Figure 5-29). The lowest yield acceleration for all possible failure surfaces passing through the slope or embankment should be used in the Newmark analysis.

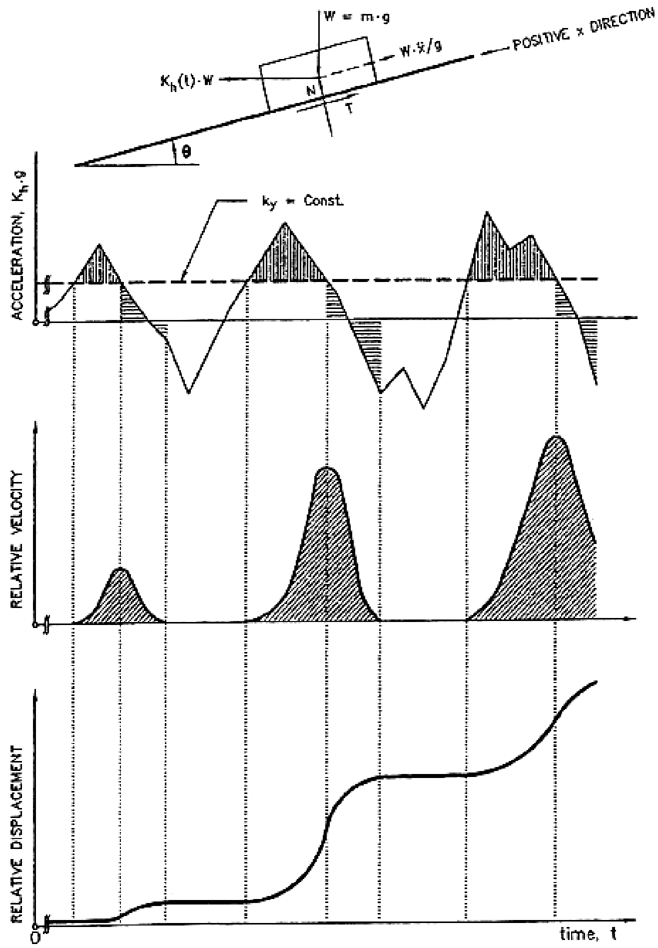


Figure 5-28: Basic Elements of Newmark Deformation Analysis (Matasovic, et al., 1997)

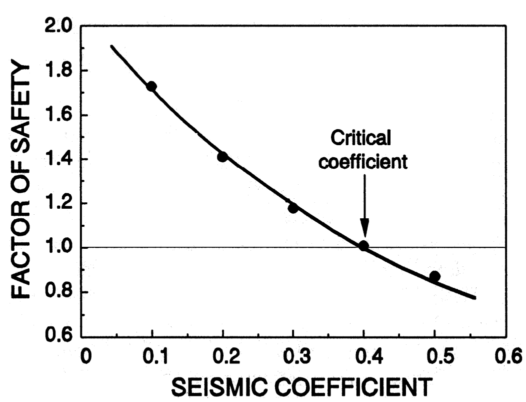


Figure 5-29: Variation of Factor of Safety with Horizontal Seismic Coefficient, k_h.

To include both pseudo-static stability and permanent deformation considerations, Chapter 7 of FHWA HI-99-012 "Geotechnical Earthquake Engineering" (Kavazanjian, et. al., 1999) proposes an unified approach by combining the seismic coefficient-factor of safety (pseudo-static stability) and permanent seismic deformation analysis methods for evaluation of seismic stability of slopes and embankments: (1)

the seismic coefficient-factor of safety approach; and (2) the permanent seismic deformation approach. First, a seismic coefficient-factor of safety analysis is performed using a suitably conservative value for the seismic coefficient. Then, if the seismic coefficient-factor of safety analysis results in an unacceptable factor of safety, a permanent seismic deformation analysis is performed. The step-by-step procedures for the unified approach (Kavazanjian, et. al., 1999) are:

Step 1: Reinterpret the cross-sections analyzed in the static stability analysis and assign appropriate dynamic residual strength parameters. In cases where it is not clear whether drained or undrained shear strength parameters are appropriate for the dynamic analysis, follow guidelines presented in Duncan (1992) or use a composite consolidated drained-consolidated undrained strength envelope proposed by the Corps of Engineers (Figure 5-30) for pervious soils and the consolidated undrained strength envelope for silts and clays. For fully saturated silts or clays of low sensitivity, multiply the undrained peak shear strength by 0.8 for the analysis. For sensitive soils, residual shear strength is often used to provide a conservative basis for design.

Step 2: Select a seismic coefficient, k_s (based upon the work of Hynes and Franklin as discussed in FHWA HI-99-012 “Geotechnical Earthquake Engineering”). If a permanent seismic deformation of 1 m (3.3 ft) is acceptable, a value of k_s equal to 0.5 A a_{max}/g, where a_{max} is peak horizontal acceleration at the ground surface, may be used for embankments. If a site response analysis has been performed to evaluate the peak average acceleration of the failure mass, a value of k_s equal to 0.17 A a_{max}/g may be used. For natural and cut slopes, where amplification effects are expected to be minimal, a value of k_s equal to 0.17 A a_{max}/g may also be used.

Step 3: Perform the pseudo-static stability analysis. If the minimum factor of safety exceeds 1.0, the seismic stability analysis is completed. (For a more critical slope or project, a higher minimum factor up to 1.2 may be warranted).

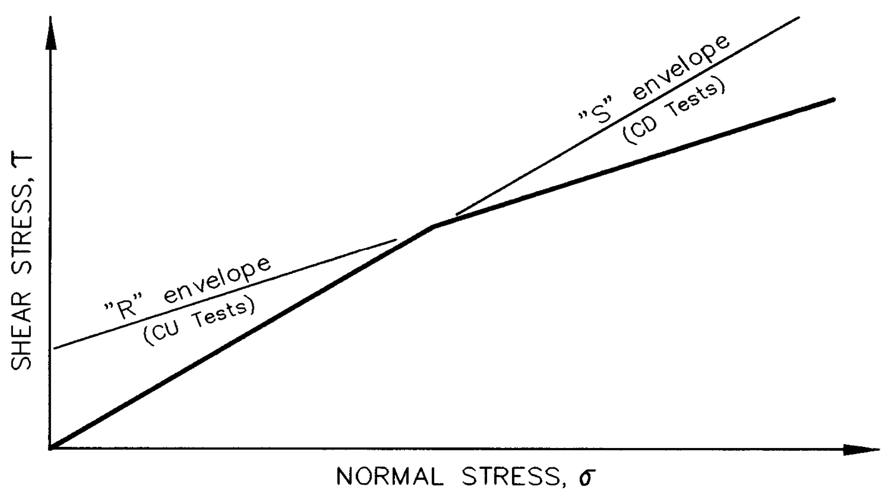


Figure 5-30: Composite Shear Strength Envelope (USACE, 1970).

Step 4: If the pseudo-static factor of safety is less than 1.0, perform a Newmark deformation analysis. This is done using the following three steps:

1) Calculate the yield acceleration, k_y. The yield acceleration is calculated using a trial and error procedure in which the critiecal seismic coefficient is varied until $FS_{min} = 1.0$ is obtained as shown in Figure 5-29.

Calculate the permanent seismic deformation. The permanent seismic deformation may be calculated using either simplified design charts (e.g., Figure 5-31) or by performing a formal time-history analysis in which the excursions of the average acceleration time history above the yield acceleration are double integrated.

3) Compare the calculated permanent seismic deformation to the allowable maximum permanent displacement, u_{max}.

Makdisi and Seed (1978) developed the seismic deformation chart shown in Figure 5-31 from the results of two-dimensional finite element analyses of embankments. This chart includes the effect of amplification of seismic motions by the embankment and provides upper and lower bounds on the permanent deformation as a function of magnitude. To calculate the permanent seismic deformation using Figure 5-31, the following procedure should be used:

1) Calculate the yield acceleration for each potential failure surface of interest using limit equilibrium analysis as described previously in this section;

2) Calculate the peak average acceleration for each failure surface of interest based upon the seismic response considerations described in FHWA HI-99-012 “Geotechnical Earthquake Engineering” (Kavazanjian, et. al., 1999);

3) Calculate the ratio of the yield acceleration to the peak average acceleration for each failure surface of interest and evaluate the permanent seismic deformation from the appropriate curve in Figure 5-31.

The results of the permanent seismic deformation analysis must be compare to the criterion established for acceptable deformations to determine if the seismic performance is satisfactory. The criterion for satisfactory performance may depend on both the system component analyzed and the geometry of the failure surface. Cut slopes may be able to sustain several meters of permanent seismic displacement without jeopardizing the structural components of a highway system. Highway embankments with approach slabs may be able to accommodate substantial deformation perpendicular to the alignment of the approach slab but may not be able to sustain significant deformation parallel to the slab alignment. Establishing how much deformation a system component can accommodate in a seismic event is usually determined by the design engineer.

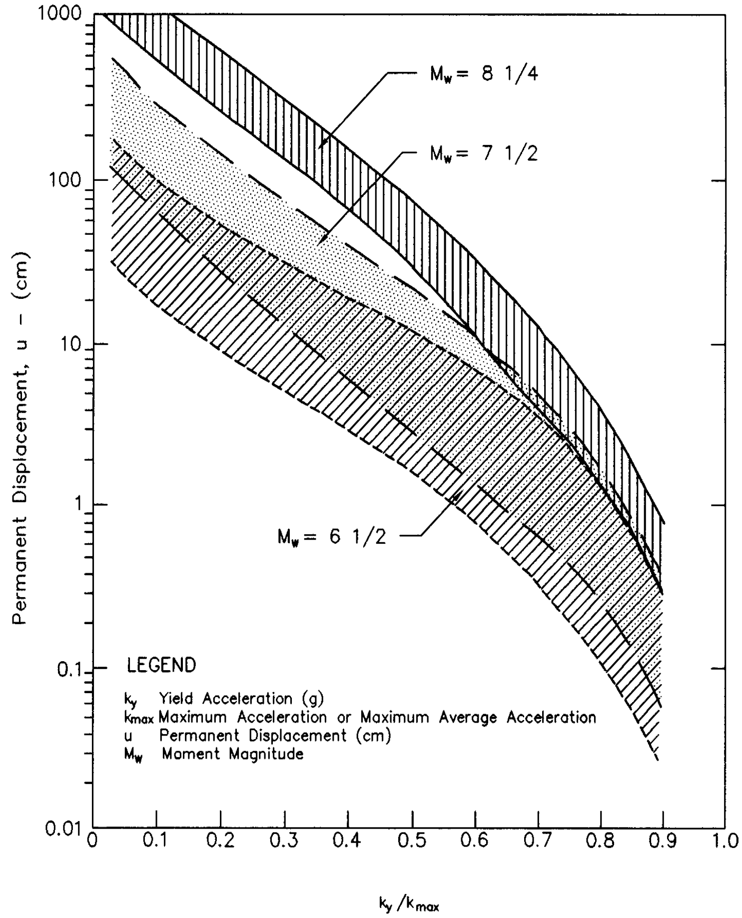


Figure 5-31: Permanent Displacement Versus Normalized Yield Acceleration for Embankments. (After Makdisi and Seed, 1978).

5.12.3 Post-Liquefaction Analysis

During strong earthquake shaking, saturated cohesionless soils may experience a sudden loss of strength and stiffness, sometime resulting in loss of bearing capacity, large permanent lateral displacements, landslides, and/or seismic settlement of the ground. Liquefaction is a major cause of structure damages and slope instabilities induced by strong ground motions. Therefore, the risk of potential liquefaction for soils within and/or underlying a soil slope is an important consideration in evaluating the overall performance of soil slope and embankment. Seed et al. (2001) presented the step-by-step procedures for post-liquefaction analysis as shown in Figure 5-32. FHWA HI-99-012 "Geotechnical Earthquake Engineering" (Kavazanjian, et. al., 1999) discusses the theory and procedures of evaluating the liquefaction potential.

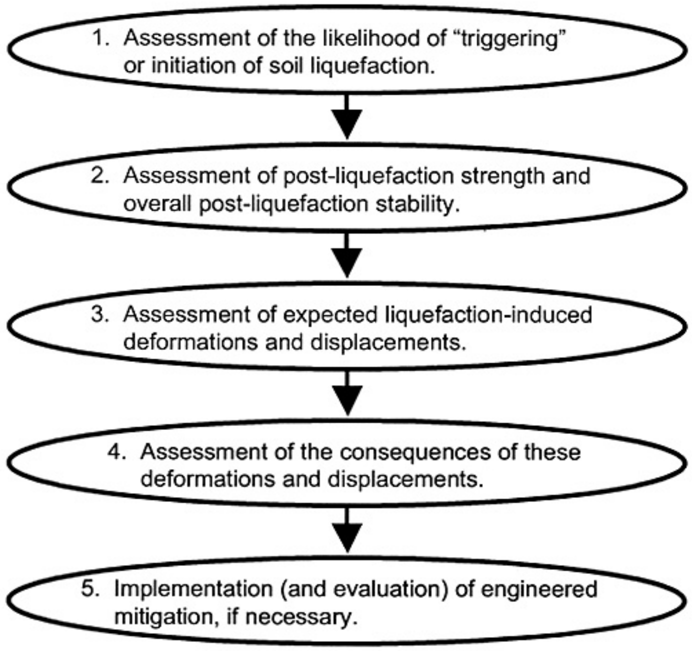


Figure 5-32: Procedures for Post-Liquefaction Analysis (Seed et al., 2001)

Liquefaction is often defined as sandy soils (in some cases, also silty and gravelly soils) loss strength and stiffness due to cyclic pore pressure generation induced by earthquake events. If liquefaction occurs, excessive stress and deformation may develop along the liquefied soils, and result in various types of slope failures as shown in Figure 2-13 and 2-14. Note that the failure type for post-liquefaction analysis can be very similar to the soils with high sensitivity or loss of strength due to monotonic shearing and/or remolding.

The potential for a liquefaction-induced slope failure as shown in Figures 2-13 and 2-14 may be analyzed using limit equilibrium analyses by employing residual shear strengths in the potentially liquefiable zones. In this type of post-earthquake stability assessment, the seismic coefficient should be set equal to zero (Marcuson, et al., 1990). If the residual shear strength is conservatively assessed using minimum values of SPT blow counts as shown in Figure 5-33 (or CPT tip resistance) within the potentially liquefiable layer(s), a factor of safety of 1.1 may be considered as acceptable. Evaluation of residual shear strength for post-liquefaction stability analyses is discussed in Chapter 8 of FHWA HI-99-012 "Geotechnical Earthquake Engineering" (Kavazanjian, et. al., 1999). Readers are also referred to Seed et al., 2001.

5.14 THREE-DIMENSIONAL (3-D) LIMIT EQUILIBRIUM ANALYSIS

As discussed in the previous sections in this chapter, the stability of slopes and embankments is typically modeled as a two-dimensional (2D) problem. However, the reality is that more often than not it is a three dimensional problem (3D). A majority of the 2D slope stability analyses are performed on sections of a slope with infinite width. The assumption that the slope may be modeled as 2D is reasonable at the center of the slide, but at the sides of the slide, shear stresses will be quite different from those at the center, because of the end restraints and a 2D model will not accurately predict the stability. Another 3D effect is the plan curvature of the slope as shown in Figure 5-34.

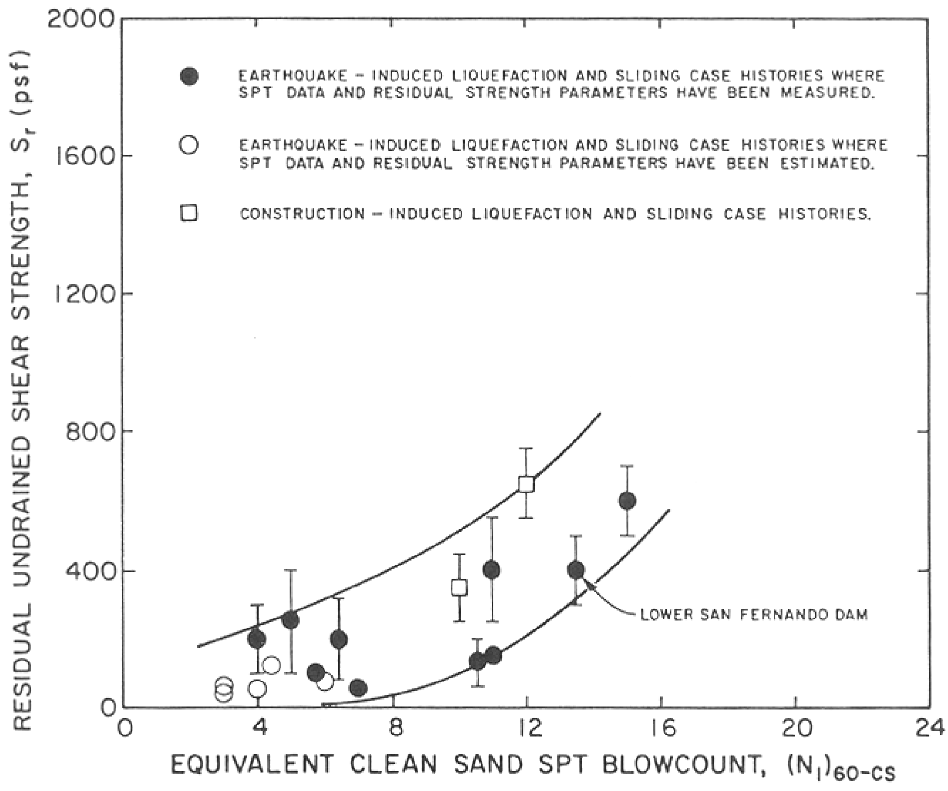


Figure 5-33: Recommended Relationship between Undrained Residual Strength and Corrected SPT Blowcounts (Seed and Harder, 1990)

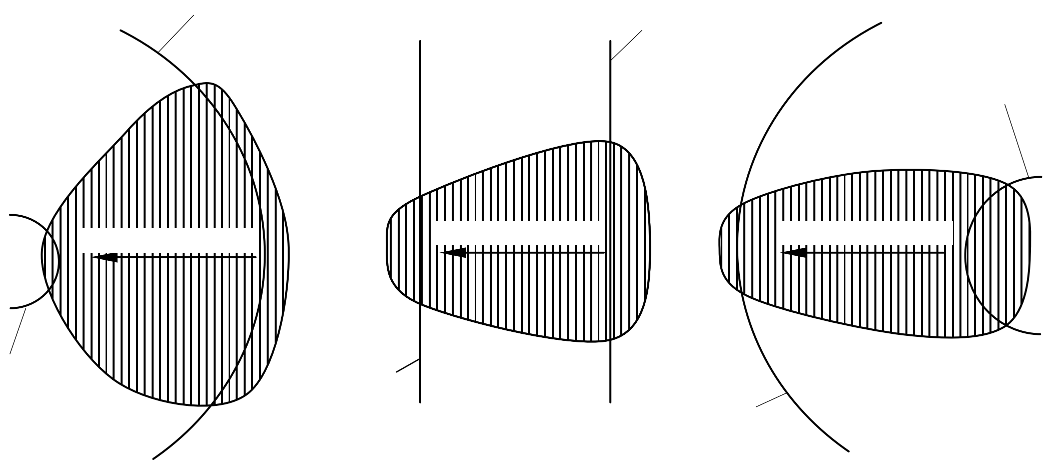


Figure 5-34: Plan Curvature of Slopes (a) Concave Slope (b) Straight Slope, (c) Convex Slope (after FHWA, 1994)

TABLE 5-6
3D SLOPE STABILITY ANALYSES SUMMARY (from Duncan 1992)

Authors	Method	Strength	Geometry of Slope/Slip Surface	3D Effects Found
Anagnosti (1969)	Extended Morgenstern and Price	C, ϕ	Unrestricted/ unrestricted	$F_3 = 1.5\ F_2$ in one case
Baligh and Azzouz (1975)	Extended circular arc	$\phi = 0$	Simple slopes/surfaces of revolution	$F_3 > F_2$
Giger and Krizek (1975)	Upper bound theory of perfect plasticity	C, ϕ	Slopes with corners/log spiral	$F_3 > F_2$
Giger and Krizek (1976)	Upper bound theory of perfect plasticity	C, ϕ	Same as above with loads on top of slope	$F_3 > F_2$
Baligh, et al. (1977)	Extended circular arc	$\phi = 0$	Simple loaded slopes/surfaces of revolution	$F_3 > F_2$
Hovland (1977)	Extended Ordinary Method of Slices	C, ϕ	Unrestricted/unrestricted	$F_3 < F_2$ for some cases
Azzouz at al. (1981)	Extended Swedish circle	$\phi = 0$	Four real embankments/surface of revolution	$F_3 = 1.07$ to $1.3\ F_2$
Chen and Chameau (1982)	Extended Spencer, and finite element	C, ϕ	Unrestricted/unrestricted	Spencer results are similar to FEM
Chen and Chameau (1983)	Extended Spencer	C, ϕ	Unrestricted/unrestricted	$F_3 < F_2$ for some cases
Azzouz and Baligh (1983)	Extended Swedish circle	$\phi = 0$	Same as Baligh and Azzouz (1975) with loads on top	$F_3 > F_2$
Dennhardt and Forster (1985)	Limit equilibrium with assumed normal stresses	C, ϕ	Slopes with loads/unrestricted	$F_3 > F_2$
Leshchinsky, et al. (1985)	Limit equilibrium and variational analysis	C, ϕ	Unrestricted	$F_3 > F_2$
Ugai (1985)	Limit equilibrium and variational analysis	$\phi = 0$	Vertical slopes/cylindrical	$F_3 > F_2$
Leshchinsky and Baker. (1986)	Limit equilibrium and variational analysis	C, ϕ	Slopes constrained in 3rd dimension/unrestricted	$F_3 > F_2$ for $c > 0$, $F_3 = F_2$ for $c = 0$
Baker and Leshchinsky (1987)	Limit equilibrium and variational analysis	C, ϕ	Conical heaps/unrestricted	$F_3 > F_2$
Cavounidis (1987)	Limit equilibrium	C, ϕ	Unrestricted/unrestricted	F_3 must be $> F_2$
Hungr (1987)	Extended Bishop's Modified	C, ϕ	Unrestricted/surfaces of revolution	$F_3 > F_2$
Gens, et al. (1988)	Extended Swedish circle	$\phi = 0$	Simple slopes/surfaces of revolution	$F_3 > F_2$
Leshchinsky and Mullett (1988)	Limit equilibrium and variational analysis	C, ϕ	Vertical slopes with corners/unrestricted	$F_3 > F_2$
Ugai (1988)	Extended Ordinary Method of Slices, Bishop's Modified, Janbu, and Spencer	C, ϕ	Unrestricted/unrestricted	$F_3 > F_2$, except for OMS
Xing (1988)	Limit equilibrium	C, ϕ	Unrestricted/ellipsoidal	$F_3 > F_2$
Michalowski (1989)	Kinematical theorem of limit plasticity	C, ϕ	Unrestricted/unrestricted	$F_3 > F_2$
Seed, et al. (1990)	Ad hoc 2D and 3D	C, ϕ	One particular case, the Kettleman Hills failure	$F_3 < F_2$
Leshchinsky and Huang (1991)	Limit equilibrium and variational analysis	C, ϕ	Unrestricted/unrestricted	$F_3 > F_2$

Slopes with concave plans are inherently more stable that those with convex plans. But concave slopes tend to impede drainage and result in higher pore pressure, and therefore, reduce the difference in factor of safety.

A large number of studies of three-dimensional slope stability problems have been performed based on the limit equilibrium methods discussed in Section 5.9. Duncan in 1992 summarized about 24 three-dimensional slope stability studies as listed in Table 5-6 and provided the following three conclusions:

It is clear the factor of safety for 3D analysis is higher than the factor of safety for 2D analysis, i.e. $F_3 > F_2$. The studies that showed otherwise, such as those by Hovland (1977), Chen and Chameau (1983) and Seed, et al. (1990), all appear to incur significant potential inaccuracies and errors. Cavounidis (1987) concluded that "Methods that give F_3/F_2 ratios which are smaller than unity either compare inappropriate factors or more probably, contain simplifying assumptions which neglect important aspects of the problem."

1. Hutchinson and Sharma (1985) and Leshchinsky and Baker (1986) suggested that for slopes in homogeneous cohesionless soils, the factors of safety generated by the 2D and 3D analyses should be the same. This is due to the fact that the critical slip surface in these soils is a shallow plane parallel to the surface of the slope.

2. Azzouz, et al. (1981), and Leshchinsky and Huang (1991) noted that, during strength back calculations, the calculated values will be too high if the 3D effects are not taken into considerations.

5.15 NUMERICAL ANALYSIS

The limit equilibrium method allows engineers to evaluate the stability of slopes quickly, however, it only provides an index (factor of safety) of the stability of a slope. It does not consider the stress-strain behaviors of the soils, and thus does not calculate the deformation of a slope. Furthermore, the stability analysis based on the limit equilibrium methods does not differentiate whether a slope is (i) of a newly constructed embankment, (ii) of a recent excavation, or (iii) of an existing natural slope. The stresses within soil slopes are strongly influenced by K_o, the ratio of lateral to vertical normal effective stresses, but conventional limit equilibrium procedures ignore this important issue (Chowdhury, 1981). In reality, the stress distributions within these mentioned conditions would be different and, therefore, significantly influence their stability.

Since 1966 when Clough and Woodward first use of numerical analysis for soil slopes and embankments, with increasing computing power, speed, and storage, and decreasing cost of personal computer, numerical analysis has gained popularity in analyzing slope and embankment problems including slope excavation, embankment construction, landslide stabilizations, reinforced slope design, etc. Numerical analysis using Finite Element Method (FEM) and Finite Difference Method (FDM) are often used to predict the static and dynamic behaviors and deformations of soil slopes and embankments, or to analyze special problems that involve in understanding soil structure interaction (SSI), sequence of construction (excavation). FEM and FDM are sometime used to address many of the deficiencies that are inherent with the limit equilibrium methods that were mentioned above. Duncan (1992) summarized the advantages of the numerical analysis such as FEM including:

FEM has been successfully used to calculate stresses, movements and pore pressures in embankments and slopes.
FEM has been used for analyses of conditions during construction, and also following construction, as consolidation or swelling occur and excess pore pressures dissipate,

FEM has been used to investigate the likelihood of cracking, hydraulic fracturing, local failure, and overall stability of slopes.
FEM is possible to model many complex conditions with a high degree of realism and account for complex geometries, a variety of loading conditions, non-linear stress-strain behavior, non-homogeneous conditions and soil structure interactive effects,
FEM can consider changes in geometry during construction of an embankment or an excavation.

For numerical analysis, the soil slope and mass to be analyzed is divided into a number of elements connected at their common nodal points. Figure 5-35 illustrates a typical example of FDM analysis.

However, the quality of the FEM is directly dependent on the ability to obtain reliable information including:

Nonlinear stress strain behavior including the initial stress conditions; the strength and nonlinear stress-strain behavior of the soils; and the sequence of construction operations or other loading conditions to be represented by the analysis.

Stress in the Soil

<u>Stress and strain parameters</u>

The stress-strain parameters for embankment designs may be collected from laboratory tests. For excavations and natural slopes, the stress-strain parameters can be developed on the basis of high quality field tests that are supported further by field observations.

Although the FEM/FDM presents a very powerful technique for the geotechnical engineer, paradoxically, it also introduces complexities that have resulted in its limited use to solve practical problems. Wong (1984) mentions the difficulty associated in developing a factor of safety against failure. In a conventional limit equilibrium approach, failure may be described as the condition where the driving forces (or moments) exceed the resisting forces (or moments) and is usually manifested by a factor of safety of less than unity. In the FEM, the soil is modeled as a set of discrete elements and the failure condition will be a progressive phenomenon where not all elements fail simultaneously. So failure can span a wide range that extends from the point where yield occurs first to the final failure state where all elements have effectively failed. Some of the popular failure criteria (Wong, 1984) are:

> ***Bulging of slope line*** (Snitbhan and Chen, 1976) that is described by the horizontal displacements of the surface of the slope. This criterion is established by specifying a maximum tolerable limit for these horizontal displacements.
>
> ***Limit shear*** (Duncan and Dunlop, 1969) condition where the computed FEM stresses along a potential failure surface are used directly to estimate a factor of safety. This FS value would correspond to the ratio of available strength along the failure surface compared to stresses calculated using the FEM.
>
> ***Nonconvergence*** (Zienkiewicz, 1971) of the solution may be indicative of a collapse of the elements under the imposed loading conditions.

Depending on which failure criterion is selected, the difference in the failure loads can be significant. More significantly, it is difficult to quantify the level of stability of a stable slope, which is always the target of design. With the lack of an obvious failure criterion, the interpretation of FEM results is still a problem and the user must rely on experience and intuition to understand the ability of the numerical model to predict the behavior of the real physical model of the slope. In view of the uncertainty and a lack of familiarity with the FEM, the complex approach is not used for the design and analysis of typical highway slopes and embankments.

The theory of the finite element and finite difference methods are not covered in this Manual and can be a separate book itself. Many computer codes and software are readily available for performing numerical analysis for soil slope and embankments as discussed in Chapter 6. In addition to the costs of computer and software, engineering time required to develop property values, to perform the computer analysis, and to evaluate the results can be substantial. Readers are referred to USACE (1995) and Duncan (1992) for detailed discussions and references.

CHAPTER 6
COMPUTER SOFTWARE FOR SLOPE STABILITY ANALYSIS

Many computer programs currently available (see Section 6.1.1 for a partial listing) for analyzing the stability of slopes are for two-dimensional (or plane strain) problems. Sections 6.2 through 6.4 introduce three computer programs, XSTABL, UTEXAS3, and ReSSA which are commonly used in highway engineering projects. Only a few are capable of handling three-dimensional problems and some of these are briefly described in Section 6.5. More sophisticated computer programs are also available for performing stability and deformation analyses of soil slopes and embankments based on finite element or finite difference method. A partial listing of the available computer programs for numerical analysis is presented in Section 6.6

Several web sites provide comprehensive listings of available computer software for soil slope stability analysis, such as the Geotechnical and Geoenvironmental Software Directory at www.ggsd.com, and Geotechnical engineering Virtual Library (GVL) at http://www.ejge.com/GVL/soft-gvl.htm, etc.

6.1 COMPUTER SOFTWARE FOR TWO-DIMENSIONAL SLOPE STABILITY ANALYSIS

All of the limit equilibrium methods discussed in Chapter 5 have been implemented within many computer programs available for personal computers. With ever-increasing computing powers and CPU speed of personal computers (PCs), slope stability analyses using more sophisticated limit equilibrium methods such as Spencer's and Janbu's methods (Section 5.9) are performed regularly by geotechnical professionals using PCs. Within a few minutes (or seconds), hundreds of possible failure surfaces can be located and checked thoroughly and accurately. Many computer programs currently available (see Section 6.1.1 for a partial listing) for analyzing the stability of slopes are for two-dimensional (or plane strain) problems.

However, as discussed in Chapter 5, computer software can only be used as a powerful tool, and results can only be as reliable as the input data that define the conditions analyzed. Duncan (1992) cautioned geotechnical engineers must have a thorough mastery of soil mechanics and soil strength (Chapter 4), a solid understanding of the computer programs they use, and the ability and patience to test and judge the results of their analyses to avoid mistakes.

As discussed in Section 5.11, when calculating factor of safety using limit equilibrium methods that satisfy all condition of equilibrium (e.g. Janbu's, Spencers', Morgenstern-Price's, etc. see Table 5-5) the uncertainties due to approximation and assumptions generally amount to 15% or less, but the margin for error in evaluating shear strengths and stresses may be considerably greater.

Duncan (1992) summarized the following three sources that result in most common mistakes in using computer analyses:

Failure of the person performing the analysis to understand soil mechanics well enough to know how to define water pressure, unit weights, and shear strengths appropriately for the analysis

Failure of the person performing the analysis to understand the computer program well enough to define these quantities correctly in the input

Failure of the person performing the analysis, and the person reviewing the results to check the results and properly evaluate their reasonableness.

6.1.1 Common Techniques for Locating Critical Surfaces

As discussed above and in Section 5.3 “Factor of Safety Concepts”, a soil slope must be designed with a minimum factor of safety greater than or equal to the recommended minimum factor of safety. At the time when the slope stability concepts were developed, it requires extensive engineering experience and substantial amount of hand-calculation time to locate the slip surface that has the lowest factor of safety, i.e. the most critical surface. Nowadays, with detailed analysis of slope stability being performed regularly using personal computers, hundreds of surfaces can be generated and examined within seconds to locate the most critical slip surface that has the lowest factor of safety.

Large number of computer techniques have been developed to automate as much of this process as possible. A variety of different procedures have been used for locating the critical circle, or the critical non-circular slip surface.

The problem of locating the critical circular slip surface is the less difficult problem. Most computer programs use systematic changes in the position of the center of the circle and the length of the radius to find the critical circle. For conditions where the geometry is complex, as in most real problems, local minimum may exist as shown in Figure 6-1. If the search command was executed focusing in a local area, and the minimum factor of safety resulted from a computer run might be a local minimum and misleading. Therefore, it is absolutely necessary to perform multiple searches using different starting points and different searching strategies, to be sure that the overall minimum value of FS has been found.

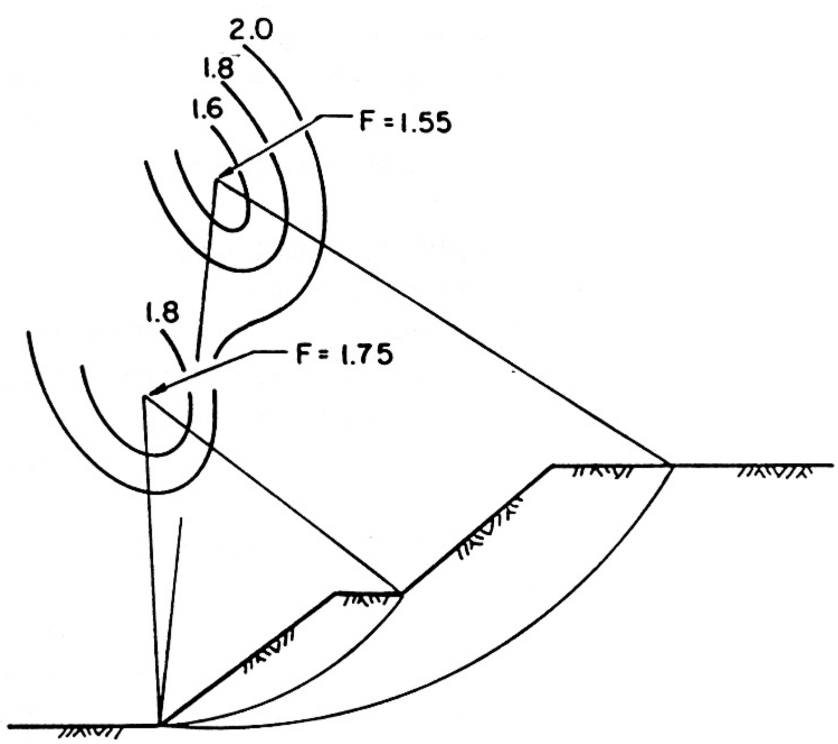


Figure 6-1: Slope with Complex Factor of Safety Contours and Local Minimum FS (Duncan, 1987)

The problem of locating the critical noncircular surface is more complex. Most software programs are applicable to any method of analysis that can be used to calculate the factor of safety for noncircular slip surface, and a variety of different approaches have been developed and used. Engineering judgment is required to select the approximate depth and length of the base slide for the initial search. Most importantly, enough trial surfaces along each possible depth must be analyzed to determine the minimum factor of safety.

6.1.2 Two-Dimensional Slope Stability Computer Programs

Table 6.1 presents a list of the representative 2D slope stability computer programs for slope stability analyses based on the limit equilibrium methods. They have been classified into four major groups based on where the programs were first developed (Purdue University, University of Texas, University of California, and independent commercially developed programs).

TABLE 6-1
LISTING OF REPRESENTATIVE 2D SLOPE STABILITY ANALYSIS PROGRAMS

Program Groups	Comments
STABL Programs: PC_STABLE, PCTABL 5M, XSTABL and GEOSLOPE	These are the subsequent versions which are a superset of the initial program originated from Purdue University in 1975 (Siegel, 1975). The latest version XSTABL includes Simplified Bishop and Janbu, Spencer, rigorus Janbu, Lowe-Karafiath, and other methods based on extension of Spencer's method.
SSTAB2, UTEXAS, UTEXAS2, and UTEXAS3	These are programs from the University of Texas. They are originated from SSTAB1. The latest version UTEXAS3 offers Spencer, simplified Bishop, US Army Corps of Engineers' Modified Swedish method and Lowe-Karafiath's method.
STABR, STABGM, SLOPE8R and GEOSOFT	These are the programs from the University of California at Berkeley. They incorporate at least one of the following analysis methods: Ordinary Method of Slices, Simplified Bishop and Spencer.
C-SLOPE, SLOPE/W, ReSSA, CLARA, GALENA, GSLOPE, and TSLOPE	These are independent commercially developed programs. Slope-W, one of the most comprehensive programs, includes methods of Fellenius (OMS), Simplified Bishop, Simplified Janbu, Spencer, Morgenstern-Price, Corps of Engineers, Lowe-Karafiath, General Limit Equilibrium. It can also determine the FS based on finite element (from a companion program SIGMA/W) computed stresses. ReSSA incorporates metallic and geosynthetic reinforcement using Bishop and Spencer methods.

The above compilation is only a limited summary listing of the majority of currently available domestic computer programs. Many other available programs that are written abroad, however, are not included due to limited usage in the United States.

The programs **XSTABL** and **UTEXAS3** are widely used by geotechnical engineers for transportation related projects. **XSTABL** is a DOS-based computer software program selected by Federal Highway Administration (FHWA) for stability analysis based on the limit equilibrium methods discussed in Chapter 5. In addition, FHWA also recommends **ReSSA** (see FHWA-NHI-00-0043), which is a Windows-based slope stability program using limit equilibrium methods that has the capabilities to analyze slopes and embankments both with and without soil reinforcement (see Chapter 7).

In view of the programs' popularity and availability to State DOT's, **XSTABLE**, **UTEXAS3**, and **ReSSA** are discussed in detail in Sections 6.2 through 6.4. A comparison of the **XSTABL** and **UTEXAS3** programs is presented in Section 6.3.6

6.2 XSTABL SOFTWARE

XSTABL is a comprehensive slope stability analysis program developed by Dr. S. Sharma based on the STABL programs originally developed at Purdue University. It performs a two dimensional limit equilibrium analysis to compute the factor of safety for a layered slope. **XSTABL** (Version 5) is a DOS-based program which provides an integrated environment for performing slope stability analyses on an IBM personal computer, or compatible. The program combines an intuitive user-friendly interface and the analytical features of the popular slope stability program, STABL. Graphical screen plots may be saved for later printing using a separate word processing program such as Microsoft Word. The menu-driven interface provides intuitive access to descriptive data tables that allow the user to enter, edit, or review slope data quickly. While developing the slope data, the user may view the assembled slope geometry and access context-sensitive help. It is expected that this approach will reduce potential errors and provide a more appropriate data preparation method for the less-familiar user.

6.2.1 Analytical Features

XSTABL provides the Generalized Limit Equilibrium (GLE) method allowing factors of safety to be calculated for force and moment equilibrium or for force equilibrium only, using different interslice force angle distributions including Spencer's, Morgenstern-Price, or one of the methods proposed by the Corps of Engineers. The program may be used either to search for the most critical circular, noncircular or block shaped surface or alternatively, used to analyze a single circular or noncircular surface. However, only the modified Bishop and Janbu methods of analysis are used to calculate the factors of safety for a search that is used to isolate and locate a potentially critical surface. Single surfaces may be analyzed using several methods that include Spencer, rigorous Janbu, Lowe and Karafiath, as well as other methods based on an extension of Spencer's Method.

The critical surface is identified by automatically generating and analyzing failure surfaces between defined initiation/termination ranges or by connecting points randomly located within search boxes specified by the user. This approach minimizes the required input parameters and can be effectively used to confine the surface generation within a narrow, well-defined zone. The soil strength along the failure surface may be described as either conventional (i.e., C, ø), undrained or non-linear Mohr Coulomb and can be either isotropic or anisotropic. The undrained strengths are assigned as a function of the vertical effective stress. Other features that have been added to **XSTABL** include the correct implementation of the phreatic surface and the use of a pore water pressure grid. Other useful features, such as anisotropic strength properties, boundary surcharge loads, and limiting boundaries, available in the original version of STABL, have been retained in this program.

6.2.2 Pore Water Pressure

For effective stress analyses, pore water pressures may be simulated by specifying:

1. A piezometric surface
2. Multiple phreatic surfaces
3. Pore water Pressure Grid
4. Constant pore pressure

5. Pore pressure parameter, r_u

6.2.3 Program Availability

A license to use **XSTABL** for slope stability analysis may be obtained from:

Dr. Sunil Sharma
Interactive Software Designs, Inc.
953 N. Cleveland Street
Moscow, ID, 83843
Tel: (208)-885-6403
E-mail: ssharma@uidaho.edu
Web site : www.xstabl.com

6.3 UTEXAS3 SOFTWARE

UTEXAS3 is a general purpose computer program for slope stability analysis that has been developed by Dr. S.G. Wright of University of Texas, Austin. The program has many powerful features for specifying slope geometries and shear strengths and makes available several methods for computing the factor of safety. However, the program is text-based and requires that the user prepare input data files independently prior to using a command line approach for the analysis.

The required input data is described within the context of *groups of data*, labeled GROUPS A-K. The output from UTEXAS3 is considerable and detailed, being grouped into readily identifiable *tables* for convenience. All 38 possible output tables are discussed in the UTEXAS3 User Manual. The input file may include data that requests a single analysis or multiple analyses of the same slope. Options can be *turned on or off* depending on the load cases under consideration.

6.3.1 Methods of Analysis

The program offers the following four methods for calculating the factor of safety:

1. Spencer's procedure (Spencer, 1967; Wright, 1969)
2. Bishop's Simplified procedure (Bishop, 1960)
3. U.S. Army Corps of Engineers' Modified Swedish procedure (Corps of Engineers, 1970)
4. Lowe-Karafiath's (1960) procedure

UTEXAS3 employs the Spencer procedure as a default for all analyses unless the user selects one of the other three methods. This procedure was selected as it satisfies complete static equilibrium for each slice and is the only statically correct method available in the computer program. Options for controlling the convergence tolerances for the factor of safety computations are provided if the available default values appear to be overly restrictive.

6.3.2 Geometry and Stratigraphy

The surface and subsoil geometry are described by a series of straight *profile* lines. The soil below each of these profile lines is specified and indexed to a unique set of soil properties (for example, shear strength, unit weight, etc.). UTEXAS3 offers a practical approach for defining the geometry of the surface of the slope. Users may specify the surface using profile lines, or by using an alternative option where a surface is superimposed onto a predefined subsurface profile. This allows users to readily analyze *cut-slopes* where the designer can concentrate on selecting a design slope rather than reformulating the input data for different slope geometries.

6.3.3 Pore Water Pressures

UTEXAS3 provides several options for specifying pore water pressures (pwp) for use with effective stress analyses. The available options include:

Constant pwp
Pore pressure ratio, r_u
Piezometric line
Discrete user-defined values of pore water pressure, head, or r_u coefficients at user-defined points.

A procedure for using the phreatic line to estimate pore water pressures is not included in UTEXAS3. Thus users often (incorrectly) use the piezometric option to simulate the geometric location of the phreatic surface within the slope.

6.3.4 Soil Properties

The shear strength properties of the soils within the slope may be defined using:

Isotropic strength (c, ϕ)
Shear strength (s_u) varying linearly with depth
Anisotropic strength given as a function of the orientation of the slice base (c_i, ϕ_i)
Nonlinear Mohr-Coulomb envelope, and
Two-stage strength envelope. The two-stage option provides the user with a method for a direct analysis of slopes subjected to undrained loading after a long period of time. This method calculates the effective stresses using drained conditions (for example, long-term) and then reanalyzes the slope using undrained strengths based on the previously calculated effective stresses along the failure surface.

6.3.5 Program Availability

A license to use UTEXAS3 for slope stability analysis may be obtained from:

Dr. Steven Wright
Shinoak Software
3406 Shinoak Drive
Austin, TX 78731
E-mail **:** swright@mail.utexas.edu
Web site: http://www.shinoak.com/

6.3.6 Comparison between XSTABL and UTEXAS3

FHWA, 1994 listed a comparison of XSTABL and UTEXAS 3 features as shown in Table 6-2.

TABLE 6-2
COMPARIONS OF XSTABL AND UTEXAS 3 PROGRAM FEATURES (FHWA, 1994)

XSTABL	UTEXAS3
ANALYTICAL METHODS: Simplified Bishop Simplified Janbu Spencer and rigorous Janbu available in version 5.0 Randomized search for circular surfaces noncircular surfaces Block surfaces may be generated using assigned search boxes Analysis of individual surfaces	ANALYTICAL METHODS: Spencer's Method (default) Simplified Bishop Corps of Engineers Lowe and Karafiath Two or three stage analysis using different soil strengths Grid based search for circular surfaces noncircular surfaces (note: wedge surfaces can also be generated by this option) Analysis of individual surfaces
SLOPE GEOMETRY Surface and subsurface boundary segments associated with underlying soil units Boundaries that limit surface generation (e.g. bedrock) may be specified to increase efficiency of the search process. Slopes may be right or left facing Slope coordinates must be positive (i.e. in first quadrant)	SLOPE GEOMETRY Specified with profile lines that can be associated with underlying soil units Slope surface profile may be superimposed onto the ground model (to simulate a cut) Slopes may be right or left facing Slope coordinates may be negative Vertical tension crack, with and without water
SOIL STRENGTH Isotropic parameters Anisotropic strength properties Non-linear Mohr-Coulomb envelop	SOIL STRENGTH Isotropic parameters Anisotropic strength properties Shear strength varies linearly with depth Non-linear Mohr-Coulomb envelop
PORE PRESSURES Phreatic surface(s) Piezometric surface(s) Pore pressure grid Pore pressure ratio, r_u Constant pore pressure in layer	PORE PRESSURES Piezometric surface(s) Pore pressure grid Pore pressure ratio, r_u Pore pressure ratio, r_u, grid Constant pore pressure in layer
EXTERNAL LOADS	EXTERNAL LOADS

Inclined pressures may be imposed at the surface of the slope	Inclined pressures may be imposed at the surface of the slope Negative pressures may be specified Concentrated (point) forces may act: at surface of slope within the slope
EARTHQUAKE ANALYSIS Horizontal and vertical seismic coefficients for pseudo-static earthquake loadings	EARTHQUAKE ANALYSIS Horizontal seismic coefficient for pseudo-static earthquake loadings
REINFORCED SLOPES Program can calculate maximum external reinforcing force required to maintain a specified factor of safety	REINFORCED SLOPES Individual lines of reinforcement (i.e. forces) may be specified within the slope
OUTPUT High quality on-screen and hardcopy plots of: slope geometry all generated failure surfaces critical failure surfaces Plots are suitable for direct printing without any additional software or may be readily included into reports using popular word processors Concise, informational output for search analysis Slice data output following analysis of single surfaces	OUTPUT Output written to text file Pre and post processor program (GRAPHIC3) for plot preparation available
OTHER FEATURES User-friendly Accommodates and displays British or SI system of units Easy to learn (especially for current STABL users) Complete interactive environment for data preparation and analysis Context sensitive help Excellent documentation Well tested (by U.S. Forest Service)	OTHER FEATURES Requires a text editor for creating file for input data Uses "command words" to identify groups of data Possible to prepare an input file for multiple analyses Detailed and extensive output Average documentation Well tested (by Corps of Engineers)

6.4 RESSA SOFTWARE

Version 1.0 of ReSSA was licensed exclusively to FHWA and its main purpose is to design and analyze reinforced slopes (i.e., geosynthetic or metallic reinforcement). This is a graphically rich program enabling the user to visualize intermediate results, such as normal stress distribution over the slip surface or the thrust line. As a result, judgment about the 'reasonableness' of the results is made easier. Slope geometry can be input using mouse functions or by coordinates inserted in a spreadsheet-like table. Each dialog in ReSSA includes its specific Help. To facilitate the input of data, a special menu allows for quick access to each category of data. Built-in reports as well as an option to print graphical results or to prepare graphical files compatible with AutoCAD® makes the use of ReSSA convenient. Although ReSSA is typically used to analyze reinforce slopes, Version 2.0 of ReSSA program includes option to analyze general soil slope without reinforcement.

6.4.1 Methods of Analysis

The program offers the following four methods for calculating the factor of safety:

Bishop Method. In the case of reinforced slopes, the AASHTO approach of incorporating reinforcement force is available.

Spencer Method using 2- and 3-part wedge failure mechanisms. The 2-part wedge is useful when assessing failure along reinforcement layers. The 3-part wedge is useful when assessing deepseated failures or failures through the reinforcement and the retained soil.

The user can invoke either or both methods for each problem. The distribution of the safety factors for various slip surfaces is displayed graphically so that the user can decide whether the genuine minimum was captured. The accuracy of the results for Spencer Method is displayed; in Bishop's, the number of iterations and the various components of the factor of safety equation is presented in case verification via hand calculations is needed.

6.4.2 Geometry and Stratigraphy

The surface and subsoil geometry are described by a series of straight *profile* lines entered using mouse functions or via a table. The soil below each of these profile lines is specified to a unique set of soil properties. ReSSA classifies geometry as simple, semi-simple and complex. The simple geometry can be input quickly by specifying the height, inclination, sloping toe, and backslope angle. The semi-complex geometry is useful when multitiered slopes are specified; simple data enables the user to define up to 10-tiered slopes, each represented by a different geometry. The complex geometry requires more complex input data; it can be defined by up to 100 vertical sections. Recent additions to ReSSA (2.0) simplifies the input data for complex geometries as compared with the original version 1.0. Graphical display of the input geometry eases the process. The user can input tension cracks, various surcharge loads, and earthquake loading. Reinforcement properties follow AASHTO 98.

6.4.3 Pore Water Pressures

ReSSA provides several options for specifying pore water pressures (pwp) for use with effective stress analyses. The available options include:

Phreatic surface defined by up to 100 points

Up to 20 lines, each having a different piezometric heads (representing, for example, piezometer readings)

ReSSA enables the user to use total stress analysis (i.e., shear strength of soil corresponds to undrained conditions), effective stress analysis (shear strength corresponds to drained conditions) where pwp is needed, and mixed stress analysis (i.e., for cases where failure is expected to be under undrained conditions in some layers while exhibiting drained conditions in others). An example where mixed analysis could be used is a reinforced slope composed of granular soil over soft soil; if the water table is within the reinforced mass, this free-draining material will exhibit drained conditions at failure relative to the low permeability, undrained foundation.

6.4.4 Soil and Reinforcement Properties

Up to 25 different soils can be represented in ReSSA. Each soil possesses unit weight, internal angle of friction and cohesion. Reinforcement strength follows AASHTO 98 guidelines. For metallic reinforcement corrosion is accounted for when determining the allowable strength. For geosynthetic reinforcement creep, construction damage and aging are considered when assessing the allowable strength.

6.4.5 Safety Map

Recent modifications of ReSSA (2.0) enables the user to draw a color coded map showing the distribution of FS within the soil mass. It follows a procedure presented by Baker and Leshchinsky (2001). This feature makes it easier to visualize whether the minimum factor of safety was indeed captured. Furthermore, it provides a diagnostic tool as to the stability status of the slope. That is, one is not looking only at the critical slip surface but rather at a zone that is potentially unstable. Such a perspective is useful if remedial actions are needed; e.g., zone within which reinforcement is needed. It also shows how effective a designed slope is; i.e., whether FS within the slope is as close as possible to uniformity.

6.4.6 Program Availability

ReSSA(1.0) is licensed by ADAMA Engineering to FHWA. Limited number of copies is available to state DOT's. A license to use ReSSA (2.0), which is suitable to deal also with unreinforced slopes and more complex failure mechanisms than ReSSA(1.0), may be obtained from:

ADAMA Engineering, Inc.
33 The Horseshoe
Newark, DE 19711
E-mail **:** adama@GeoPrograms.com
World Wide Web: http://www.GeoPrograms.com/

6.5 COMPUTER SOFTWARE FOR THREE-DIMENSIONAL SLOPE STABILITY ANALYSIS

A large number of studies of three-dimensional slope stability problem have been performed in the past 30 years. Duncan in 1989 stated that the 3D slope stability analyses have progressed far enough to conclude that the factor of safety using 3D analyses based on limit equilibrium approach, will most likely be greater than, or equal to the factor of safety calculated using 2D analyses. Therefore, three-dimensional analyses are performed only in a few specific projects and mostly for post failure analyses (to back-calculate the mobilized shear strengths). Since the 3D slope stability analysis is not yet well adopted by the general geotechnical engineering community, very few general all-purpose 3D analysis programs are commonly used. Table 6.3 lists a few of the currently available 3D slope stability programs.

As shown in Table 6.3, CLARA 2.31 is the most versatile of the three. It can analyze failure surfaces of any shape, satisfies more conditions of equilibrium and has more 3D analysis options. Besides the three programs listed above, there are other 3D analysis programs such as STAB3D (Baligh and Azzouz 1985), LEMIX&FESPON (Chen and Chameau 1983), BLOCKS (Lovell 1984), and F3SLOR&DEEPCYL (Gens et al. 1988) which have been used in the past, however, they are all quite limited in their capabilities. They can only analyze failure surfaces with a cylindrical shape. Note that all available 3D slope stability programs assume that the entire failure mass slides in the same direction at the same rate.

TABLE 6-3
REPRESENTATIVE COMMERCIALLY AVAILABLE 3D SLOPE STABILITY PROGRAM
(From Stark et al., 1998)

Program	Theoretical Basis for 3D Model	Static Assumptions	Equilibrium Condition Satisfied	Comments
CLARA 2.31 (Hungr 1988)	Bishop's simplified method or Janbu's simplified method	Vertical inter-column shear forces are zero	Vertical force and moment	Can analyze any failure surface shape; utilizes linear Mohr-Coulomb envelope or parabolic envelope [proposed by Hoek and Brown (1980)]; 3D critical failure surface search for circular surface only; can calculate yield seismic coefficient.
TSLOPE3 (Pyke 1991)	Los Angeles County method	Inter-column forces are zero	Horizontal force	Can analyze any failure surface shape; utilizes linear or non-linear Mohr-Coulomb envelope or parabolic envelope [proposed by Hoek and Brown (1980)]; cannot search for 3D critical failure surface; can calculate yield seismic coefficient.
3D-PCSTABL (Thomaz 1986)	Method of slices	Inter-slice forces on the sides of the column have the same inclination; intercolumn shear forces are parallel to the column base.	All forces and moments	Can analyze symmetrical failure surface only; utilizes linear Mohr-Coulomb envelope; can search for 3D critical failure surface; can calculate yield seismic coefficient.

6.6 COMPUTER SOFTWARE FOR NUMERICAL ANALYSIS (FEM AND FDM)

As discussed in Chapter 5, the conventional limit equilibrium methods are the most commonly used methods to analyze the stability of a soil slope. However, these methods are only capable of calculating the factor of safety of a slope, and not capable of predicting the deformation. Therefore, the deformation analysis for slopes and embankments is mostly performed by numerical analysis using finite element or finite difference methods. Numerical analysis require definition of initial stress condition, stress-strain relationship (non-linear), and the loading or unloading sequence to compute stresses and displacements. Therefore, if a horizontal (and vertical) displacement criteria is established for the surface of the slope, finite element and finite difference programs with appropriate soil properties input capability, can be used to estimate the stability of slopes. However, the maximum tolerable displacement limit may be difficult to establish since it depends on the soils, the physical configuration/dimensions of the slope and other factors. Only a few finite element programs have incorporated this capability of calculating the factor of safety for slopes. Some of these programs are listed in Table 6.4. For programs that do not have this capability the overall factor of safety should be checked with limit equilibrium analysis and the deformation of the slope or embankment with the finite element or finite difference analysis.

TABLE 6-4
PARTIAL LISTING OF CURRENTLY AVAILABLE FINITE ELEMENT AND FINITE DIFFERENCE SLOPE STABILITY ANALYSIS PROGRAMS

Program	Comment
GFA2D	This is perhaps the simplest finite element program currently available and it is free and was written by Bang of South Dakota School of Mines and Technology, Department of Civil Engineering. The local safety factor of each element is calculated from the non-linear Mohr shear failure criterion. The program searches throughout the defined searching zone for the most likely failure surface. The global safety factor is defined as the weighted sum of the local element (on the final failure surface) safety factors. Because of its simplicity, the capability of this program is limited when compare to other commercially available products. For example, the maximum number of elements and nodes are limited to 200 and 300 respectively.
PLAXIS	This is a general-purpose finite element package for the analysis of deformation and stability in geotechnical engineering projects. The non-linear Mohr-Coulomb model is used for slope stability analysis. The factor of safety is computed using a c-ϕ reduction procedure. Input of soil layers, structures, construction stages, loads and boundary conditions are all performed graphically. Finite element mesh is automatically generated. Pore pressure distributions may be generated on the basis of a two-dimensional groundwater flow analysis. Initial stresses may be generated by specifying K_o or by using gravity loading. The Software may be upgraded to include dynamic and 3D analyses.
TELSTA	This is a general purpose, non-linear finite element program for the analysis of soil and soil-structure interaction problems. Modified "Duncan and Chang" hyperbolic model of non-linear soil behavior (which includes modeling of unloading and reloading) is utilized. Allows specification of initial stresses and compaction-induced stresses. Mesh generation is semi-automatic. Both local and overall factors of safety are computed along specified slip surfaces.
ZSOIL STAB98	This is a scaled down version of the general geotechnical engineering package Z_SOIL.PC. Its capability is narrowed to stability and ultimate load analyses. Non-linear model options include Mohr-Coulomb, Rankine, Von Misès and Drucker-Prager. Allows application of initial stresses. Transient groundwater flow with time-dependent free surface can be incorporated. Mesh generation is semi-automatic. Slope stability factors of safety are obtained based on the c-ϕ reduction algorithm. The full scale version Z_SOIL.PC can be upgraded to include 3D analysis.
FLAC	The Fast Lagrangian Analysis of Continua (FLAC) program uses a unique analysis process employing the finite difference method. The process and program were developed by Itasca Consulting Group in Minneapolis, MN. It allows for consideration of large deformations and discontinuities and hence, ideal for geotechnical stability problems. It has many constitutive laws built-in for describing soil behavior, however, the user can input any desired other relationship. The program allows for simulation of construction and clearly displays zones within the soil that reached a plastic state. FLAC can be a powerful tool for assessing the stability of slopes and settlement of embankments; however, input data and modeling must be done carefully by a knowledgeable analyst. Current version (4.0) is Windows-based allowing for quick generation of input data as well as graphic-rich presentation of results. FLAC has 3D, dynamic, and creep options.

Currently PLAXIS is one of the most versatile geotechnical finite element analysis programs commercially available and is quite well received within the geotechnical engineering community. Besides slope stability, both PLAXIS and Z_SOIL.PC will also calculate displacements including consolidation and creep. Both can be upgraded to perform 3D slope stability analysis but only PLAXIS also includes the dynamic analysis capability.

FLAC is a two-dimensional explicit finite difference code, which simulates the behavior of soil, rock or other geotechnical materials which may undergo plastic flow when their yield limits are reached and considers large deformations and discontinuities. It also includes structure elements such as sheet piling, rock bolts and tunnel liners that interact with the surrounding geological materials. FIAC is a very popular and powerful tool for numerical modeling to assess geotechnical stability and settlement problems.

CHAPTER 7
LANDSLIDE RECOGNITION AND INVESTIGATION

7.1 GENERAL

In the previous seven chapters, we have discussed the principals and methodologies for the investigation, analysis, and design of stable (safe) slopes in soils. However, landslides in different scales do occur and are increasingly more common all around the country. They are typically related to natural phenomena such as increased precipitation, flooding and earthquake, or man-made activities such as increased urbanization and development. Landslides often cause restrictions and disruptions of traffic flow, thus pose serious threats to highways and structures that provide general transportation as well as support fisheries, tourism, timber harvesting, mining, and energy production. Landslides also can create serious safety hazards and result in significant economic losses.

Cruden in 1991 defined the term "landslide" as "the movement of a mass of rock, debris or earth down a slope", which basically includes all types of slope movements such as slide, flow, spread, fall, and topple as described in Chapter 2. Therefore, by this definition, a landslide is not literally a sliding failure in land but is generally any mass movement of soil or rock down a slope. So far, the techniques and methodologies discussed in the previous chapters in general can be applied to the investigation and analysis of mass movement in soil slopes, i.e. landslides. However, in order to determine feasible and practical remedial measures to restore the displaced soil slopes to their original function and to minimize possible reoccurrences, supplemental investigation techniques must be applied to understand the extent and causes of the movement. In this chapter, we focus on recognizing and investigating landslides that consist of soil masses moving downslope, where the mass movement is driven by gravitational or seismic forces. The soil mass movements are predominately comprised of rotational slide, translation slide, flow and spread types of movement as discussed in Chapter 2. Fall and topple types of slope movement or failure typically result from unstable rock slopes, and are covered in NHI 132035 "Rock Slopes" in details.

Many highway agencies regard the maintenance of slopes and the restoration and correction of slides on highways as a major and continuing problem involving considerable expenditures of funds (Hopkins et al, 1988). Landslides constitute a major geologic hazard because they are widespread, occurring in all 50 states, and cause $1-2 billion in damages and more than 25 fatalities on average each year nationwide (Schuster, 1996). The Federal Emergency Management Agency reported that the most expensive landslide in U.S. history occurred in Thistle, Utah, in spring, 1983. It reached .8 km (1/2 mile) from top to bottom and ranged in width from 300 m (1,000 feet) to about 1.6 km (1 mile). Total costs attributable to the landslide exceeded $500 million.

Landslides in soil slopes can occur in almost any landform, and are common throughout the Appalachian region, New England, and West Coast as well as many other areas of the nation. The greatest hazard in the eastern part of the U.S. is from sliding of clay-rich soils; related damages in urban areas such as Pittsburgh, PA, and Cincinnati, OH, are among the greatest in the U.S. Landslides also occur across the Great Plains and into the mountain areas of the western U.S. in weathered shales and other clay-rich rocks particularly where there are steep slopes, periodic heavy rains, and vegetation loss has occurred after wildfires. Earthquakes and volcanoes also cause landslides; the catastrophic 1980 eruption of Mount St. Helens in Washington was preceded by the development of a large landslide on the north side of the volcano. The Northridge earthquake in 1994 in the San Fernando Valley triggered thousands of landslides in the Santa Susanna Mountains north of the epicenter. Table 7-1 identifies a key to landforms and their susceptibility to landslides.

TABLE 7-1
KEY TO LANDFORMS AND THEIR SUSCEPTIBILITY TO LANDSLIDES (Schuster and Krizek, 1978)

Topography	Landform or Geologic Materials	Landslide Potential*
I. Level Terrain		
A. Not elevated	Floodplain	3
B. Elevated		
1. Uniform tones	Terrace, Lake bed	2
2. Surface irregularities, sharp cliff	Basaltic plateau	1
3. Interbedded – porous impervious layers	Lake bed, coastal plain, sedimentary plateau	1
II. Hilly Terrain		
A. Surface drainage not well integrated		
1. Disconnected drainage	Limestone	3
2. Deranged drainage, overlapping hills, associated with lakes and swamps (glaciated areas only)	Moraine	2
B. Surface drainage well integrated		
1. Parallel ridges		
a. Parallel drainages, dark tones	Basaltic hills	1
b. Trellis drainage, ridge-and-valley topography, banded hills	Tilted sedimentary rocks	2
c. Pinnate drainage, vertical-sided gullies		
2. Branching ridges, hilltops at common elevation	Loess	2
a. Pinnate drainage, vertical-acid gullies		
b. Dendritic drainage	Loess	2
(1) Banding on slope		
(2) No Banding on slope	Flat-lying sedimentary rocks	2
(a) Moderately to highly dissected ridges, uniform slopes	Clay shale	1
(b) Low ridges, associated with coastal features		
(c) Winding ridges connecting conical hills, sparse	Dissected coastal plain	1
vegetation	Serpentinite	1
3. Random ridges or hills		
a. Dendritic drainage		
(1) Low, rounded hills, meandering streams		
(2) Winding ridges connecting conical hills, sparse	Clay shale	1
vegetation	Serpentinite	1
(3) Massive, uniform rounded to A-Shaped hills		
(4) Bumpy topography (glaciated areas only)	Granite	2
	Moraine	2
III. Level to hilly, transitional terrain		
A. Sleep slopes	Talus, colluvium	1
B. Moderate to flat slopes	Fan, delta	3
C. Hummocky slopes with scarp at head	Old slide	1

Note: Table extracted from TRB-SR176, 1978

*1 = Susceptible to landslides; 2 = susceptible to landslides under certain conditions; and 3 = not susceptible to landslides except in vulnerable locations.

Figure 7-1: Landslide in La Conchita, California (USGS)

7.2 LANDSLIDE FUNDAMENTALS

The first step to recognize and investigate landslides involves observation of the landslide fundamentals such as landslide features, dimensions, volume, rate of movement, etc. Since rock slopes are discussed separately in NHI 132035 Course, "Rock Slopes" Reference Manual, only soil (earth and debris) slope movement is discussed in this Chapter. In addition, as discussed previously, a landslide may consist of one or more of the following types of movement, slide, flow, spread fall, and topple, as described in Chapter 2. This chapter focuses on slide, flow and spread types of landslide movement, which are more predominant in soil slopes than topples and fall.

7.2.1 Landslide Features and Dimensions

Landslide features and dimensions are the most important observations that must be made during the initial site reconnaissance and study before detailed analysis and study can proceed. Landslide features are best illustrated by the earth slide-earth flow diagram developed by Varnes, 1978 (Figure 7-2). The International Association of Engineering Geology (IAEG) Commission on Landslide and Other Mass Movement has further defined each of the features and the dimension of a landslide schematically in 1990 (Figures 7-3a and 3b, respectively). Definitions for the landslide features and dimensions are provided Table 7-2 and 7-3, respectively. Readers are referred to IAEG Bulletin "Suggested Nomenclature for Landslides" as listed in the references for detailed discussion.

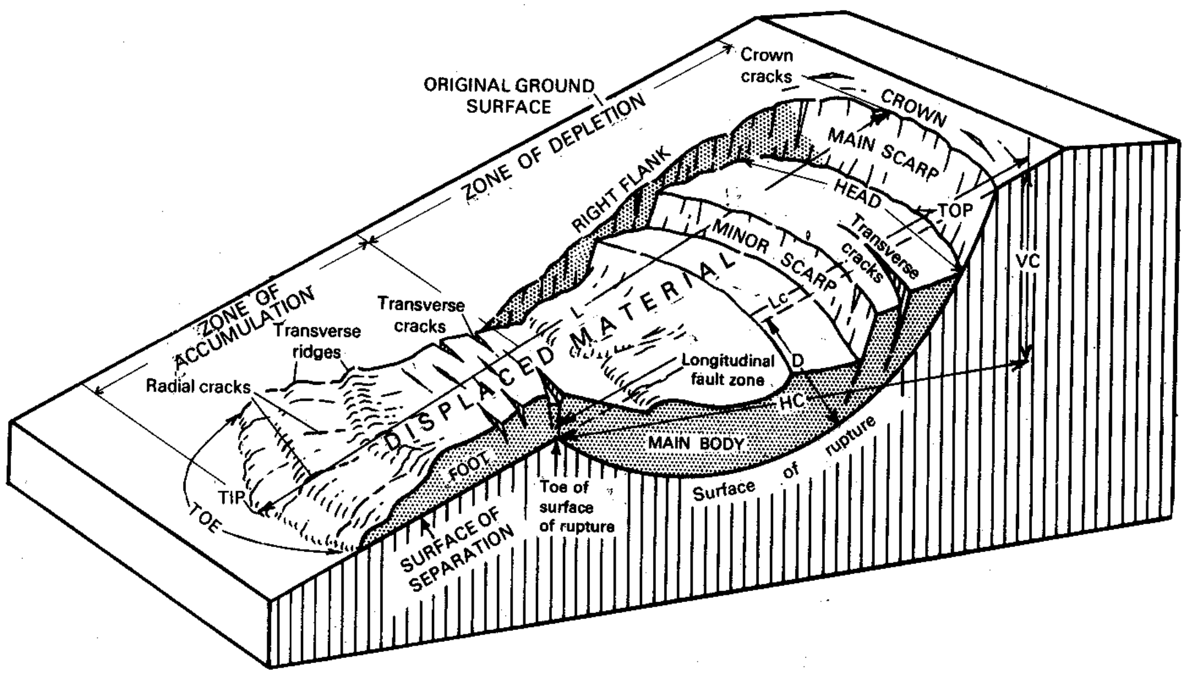
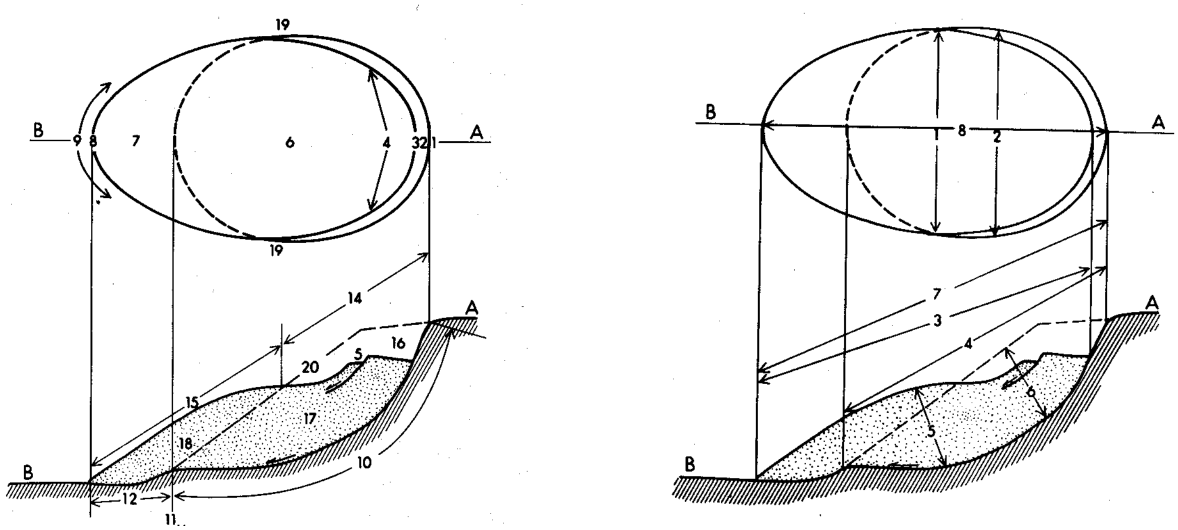


Figure 7-2: Block Diagram Of Idealized Complex Earth Slide-Earth Flow (Varnes, 1978)



(a) (b)

Note: The numerical annotations for 7-3a and 3b are listed in Tables 7-1 and 7-2, respectively.

Figure 7-3: Landslide Features And Dimensions (IAEG, 1990)

TABLE 7-2
DEFINITIONS OF LANDSLIDE FEATURES (IAEG Commission on Landslide, 1990) (Refer to Figure 7-3a)

No.	Feature Name	Definition
1	Crown	The practically undisclosed 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 7-3
DEFINITIONS OF LANDSLIDE DIMENSIONS (IAEG Commission on Land Slide, 1990) (Refer to Figure 7-3b)

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 crow to tip of landslide through points on original ground surface equidistant from lateral margins of surface of rupture and displaced material.

7.2.2 Landslide Volume

Based on the features and dimensions observed in the field, a rough estimate of the volume of the landslide can be made. This estimate is based on the assumption that the landslide shape can be approximated as a spoon shape corresponding to one-half an ellipsoid:

$$VOL_{ls} \approx 1/6\ (\pi\ D_r\ W_r\ L_r)$$

Where the VOL_{ls} is the approximate volume of the ground displaced by a landslide, and the D_r, W_r, and L_r are defined in Table 7-2 and shown in Figure 7-2b. The displaced materials generally dilate and expand by the movement. As a result, the materials displaced by a landslide swell, and the volume increases to

$$VOL_{ls\text{-}swell} \approx 1/6\ (\pi\ D_d\ W_d\ L_d)$$

The dimensions mentioned in Table 7-2 can mostly be measured by land survey except for the depths of displaced mass (D_d) and surface of rupture (D_r), and the length of surface of rupture (L_r). McGuffey et al. (1996) suggested that the D_d is rarely greater than the W_d, and the maximum depth of the failure surface is often approximately equal to the distance from the break in the original ground surface slope to the most uphill crack or scarp (L_r).

7.2.3 Rate of Movement

An active landslide of any type can be moving rapidly at a speed in excess of 10 m (33 feet) per second, or slowly at a rate of less than few centimeters per year. Cruden et al (1991) proposed a scale for landslide velocity (Table 7-3). In general, estimation of landslide velocity can be made by the following means:

- Repeated surveys of the positions of displaced mass
- Reconstruction of the trajectories of portions of the displaced mass
- Eyewitness observations, and
- Continuous monitoring using geotechnical instrumentation techniques

Cruden et all (1991) also purposed to correlate the rate of the movement to the probable destructive significance of landslide as shown in Table 7-4. Mudflow movements may occur where the initial slide has remolded the soil into a broken and softened mass. Therefore, in general these movements are faster than post-failure movements.

TABLE 7-4
LANDSLIDE VELOCITY SCALE AND PROBABLE DESTRUCTIVE SIGNIFICANCE
(After Cruden and Varnes, 1992)

Velocity Class	Description	Velocity (mm/sec)	Probable Destructive Significance
7	Extremely rapid	5×10^{3} or above	Catastrophe of major violence; buildings destroy by impact of displaced material; many deaths; escape unlikely
6	Very Rapid	$50 - 5 \times 10^{3}$	Some lives lost; velocity too great to permit all persons to escape
5	Rapid	0.5 – 50	Escape excavation possible; structures, possessions, and equipment destroyed
4	Moderate	$5 \times 10^{-3} - 0.5$	Some temporary and insensitive structures can be temporarily maintained
3	Slow	$5 \times 10^{-5} - 5 \times 10^{-3}$	Remedial construction can be undertaken during movement; insensitive structures can be maintained with frequent maintenance work if total movement is not large during a particular acceleration phase
2	Very Slow	$5 \times 10^{-7} - 5 \times 10^{-5}$	Some permanent structure undamaged by movement
1	Extremely Slow	Below 5×10^{-7}	Imperceptible without instruments; construction possible with precautions

mm/sec = .04 inch/sec

7.3 LANDSLIDE TRIGGERING MECHANISM

The geological, topographical, climatic, and other factors that cause landslides are similar to those factors affecting the slope instability as discussed in Chapters 3 and 4. Landslides can have one or more causes including ground causes, morphological causes, physical causes, and man-made causes as discussed in Chapter 3 and as listed in Table 3-1 (Cruden et al, 1992). In summary, typical causes of landslides in highway soil slopes include:

- Excessive slope
- Low strength foundation
- Removal of loading
- Increase loading
- Environmental factors
- Poor handling
- Unsuitable materials
- Undetected and Unfavorable geologic features
- Liquefaction
- Wildfires

Although a landslide might be induced by one or combination of the above typical causes, usually there is only one that triggers the movement. Varnes in 1978 defined the triggering mechanism of a landslide as an external stimulus that induces a near-immediate response. For instance, landslides commonly occur on slopes with the aforementioned adverse causes during or after major natural disasters such as storms, earthquakes and floods. Rapid expansion of land development and highways has also increased the incidence of landslide disasters.

Wieczorek in 1996 listed several triggering mechanisms for landslides and concluded that the common landslide triggers, including intense rainfall, rapid snowmelt, water-level changes, Volcanic eruptions, and strong ground shaking during earthquakes, are probably directly responsible for the majoirity of landslides worldwide. The triggering mechanisms more commonly occurred in transportation projects include the following:

- Intense rainfall
- Water level changes
- Earthquakes
- Human activity
- Stream erosion
- Volcano eruption (not discussed in this Chapter)
- Rapid snowmelt (not discussed in this Chapter)

7.3.1 Intense Rainfall

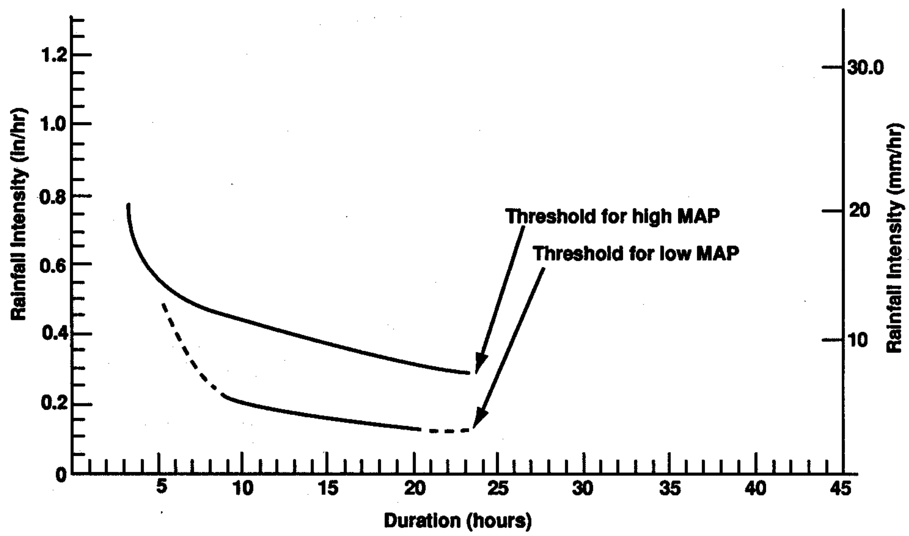
Most landslides occur after or during intense rainfall produced by storms. During 1997-98, numerous storms linked to El Niño triggered hundreds of landslides in California that caused casualties, damaged residences, or closed major highways and local roads.

Landslides in soils and weathered rock often are generated on steep slopes during the more intense parts of storms where the combined thresholds of intensity and duration of rainfall are necessary to trigger them. 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. Those landslides provide an identification of landslide–triggering rainfall thresholds based on both rainfall intensity and duration as shown in Figure 7-4. In addition to the rainfall intensity (mm/hr) and duration of a storm, a landslide can also be triggered due to high cumulative rainfall.

7.3.2 Water Level Changes

As discussed in previous chapters, ground water contributes greatly to many landslides and slope instability. An increase in the groundwater table (for instance, 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 landslide. Rising groundwater levels can also accelerate active landslide movement. In addition to excessive precipitation, water level increase 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 is another potential source of excess water.

The sudden lowering of the ground water level (rapid drawdown) against a slope can trigger landslides, especially along coastlines and on the banks of lakes, reservoirs, canals, and rivers.



Note: MAP – Mean Annual Precipitation

Figure 7-4: Rainfall Thresholds that Triggered Abundant Landslides in San Francisco Bay Area, CA.

7.3.3 Earthquakes

Loose, saturated sands are particularly vulnerable to liquefaction during earthquakes, which leads to flow slides or unstable foundation conditions for overlying 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 landslides throughout an area of 14,000 Km^2 (5,400 square miles). In addition to causing at least tens of millions of dollars of damage to houses, other structures, and utilities, landslides 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).

During earthquakes, thin lenses of loose saturated silts and sands may cause an overlying sloping soil mass to slide laterally along the liquefied layer (spreading failure). A zone of soil at the back end of the sliding mass sinks into the vacant space formed as the mass translates, resulting in a depressed zone known as graben (Figure 2-13 and 2-14). Buildings in the graben normally are subjected to large differential settlements (Figure 2-15).

Major slide movements also can occur in clay deposits during earthquakes. In some cases, clay deposits often contain sand lenses, and liquefaction of these lenses may well contribute significantly to the slide development, such as the earthquake-induced landslide along the coastline of the Turnagain Heights area of Anchorage during the Alaskan Earthquake in 1964 (Seed and Wilson, 1967).

7.3.4 Human Activity

Human activity can often be the trigger of serious slope stability problems. Examples of how man can alter site conditions and reduce the stability of natural slopes are given below:

- Weak strata or stratification planes overloaded with new fill or 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 toe 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.

7.3.5 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, especially in loess deposits, the processes of which includes:

- Subsurface seepage with tunnel formation and ultimate collapse into gullies;
- 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 and 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.

7.4 INVESTIGATION OF ACTIVE LANDSLIDE

General field and laboratory investigative techniques for analysis and design of stable soil slopes are discussed in Chapter 3. However, investigations of stable slopes and stable ground, even those with inactive landslide deposits, may be more methodical than investigation of active slide areas because the time constraint and conditions of urgency. Once a landslide has developed (activated), either during construction of a facility or subsequently, the investigation is undertaken to diagnose the factors affecting the movements and to determine what corrective measures are appropriate for preventing or minimizing further movements and development.

Many attempts have been made to define a rigid process for planning an investigation for an active landslide. In general, the flowchart shown in Figure 3-4 for planning of site geotechnical investigation for the design of soil slope (Modified from Mintzer, 1962) is mostly applicable. Sowers et al (1978) suggested a rather extensive and complete checklist for planning a landslide investigation as shown in Table 7-5. However, it is not expected that any single landslide investigation would involve all the items on the list (Turner et al. 1991).

Basically, the information needed by a design team 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

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

- Site reconnaissance
- Engineering geological mapping
- Aerial photo study
- Subsurface Exploration, and
- Geotechnical instrumentation

Initial site reconnaissance and engineering geological mapping should be performed as soon after the landslide occurs as possible. An engineering geologist or geotechnical engineer is usually well equipped to identify and interpret the observable features based on his or her experience in the local geology and landslide feature observation. In most cases, landslides do not occur without some advance warning. Therefore, if a slide is discovered in the early stages, steps might be taken to prevent further movement which in term will limit the amount of sliding mass involved and the cost of remediation. Hopkins et al in 1988 identified twelve common signs of movements in highway soil slopes as listed in Table 7-6.

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 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 7-5).

TABLE 7-5
CHECKLIST FOR PLANNING A LANDSLIDE INVESTIGATION (Sowers and Royster, 1978)

I. TOPOGRAPHY
- A. Contour Map
  - 1. Land form
  - 2. Anomalous patterns (jumbled, scraps, bulges)
- B. Surface Drainage
  - 1. Continuous
  - 2. Intermittent
- C. Profiles of Slope
  - 1. Correlate with geology (II)
  - 2. Correlate with contour map (IA)
- D. Topographic Changes
  - 1. Rate of change by time
  - 2. Correlate with groundwater (III), weather (IV), and vibration (V)

II. GEOLOGY
- A. Formations at Site
  - 1. Sequence of Formations
  - 2. Colluvium
    - a. Bedrock contact
    - b. Residual Soil
  - 3. Formations with bad experience
  - 4. Rock minerals susceptible to alteration
- B. Structure: Three-Dimensional Geometry
  - 1. Stratification
  - 2. Folding
  - 3. Strike and dip of bedding or foliation
    - a. Changes in strike or dip
    - b. Relation to slope and slide
  - 4. Strike and dip of joints with relation to slope
  - 5. Faults, breccia, and shear zones with relation to slope and slide
- C. Weathering
  - 1. Character (chemical, mechanical, and solution)
  - 2. Depth (uniform or variable)

III. GROUNDWATER
- A. Piezometric Levels Within Slope
  - 1. Normal
  - 2. Perched levels, relation to formations and structure
  - 3. Artesian pressure, relation to formations and structure
- B. Variations in Piezometric Levels [correlate with weather (IV), vibration (V), and history of slope changes (VI)]
  - 1. Response to rainfall
  - 2. Seasonal fluctuations
  - 3. Year-to-year changes
  - 4. Effect to snowmelt
- C. Ground Surface Indications of Subsurface Water
  - 1. Springs
  - 2. Seeps and damp areas
  - 3. Vegetation differences
- D. Effect of Human Activity on Groundwater
  - 1. Groundwater utilization
  - 2. Groundwater flow restriction
  - 3. Impoundment and additions to groundwater
  - 4. Changes in ground cover and infiltration opportunity
  - 5. Surface water changes
- E. Groundwater Chemistry
  - 1. Dissolved salts and gases
  - 2. Changes in radioactive gases

IV. WEATHER
- A. Precipitation
  - 1. Form (rain or snow)
  - 2. Hourly rates
  - 3. Daily rates
  - 4. Monthly rates
  - 5. Annual rates
- B. Temperature
  - 1. Hourly and daily means
  - 2. Hourly and daily extremes
  - 3. Cumulative degree-day deficit (freezing index)
  - 4. Sudden thaws
- C. Barometric Changes

V. VIBRATION
- A. Seismicity
  - 1. Seismic events
  - 2. Microseismic intensity
  - 3. Microseismic changes
- B. Human Induced
  - 1. Transport
  - 2. Blasting
  - 3. Heavy machinery

VI. HISTORY OF SLOPE CHANGES
- A. Natural Process
  - 1. Long-term geologic changes
  - 2. Erosion
  - 3. Evidence of past movement
  - 4. Submergence and emergence
- B. Human Activity
  - 1. Cutting
  - 2. Filling
  - 3. Changes in surface water
  - 4. Changes in groundwater
  - 5. Changes in vegetative cover, clearing excavation, cultivation, and paving
  - 6. Flooding and sudden drawdown of reservoirs
- C. Rate of Movement
  - 1. Visual accounts
  - 2. Evidence in vegetation
  - 3. Evidence in topography
  - 4. Photographic evidence
    - a. Oblique
    - b. Stereo aerial photographs
    - c. Aerial photographs
    - d. Spectral changes
  - 5. Instrumental data
    - a. Vertical changes, time history
    - b. Horizontal changes, time history
    - c. Internal strains and tilt, including time history
- D. Correlations of Movements
  - 1. Groundwater [correlate with groundwater (III)]
  - 2. Weather [correlate with weather (IV)]
  - 3. Vibration [correlate with vibration (V)]
  - 4. Human activity [correlate with human-induced vibration (V.B)]

TABLE 7-6
COMMON SIGNS OF MOVEMENTS IN HIGHWAY SOIL SLOPES (After Hopkins et al, 1988)

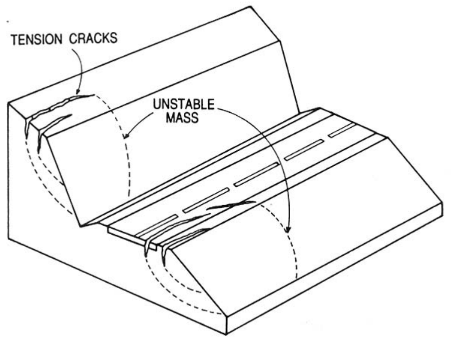
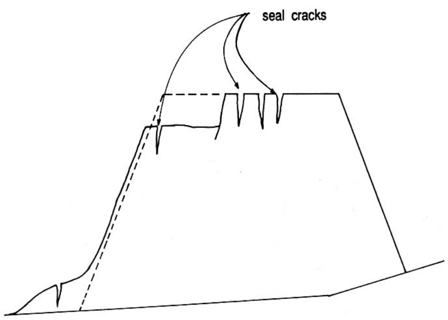
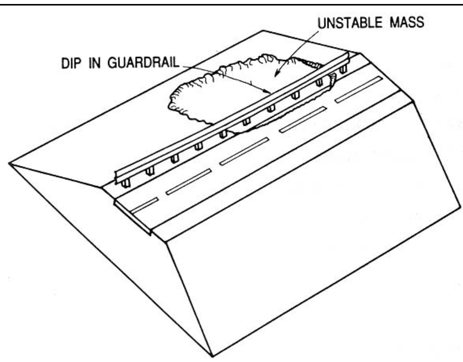
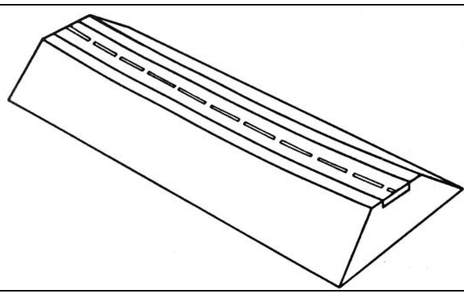
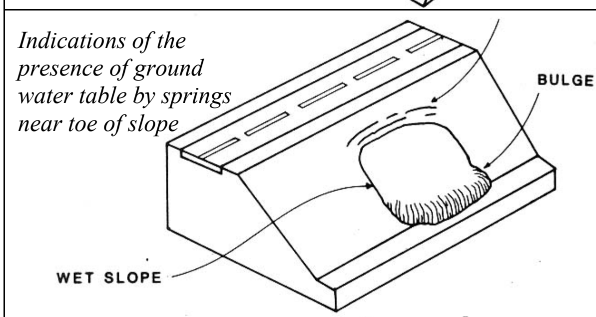
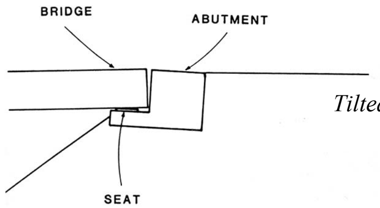
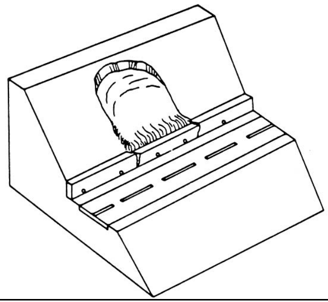
	***1. Tension cracks in roadway or on slopes above roadway:*** Tension cracks in the roadway indicate the natural slope or cut slope is in the early stages of movement. These cracks permit water to enter and further soften material along the failure plane as well as add additional water pressure to the moving mass.
	***2. Escarpments in or above Roadway:*** Escarpments indicate that the mass of soil or rock has already failed and moved. Some landslides develop more than one escarpment, as the soil mass often has a tendency to move in "blocks"
	***3. Sunken Guardrail:*** 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 embankment and involves only the shoulder but not the driving lanes. 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 guardrails at bridge approaches indicate that the approach embankment and/or foundation have settle or that the embankment is creeping.
	***4. Dips in Grade:*** Dips in grade in all driving lanes may be associated with slumping or creeping of the embankment under its own height. Dips in grade also may be associated with culverts located under large fills.
	***5. Bulges above, on, or below Roadway:*** Minor bulge above, on, or below roadway may indicates considerate movement already has occurred and that movement will probably continued until complete or retrogressive failure occurs.

TABLE 7-6 (Cont'd)
COMMON SIGNS OF MOVEMENTS IN HIGHWAY SOIL SLOPES (After Hopkins et al, 1988)



Tilted bridge abutments



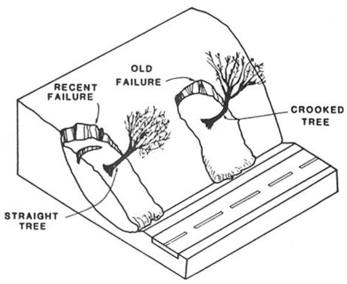
Cracks in retaining wall or tilted retaining wall

6. *Changes in Structures*

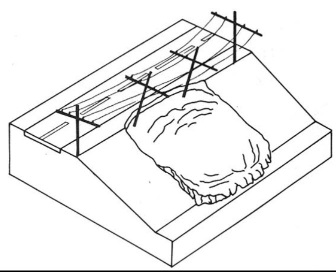
a. *Bridges:* Bridges abutments that tilt in relation to the bridge beams or abutments that move toward the ends of the bridge beams indicate that the approach embankment is moving or creeping toward the bridge. Settlement of bridge approach pavement slabs indicates that the approach embankment is settling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

b. *Retaining Walls:* Retaining walls are structures used to hold back soils slopes. However, if the soil continues to move excessively, the wall will tilt from the vertical and in sever cases will actually overturn. Cracks in retaining walls also may be evidence of soil movement behind the wall.

c. *Building:* Buildings located in slide areas may provide evidence of earth movements. The most notable evidence are cracks in foundations or in masonry walls. Buildings also may rise or fall in elevation, depending on their locations in the slide area.



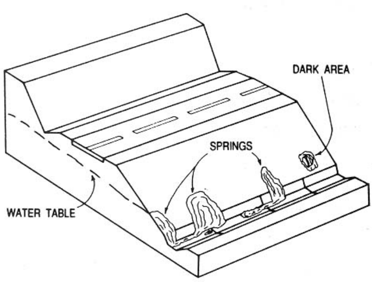
Tilted or curved trees



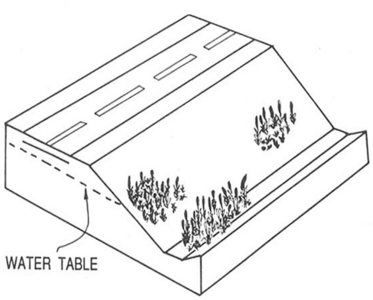
Tilted telephone poles

7. *Changes in Features*

More subtle signs of earth movements may include trees that are tilted from vertical. Tilted 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 tilted out of alignment are also good indicators of earth movement.



Presence of groundwater table by springs near toe of slope



Cattails or willow trees warn of subsurface seepage

8. *Poor Subsurface Water Drainage:*

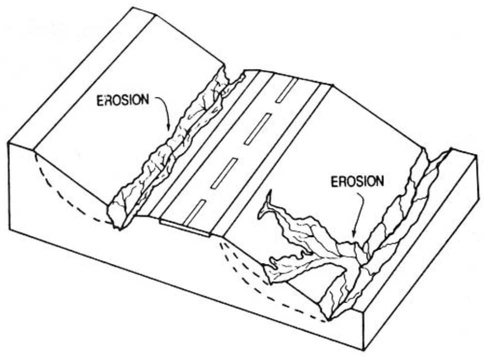
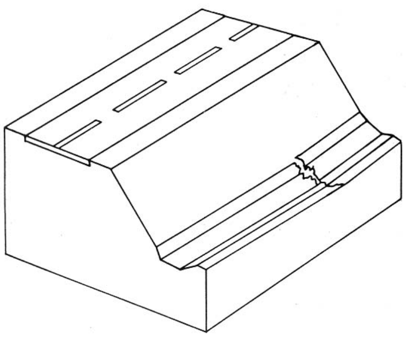
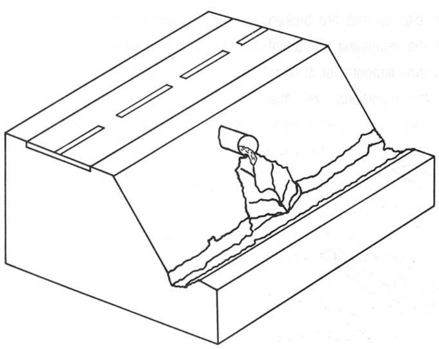
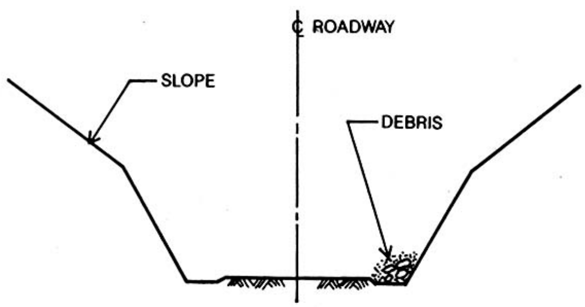
a. *Springs on or at Toes of Slopes:* Springs indicate the presence of the ground-water table as it intercepts the ground surface. Springs also may indicate where water from a water-bearing rock formation has saturated a portion of an embankment or cut slope. Areas around springs will be particularly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

b. *Light and Dark Areas on Slopes:* Differences in color may indicate distinct differences in the amount of water from one area of the slope to another. Again, areas containing the greater amounts of water will be more vulnerable to landslides.

c. *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.

d. *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 of a slope that stay green in dry seasons are sometimes indications of subsurface water.

TABLE 7-6 (Cont'd)
COMMON SIGNS OF MOVEMENTS IN HIGHWAY SOIL SLOPES (After Hopkins et al, 1988)

 *Erosion of toe of cut slope and toe of fill*	***9. Erosion*** a. *Toe of Embankment Slopes:* Surface water from paved ditches or other drainage structures may erode the toe of an embankment, removing supporting soil and causing a landslide. b. *Toe of Cut Slopes:* 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 landslides. c. *On slopes of Embankments:* Surface water from broken paved ditches or other drainage structures often is the source of this erosion. Poor maintenance practices are usually the cause of this type of erosion.
 *Broken paved ditch*  *Slope erosion caused by discharge from drainage structure*	***10. Poor Surface Water Drainage:*** a. *Blocked Culverts:* A culvert that does not permit water to flow properly may cause water to pond next to the toe of embankments. This condition will facilitate saturation of the embankment toe, cause the soil to lose strength and hinder the ability of the soil at the toe to resist the weight of the soil higher on the slope. b. *Broken Paved Ditches:* Paved ditches that are broken permit surface water to flow under the remaining portion of the ditch. This may erode the embankment or it may permit surface water to saturate portions of the embankment. c. *Water Ponding above, below, on, and in Median of 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 weakened failure plane in an embankment. Water ponding at the toe of the embankment will weaken the toe. d. *Drainage Structures Discharging onto Slopes:* Pipes, culverts, ditches, or other drainage structures that permit water to flow onto an unprotected embankment slope may be a major factor in causing landslides. Water from these structures may saturate a soil structure or severely erode the slope.
	***11. Debris on Roadway:*** Debris of soil or rock on the roadway may indicate an unstable slope above the roadway.

Hopkins et al. (1988) proposed a detailed multi-page report which is included in the Appendix A of the FHWA Slope Maintenance Manual. 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, and
- 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 7-5: Proposed Standard Landslide Report Form

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 or to some change in lithology or slope angle.

Boring and other direct investigative techniques for ***subsurface investigation*** for stable slopes as discussed in Chapter 3 and detailed in 132031A "Subsurface Investigation" can be tailored for landslide investigation. It is preferable that the investigation be performed after the site reconnaissance and ground survey, and before the selection of instrumentation. Longitudinal 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. Knowledge of the subsurface strata may suggest where the failure surface is as discussed in the previous chapters.

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) also recommends 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 into the overall geotechnical evaluation.

To supplement the subsurface investigation, ***geotechnical instrumentation*** can be installed to measure the slope behavior and characteristics so that any necessary remedial measure may be taken. These instruments can help defining parameters such as ground water level and pressure, surface and subsurface movement. Furthermore, for a moving landslide, inclinometers (Figure 7-6) may detect the actual failure plane(s). Chapter 10 discusses briefly about the applicable Geotechnical instruments for soil slopes and embankments, including inclinometers, piezometers, tiltmeter, etc. Dunnicliff (1998) provides guidelines for planning instrumentation for monitoring landslides in soils in Section 4 of Chapter 8 "Instrumentation of Soil Slopes and Embankments" of 132041 "Geotechnical Instrumentation." Table 7-7 lists the suitable Geotechnical instrumentations for monitoring landslides in soil. Readers are also referred to Mikkelsen (1996) for detailed descriptions for field instrumentation and measurement techniques associated with permanent borehole installations. Geotechnical instrumentation can also be installed to monitor the post-landslide conditions. Especially, if early signs of movement are observed as shown in Table 7-6, it may provide a forewarning to instability, thus the remedial measures can be implemented before critical situations arise.

7.5 LANDSLIDE ANALYSIS AND MITIGATION

The focus of this chapter is primarily landslide definition, recognition and investigation. The methods of examination and analysis of landslides are identical to the methods discussed in Chapter 5. However, during the investigation, attempts must be made to identify the failure surface for modeling and back analysis. Once the failure surface is identified as shown in Figure 7-7, the shear strength along the identified failure surface at the time of failure can be back calculated using the methods described in Chapter 5 and checked against the laboratory testing results. Especially, the strength properties of many types of natural soils are difficult to determine by the means of laboratory testing, and can only be estimated effectively by back analysis. Duncan in 1996 suggested a three-step process:

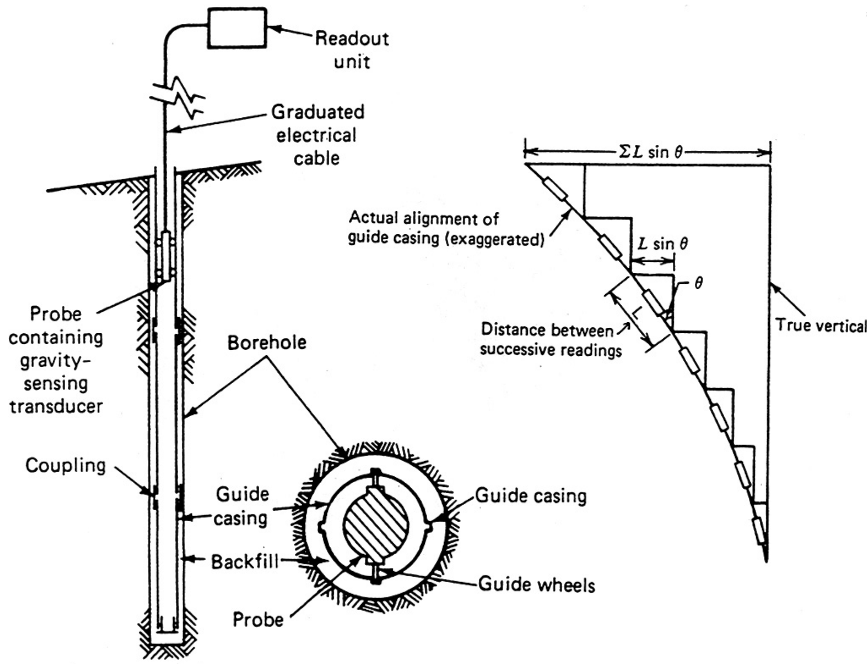


Figure 7-6: Principal of Inclinometer Operation (After Dunnicliff, 1998)

TABLE 7-7
SUITABLE INSTRUMENTS FOR MONITORING LANDSLIDES IN SOIL (DUNNICLIFF, 1998)

Geotechnical Question	Measurement	Suitable Instruments
What are the post-landslide condition?	Surface deformation	Surveying methods Tiltmeters Metallic time domain reflectometry
	Subsurface deformation	Inclinometers Shear plane indicators In-place inclinometers Metallic time domain reflectometry
	Groundwater pressure	Vibrating wire or pneumatic piezometers
Is the slope stable in the long term?	As for "What are the post-landslide conditions?"	As for "What are the post-landslide conditions?"
	Precipitation	Rain gages Snow Stakes
	Load in tiebacks	Load cells

7.5 LANDSLIDE ANALYSIS AND MITIGATION

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1. The best estimates possible must be made of the soil strengths and unit weights using the information at hand. Laboratory tests and strength correlations provide an effective basis for these estimates. The slope geometry and phreatic conditions at the time of failure must also be established.
2. The slide should be analyzed using the estimated properties. If the calculated factor of safety is equal to 1.00, the properties and conditions represent a reasonable model of the slide. If the calculated factor of safety is not equal to 1.00, the strengths are adjusted until FS=1.00. The adjustment ratio need not be the same for all soils involved in the slide. Logically, large adjustments should be made for strengths that are considered to involve greater degrees of uncertainty.
3. When values of soil strength have been determined that give FS=1.00 for the conditions are the time of failure, these strengths are used to evaluate repair measures.

These strengths will then be required for analysis and selection of a proper mitigation method.

Table 7-8 lists same ground methods for stabilizing landslides. Chapter 9 reviews the stabilization methods that are available both during the design of soil slopes and embankments and during the mitigation of a failure of a soil slope or embankment. Readers are also referred to the Transportation Research Board Special Report No. 247, “Landslides, Investigation and Mitigation” for more complete discussions.

TABLE 7-8
GENERAL LANDSLIDE STABILZATION METHODS

Category	Principle	Stabilizing Methods
Geometric methods	Change the geometry of the slope of the sliding mass	Flattening the slope Removing part of the slope Unload the top of the slope Construct a toe berm
Hydrological methods	Lower the groundwater table or reduce the water content of the sliding mass	Surface Drains Horizontal Drains Vertical Drains Drainage Galleris Electro-osmosis Inverted Filters Erosion Protection Measures
Chemical methods	Increase the shear strength of the sliding mass	Grouting Lime and Cement Columns Soil Mixing Freezing Compaction
Mechanical methods	Reduce or resist the external force causing the landslide	Geofabric Reinforcement Rock Bolts Soil Nailing Mini-Piles

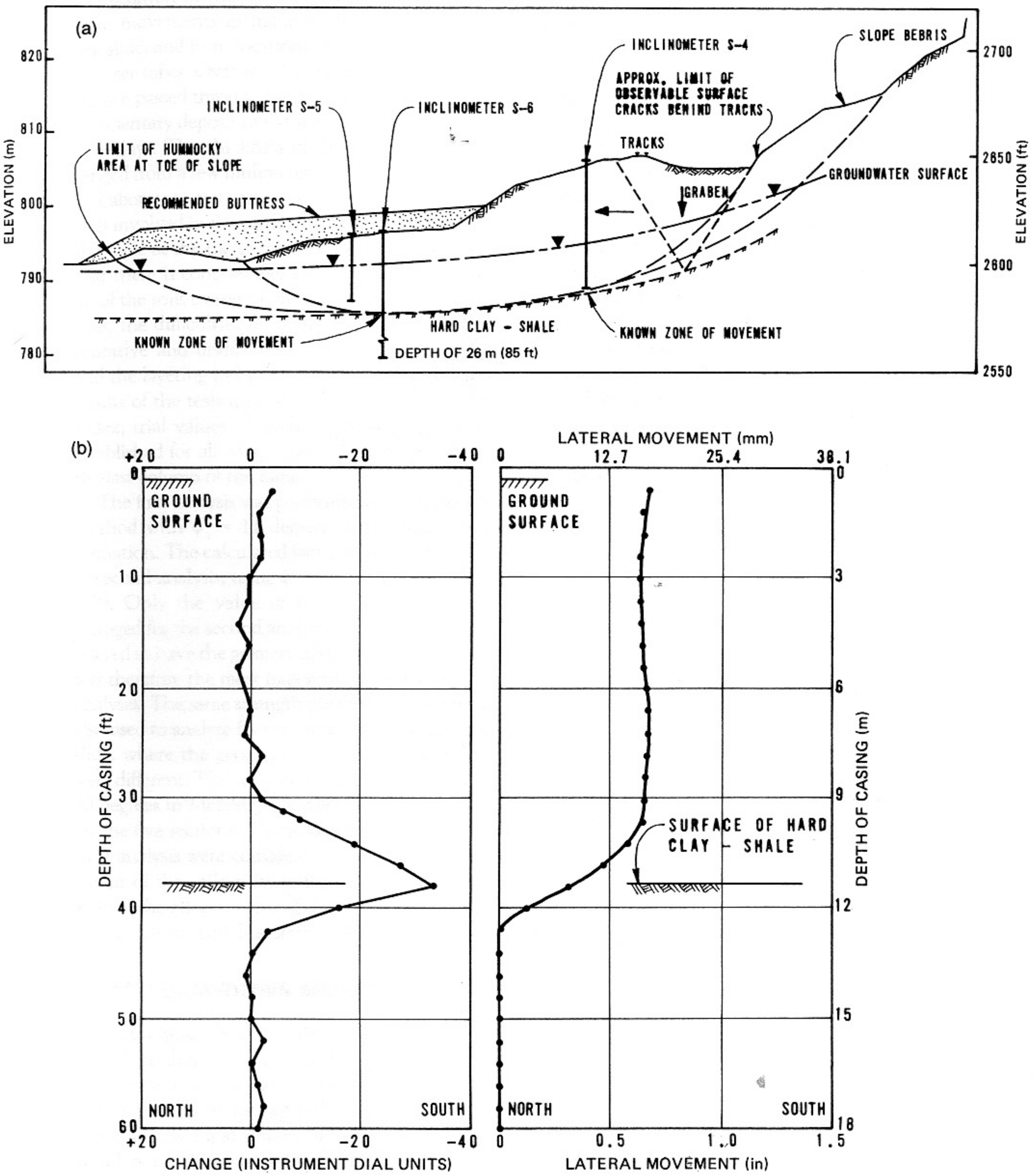


Figure 7-7: Inclinometer Location and data, Fort Benton landslide, Montana (Wilson and Mikkelsen 1978): (a) section through Fort Benton landslide; (b) movement of inclinometer S-5 at toe of slope.

CHAPTER 8
HIGHWAY EMBANKMENTS

8.1 INTRODUCTION

Highway embankments have typically been constructed at roadway and water crossings, grade separations and in mountainous or hilly terrain. However, because of limited access and other geometric requirements, embankments for modern highways are routinely built in areas of flat topography. The design of highway embankments should address not only the stability and settlement of the embankment but also: safety, performance requirements, construction costs, maintenance costs, environmental aspects, aesthetic aspects, and convenience and accessibility. Table 8-1 lists factors involved in design and construction of highway embankments (after Holtz, 1989).

Obviously, performance and safety considerations are especially important for geotechnical engineers. However, highway embankments are often considered to have failed if the roadway pavement surface becomes damaged or uneven. The failure can be caused by settlement of the embankment material and/or foundation soil, minor surface sloughing, or in the case of a catastrophic failure by deep seated global stability failure where the failure surface passes through the embankment foundation. In the latter case, the entire roadway may be demolished. In most cases, however, the failure is generally subtle and is usually caused by creep/consolidation of the embankment, its foundation or both.

TABLE 8-1
FACTORS INVOLVED IN DESIGN AND CONSTRUCTION OF HIGHWAY EMBANKMENTS
(After Holtz, 1989)

ITEM	REMARKS
Additional Construction Costs	Substantial; may be as much as several million dollars per mile.
Safety and Public Relations	Excessive postcontruction differential settlements may require taking part of roadyway out of service for maintenance: • Serious safety hazard for heavily traveled roads. • Major inconvenience – public-relations problems.
Maintenance Cost	May be large: • More expensive construction may minimize post construction maintenance. • Maintenance costs are sometimes regarded as deferred construction cost.
Environmental Considerations	May determine type of highway construction and possible alternatives for foundation treatment.
Foundation Stability During Construction	Detailed subsurface investigations, laboratory and in situ tests, and design studies treatment.
Tolerable Postconstruction Total and Differential Settlements	Appropriate criteria not well formulated; subjective; depends on engineering and public attitudes.
Structure Vs. Embankment	An important decision affecting both construction and maintenance costs.
Construction Time Available	Some alternatives may be eliminated by need for early completion date.

There are many causes for the instability of an embankment. However, most difficulties with embankments come from problematic foundations. Since engineers generally have good control of the embankment material and the construction, the majority of embankment problems are associated with a weak/poor foundation rather than the embankment itself. Failure within the foundation may be influenced by inadequate or inappropriate site investigations, inappropriate design for the foundation conditions or improper implementation of the design during construction. In addition to the common rotational and translational sliding failures, Sowers (1979) illustrated typical embankment foundation problems as shown in Figure 8-1. Other common embankment instability problems are discussed in Chapter 2.

It is not difficult to construct an embankment with fill that is strong, free from volume change, and incompressible. However, such ideal materials may not be readily or economically available at the project site. Alternatively, if an embankment is designed using the locally available lesser quality fill material, instability may occur within the embankment itself due to poor compaction, substandard construction practices and/or poor quality control procedures.

To successfully build an embankment, proper materials, design and construction considerations must be incorporated in the geotechnical design from the planning of the site investigations through the completion of construction. A proper embankment design should consider all foundation conditions, fill materials and construction methods. In addition, the design criteria for an adequate highway embankment must be flexible and take into account long-term considerations and maintenance.

Therefore, the level of embankment design required for features with critical conditions should be more detailed and involved than that required for less critical or routine design. Holtz (1989) listed critical and non-critical conditions for embankment design and construction as shown in Table 8-2.

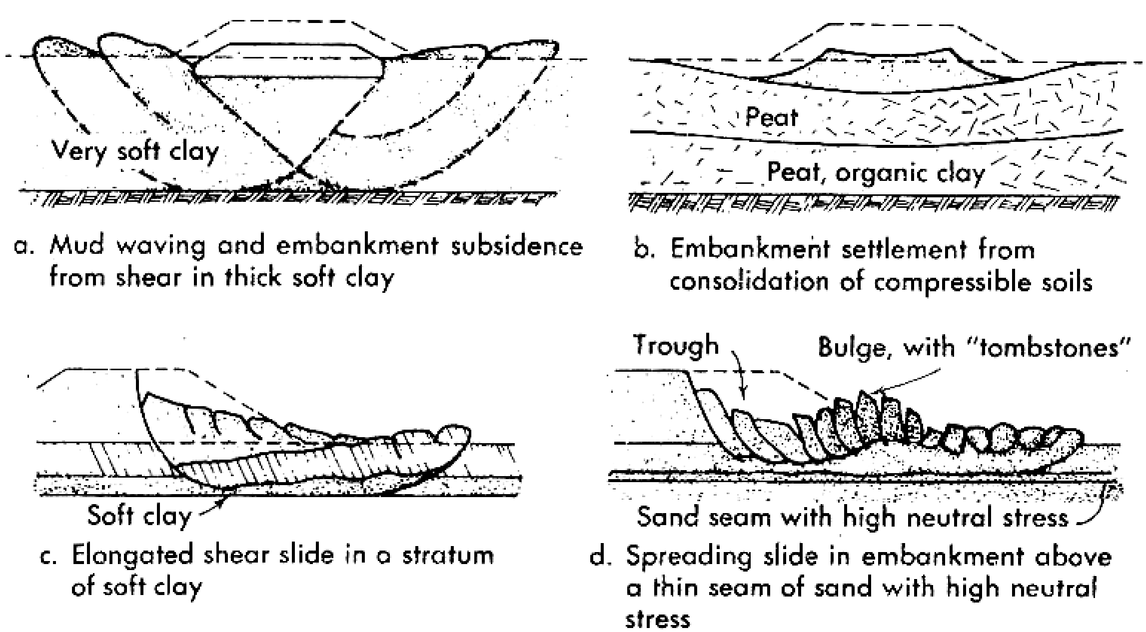


Figure 8-1: Typical Embankment Foundation Failures in Addition to the Typical Rotational Sliding Failure (Sower, 1979).

TABLE 8-2
CRITICAL AND NONCRITICAL CONSIDERATIONS OF EMBANKMENT DESIGN AND CONSTRUCTION (after Holtz 1989)

	Critical	**Non-critical**
Stability	Large, unexpected, catastrophic movement	Slow, creep movements
	Structures involved	No structures involved
	Evidence of impending failure or existing landslides	No evidence of impending failure or existing landslides
Settlement	Large total and differential settlement	Small total and differential settlement
	Occur over relatively short distances	Occur over large distances
	Rapid direction of traffic	Slow transverse to direction of traffic
Repair	Repair cost much greater than original construction cost	Repair cost less than original construction cost
Maintenance	Long-term maintenance cost is high	Long-term maintenance cost is low

In this chapter, the design, construction and performance of highway embankments are presented. The materials, equipment and procedures used in construction of earthfill, rockfill, mechanically stabilized earth, and lightweight fill embankments are discussed in detail. Stability and settlement analyses of the embankment and its foundation are outlined and acceptable safety factors are recommended.

8.2 GEOTECHNICAL DESIGN OF HIGHWAY EMBANKMENT

The geotechnical design of an embankment basically involves the determination of stability of the embankment and the embankment slopes, and settlement of the embankment material and its foundation under the weight of the embankment fill and any superimposed surcharge loads. The analysis and design fundamentals are described in previous chapters 1 through 6. If the embankment is on a soft foundation the design will also require a bearing capacity analysis and a lateral spreading analysis. In general, the major steps in the geotechnical design of an embankment are listed below:

Step 1: Determine the surface and subsurface characteristics (site exploration program)

Step 2: Determine performance requirements of the embankment

Step 3: Determine the types and the material properties for the embankment fill materials

Step 4: Perform embankment stability analysis

Step 5: Perform settlement analysis of foundation soils

Step 6: Perform settlement analysis of embankment material

Step 7: If factor of safety against instability is low, or if the calculated total and/or differential settlement is excessive, re-design the embankment geometry or materials, or select a foundation improvement or stabilization technique and repeat Steps 4 through 6

Step 8: Prepare plans and specifications.

8.3 SUBSURFACE INVESTIGATION

As discussed in Chapter 3, adequate knowledge of subsurface conditions often is the key to successful embankment construction and performance. Detailed subsurface investigations are routinely performed for highway route selection and the design of the embankment sections. FHWA-NHI-01-031 "Subsurface Investigation" (Mayne et al., 2001) presents details of the field and laboratory investigation techniques used on highway projects. It also discusses data presentation and interpretation, and correlation of soil and rock parameters.

8.3.1 Field Investigations

Both geophysical surveys and soil boring explorations can be used. The most common geophysical methods are seismic refraction and electrical resistivity. These techniques are mostly used for determining the position of the groundwater table, the depth to rock, and the delineation of various rock strata. They are less effective in differentiating among soil types and do not provide for visual identification of the subsurface materials. Thus, when these procedures are used, they are almost always supplemented by a boring program.

The spacing between borings will generally depend on the geologic conditions at the site and the guidelines of the transportation agency. For long embankments, the maximum interval between borings can range from 75 to 300 m (about 250 to 1000 feet). AASHTO recommends spacings of 60 to 120 m (200 to 400 feet) when embankment is longer that 60 m (200 feet). In problem areas, the spacing between borings will be reduced significantly when defining the extent of weak deposits.

The required depth of borings is usually related to either the width or the height of the embankment. Considering the depth to which significant stresses are produced by embankment loads, the borings should at least extend to depths equal to one half the embankment's width (i.e., the distance from the center line to the toe of the slope). Alternatively, the borings should be drilled to a depth of twice the height of the embankment. The width criterion is fundamentally more correct, but the height criterion is easier to use in practice. These criteria can be modified by experience with local geologic conditions. Also borings may be terminated at shallower depths when firm bedrock is encountered. On the other hand, borings should never be terminated in a soft soil deposit. In such case, the borings should be continued until firm material is located.

8.3.2 Laboratory Investigations

Routine laboratory investigations include grain size analyses and Atterberg limits tests for classification of all soils used in the construction of the embankment. The shear strength and compressibility of the foundation soils are of particular importance in the embankment design. Where deep deposits of soft to medium stiff clays or loose sands are present, ground improvement techniques (i.e., geosynthetic reinforcement, vibrocompaction, stone columns, dynamic compaction etc., Chapter 9) may be necessary and the laboratory testing program should be designed to provide the data necessary for the design of an appropriate ground improvement program. The laboratory test data required may include strength, compressibility, volume change, swell potential, resilience, permeability and compaction. Descriptions of all these tests are included in FHWA-NHI-01-031 "Subsurface Investigation" (Mayne et al., 2001).

8.4 TYPES OF HIGHWAY EMBANKMENT MATERIALS

Embankments can be built with various types of material – soil, rock, shale, and lightweight materials, etc. The selection of the material used is mostly dependent on the suitable materials that are available locally and therefore can be obtained economically. Suitable materials generated from slope cutting or any excavation associated with the project (or other nearby projects) can be a good source for the embankment fill. If suitable materials cannot be found locally, fill materials will have to be imported, likely at a higher construction cost. In certain instances, however, the unsuitable materials at the site may have to be used which would require treatment/stabilization. The overall cost for this case may be higher than that for the case of imported backfill material.

8.4.1 Soil Fill Embankment

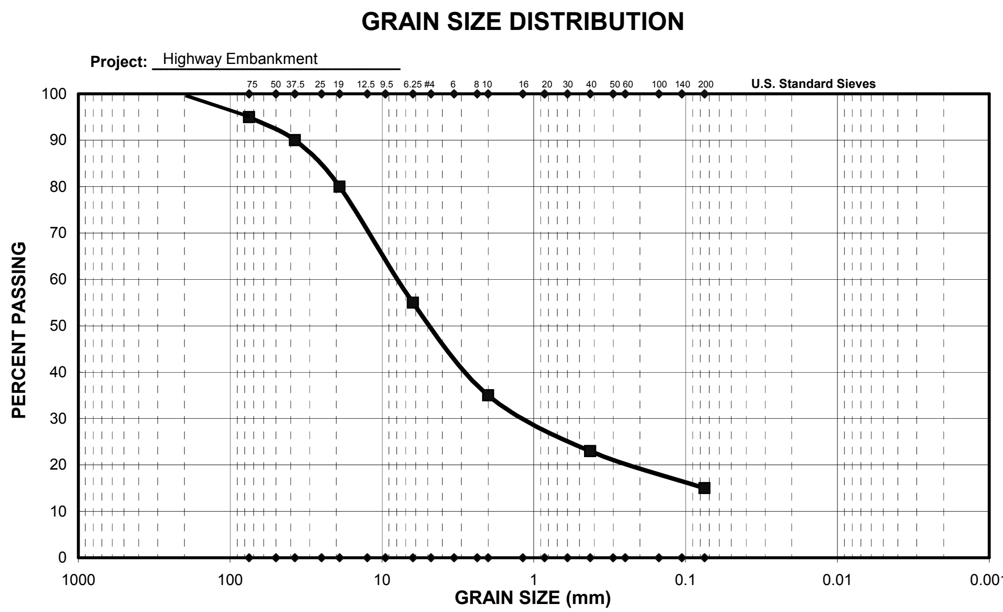
Soil fill embankments are the most common type of embankments used in highway construction. Most soils can be used for embankment construction as long as they are insoluble and substantially free of organic matter, trash, sod, peat and similar materials. However, typical rock flours and clays with high liquid limit or deformation characteristics should generally be avoided. In general, cohesionless (granular) fill is preferred over cohesive (fine-grained) soils.

If a cohesive soil is available at a low cost, and it can be readily brought to the moisture range for optimal compaction and will not cause excessive settlements, it may be used for embankment construction. Note that cohesive soils with high water contents are likely to cause high pore pressures during compaction and ultimately lead to higher long-term settlements. Also, groundwater is more likely to accumulate within an embankment constructed with cohesive materials than with cohesionless soils, and will require a more sophisticated subsurface drainage system. The lower cost of the cohesive materials may not outweigh the cost associated with more difficult construction, with higher settlement potentials and the need to install subsurface drainage systems.

In general, soils with a wide range of particle sizes are preferable to soils with uniform particle sizes. The soils with wider particle size distributions are usually stronger, less susceptible to piping, erosion, compression and liquefaction. However, the presence of cobbles and boulders can increase the cost of construction since stone with dimensions larger than one half of the compacted layer must be removed to ensure proper compaction. A good grain-size-distribution curve for an embankment fill (with a loose lift thickness of 300 mm (12 in)) is shown in Figure 8-2.

In areas where frost-heave can be problematic, select sand and gravel fill should be used near the top of the embankment to provide a fast drainage pathway without piping. The select fill may be processed from the local available materials or imported from elsewhere.

For soil fill to be used in embankments, their strength parameters and deformation characteristics must be established. The samples for testing should be prepared similar to the way that the fill will be placed. For example if the fill in the embankment is to be compacted to 95 percent of the maximum proctor dry density, the sample for testing should be prepared similar to that of the proctor sample. The number of tamps per layer will have to be reduced to simulate the 95 percent density. The samples should be prepared at moisture contents that are expected at time of placement. It is important that the sample be prepared in even layers to avoid significant density variations within the sample.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Source	Sample No.	Depth (m)	Classification		WC	LL	PL	PI
LI	1	0.6 to 1.2	SM-SC	Silty/Clayey brown SAND with gravel	15	22	17	5

Figure 8-2: A Typical Good Grain Size Distribution Curve for Embankment Fill

The following tests are typically required to develop the necessary information for use during the design of the embankment:

Test	Test Parameters
Direct Shear Test	Effective internal friction angle (ϕ') (chesionless soils)
Triaxial Test	
Unconsolidated Undrained	Undrained shear strength (s_u)
Consolidated Undrained (with pore pressure)	Effective internal friction angle (ϕ') and cohesion (c')
1-D Consolidation	Coefficient of consolidation (c_v), compression index (C_c)
Mechanical Sieve Analysis	Grain size distribution
Atterberg Limits	Plasticity, liquid limit, natural water content, plasticity index

The above tests are described in FHWA-NHI-01-031 “Subsurface Investigation” (Mayne, et al., 2001) Table 8-3 presents typical properties of compacted soils used in embankment construction. These properties can be used in the stability and settlement analyses of the embankment and its foundation, as well as the design of structures supported by the embankment or embedded in it.

TABLE 8-3
TYPICAL PROPERTIES OF COMPACTED SOILS (AFTER NAVFAC DM-7.2, 1986)

Group Symbol	Soil Type	Range of Maximum Dry Unit Weight kN/m^3 (pcf)[1]	Range of Optimum Moisture Percent[1]	Typical Strength Characteristics[2]			
				Cohesion (as compacted) kPa (psf)	Cohesion (saturated) kPa (psf)	ϕ' (Effective Stress Envelope Degree)	Tan ϕ'
GW	Well graded clean gravels, gravel-sand mixture.	19.6 – 21.2 (125 – 135)	11 -8	0	0	>38	>0.79
GP	Poorly graded clean gravels, gravel-sand mix.	18.1 – 19.6 (115 – 125)	14 - 11	0	0	>37	>0.74
GM	Silty gravels, poorly graded gravel-sand-silt.	18.9 – 21.2 (120 – 135)	12 -8	ID	ID	>34	>0.67
GC	Clayey gravels, poorly graded gravel-sand-clay.	18.1 – 20.4 (115 – 130)	14 -9	ID	ID	>31	>0.60
SW	Well graded clean sands, gravelly sands.	17.3 – 20.4 (110 – 130)	16 -9	0	0	38	0.79
SP	Poorly graded clean sands, sand-gravel mix.	15.7 – 18.9 (100 – 120)	21 - 12	0	0	37	0.74
SM	Silty sands, poorly graded sand-silt mix.	17.3 – 19.6 (110 – 125)	16 - 11	50 (1050)	20 (420)[4]	34	0.67
SM-SC	Sand-silt clay mix with slightly plastic fines.	17.3 – 20.4 (110 – 130)	15 - 11	50 (1050)	14 (300)[4]	33	0.66
SC	Clayey sands, poorly graded sand-clay mix.	16.5 – 19.6 (105 – 125)	19 - 11	74 (1550)	11 (230)[4]	31	0.60
ML[3]	Inorganic silts and clayey silts.	14.9 – 18.9 (95 – 120)	24 - 12	67 (1400)	9 (190)	32	0.62
ML-CL[3]	Mixture of inorganic silt and clay.	15.7 – 18.9 (100 – 120)	22 - 12	65 (1350)	22 (460)	32	0.62
CL[3]	Inorganic clays of low to medium plasticity.	14.9 – 18.9 (95 – 120)	24 - 12	86 (1800)	13 (270)	28	0.54
OL[3]	Organic silts and silt-clays, low plasticity.	12.6 – 15.7 (80 – 100)	33 - 21	ID	ID	ID	ID
MH[3]	Inorganic clayey silts, elastic silts.	11.0 – 14.9 (70 - 95)	40 - 24	72 (1500)	20 (420)	25	0.47
CH[3]	Inorganic clays of high plasticity.	11.8 – 16.5 (75 – 105)	36 - 19	103 (2150)	11 (230)	19	0.35
OH[3]	Organic clays and silty clays.	10.2 – 15.7 (65 – 100)	45 - 21	ID	ID	ID	ID

Notes: ID: Insufficient data, [1]Obtained from Standard Compaction test results, [2]Strength characteristics are for effective strength envelopes and are obtained from USBR data, [3]These soil types are generally not used as wall backfill, and [4]Cohesion values are reported in NAVFAC (1986); however use of nonzero cohesion value shall be used with caution as described in this chapter.

8.4.2 Rockfill Embankments

Rock materials generated from project excavation can be used for embankment construction. Although used mainly for construction of dams, dikes and water barriers, rockfill embankments are built along highways, particularly when the embankment is located in mountainous terrain, or subjected to erosion from the actions of an adjacent water body (waves, currents, floods, etc.).

A great variety of rock types within the igneous, metamorphic and sedimentary groups can be used for embankment construction. They range from hard, durable rock such as granite and quartzite to weaker rock types like graywacke, sandstone and shale.

However, in general, hard and durable rock types are preferred. They should be able to withstand disintegration due to weathering and frost-heave. No excessive breakage should occur due to quarrying/excavation, loading, hauling and placement operations. The rock should not contain minerals that may breakdown due to weather action or chemical attack, which in turn causes the disintegration of the rock.

The rockfill should consist of a relatively wide range of stone sizes. The stones should be angular but with relatively uniform shapes – one of the stone's three dimensions should not be significantly larger than the other two. These materials in general will settle less than those composed of flat elongated stones, which tend to bridge and then break under stress induced by the overlying fill. Rounded boulders and cobbles are acceptable as long as they are scattered throughout the mass. Rocks that break down to smaller sizes during excavation, transportation and placement/compaction should not be used as rockfill. These non-durable materials should be treated as soils.

Other laboratory tests should also be performed to evaluate the rock materials as a rockfill. These tests include unconfined strength, abrasion resistance, freeze-thaw characteristics, percent of water absorption and petrographic analysis (for identification of minerals that can be weathered easily).

Sound rock is ideal for compacted rock-fill, and some weathered or weak rocks may be suitable as well. Rocks that break down to fine sizes during borrow excavation, placement, or compaction are unsuitable as rock-fill, and should be treated as soil-fill. Rockfill composed of a relative wide gradation of angular, bulk fragment settles less than if composed of flat, elongated fragments that tend to bridge and then break under stresses imposed by overlying fill. Rockfill materials used in highway embankments should be suitably graded with a maximum particle size not more than 90 percent of the lift thickness (0.5 to 1.0 m (1.6 to 3.2 ft)); with not more than 50 percent passing the 25 mm (1 in) sieve, and not more than 6 percent passing the 0.075 mm (.003 in) sieve.

The material properties of rockfill are difficult to obtain because of the size of the particles. Large diameter triaxial test machines are not readily available. As it is a cohesionless material, the strength of rock fill may be expressed by its friction angle. Lightly compacted slopes of angular rockfill are commonly sloped as steep as 1H:1V, demonstrating a friction angle of up to 45^{o}. Test quarry with associated test fill might be useful to determine rock fill properties where there are questions about the suitability and behavior of the proposed rock. Leps in 1970 summarized the laboratory and field results of the rock fill materials from many references. Some of the findings included:

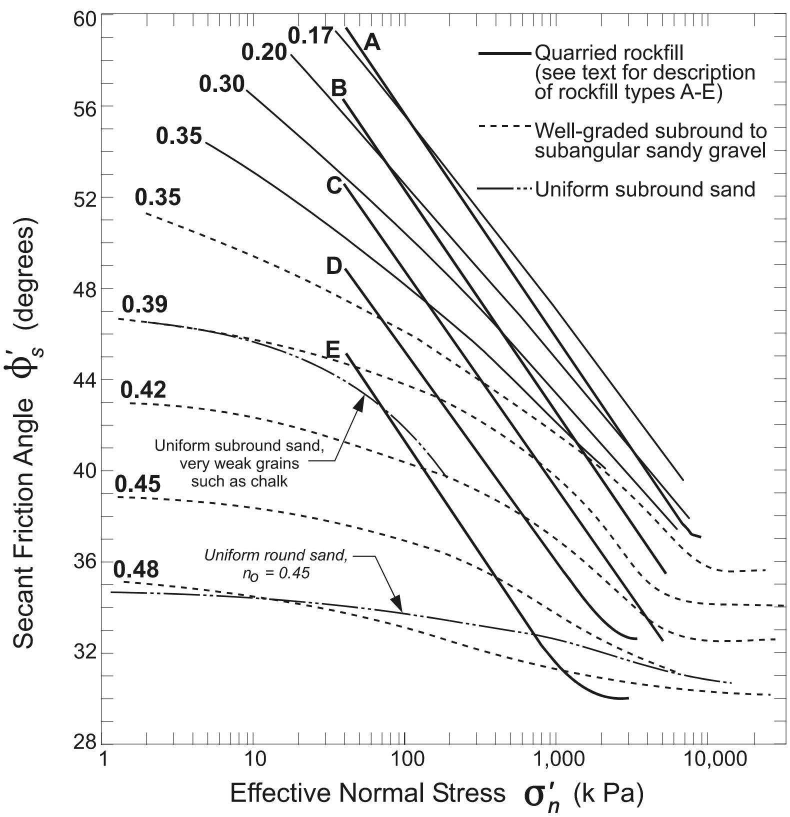
Strength and deformation characteristics of the actual rockfill materials can be simulated by small scale tests.

At any given confining pressure, the angle of internal friction decreases by a small but significant amount as the particle size of the specimen increases.

Rockfill materials with well-graded and round-shaped particles are superior to those with uniformly graded angular particles. This effect is more pronounced when the confining pressure is high.

For any given particle size, the angle of internal friction decreases as the confining pressure increases.

Figure 8-3 shows typical ranges of friction angle for rockfills, gravels, and sands over a wide range of confining stresses and with initial porosities ranging from 0.17 to 0.48. To use this figure, the engineer must select the range of confining stresses that the granular soil will be subject to in the field, thus for embankment loading, the friction angle of the rockfill soil will vary over the range of confining stresses. A conservative single value can be selected based on calculating confining stresses at the bottom, middle, and top of the backfill soil and then averaging. The rockfill grades for quarried materials shown on Figure 8-3, are summarized in Table 8-4 (A through E). Nonetheless, for an average rockfill material, its internal friction angle is unlikely to be less than 35° except at very high normal pressures as shown in Figure 8-3 so it can be used as a conservative parameter when project specific testing is not feasible. In addition, the dry density of rockfill could be as high as 20.4 kN/m^3 (130 pcf) or above, or as low as 14.9 kN/m^3 (95 pcf), depending on the types of rocks, gradation distribution, and the methods of excavations (Lep, 1988). It is common to assume a dry density for approximately 18-19 kN/m^3 (115 to 120 pcf).



Note: 1 kPa = 6.89 psi

Figure 8-1: Typical Ranges of Friction Angle for Rockfills, Gravels, and Sands (after Terzaghi et al., 1996).

TABLE 8-4
UNCONFINED COMPRESSIVE STRENGTH OF ROCK PARTICLES FOR ROCK FILL GRADES SHOWN IN FIGURE 8-3

Rockfill Grade	Particle Unconfined Compressive Strength	
	(MPa)	(psi)
A	≥ 220	≥ 32,000
B	165 to 220	24,000 to 32,000
C	125 to 165	18,000 to 24,000
D	85 to 125	13,000 to 18,000
E	≤ 85	≤ 13,000

Rock materials which are susceptible to slaking, have very low compressive strength, exhibit soil-like characteristics, weather rapidly, or fail to meet the recommended gradation requirements, would not be acceptable as rockfill. If such material is encountered during excavation, it may still be used in embankment construction. In this case, however, the embankment must be designed as a conventional earthfill embankment.

Two index tests can be used to identify non-durable rockfill: the slake-durability test and the simple jar-slake test. The simple jar-slake test can be performed on many core pieces as a rapid screening test. Oven dried pieces are soaked in water, and a jar-slake index I_J is assigned using the descriptive behavior noted in Table 8-5:

TABLE 8-5
JAR-SLAKE INDEX I_J (From FHWA Report No. FHWA-TS-80-219)

Jar-Slake Index I_J	Descriptive Behavior
1	Degrades into a pile of flakes or mud
2	Breaks rapidly and/or forms many chips
3	Breaks rapidly and/or forms few chips
4	Break slowly and/or forms several fractures
5	Breaks slowly and/or develops few fractures
6	No change

An I_J value of 1 or 2 indicates an obvious soft non-durable shale. Values greater than 2 require slake-durability testing.

In the slake-durability test, ten 19 to 25 mm (.75 to 1 in) pieces of oven dried, unweathered shale are placed in a wire-screen drum that is submerged in water and rotated at 20 rpm for 10 minutes. The procedure is repeated on the material remaining in the drum, again after oven-drying. The slake-durability index I_D is the percent of material retained after the two-cycle test.

$$I_D = (\text{Dry wt after 2 cycles/Dry wt before test}) \times 100$$

The index descriptions are shown in Table 8-6.

TABLE 8-6
THE SLAKE-DURABILITY INDEX I_D (From FHWA Report No. FHWA-TS-80-219)

Slake-Durability Test, Two Cycles on Oven dry Samples		
I_D (1) (% Retained)	Type of Retained (2) Wet Material	Shale Classification
<60	T2, T3	Soil-like, Nondurable
60 to 90	T1S, T3	Soil-like, Nondurable
	T1H, T2	Intermediate(3) Hard, Nondurable
>90	T1S, T3	Soil-like, Nondurable
	T1H, T2	Rock-like, Durable

(1) Different Limiting values may be justified on basis of local embankment performance experience.
(2) Type T1 – No significant breakdown of original pieces.
Type T1S – Soft, can be broken apart or remolded with fingers.
Type T1H – Hard, cannot be broken apart.
Type T2 – Retained particles consist of large and small hard pieces.
Type T3 – Retained particles are all small fragments.
(3) Requires special procedures to assure good drainage and adequate compaction (95% Standard Proctor) for loose lift thickness up to 0.6 m (24-in) maximum.

More detail descriptions of these two index tests and the design and construction of shale embankments are presented in the report "Design and Construction of Shale Embankments: Summary" U.S. Department of Transportation, Federal Highway Administration, Report No. FHWA-TS-80-219.

8.4.3 Shale Embankments

In areas where shale and other weak fine-grained sedimentary rock are abundant, they are commonly used as embankment fill material. However, the use of these relatively weak materials has led to excessive settlements and in some cases sliding failures. The main cause of the excessive settlement and sliding problems has been in the method of placement. These weak rocks were placed as rockfill but upon wetting they softened and turned into weak soil. Also, the presence of some durable shale in the mix may have prevented adequate compaction, increasing the potential for sliding and excessive settlement.

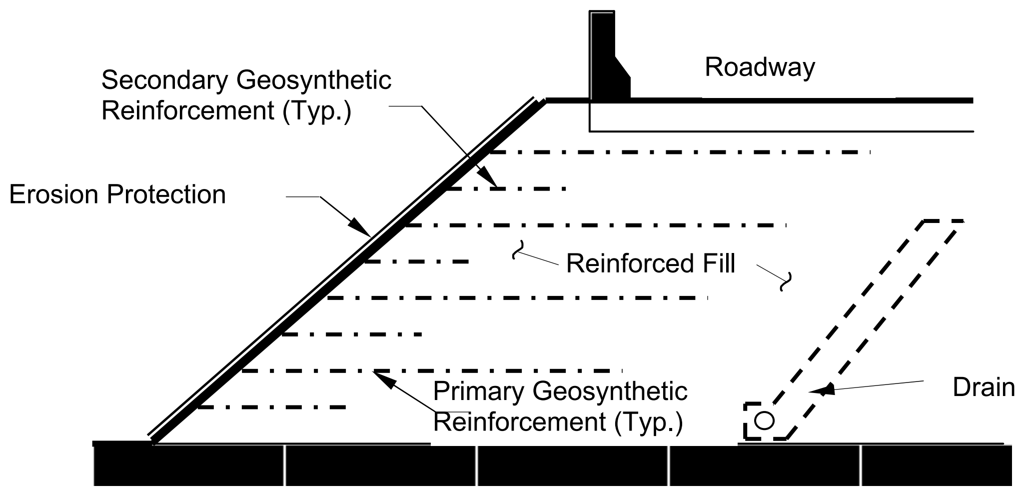
To avoid these settlement and sliding problems, it is essential to separate the durable shale fragments from the non-durable ones. After separation, the competent shale can be placed in thicker lifts and the weak shale should be broken down and compacted as earthfill. For more information the design and construction of shale embankments see FHWA TS-80-219 *Design and Construction of Shale Embankments.*

Laboratory compaction tests (AASHTO T99 – Standard Proctor and T 180 – Modified Proctor; the current trend is to use T180 because it offers a wider range of densities) should be performed on shales that are soil-like to determine their optimum water content and maximum density to be used in design. Soaked-compression index tests on compacted samples can provide a relative measure of the expected performance of high embankments where settlement of the fill materials can be considerable. Samples compacted to various densities are subjected to a vertical pressure equivalent to that generated by one-half of the fill height, then allowed to soak until the measured compression stabilizes. The increase in density and decrease in compression due to compaction can be related to the slake-durability index. The slake durability test may be used to evaluate the effectiveness of greater compaction in reducing settlements to tolerable limits. During construction, laboratory tests similar to those performed for earthfill embankments are usually performed. These tests are discussed in the Chapter 10.

8.4.4 Reinforced Soil Slope Embankment

A reinforced soil slope (RSS) embankment is a formed of Mechanically Stabilized Earth (MSE) that incorporates planar reinforcing elements within a man-made slope, with the inclination of the slope face less than 70 degrees. Multiple layers of reinforcement are placed in the slope during embankment construction to reinforce the soil and increase slope stability. A typical reinforced embankment configuration is shown in Figure 8-4.

A RSS system has three basic elements: geosynthetic or metallic reinforcement, embankment fill and surface treatment. Numerous geosynthetic reinforcements and facing systems are available. The embankment fill may be either granular or cohesive material. The concept of soil reinforcement, details of the basic elements of a reinforced-soil structure, design methods, construction details and durability issues are discussed in detail in NHI 132036A – Earth Retaining Structures, and FHWA NHI-00-043, by Elias, Christopher and Berg (2000) "*Mechanically Stabilized Earth Walls and Reinforced Soil Slopes.*"



(a)

Figure 8-4: Geosynthetic Reinforced Embankment

Reinforcement Types

RSS systems may be designed and constructed with any of the currently available reinforcement products. The RSS system is often described by the reinforcement geometry, stress transfer mechanism, reinforcement material and the extensibility of the reinforcement material. Three types of reinforcement geometry are available:

Linear unidirectional – strips, including smooth or ribbed steel strips, or coated geosynthetic strips over a load-carrying fiber.
Composite unidirectional – grids or bar mats characterized by grid spacing greater than 150 mm (6 in).
Planar bidirectional – continuous sheets of geosynthetics, welded wire mesh, and woven wire mesh. The mesh is characterized by element spacing of less than 150 mm (6 in).

The reinforcement is also typically characterized with respect to the material from which they are manufactured:

Metallic reinforcement – typically mild steel which is usually galvanized or epoxy coated.
Nonmetallic reinforcement – generally polymeric materials consisting of polypropylene, polyethylene, or polyester.

There are two classes of extensibility:

Inextensible reinforcement – the deformation of the reinforcement at failure is much less than the deformability of the soil (e.g., metal).
Extensible reinforcement – the deformation of the reinforcement at failure is comparable or even greater than the deformability of the soil (e.g., geosynthetic)

Advantages

RSS embankments can be constructed at an angle steeper than could otherwise be safely constructed with the same soil (with the existence of a firm foundation). This would result in savings of materials and right-of-way. Right-of-way savings can be a substantial benefit, especially for construction of new roads or road widening projects in urban areas where acquiring new right-of-way is always expensive or, in some cases, unobtainable.

Due to inherent conservatism in the design of embankment RSS, they are actually safer than flatter slopes of conventional embankments designed at the same factor of safety. As a result, there is a lower risk of long-term stability problems developing in the slopes. Such problems often occur in compacted fill slopes that have been constructed to low factors of safety and/or with marginal materials (e.g. deleterious soils such as shale, fine grained low cohesive silts, plastic soils, etc.). The reinforcement may also facilitate strength gains in the soil over time from soil aging and through improved drainage, further improving long-term performance.

Reinforcement placed at the edges of a compacted slope can provide lateral resistance during compaction. The increased lateral resistance allows for an increase in compacted soil density over that normally achieved and provides increased lateral confinement for the soil at the face. Even modest amounts of reinforcement in compacted slopes have been found to prevent sloughing and reduce slope erosion. Edge reinforcement also allows compaction equipment to more safely operate near the edge of the slope. The effects of compaction on the performance of MSE systems is studied in 132036A – Earth Retaining Structures.

Limitations

The following general limitations may be associated with RSS projects:

- Suitable design criteria are required to address corrosion of steel reinforcing elements, deterioration of geosynthetic elements due to exposure to ultra violet rays, chemical attack, heat and other potentially degrading elements in the ground.

- Since certain systems require select granular fill, they may become uneconomical if granular borrow sources are not readily available.

- Maintenance of (e.g., grass mowing) of steep side slopes may require special equipment.

8.4.5 Lightweight Fill Embankments

Lightweight materials such as expandable shale, oyster or clam shell, geofoam, boiler slag, wood chips etc. are sometimes used to construct embankments over soft compressible soils. The lower density of these materials as compared to that of conventional earthfill will result in reduced settlement of the subgrade soils, increased stability of the embankment and reduce the potential impact on structures and utilities present at the site. The loose dry unit weight of expanded shale, for instance, typically varies from 7 to 10 kN/m^3 (45 to 64 pcf), while its compacted dry unit weight (Standard Proctor) ranges from 11 to 13 kN/m^3 (70 to 83 pcf). The unit weight of geofoam can be as low as 0.3 kN/m^3 (2 pcf) (FHWA SA-98-086R "Ground Improvement Technical Summaries"). Figure 8-5 shows construction of bridge approach embankment using geofoam materials, which are the molded expanded polystyrene blocks under the backhoe.

Figure 8-5: I-93 Ramp – Central Artery Project (Big Dig), Boston, MA

In addition to their light unit weight advantage, the gradation of some of these materials (i.e., expanded shale, boiler slag) may be easily controlled to produce a free-draining backfill, thereby avoiding development of water pressures on retaining structures, abutments, culverts or utilities placed adjacent to, or embedded within the embankment. Before using these materials in construction, however, laboratory testing should be performed to define their unit weight, shear strength, corrosivity, and durability (soundness and resistance to abrasion). The lightweight materials must have adequate hardness and durability to resist degradation during placement, compaction, and over the life of the embankment.

Lightweight materials may not be suitable for all cases of embankment construction. For example, the resistance of geofoam material to flotation when flooded, or shredded tires to spontaneous combustion, or wood chips to decomposition should be carefully evaluated before their use. In general, the lightweight materials are more expensive than conventional backfill and are used only in specialized cases where the overall construction cost of the project warrant their use. Table 8-6 provides a range of densities for the various lightweight fill materials and approximate costs.

TABLE 8-6
DENSITIES AND APPROXIMATE COST FOR VARIOUS LIGHTWEIGHT FILL MATERIALS (FHWA NHI-04-001)

Fill Type	Range in Density (kg/m^3)	Range in Specific Gravity	Approximate Cost $\$/m^3$
Geofoam (EPS)	12 to 32	0.01 to 0.03	40.00 to 85.00[1]
Foamed Concrete	320 to 970	0.3 to 0.8	40.00 to 55.00
Wood Fiber	550 to 960	0.6 to 1.0	12.00 to 20.00[1]
Shredded Tires	600 to 900	0.6 to 0.9	20.00 to 30.00[1]
Expanded Shale and Clay	600 to 1040	0.6 to 1.0	40.00 to 55.00[2]
Flyash	1120 to 1440	1.1 to 1.4	15.00 to 21.00[2]
Boiler Slag	1000 to 1750	1.0 to 1.8	3.00 to 4.00[2]
Air Cooled Slag	1100 to 1500	1.1 to 1.5	7.50 to 9.00[2]

1 Price includes in-state transportation cost

2 FOB Plant

3 $Kg/m^3 = 0.0624\ lb/ft^3$

8.5 EMBANKMENT GEOTECHNICAL ANALYSIS

8.5.1 Stability Analysis

<u>Earth Embankments</u>

The embankment stability is usually analyzed using conventional "limit equilibrium" methods of slope stability analysis in which the available shearing resistance along an assumed two dimensional surface (and in some rare cases, three-dimensional) of sliding is compared with the shear stress along the same surface to determine the available factor of safety against sliding. Chapter 5 discusses various methods of slope stability analysis that follow the basic principle of limit equilibrium. The following minimum factors of safety are usually adopted in the design of earthfill embankment slopes:

FS	=	1.25	for static loading conditions – highway side slopes
FS	=	1.30	for static loading conditions – end slopes, slopes with retaining structures
FS	=	1.50	for static loading conditions – for highway slope using fine grained soils (cohesive)
FS	=	1.1	for seismic loading conditions (Pseudo-static analysis)

The shear strength and compressibility of the foundation soils are of paramount importance, particular for high embankments. Weak foundation soils, for instance, may require flatter than normal slopes which would affect the construction cost of the project and right-of-way requirements. Where deep deposits of soft to medium stiff clays, organic soils, or loose sands are present, high embankments may not be feasible without employing extensive ground improvement measures. Ground improvement techniques used for improving embankment performance are discussed in Chapter 9, and covered in detail in 132034A – Ground Improvement Techniques.

The side slopes of earthfill embankments typically range between 1V:2H and 1V:3.5H. Where the foundation soils are weak, however, much flatter slopes may be required which would affect the cost, or even the feasibility, of constructing the highway embankment within the right-of-way available for the project. Soil reinforcement may be used to increase the stability of the embankment and may allow for the construction of steeper side slopes. Generally, for embankments constructed of fine-grained soils, without reinforcement, the higher the embankment, the flatter the slopes.

Rockfill Embankments

Although high rockfill embankments have been used in highway construction, the available literature provides little information on the design and performance of such embankments. However, extensive information is available on rockfill dam projects, and much of this information can be directly applied to highway embankments.

In general, the design of a rockfill embankment is essentially empirical and is mostly based on experience and judgement (Cooke, 1984). The conventional limit equilibrium slope stability analysis, which computes the stability of a sliding mass along a curved or planar surface, is not appropriate since, based on experience and theory, such a shear slide cannot occur within the rockfill (Sherard and Cooke, 1987).

Based on data compiled by Cooke (1984), Sherard and Cooke (1987), Smith, et al. (1963) and USBR (1987), from a large number of rockfill dams, embankment slopes ranging between 1V:1.3H to 1V:1.5H have been used. When the rockfill dam has an earth-core material, flatter slopes of 1V:1.6H to 1V:1.8H are reported to have performed successfully (Dascal, 1987).

8.5.2 Settlement Analysis

The settlement of an embankment is a combination of the settlement of the embankment 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 fill and quite often it is much larger. This is particularly true when the embankment is resting on soft compressible soils.

Embankment Fill Settlement

The embankment fill settlement is due to the reduction of the voids in the fill caused by the dead weight of the embankment, the traffic loading and possibly the weather. 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 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 usually can be ignored because by the time the embankment is completed, most of the elastic settlement has already occurred. The additional elastic settlement due to traffic loading is insignificant when compared to that caused by the loading of the embankment.

For an embankment constructed with cohesive soils, if compaction is performed wet of the optimum water content, a high pore pressure is likely to result. Dissipation of the excess pore pressure over time may lead to excessive consolidation settlement. This placement condition also results in lower undrained shear strength, s_u, values. Compaction slightly dry of the optimum water content, on the other hand, will result in a higher strength and lower consolidation, and is therefore desirable. However, compaction performed at a water content substantially dry of the optimum may lead to higher settlement upon saturation.

When a cohesive fill is placed initially, it is partially saturated. Upon loading of the material, the elastic settlement will occur and the air within the voids will be displaced. This is assumed to happen quite rapidly. Once the air has been displaced, the cohesive soil is assumed to become saturated and the consolidation settlement can be estimated using the one dimensional consolidation theory presented in Chapter 4.

Soil fill embankments generally exhibit greater magnitudes of fill settlement than do embankments constructed of compacted rockfill due to the higher compressibility of the soil 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.**

Embankment Foundation Settlement

If an embankment is constructed over a weak and compressible foundation (e.g. soft clay or peat), the primary and secondary consolidation settlements over time will be substantial. These settlements will typically occur over an extended period of time, especially in organic soils where secondary consolidation also occurs. Consolidation settlements can be estimated based on laboratory consolidation tests performed on samples collected from these materials (Chapter 5). The amount of settlement and the finished grade elevation must be considered when calculating the total load from the embankment on the foundation soil. Therefore, the settlement calculation will be an iterative one. If the settlements estimated are too excessive, ground improvement can be used to improve the magnitude and time rate of settlement. Various ground improvement techniques are presented in Chapter 9.

Elastic settlement due to the embankment load can generally be neglected since there is usually a gap between the completion of the embankment and the start of the roadway construction. At the start of the pavement construction, the elastic settlements are likely to be completed. Elastic settlements caused by the pavement and traffic loading are generally insignificant when compared with those generated by the embankment load.

Settlement Calculation

The method of settlement analysis of foundation soils are discussed in Chapter 4, and in more detail in FHWA NHI-01-023 "Shallow Foundations" (Arman, et al, 2001). The basic requirements for calculating the settlement of an embankment foundation is the knowledge of the compressibility of the foundation soil, and the increase in stress transmitted to the foundation by the embankment loading. The stress distribution due to embankment loading can be estimated using elastic theory, taking into account the geometry of the embankment. Typical stress distribution curves used to calculate the foundation settlement are provided in Chapter 4.

Once the stress increase in any foundation layer is determined, the settlement of that layer can be calculated using conventional settlement analysis procedures developed for shallow foundations on cohesionless or cohesive soils. The total settlement of the foundation soil in cohesionless soils, or total primary consolidation in cohesive soils, is equal to the sum of the settlements calculated for the different compressible layers beneath the embankment. In soils with high secondary consolidation characteristics, the settlement due to secondary compression of the soil is added to the initial and primary consolidation settlements to estimate the overall long-term settlement of the embankment foundation. Secondary compression of cohesive soils is discussed in Chapter 4 and in more detail in FHWA-NHI-01-023 "Shallow Foundations" (Arman, et al., 2001). The generalized step-by-step procedure for determining the settlement of the foundation soil (Clay) due to embankment loading is provided below:

Step 1: Determine the depth of the zone of influence of the embankment.

Step 2: Compute the initial effective vertical stress within the clay layer.

Step 3: Compute the change in effective stress due to the construction of the embankment.

Step 4: Compute the final effective stresses in the clay layer.

Step 5: Determine the compression index C_c from reconstruction of the virgin field consolidation curve from laboratory 1-D consolidation test (Table 4-6).

Step 6: Compute the immediate settlement of the foundation soil using equation 4-16.

Step 7: Determine if the clay is normally consolidated or overconsolidated.

Step 8: Compute consolidation settlement using equation 4-17 or 4-19 depending on if the clay layer is normally or over consolidated.

Step 9: Compute the total settlement of the foundation soil.

Step 10: Determine the time to reach 90% of primary settlement.

Step 11: Compute the secondary settlement using equation 4-23.

Impacts of Embankment Settlement

Excessive embankment settlements may pose serious negative effects on the performance of the highway. Specific examples include:

Bump at the end of the bridge due to differential settlement of the foundation and/or embankment fill with respect to the bridge abutment foundation (typically pile foundation). Loss of smooth transition between approach embankments and bridge abutments.

Increased loads on adjacent pile foundations due to downdrag forces caused by the downward movement of the surrounding soil.

Cracking of the pavement along the highway cross-section, since the settlement induced at the center is usually greater than that at the shoulders.

Appearance of differential settlement in the longitudinal direction, due to non-uniformity of the foundation soils along long stretches of the highway. This may affect the performance of the road (roller-coaster effect), its pavement, and surface drainage.

Reduction in the height of the embankment, which is important when the road has to cross a flood zone.

Damage to culverts, other drainage systems and utilities.

8.6 EMBANKMENTS ON SOFT GROUND

The construction of embankments on soft compressible cohesive soil foundations require special design considerations. Since these projects generate positive excess pore water pressures within the underlying foundation soils the most critical stability condition occurs during the actual construction. After construction is complete as the excess pore water dissipates, the foundation soil consolidates and increases in strength and hence the factor of safety against a shear induced failure increases. Thus with time the foundation soil strength increases and the critical time from a stability stand point is at the end of construction.

The design of embankments over soft soils requires that the stability analysis of the embankment consider the short-term or end of construction case. For this case both construction and failure occurs rapidly enough to preclude significant drainage of the foundation soil. Since there is negligible change in the water content, the in-situ undrained shear strength of the cohesive soil controls stability during construction. It represents the critical condition for loading problems since the factor of safety increases with time due to consolidation.

One technique that is often used during the design and construction of embankments over soft soil to increase the stability of the embankment during construction is to stage the construction. Staged construction involves building the embankment in stages and allowing the a portion of the excess pore water pressure that develops as a result of the load placed on the foundation from that stage of construction to dissipate. After dissipation of a portion of the excess pore water pressure occurs the next stage of construction begins. At this time the strength of the foundation soil has increased as a result of the consolidation that has occurred from the previous stage of construction.

The stability analysis of embankments over soft compressible foundations using staged construction would typically involve the following:

Step 1: Evaluate the stability for the stage 1 loading assuming no drainage during construction (the unconsolidated undrained case). For this analysis the initial stress history of the soft soil governs the available undrained shear strength of the foundation soil.

Step 2: Determine the increase in shear strength of the foundation soils during the time interval between the first and second stage of construction.

Step 3: Evaluate the stability of the second stage of construction. The stability analysis should be a consolidated undrained (CU) analysis.

Step 4: Determine the increase in shear strength of the foundation soils during the time interval between the second and third stage of construction.

Step 5: Evaluate the stability of the third stage of construction. The stability analysis should be a consolidated undrained analysis.

Step 6: Repeat steps 4-5 for each stage of construction.

The estimation of the increase in shear strength during each stage of construction is beyond the scope of this manual. Some discussions are included in Chapter 4, however, for a detailed discussion on this topic the readers are referred to Ladd (1991). Part of the success of staged construction is the monitoring of pore water pressure development and dissipation during construction. Section 10.3 provides specific guidance for construction monitoring of embankments over soft soils.

8.6.1 Preloading and Wick Drains

Often times when constructing embankments over soft soils either the amount of settlement or the time for the settlement to occur is greater than the project requirements will allow. A relatively economical method to improve weak cohesive foundation soils in advance of construction is to preload (or surcharge) the soil. The foundation soils may be preloaded by placing fill (soil or rock) across the site or by lowering the groundwater table. The objectives of preloading the foundation soil are:

Reduce the settlement that would occur from construction of the new structure by temporarily surcharging the site.

Improve the shear strength of the foundation soil by increasing the density, reducing the void ratio, and decreasing the water content.

The amount and duration of the preload should be determined to reduce the post preload settlement of the embankment to an acceptable amount. The amount of consolidation settlement of a soil is a function of the applied load, and the void ratio of the clay. The time that it takes to consolidate clay is a function of the permeability of the clay, and the length of the drainage path. Normally a preload is selected that is greater than the estimated weight of the proposed structure so that the majority of settlement occurs prior to building the structure. Figure 8-6 shows a typical settlement curve for uniformly loaded normally consolidated clays. The figure also shows the settlement curve for the same clay preloaded to a higher vertical stress than expected from the structure. The preload is maintained until the settlement at the site reaches the settlement that would be caused by the weight of the structure. Because the preload was larger than the design load of the structure, the time required to preload the site is less than the time that it would take the soil to consolidate this amount under the structure load.

The steps in designing a preload are listed below:

Step 1: From laboratory consolidation test determine the e log σ curve and estimate the change in void ratio that results from the added weight of the structure.

Step 2: Determine the consolidation settlement of the structure without the preload ($S_{c\text{-embankment}}$).

Step 3: Determine c_v from laboratory consolidation test.

Step 4: Calculate the time to achieve 90% - 95% consolidation.

Step 5: Determine the amount of preload that is available to load the site.

Step 6: Determine the consolidation settlement of the preload ($S_{c\text{-preload}}$)

Step 7: Check the global stability of the site with the addition of the preload fill.

Step 8: Determine the percent consolidation of the preload that would be required to be equal to the design settlement of the structure without the preload ($S_{c\text{-embankment}} / S_{c\text{-preload}}) \times 100$).

Step 9: Determine the time to achieve this consolidation ($t_{preload} = T \times c_v \div H^2$).

The preload may be removed at the time ($t_{preload}$) determined in step 9. The placement and removal of the preload will have transformed the soft foundation soil from a normally consolidated deposit into an overconsolidated deposit. Following the preload, the foundation soil will have desirable characteristics of an overconsolidated soil as compared to a normally consolidated soil. It will be less compressible and stronger. The bearing capacity of the soft soil will have been increased and the settlements that will have resulted from the construction of the structure will have been significantly reduced. It must be noted, however, the short-term undrained slope stability must be maintained during preloading as discussed in the previous section.

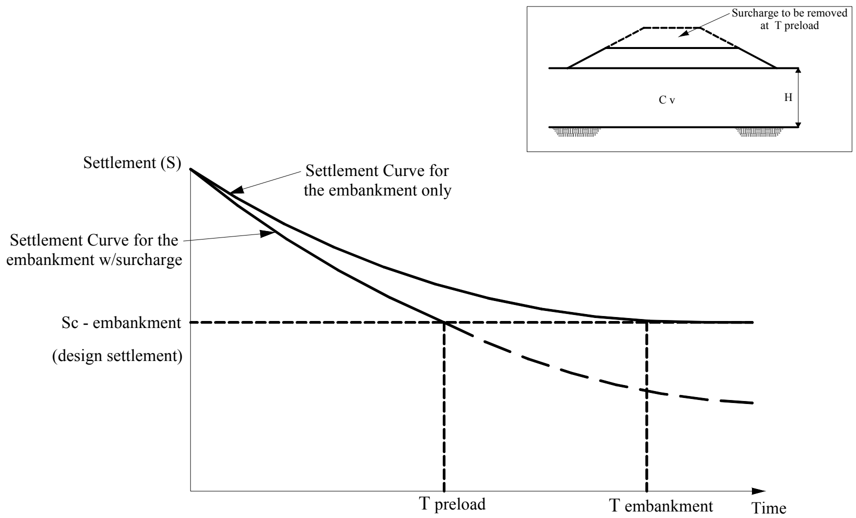


Figure 8-6: Preload Settlement Curve

8.6.2 Wick Drains

Often, there either is not sufficient time for preloading, or not enough soil available to preload the site sufficiently to achieve the desired quantity of consolidation. Wick drains, prefabricated vertical drains, (see FHWA NHI-04-001 "Ground Improvement Methods" for a detailed discussion of wick drains) may be used in conjunction with preloading to accelerate the consolidation process. Wick drains are band-shaped (rectangular cross-section) products consisting of a plastic core surrounded by a geotextile jacket. Figure 8-7 shows a cross section of a wick drain and the installation mandrel. Wick drains function by allowing porewater in the soil to seep into the drain for collection and transmittal up and down the length of the core. The size of a wick drain is typically 100 mm (4 in) wide by 3 to 9 mm (.1 to .4 in) thick. Figure 8-8 shows a typical wick drain installation for an approach embankment at a bridge (with or without additional surcharge).

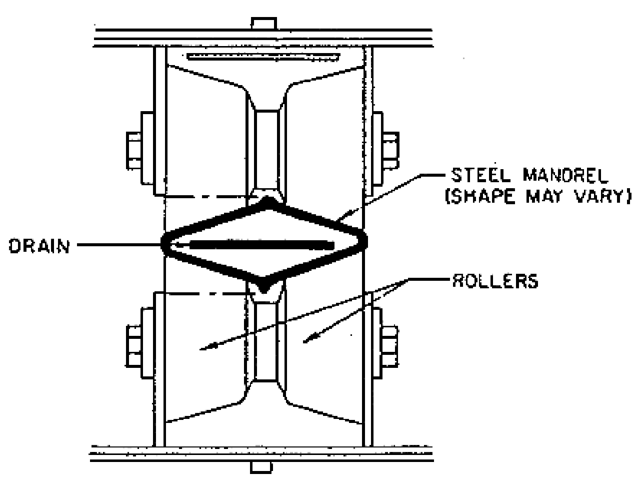


Figure 8-7: Cross Section of Mandrel and Drain (Elins et al., 2001)

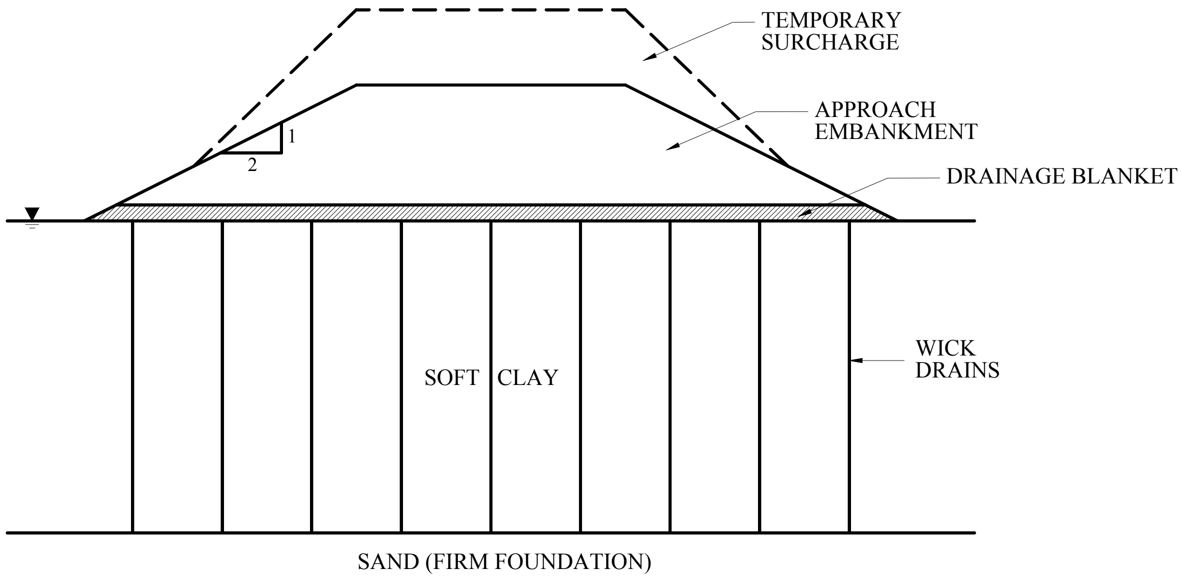


Figure 8-8: Typical Wick Drain Installation

The design of a wick drain system, used to accelerate the time to reach a specified consolidation, starts with the development of the settlement analyses for the structure without the wick drains. This is necessary to determine the total magnitude and the time rate of settlement under the final load (step 1- 9 above). The design of the wick drains is based on procedures for radial consolidation developed by Barron (1948). One dimensional consolidation theory assumptions were applied to Barron's equation to develop the design method for wick drains. The time (t) required to achieve any average degree of consolidation ($\bar{U}_h$) is determined as follows:

$$t = D^2 F(n) \ln[\, 1 \div (1 - \bar{U}_h)] \div 8c_h \qquad (8\text{-}1)$$

Where:
- $F(n)$ = drain spacing factor = ln (D/d) – 0.75 (simplified)
- D = diameter of the cylinder of influence of the drain (see Figure 8-8)
- d = diameter of an equivalent circular drain
- c_h = coefficient of consolidation for horizontal drainage

The horizontal coefficient of consolidation (c_h) of a soil may be estimated from laboratory consolidation tests. Even with proper laboratory techniques and high quality undisturbed samples, it is not unusual that the laboratory results are off by more than 50%. It is for this reason that designers may take a conservative approach in estimating a value for the coefficient of horizontal consolidation. Normally only the coefficient of vertical consolidation (c_v) is obtained from standard consolidation tests from which the horizontal coefficient is estimated.

High quality laboratory tests have consistently shown c_h to be at least two times c_v. A conservative approach that is often used by designers is to assume c_h is directly related to c_v, without direct measurement. For design, c_h is generally taken as 1.5 to 2 c_v.

A wick drain has a rectangular cross-section. Designers have used varying methods to estimate the diameter (d) of an equivalent circular drain. The most commonly used diameter is 60 mm (2.4 in). The diameter of the cylinder of influence of each wick drain (D) is a function of the wick drain layout. When using an equilateral pattern, D is 1.05 times the spacing between drains. In a square pattern, D is 1.13 times the spacing between drains (L) (Figure 8-9).

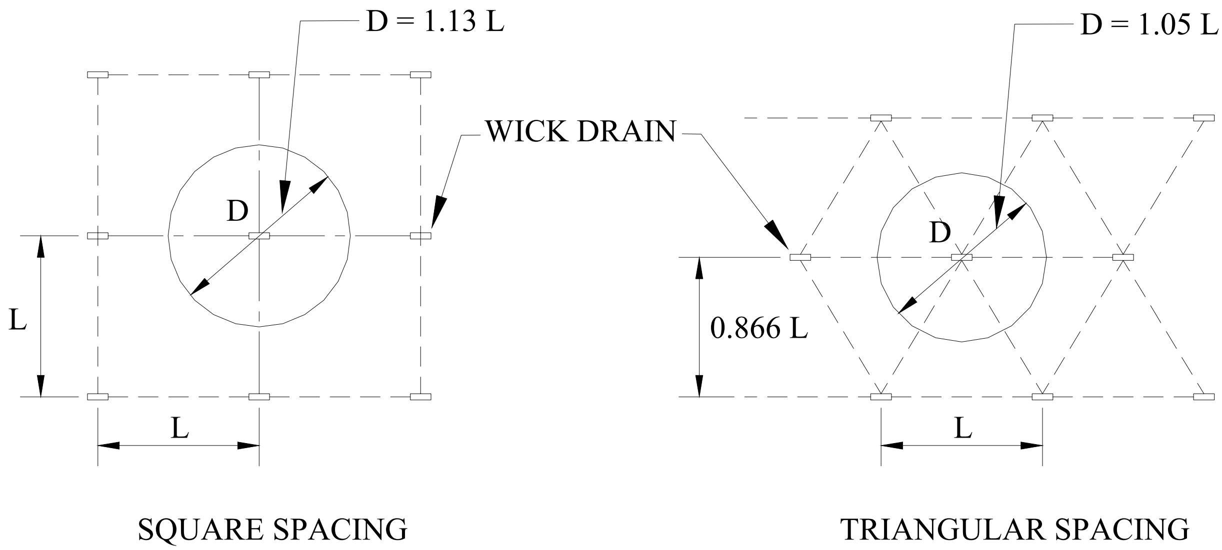


Figure 8-9: Diameters of Influences of Wick Drains (Elins et al., 2001)

Stress-Strain Incompatibility

As discussed in Chapter 2, for embankments on soft ground, the peak strengths of the embankment and the foundation soils cannot be mobilized simultaneously because of the difference in the stress-strain characteristics of the embankment and the foundation (Figure 8-10). Hence, a stability analysis performed using peak strengths of all soils may overestimate the factor of safety (FS) due to this stress-strain incompatibility, and as a result, the embankment on soft ground may fail progressively. Therefore, it is prudent for engineers to reduce the strengths of both the embankment and the foundation soil to allow for this progressive failure effect, using the reduction factors R_E and R_F as shown in Figure 8-11. The use of strength parameters reduced by these factors will help to ensure that neither the embankment nor the foundation are stressed so highly that progressive failure can begin (Duncan et al., 1987).

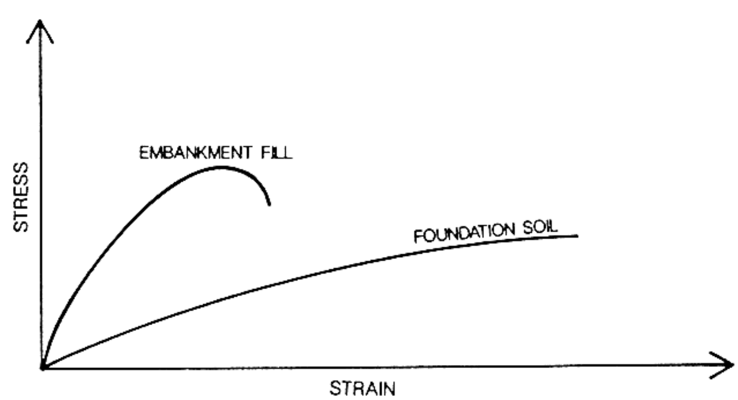


Figure 8-10: Example of Stress Strain Incompatibility

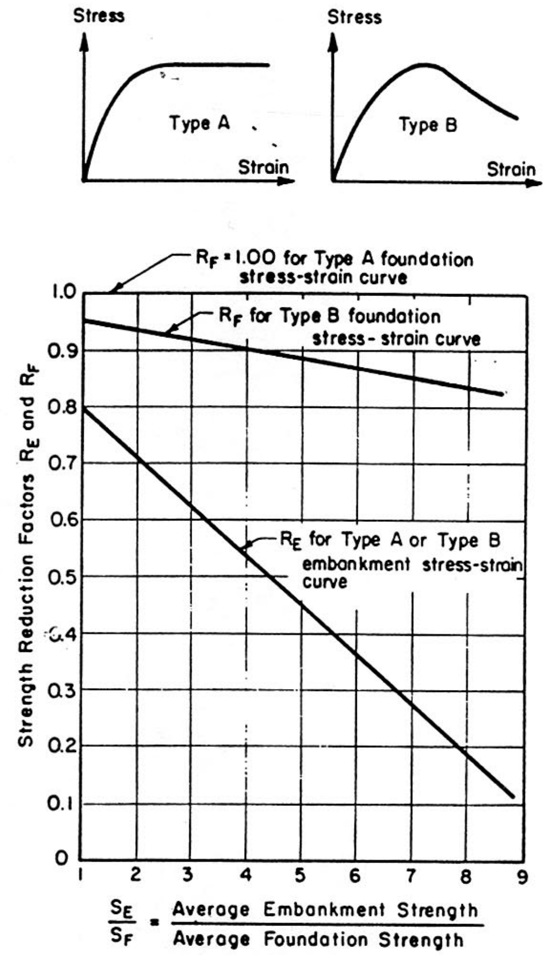


Figure 8-11: Correction Factors to Account for Stress Strain Incompatibility between the Embankment and the Underlying Soft Ground (Duncan et al., 1987)

Furthermore, due to the stress-strain incompatibility, if an embankment constructed of cohesive fill is built higher than some critical height, H_T, there will be a tendency for tensile stress to develop within the embankment, which may result in the development of tension cracks. The approximate value of H_T can be estimated using Figure 8-12.

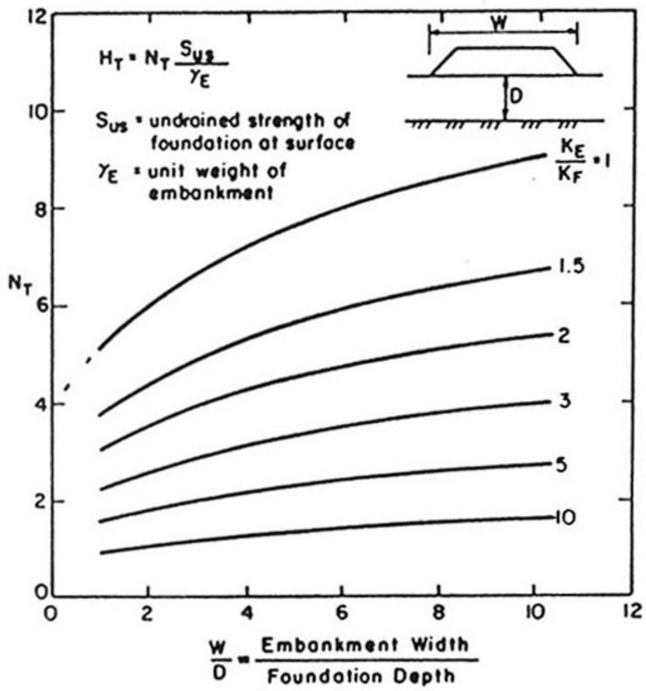
Embankments that are built higher than the calculated H_T should be analyzed assuming that the fill is cracked to a depth of H_c:

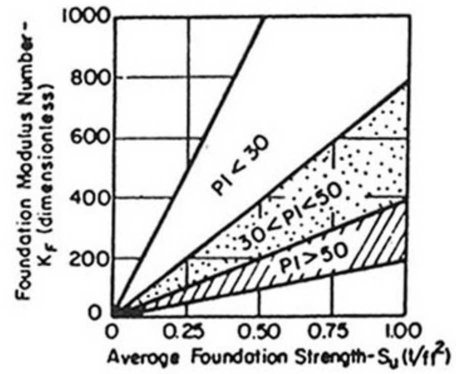
$$H_c = \frac{4c}{\gamma}\tan(45 + \phi/2) \qquad (8\text{-}2)$$

In which

H_c = crack height
c = cohesion intercept of embankment fill
ϕ = frictional angle of embankment
γ = unit weight of embankment

If H_c exceeds the embankment height, the crack should be assumed to extend through the full height of the fill in any part of the embankment, but not into the foundation (Duncan et al, 1987). Zero shear resistance should be assigned to the vertical portion of the failure surface which coincides with the cracks.





Typical values of K_E for compacted fills

Unified Class.	Compaction Water Content		
	Optimum - 3%	Optimum	Optimum + 3%
GC	300 - 1200	200-500	75 - 300
SP	400 - 1000	400-1000	400-1000
SM	300 - 750	300 - 750	300 - 750
SC	250 - 1000	150 - 600	50 - 250
ML	250 - 1000	150 - 600	50 - 250
CL	250 - 1000	100 - 400	30 - 200
CH	100 - 400	50 - 200	20 - 100

Values shown apply to fill materials compacted to dry densities from 90% to 95% of the Std. AASHO maximum. In general, the value of K_E increases with increasing dry density at a given water content.

Figure 8-12: Chart for Estimating H_T When Cracking Will Begin (Duncan et al., 1987)

8.7 LATERAL SQUEEZING OF COHESIVE SOILS UNDER EMBANKMENT LOADING

It is important to determine the amount of long-term lateral displacement or “squeeze” especially if there are pile-supported structures such as bridge piers or abutments in the area of influence. This "lateral squeeze" of the soft foundation soil can transmit excessive lateral thrust which may bend or push the piles out, causing the abutment to rotate back toward the fill, as illustrated in Figure 8-13.

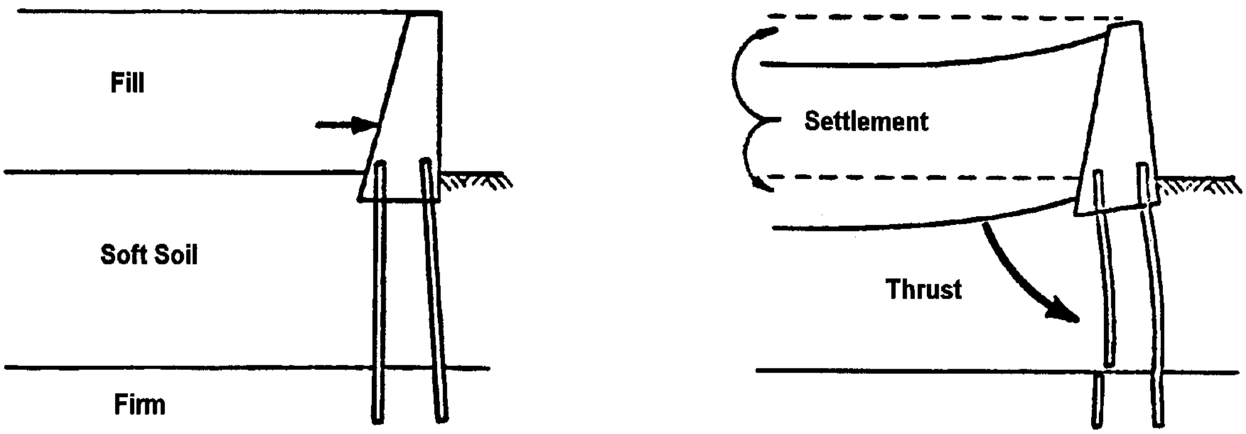


Figure 8-13: Lateral Squeeze due to Horizontal Movement of the Embankment Fill (Cheney, 2000)

Calculation of horizontal displacements in a clay mass under an embankment often required sophisticated numerical analysis. However, there are several simpler empirical calculations available to approximate the potential for lateral squeeze and the amount of horizontal displacement. Cheney et al (1982) indicated that experience has shown that, for the pile-supported retaining abutments; lateral squeeze is likely to occur if the weight of the embankment fill exceeds 3 times the undrained shear strength (S_u) of the soft soils, i.e.,

if $\gamma_{Fill} \text{ x } H_{Fill} > 3\ S_u$ (8-3)

Cheney et al. (1982) also suggested that if the fill load exceeds the three time the undrained shear strength limit, then the horizontal abutment movement that may occur can reasonably be estimated as 25 percent of the vertical fill settlement (i.e., horizontal Abutment Movement = 0.25 x Fill Settlement).

In addition to lateral squeeze, there is lateral deformation associated with vertical deformation (section 4.3.2). Tavenas et al. (1979) illustrated the relation between lateral displacement (Y_m) that develops under the toe of an embankment and long-term settlement (S) as shown in Figure 8-14 and concluded that the progress of horizontal displacement which develops under the toe of an embankment after the end of construction is likely to be proportional to the consolidation settlement S_c. Therefore, the horizontal displacement can be estimated using equation (8-4):

$Y_m = \xi\ S_c$ (8-4)

ξ is a dimensionless factor which can be obtained from Figure 8-15.

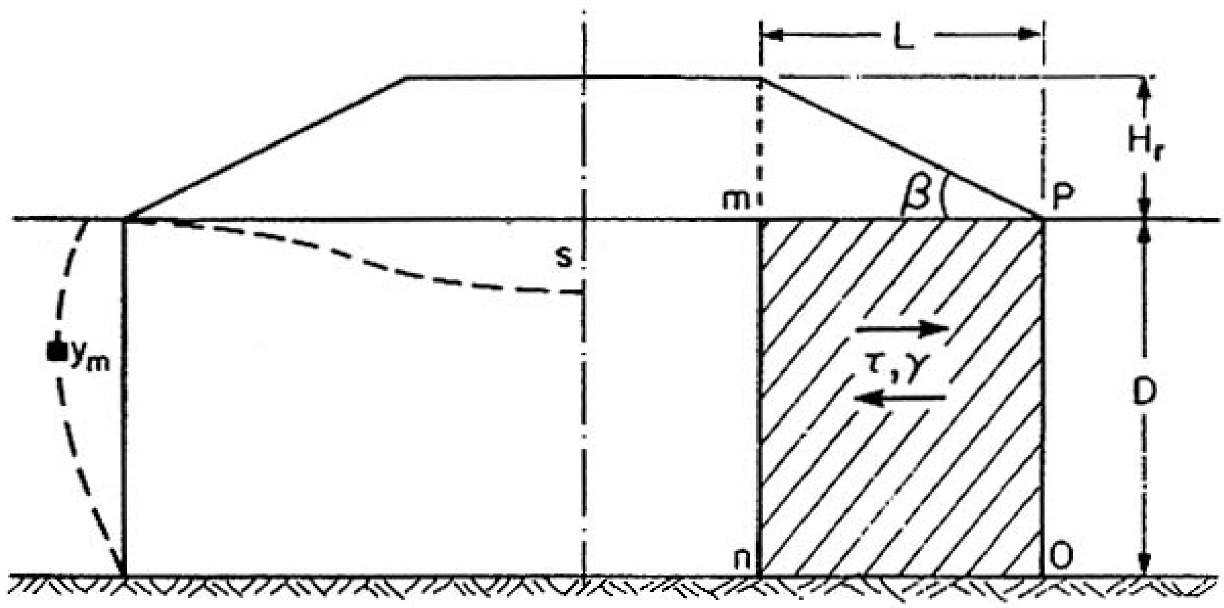


Figure 8-14: Lateral Displacement at the Toe of the Embankment Fill (Tavenas et al., 1979)

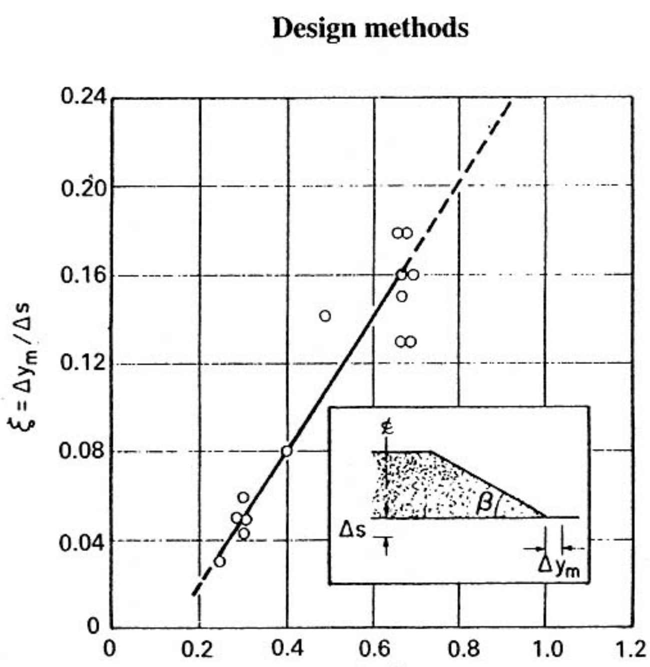


Figure 8-15: Relation between ratio of Lateral Displacement to Settlement and Slope Angle of Embankments (Tavenas et al., 1979)

Finite difference and finite element analyses have also been used to calculate the lateral deformation in the foundation soil below embankments.

8.8 TOLERABLE SETTLEMENT

Once the embankment is designed based on the safe allowable pressure, the differential displacements between locations along the length of the embankment should be estimated. If the differential displacements are excessive the embankment design may require modification or an alternative foundation system. Precise computations are not necessary (Peck, et. al. 1974) and the decision can usually be made as to the adequacy of the foundation type with knowledge of whether the differential displacement is in the neighborhood of ½, 2, or 20 inches (1.3, 5, or 50 cm). AASHTO requires that tolerable movement criteria for foundation settlement be developed considering the angular distortion between adjacent footings. Since the settlement is influenced by the size of the foundation, an iterative process may be required once the initial footing size is chosen on the basis of bearing capacity.

The tolerable movements a structure can withstand depend on the type of structure, the expected service life, and the consequences of excessive movements. In most cases the maximum angular distortion, defined as the ratio (δ'/l, uncorrected for tilt) of the differential settlement (δ') between footings to the span length (l), should be limited to some acceptable value to minimize structural limit state and serviceability problems. Excessive differential displacements can induce significant structural cracking in column-to-beam and abutment-to-beam connections. In some situations the total movements may be the important criteria to ensure serviceability issues are met. Empirical studies of the performance of bridges subjected to various maximum angular distortions between adjacent foundations concluded that the serviceability criteria would be met provided the angular distortion for simple span bridges is less than 0.005 and for continuous span bridges is less than 0.004 (Moulton 1982). Using this criteria a continuous span bridge having a span length of 15m (50 feet) can tolerate a differential settlement of 6 cm (2.4 inches) between supports.

The studies by Moulton et al., also found that bridges that experienced horizontal movement, or horizontal movement in conjunction with vertical movements, exhibited a high frequency of distress. This suggests the problems associated with bridges were to a large extent controlled by horizontal movements and the focus of predicting only vertical movements is not appropriate. It should be noted that the database consisted mainly of bridges that had experienced deformations (223 of 314). As a result, the conclusions may not be entirely applicable for all bridges.

They recommend limiting horizontal movements to 40 mm (1.5 inches) at the abutments. However, there is no convenient way to estimate horizontal movements at abutments for either shallow or pile-supported foundations. Terzaghi et al. (1996) recommend limiting the embankment surcharges to less than one-half of the bearing capacity of the subgrade soils to minimize the risk of progressive lateral movements. No systematic study has been made to our knowledge that confirms this criteria is reliable under all possible situations.

CHAPTER 9
SOIL SLOPE STABILIZING METHODS

9.1 INTRODUCTION

There are several alternates available to a design engineer when the soil slope or embankment is unstable or unacceptable settlements are anticipated. The most obvious is to flatten the slope face or reduce the height of the slope and/or embankment. However, it is not always feasible due to the right of way constraints and other requirements. Therefore, this chapter discusses briefly the following common stabilization techniques that may be considered to improve the performance of a soil slope or embankment:

- Load Reduction
- Surface and Subsurface Drainage
- Erosion Control/Slope Protection
- Butresses/Berms
- Earth Retaining System
- Deep Foundation Reinforcement
- Soil Improvement
- Preconsolidation
- Densification

For a more detailed discussion of these techniques the reader is directed to NHI 132034 – Ground Improvement Methods, FHWA-SA-98-086R “Ground Improvement Technical Summaries” and TRB Report 247 “Landslides Investigation and Mitigation”.

9.2 LOAD REDUCTION

The performance (stability and settlement) of soil slopes and embankments may be improved by using lightweight fill material. When a fill for an embankment or sideslope is placed on soft ground, the main driving force in the stability of the structure is the weight of the fill itself. The use of a lightweight fill will reduce this driving force and increase the stability of the structure. It will also reduce the settlement of the structure by decreasing the compressive load on the foundation soil. The amount of reduction in the driving force and the settlement will depend on the type of lightweight material that is used. There are many types of lightweight fill materials that have been used for roadway construction (i.e., geofoam, foamed concrete, wood fiber, shredded tires, expanded shale and clay, flyash, and boiler slag). These materials have a wide range of densities (i.e., 12 – 1500 kg/m^3 (.75-95 lb/ft^3)). Some lightweight fill materials have been used for decades while others are relatively recent developments.

Wood fiber has been used for many years by timber companies for roadways crossing peat bogs and low lying land as well as for repair of slide zones. Slags have been produced by the steel companies since the start of the iron and steel industry. Initially, the slags were stockpiled as waste material, but beginning around 1950, the slags were crushed, graded, and sold for fill materials. Geofoam is a generic term used to describe any foam material used in geotechnical engineering applications and includes expanded polystyrene (EPS), extruded polystyrene (XPS), and glassfoam. Geofoam was initially developed for insulation material to prevent frost from penetrating soils and is the lightest of the light weight fill materials.

Shredded tires are a relatively recent source of lightweight fill material. The availability of this material is increasing each year and it's use as a lightweight fill is further promoted by the need to dispose of tires. In 1995, three tire shred fills with thickness greater than 8 m (26 ft) experienced an unexpected internal heating reaction. As a result, FHWA issued an Interim Guideline to minimize internal heating of tire shred fills in 1997, limiting tire shred layers to 3 m (910 ft).

Expanded shale has been used for decades to produce aggregate for concrete and masonary units. In the 1980's lightweight fill started to be used in geotechnical applications. For a detailed description of these material and their engineering properties see NHI 132034 – Ground Improvement Methods and FHWA SA-98-086R "Ground Improvement Technical Summaries."

9.3 DRAINAGE

Drainage of both surface and subsurface water is arguably the most important stabilization measure for slopes and embankments as water is typically the trigger to slope and embankment failures. Drainage reduces the weight of the sliding mass (driving force), increases the effective weight and, therefore, the shear resistance of the soils (resisting force). It also reduces seepage forces which may cause erosion and piping of the slope or embankment.

9.3.1 Surface Drainage

Surface drainage details are a very important aspect of the slope and embankment design. Figure 9-1 illustrates a typical highway side slope with a curb and ditch detail.

If the slope/embankment surfaces cannot adequately convey the water to other safe areas without damaging the embankment slopes, erosion will occur and ultimately cause the embankment to fail. Insufficient surface drainage will also allow surface water to seep through and increase the groundwater level within the embankment. As pore pressure within the embankment increases, so does the potential of a slide.

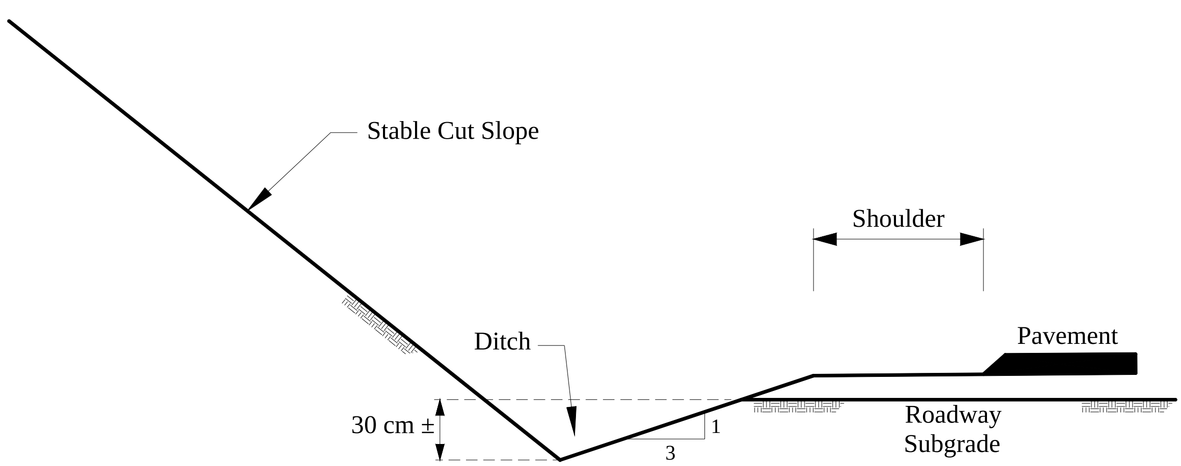


Figure 9-1 Typical Highway Sideslope with a Curb and Ditch Detail

The best way to drain highway embankments is to allow water from the pavement to drain uniformly (sheet flow) over the side slopes and free flow to the adjacent ground. However, if the spread of this water can damage the adjacent ground, drainage swales or ditches should be constructed at the toe of the slope or embankment to transport the water away from the adjacent ground.

The frequencies of a storm event to be used in runoff calculations ranges from a 10-year rainfall for minor roads to a 100-year event for major interstate highways. The first step in the analysis is to determine the estimated runoff volume. This is typically done using the rational method (FHWA HI-95-038 *Geosynthetic Design and Construction Guidelines*). The rational method gives fairly good results for small watersheds less than 50 hectares (.19 square miles). Other parameters to be considered in drainage system design include the area and shape of the catchment area, the gradient and length of the slope to be drained. Based on the runoff and the gradient of the slopes, the velocities of the sheet flow over the slopes can be estimated. Appropriate slope protection will then be designed to withstand these velocities. Two of the most commonly used slope protection methods are vegetation and riprap. Slope vegetation has been demonstrated to be very effective at protecting slopes and embankments from surface erosion. Geosynthetics may be used to protect the slope face during the establishment of the vegetation and/or to reinforce the root structure of the vegetation to increase it's resistance to prolonged periods of flow. For flow velocities and duration's that exceed the capabilities of reinforced vegetation hard armor is required. For more information on the engineering use of erosion control products see FHWA HI-95-038 "*Geosynthetic Design and Construction Guidelines.*" The longitudinal ditches, if required, at the toes of the slopes shall be designed to carry these runoffs to safe areas.

9.3.2 Subsurface Drainage

If an embankment or slope is constructed with relatively low permeability soils, significant rainwater and snowmelt can accumulate within the slope or embankment. This groundwater table fluctuates seasonally and these fluctuations can be large. To prevent groundwater within the slope or embankment from reaching a level that will lower the FS against a slope failure to an unacceptable level or even cause slope instability, a subsurface drainage system may be required to lower the groundwater to satisfactory levels. Methods commonly used to accomplish subsurface drainage are drainage blankets, trenches, cut-off drains, horizontal drains (weep holes), relief drains and drainage tunnels. It is more cost efficient to incorporate these methods into the design than to include them as remedial measures during or following construction. The design of the subsurface drainage system relies on the knowledge of the following properties of the major soil groups encountered: shear strengths, compressibility, swell and dispersions characteristics, the unit weights (dry, moist and saturated), the coefficient of permeability, the in situ water content, specific gravity, grain size distribution and the effective void ratio.

As compared to embankment slopes, natural slopes or cuts are rarely homogeneous enough to allow reliable subsurface drainage design according to simple principles of dewatering by slots and wells. The secret of success, for these structures, lies in a good understanding of the geological structure and choosing a drainage system layout that increases the probability of intersecting the major water bearing layers (Hausmann, 1992). Table 9-1 conveys the need for subsurface drainage in different conditions and illustrates the stability problems in poorly drained slopes and hillsides. The volume of water flowing from a drain is determined by permeability and hydraulic gradient. When a drain is installed, the groundwater level will be lowered, thereby reducing the hydraulic gradient. The seepage will gradually reduce from its initial values to a steady state. This flow reduction is not necessarily an indication of drain deterioration. There may not be any noticeable seepage flow from the drains in cohesive materials with low permeability; even when they are operating successfully.

TABLE 9-1
COMMON SLOPE SEEPAGE CONDITIONS AND EFFECTS ON STABILITY

Slope/Embankment Condition	**Effects on Performance (Stability)**
Naturally dry or well drained conditions	Favorable with little seepage, serves purpose of subdrainage.
Subjected to a normal flow which is uncontrolled because of rainfall	Stability reduced as pore pressures are produced. Flow is generally parallel to slope.
Subjected to vertical downward flow, encouraged by subdrainage systems	Vertical downward flow reduces pore pressures and increases stability

Monitoring is important for any subsurface drainage program. Not only should piezometers be installed to measure the pre-construction pore pressure, but also they should be monitored during and after construction to observe the effects of the subsurface drainage systems. Long-term piezometric readings can indicate reduction of drainage efficiency caused by siltation, deterioration of seals, breakdown of pumps, etc.

Readers are referred to “Subsurface Drainage for Slope Stabilization” by K. Forrester, 2001, and TRB Report 247 “Landslides Investigation and Mitigation” for detailed discussions.

Drainage Blankets

The drainage detail on Figure 9-2 is included to stabilize the in-situ groundwater profile which exists prior to construction. The inclusion of the drainage blanket prevents the development of excess pore pressures which result from the embankment construction. It is common practice to totally remove poor soil when there is a thin layer (not more than 3 meters (10 ft)) at a shallow depth (not deeper 5 meters (16.4 ft)) beneath a proposed slope or embankment. The bottom of the excavation is then typically covered with a drainage blanket consisting of a geotextile (filter) wrapping a 150 to 600 mm (6 to 24 in) thick open graded aggregate and a perforated pipe, to capture flow. To avoid blockage of holes by vegetation, the first 1.5 meters (5 ft) of the outlet end of the perforated pipe should not be perforated. To minimize surface erosion, a flexible drainage ditch should be installed to convey water flow from the outlet of the pipe.

When excavating a face for a blanket drain, Forrester (2001) suggested that final trimming should be done with light rubber-tired equipment to avoid disturbance or compaction of the face and to leave seepage outlets from the face as open as possible. If geotextile is placed directly against the face, then any depressions in the face deeper than approximately 50mm (2 in) should be filled with medium sand and smoothed over to present an even surface for geotextile placement.

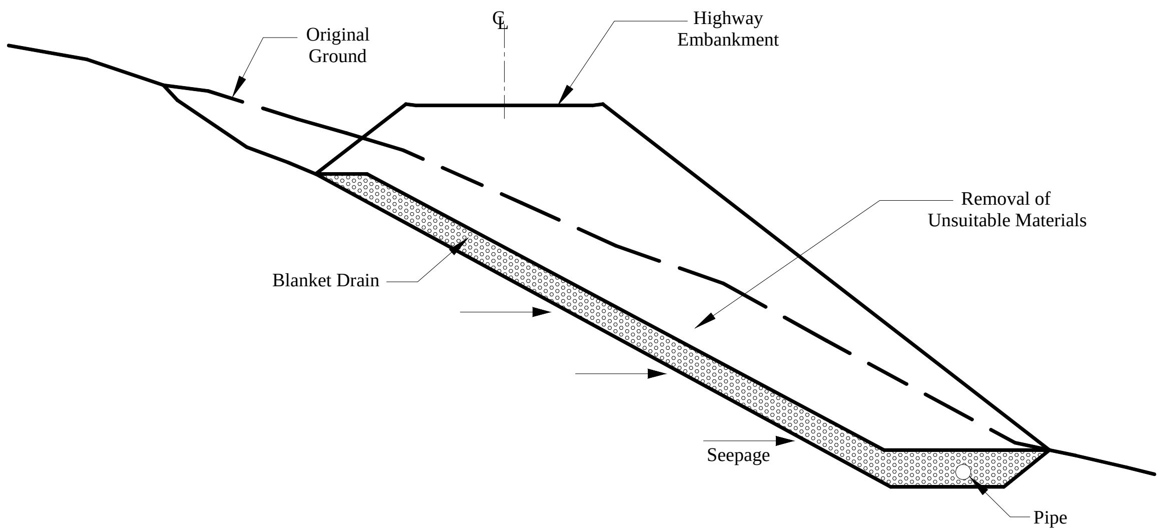


Figure 9-2 Placement of a Drain Blanket underneath Embankments Subsequent to Removal of Poor Material

Deep Trenches Drains

Deep drainage trenches are often used when subsurface water or soils of questionable strength are found at such great depths that excavation of the in-situ soils is not practicable. Drainage trenches usually are excavated at the steepest stable side slopes for the construction periods. Shoring may be required. Logically, any trench so excavated should extend below the water-bearing layer to a position where the calculated factor of safety against slope failure is acceptable. A layer of pervious material encased in a geotextile filter and an underdrain pipe are placed in the excavation before the trench is backfilled (Figure 9-3).

The number of trenches needed depends on hydrogeology and geomorphology of the slope under study. If the slope is in a natural depression of limited areal extent, one trench normal to the centerline of the road may be sufficient. In the case of large areas, an extensive system of trenches, often in the form of a herring-bone pattern, may be necessary.

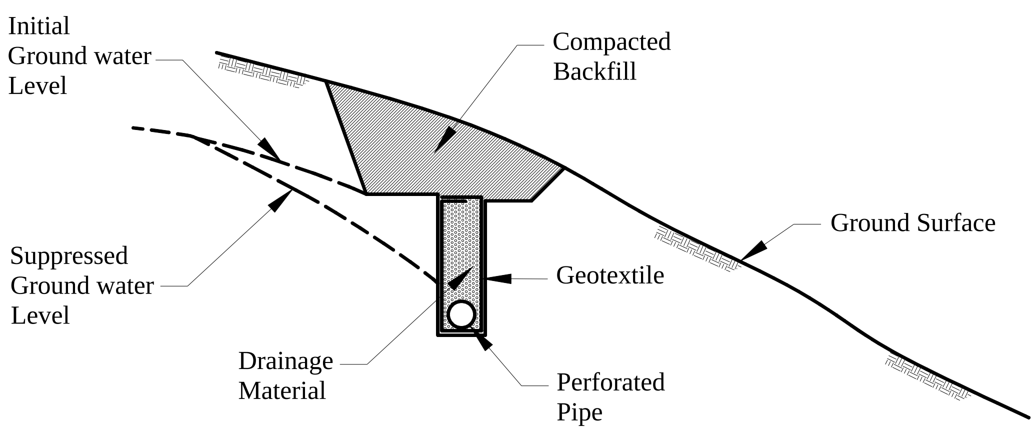


Figure 9-3 Typical Deep Trench Drain

Cut-off (Interceptor) Drains

Where shallow groundwater is encountered, cut-off drains can be used to intercept the groundwater flow. A typical layout is shown in Figure 9-4. A shallow trench is constructed and a drain is constructed in the trench. The cut-off trench intercepts the surface groundwater and removes it from the ground. Runoff from the upper slopes should be collected in drainage channels. Design the free draining material used to backfill the trenches should conform to standard filter criteria if this is not possible a geotextile filter may be used. The size of perforations in pipes should be compatible with the grain size of the backfill filter material. The stability of the slope or embankment should be checked for the condition during construction of the cut-off trench. The trench may temporarily lower the stability of the slope or embankment during construction.

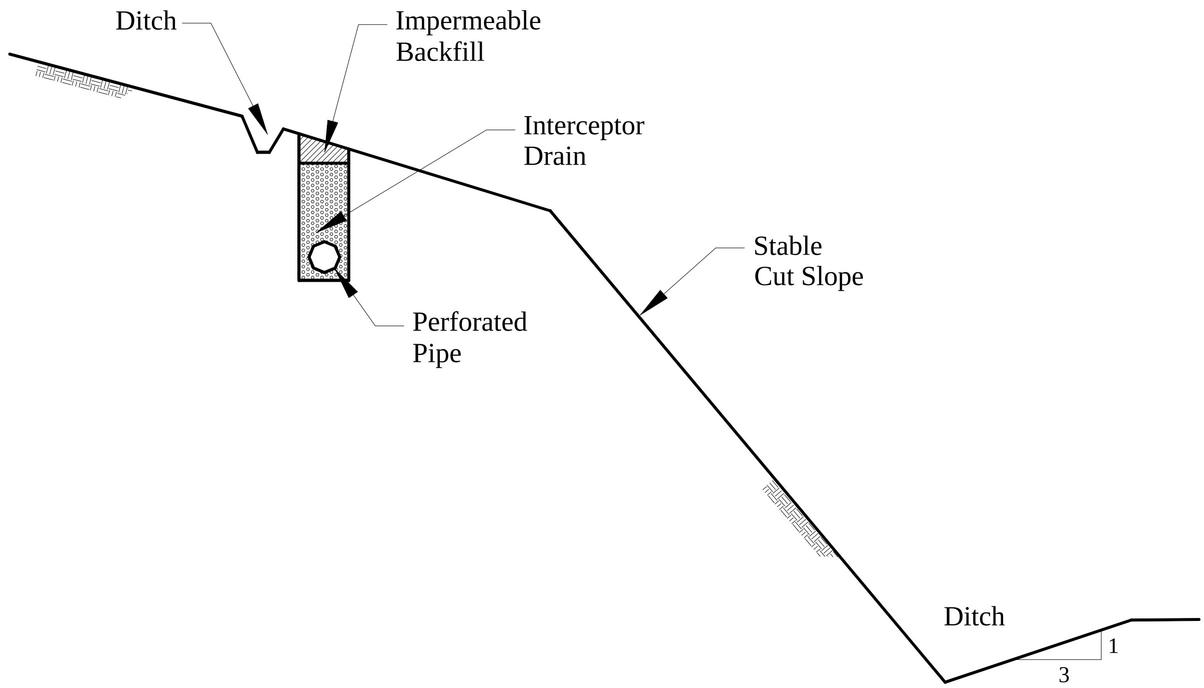


Figure 9-4 Typical Cut-off Interceptor Drain

Horizontal Drains

Horizontal drains can be used where the depth to subsurface groundwater is so great that the cost of excavating or placing drainage trenches is prohibitive. They are designed specifically to lower the seepage pressures in slopes to prevent failure. Horizontal drains provide a popular and economic mean to stabilize a soil slope or an existing landslide as shown in Figure 9-5. A horizontal drain is a small diameter, (75 to 100 mm (3 to 4 in)) hole drilled into a 5 to 10 percent grade slope and fitted with a 50 to 65 mm (2 to 2.5 in) diameter perforated pipe wrapped with a geotextile or a slotted screen pipe. Water captured by the drains is often discharged on the road shoulder ditch, or in large-scale installations, into collector pipes (200 mm (8 in) diameter). The first 1.5 meters (5 ft) of drainpipe immediately next to the outlet should not be perforated to prevent vegetation growth.

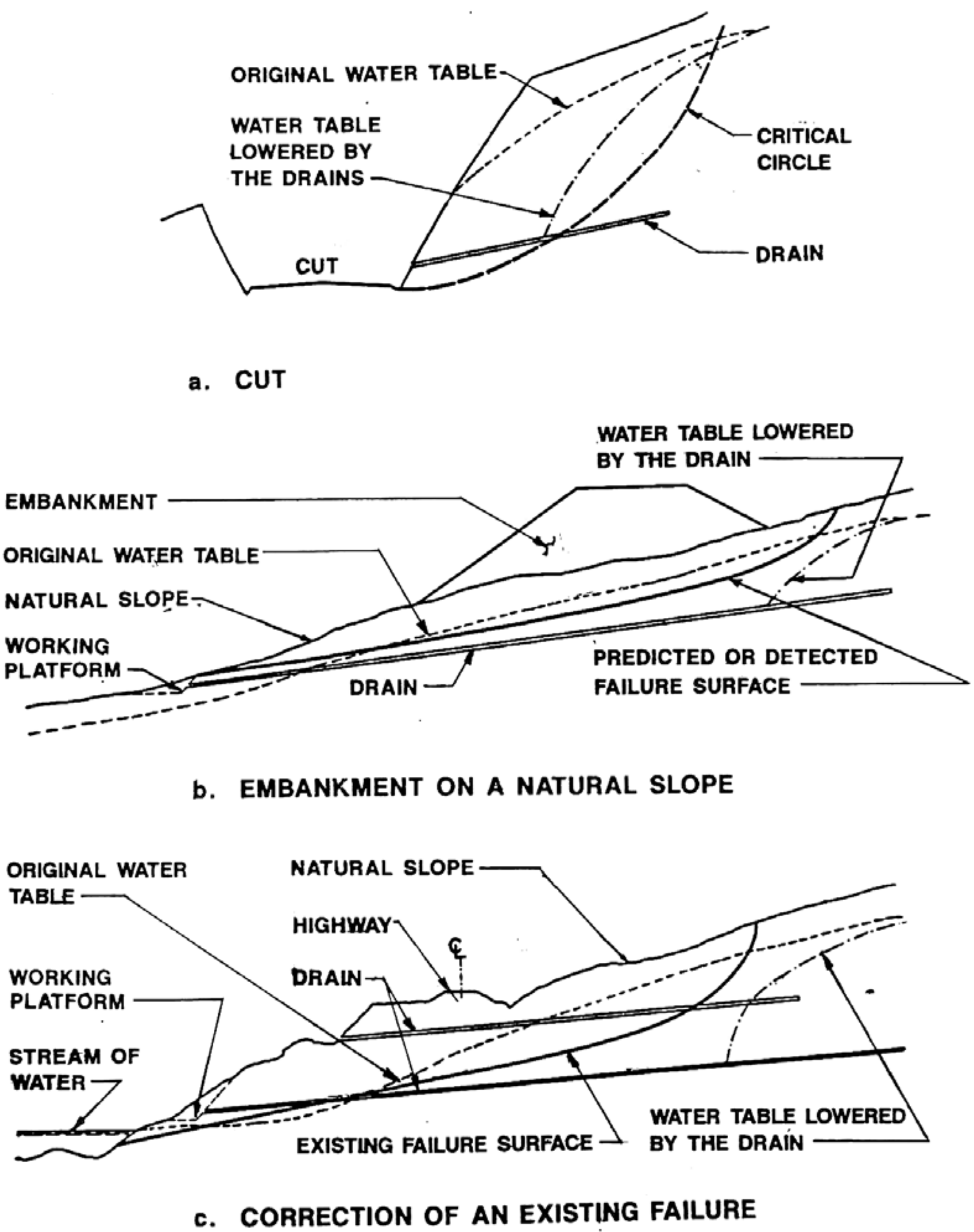


Figure 9-5 Horizontal Drains to Lower Groundwater Table in Natural Slope (Rodriguez, et al., 1988)

The length of horizontal drains largely depends on the geometry of the zone in which they are to be installed. They have been successfully installed up to lengths of 90 meters (98.4 yards). The constructable length is often controlled by the nature of the ground in which they are installed. In general, determine the length required by drawing a cross section of the slope with its probable critical circle superimposed on a geologic cross section depicting aquifers. The drains must be installed in such a way to intercept groundwater zones before the groundwater reaches the critical soil mass. By the same token, for an existing slope, the critical circle will assist the engineer in determining the layout and the length of the drains. Horizontal drains are commonly placed in a fan pattern as shown in Figure 9-6. As for spacing (horizontal and vertical), 1.5 to 5 meters (5 to 15 feet) is needed for low-permeability ground as the effect of any one drain may be relatively small. If a drain produces a high flow of water, at least one extra drain should be placed close by, within about 1 or 2m (3 to 6 feet) at the outlet. There are several good references on the design of horizontal drains, including groundwater modeling and stability analysis (FHWA-RD-72-30, FHWA-CA-TL-80/16 and FHWA Implementation Package 76-9).

Installing horizontal drains is difficult in fine silty sands and soils that contain boulders, rock fragments, open cracks and cavities. Silty sand tends to collapse and form cavities during drilling, as the initial holes are usually not cased for economic reasons. In the case of a formation with boulders, the difficulties caused by their hardness and heterogeneity often drive the drilling cost prohibitively high.

The success of a horizontal drain system is not necessarily measured by the quantity of water collected by the drains. A very permeable water-bearing stratum with free water may be intercepted by the drains, in which case the volume of flow drained may be impressive. Conversely, drains may be installed in low-permeability clayey formations where they can very efficiently lower pore pressures and greatly contribute to increased stability. The quantities of water they collect may be small. Also, the amount of water collected by drains may vary seasonally.

It must be noted that the efficiency of horizontal drains may decrease with time as soil fines and other debris plug the pores of the drains. Thus, they should be installed in such a position that they can be cleaned and flushed by pumping water into the drains. Special equipment with wire brushes and water jets mounted on tractors generally is available for this work. Check the performance of a system of horizontal drains by recording variations in the height of water table in the observation wells that are strategically placed throughout the drained zone.

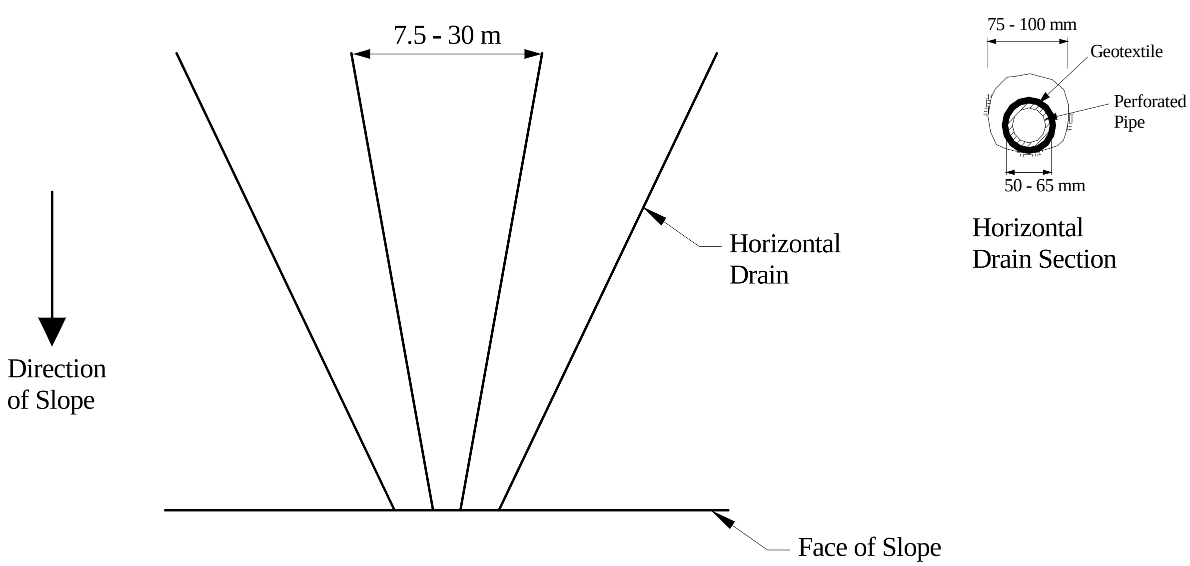


Figure 9-6 Fan-shaped Layout of Horizontal Drains in Plan View (After Forrester, 2001)

Relief Wells

Although not commonly used in highway technology, relief wells provide a useful technique for certain problems. The principal function of relief wells is to lower the water pressures in layers deep down in the subsoil, which cannot be reached by open excavation methods or horizontal drains because of cost or construction difficulties.

Relief wells are vertical holes with a diameter of about 400 to 600 mm (16 to 24 in), inside of which a perforated pipe with a 100 to 200 mm (4 to 8in) diameter is placed. The annular space is filled with filter material. A water disposal system using a submersible pump or surface pumping and discharge channels is required to dispose of the water from the wells. This disposal system may be very costly and requires constant maintenance. Alternatively, horizontal drains can be used to tap the relief wells and eliminate the water (Figure 9-7). However, this method requires a carefully designed drilling operation to physically connect the horizontal drains with the wells.

The spacing between relief wells is very important because it affects the performance and cost of the system. Spacings of 5 to 12 meters (16.4 to 40 feet) are common and many systems consist of two closely overlapped rows. The depth of relief wells depends on the unstable zone in, which stability needs to be improved. Relief wells with a depth of 50 meters (164 feet) have been built.

Summary

The Authors of this manual have been involved with the forensic engineering of dozens of slope and embankment failures. All of these failures have had one thing in common – water was the trigger to the failure. It is, therefore, important to understand the affects of water on the performance of slopes and embankments and to include drainage systems for both surface and subsurface water in the design.

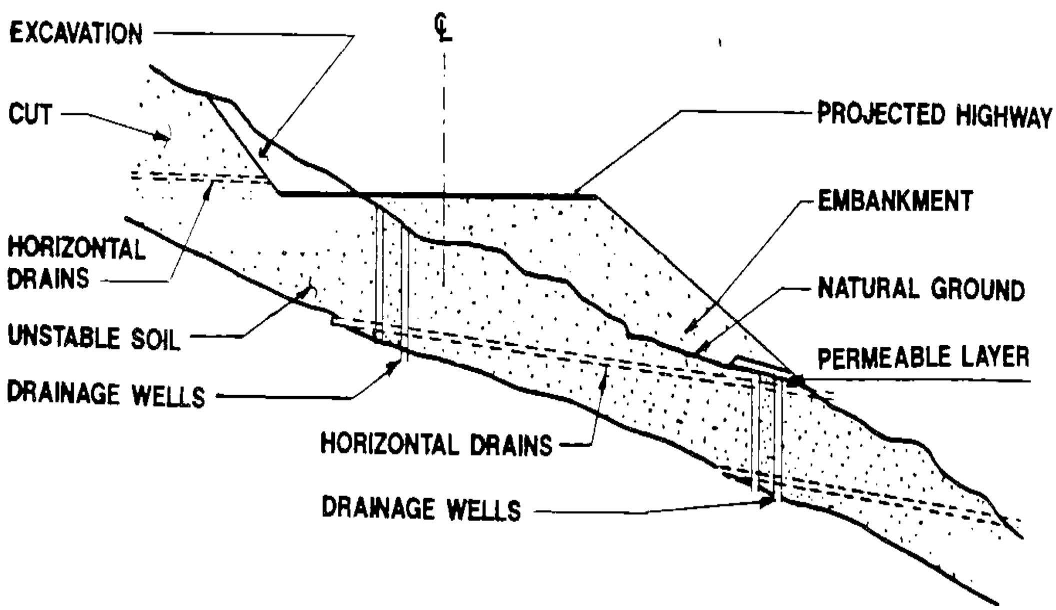


Figure 9-7: Drainage Wells Combined with Horizontal Drain (Rodriguez, et al., 1988).

9.4 EROSION CONTROL/SLOPE PROTECTION

The slope face should be protected against excessive erosion from runoff, wave action and impinging currents if the embankment/slope is constructed adjacent to a body of water. Vegetation, erosion control mats and riprap are common types of protection measures used for erosion control. Other protection methods include shotcrete, masonry, biotechnical stabilization, and gabions.

9.4.1 Vegetation

Vegetation (grass, shrubs and small trees) is highly effective for slope protection purposes. Vegetation minimizes seepage of runoff into the soil by intercepting the rainfall and stabilizes the soils by the intertwining of its roots. In addition, vegetation may have an indirect influence on deep seated stability by depleting soil moisture and attenuating depth of frost penetration.

Vegetation is multifunctional, relatively inexpensive, self repairing, visually attractive, and does not require heavy or elaborate equipment for its installation. However, there are certain limitations. Vegetation is susceptible to seasonal variation of blight and drought. It is difficult to get established on steep slopes. It is unable to resist severe scour or wave action and is slow to become established in many areas of the U.S.

Since plants and grass absorb water differently from the soil in which they grow, there are several different criteria for the selection of the most appropriate species. A general rule of thumb is to use local plants and grass that are adaptable to the local climate. Exactly what types or species are needed requires the aid of horticulture and landscaping experts. In general, vegetation species that absorb much water from the soil are the best in clayey soils to ensure a drier and stronger soil crust. On the contrary, species that absorb less water would be ideal for sandy soils because intense drying of sandy surface soils makes them more susceptible to erosion.

9.4.2 Erosion Control Mats and Blankets

As mention previously, when vegetation is used as a slope protection measure, it will take a certain period (which varies according to different species and the availability of water) for it to mature. During this period, both the seeds and the bare soil will require protection. Erosion mats/blankets, rovings and soil confinement systems are commonly used for this purpose. Erosion control mats and blankets include either natural/organic or synthetic mat/blankets.

Natural and Synthetic Mats and Blankets

Natural/organic mats and blankets generally are machine or hand woven products composed of wood excelsior, straw, or other filaments bound together with a photodegradable plastic mesh. They retain soil moisture, control surface temperature fluctuations of the soil, conform to the terrain, protect against sun burnout, and break up raindrops to minimize erosion. However, these materials are self-degradable with time so that they disintegrate after the plants grow and become well established.

Synthetic mats and blankets typically are composed of a web of continuous or stapled monofilaments bound by heat fusion or stitched between nets to provide dimensional stability. They are durable materials. However, they are susceptible to ultra-violet degradation and will become brittle with time unless protected from the sun. For detailed information on the design and installation of erosion control blankets and mats see FHWA HI-95-038 "*Geosynthetic Design and Construction Guidelines.*"

Roving

Roving is a pneumatically applied synthetic fiber covering held in place by emulsified asphalt. There are two types of roving, one is fiberglass roving – a material formed from fibers drawn from molten glass; and the other consists of polypropylene strands. Erosion-control roving is unique in the industry because of its application flexibility, which allows for any width of thickness of material to be applied. Like the synthetic mats and blankets, fiberglass and UV- treated polypropylene are resistant to UV and biodegradation and are a long-lasting (on the order of 10 years) erosion control materials.

Before installation, the area should be prepared either mechanically or manually for the application of seed, fertilizer and lime. Roving is applied using a special nozzle connected to an air compressor. As the roving feeds into the nozzle, compressed air, regulated at the nozzle to an approximately pressure of 430 kPa (62.4 lbs/in^2) propels the roving at a rate of about 0.12 to 0.31 kg per square meter (0.025 to 0.063 lb/sf) out of the nozzle, separating the strands into individual fibers. After completing the application, a tack coat of emulsified asphalt is applied at a rate of about 2 to 3 liters per square meter to assure adhesion of the fibers to one another and to the soil. Using a tack coat of black emulsified asphalt makes the roving look somewhat unappealing at first until the vegetation takes over.

Soil-Confinement Systems

Soil confinement systems generally consist of a series of honeycomb-like cells formed into a spreadable sheet or blanket. Sheets of the material are anchored and filled with soil, creating a solid pavement-like surface in areas of poor surface soil stability. The products are generally made from a high-density polyethylene material. Installation of these systems is simple and is illustrated in Figure 9-8. It must be emphasized that no matter what type of erosion control material is used, it is essential to consult with the manufacturer to ensure material selection and installation.

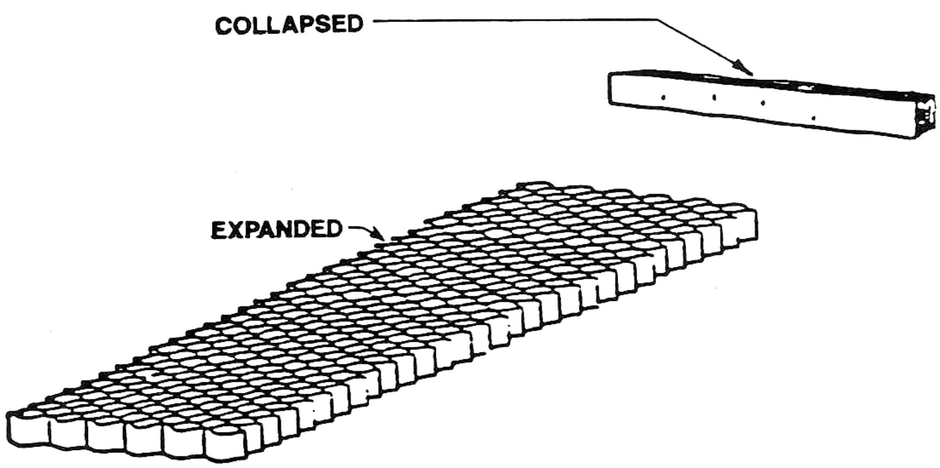


Figure 9-8: Typical Installation Instructions for Soil Confinement Systems, Following Manufacturer-Recommended Procedures (Agnew, 1991).

9.4.3 Riprap

Moving water in rivers, streams, or bays commonly cause erosion of slopes and can lead to instability if left unattended. Slopes with heavy runoff may cause similar problems. To protect slopes from water actions at the base, the general solution is to place riprap from the toe of slope to an elevation 1.0 to 1.5 meters (3.3 to 5 feet) above the mean high-water level (Figure 9-9). Similarly, riprap should be placed in areas where heavy runoff will occur. Where eroding forces are substantial, a lining of reinforced concrete with hydraulic detailing to dissipate the eroding forces from the water flow may be used.

Riprap can be dumped or hand-placed. Dumped riprap consists of large-sized rock dumped on a properly graded filter. However, the current trend is to use a geotextile as the filter/separator material. Geotextiles have a long term proven performance. They are less costly and more effective in preventing loss of soil fines through the riprap than soil transition/filter layers. Hand-placed riprap is laid with a minimum of voids and a relatively smooth top surface. A geotextile filter blanket must be provided and designed for filtration and permeability. Riprap should be hard and durable against weathering and heavy enough to resist displacement by running water or wave action. A well graded riprap is preferred to one with a narrow range of sizes. For more information on riprap design see FHWA IF-00-022 *Design of Riprap Revetments*

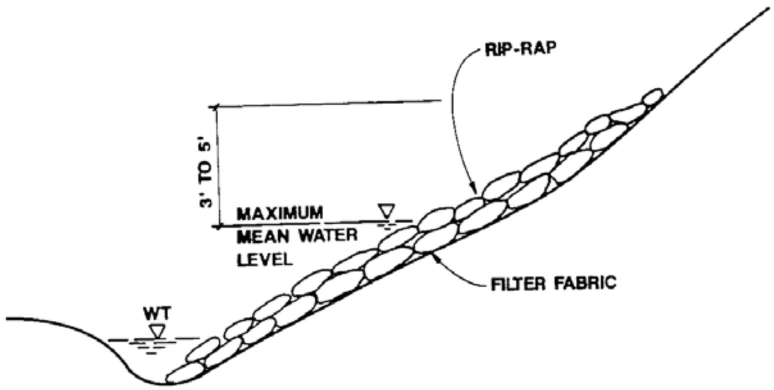


Figure 9-9 Riprap to Protect Erosion at Toe of a Slope (FHWA, 2000)

9.4.4 Shotcrete

The main purpose of shotcrete is to protect the slope from rainfall infiltration. The specifications for materials used are similar to those adopted for conventional concreting, although the aggregates are specially selected not only to meet the requirements of the finished surface but also to prevent segregation while the shotcrete is being pumped and applied. In general, the maximum grain size of the aggregate for slope stabilization should not exceed 10 mm (0.4 in). Mesh or fiber reinforcement may be provided where necessary. Admixtures and water cement rations need to be fine-tuned for successful shotcrete application. The mix should be tested in the laboratory for compatibility and strength and also tested in the field using panels and field equipment and personnel.

Careful consideration must be given to drying and consequent shrinkage cracking that occurs when shotcrete is used for slope surfacing. The performance of the field test panel, with respect to durability, permeability, and shrinkage should be assessed and, if necessary, the mix should be modified to meet all requirements. The shotcrete should be cured for not less than 7 days. To facilitate drainage, weepholes should be installed in the shotcrete surface, especially where areas of seepage are noted prior to applying the shotcrete. In areas that are susceptible to regular freeze thaw cycles shotcrete is not recommended. For more information on shotcrete see FHWA IF-03-017 *Geotechnical Engineering Circular No. 7 – Soil Nail Walls.*

9.4.5 Biotechnical Stabilization

Biotechnical stabilization was developed to stabilize surface erosion and to prevent shallow slope failures (sloughs). This technique basically combines the concept of in situ reinforcement by wood stems with surface protection by vegetation. It was pioneered by Gray and Leiser (1982) and entails the use of living vegetation, primarily cut woody-plant materials that is arranged and imbedded in the ground to prevent surficial erosion and to resist shallow mass movement.

The practical application of this technique is confined mostly to fill slopes, embankments, or buttress soil against a cut. Live cut stems and branches are used as the main soil reinforcement to provided immediate reinforcement. Secondary stabilization occurs as a result of rooting along the length of the buried stems.

In addition to its reinforcing role, the woody vegetation serves as a hydraulic drain buttressing element, and barrier to earth movement. The biotechnical stabilization technique employs a method of brushlayering during construction of the fill. A brush layer fill consists of alternating layers of earth and live, cut branches. The branches are placed in a crisscross pattern between successive lifts of compacted soil, as shown in Figure 9-10. This technique has been used successfully to repair highway slopes in the U.S. One example is along State Route 126 in North Carolina. Another example is along State Route 112 in North Massachusetts.

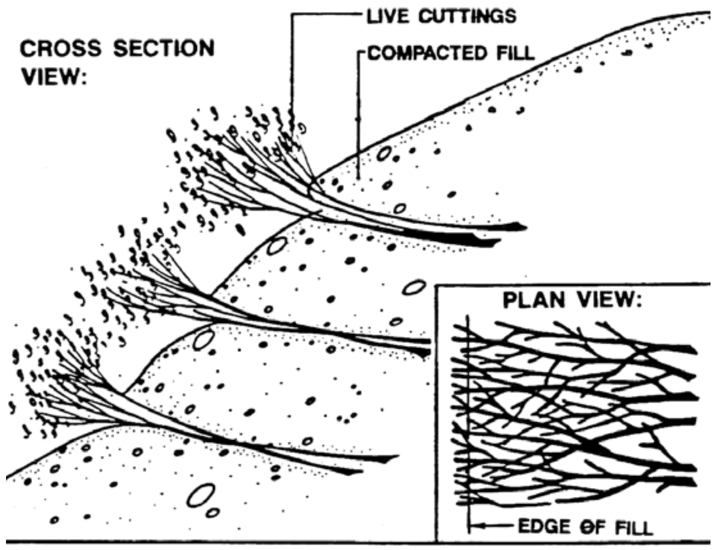


Figure 9-10 Schematic Illustration of a Brushlayer Fill Installation (Gray and Sotir, 1992)

9.4.6 Gabions

Gabions are an effective measure to protect the bottom of slopes against water actions. In addition to their resistance to water action, gabions may serve as a buttress for a slope and therefore increase the stability. A discussion of the design of a Gabion wall is presented in NHI 132036 – Earth Retaining Structures. Gabion manufacturers and suppliers can provide design guidance for slope protection and revetment applications.

9.5 BUTTRESSES/BERMS

The principle behind the use of buttresses and toe berms is to provide sufficient dead weight and increased shear resistance near the toe of the unstable slope or embankment to increase the stability to an acceptable level. The buttress must be heavy enough to provide the additional component of resistance near the toe of the slope required for stability. Figure 9-11 shows a rock buttress used to stabilize an unstable slope.

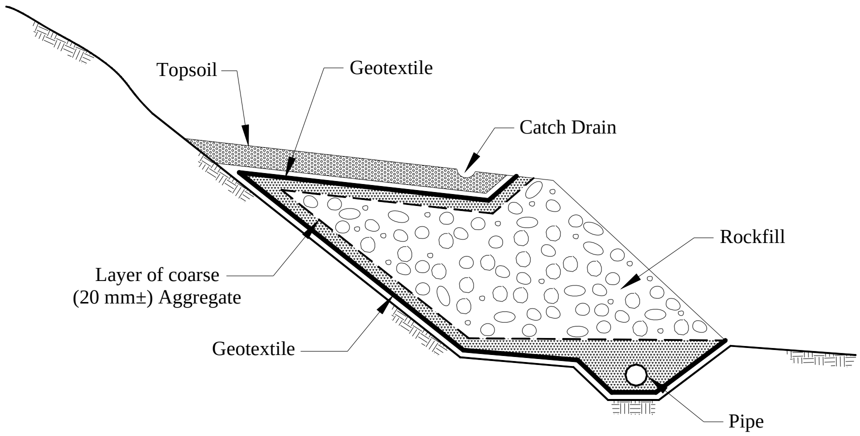


Figure 9-11 Rock Buttress (after Forrester, 2001)

The basic design of buttresses is similar to the design for external stability of conventional gravity retaining structures (NHI 132036 – Earth Retaining Structures). The buttress must be stable against:

- Overturning
- Sliding
- Bearing Failure

A settlement analysis should also be performed if the foundation is compressible to ensure that the final grade of the buttress is consistent with the geometric design requirements of the project. Internal failure modes of the buttress should also be checked to ensure that the buttress does not fail by internal shear.

9.6 EARTH RETENTION SYSTEMS

In situations where a buttress is not feasible because of geometry, cost or available right-of-way, earth retention systems (i.e., conventional retaining structures, driven H pile walls, drilled shaft walls, tieback walls, reinforced earth walls, and soil nailing) may provide a workable alternative solution. NHI 132036 – Earth Retaining Structures discusses design and construction of various earth retaining structures in details. The following paragraphs discuss briefly a few earth retaining structures commonly used for slope stabilization.

9.6.1 Conventional Concrete Gravity and Cantilever Retaining Walls

Conventional concrete gravity and cantilever retaining walls have been used for decades to increase the stability of soil slopes and embankments. For highway projects they have been used for both roadway cut and fill applications. Conventional gravity walls are rigid walls that rely entirely on their self weight to resist external forces. Considering the volume of material required for this type of wall and the related cost conventional gravity walls are typically limited to a maximum height of 3 m (10 ft). Cast in-place concrete cantilever walls are typically used in fill slopes and embankments and not in cut slope applications. In addition to their own weight this type of wall uses bending action to resist lateral earth pressures. At locations where foundation bearing capacity or settlement is inadequate a deep foundation will be required. The design of conventional and cantilever walls is covered in detail in NHI 132036 – Earth Retaining Structures.

9.6.2 H Piles Walls

H Pile (soldier beam) and lagging walls may be used to increase the stability of existing slopes or embankments and consist of steel beams, typically set at 1.8 to 3 m (6 to 9.8 ft) spacing, and lagging which spans the distance between beams. The lagging is used to retain the soil face and transmit the lateral earth pressure to the soldier beams. For permanent walls precast concrete lagging is typically used. However, timber lagging may be used as a temporary facing that is subsequently covered with cast-in-place concrete. Driven H pile walls used to stabilize slopes and embankments provide increased stability through the bending moments developed in the cantilever piles. The design of this type of wall for stabilizing slopes must consider the effect of the wall on the failure surfaces within the slope.

9.6.3 Drilled Shafts and Drilled Shaft Walls

Drilled shafts can be used to stabilize soil slopes as shown in Figure 9-12a (Reese et al. 1987). In order to do this the shafts must be able to withstand the bending stress exerted on the drilled shafts from the unstable mass, and extended a sufficient depth below the critical failure surface in order to develop enough passive resistance to withstand the driving force. Drilled shafts can also be used as a earth retention wall in various different forms. The drilled shafts may just touch each other (tangent pile wall), they may overlap each other (secant pile wall), or they may be spaced with a gap between shafts (intermittent pile wall). Figure 9-12b shows a typical application. Drilled shafts increase the stability of slopes and embankments by increasing the shear resistance across the potential failure surface. If the drilled shaft wall is an intermittent wall, the spacing of the shafts must be designed so that soil from the unstable mass does not squeeze through the gap between shafts. A detailed design procedure is provided in NHI 132036 – Earth Retaining Structures. Readers are also referred to FHWA IF-99-025 "Drilled Shafts: Construction Procedures and Design Method".

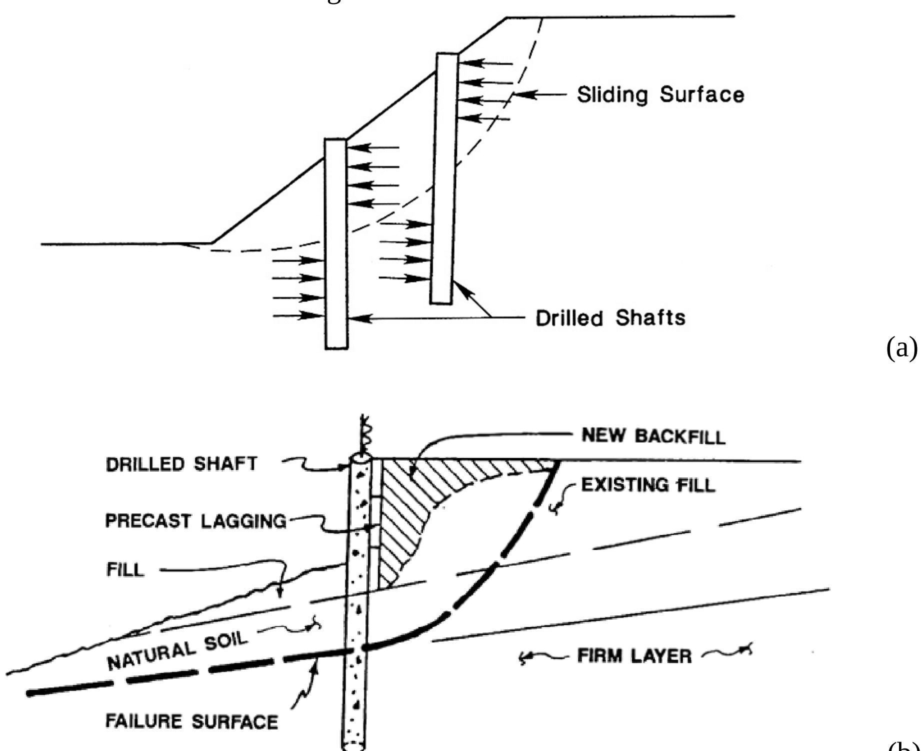


Figure 9-12 (a) Drilled Shafts for Stabilizing a Slide (Reese et al., 1987); (b) Drilled Shaft Wall (Cylinder Pile Wall) to Stabilize Deep-Seated Failure (Nethero, 1982).

9.6.4 Ground Anchor Walls

When the additional resisting force necessary to increase the stability of a slope to an acceptable level is larger than what may be provided economically with conventional gravity, cantilever, H pile and lagging or drilled shaft walls, ground anchor (i.e., tieback) walls may be used. A ground anchor wall is a structural system which uses ground anchors to apply a stabilizing force to the wall (Figure 9-13). The anchor is composed of pre-stressing steel with sheathing, and an anchorage. The anchor transmits the tensile force in the pre-stressing steel to the ground. Cement grout is used to anchor the steel. Most slope stability programs have been developed to model the effects of ground anchors on the stability of the structure.

The bond length is the resisting length of the tieback where the tieback force is transmitted to the ground behind the critical failure surface of the slope or embankment. The tendon bond length is the length of the tendon which is bonded to the anchor grout. Normally the tendon bond length is equal to the anchor length. The unbonded length of the tendon is the length which is free to elongate elastically. When used to stabilize slopes the unbonded length is typically the length of the anchor from the wall face to the critical failure surface. For a detailed discussion on the design on construction of permanent ground anchors the reader is referred to FHWA-IF-99-015 *Ground Anchors and Anchored Systems.*

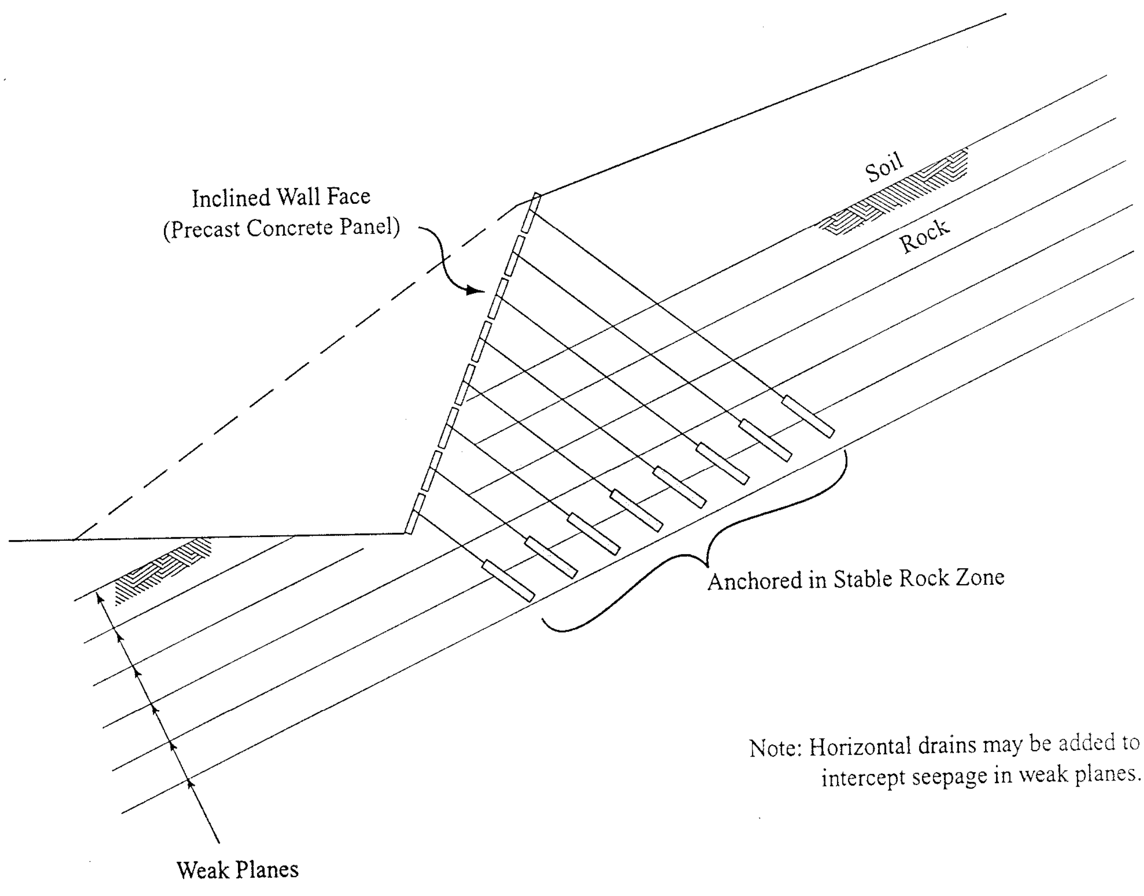


Figure 9-13 Inclined Ground Anchored Element Wall for Support of Excavation (Munfakh, 1999)

9.6.5 Soil Nail Walls

Soil nail walls are a type of in situ reinforced walls and are used in top-down construction applications. They use horizontal to subhorizontal reinforcements to improve the shearing resistance of the soil. The reinforcements, known as nails, are closely spaced and are not prestressed. The shear stresses in the ground are transferred as tensile forces in the nails through friction mobilized at the ground-nail interface. The nails develop tension as the ground deforms in response to continued excavation. The nails may also develop some bending and shear forces. However, these are generally ignored in the design. Nails may be installed by driving, drilling and grouting, and jet grouting. In the U.S. the drill and grout technique is the most common practice. Figure 9-14 illustrates the typical installation sequencing for a soil nail wall.

Soil nail walls act as gravity walls to stabilize existing slopes or embankments. The design of soil nails walls is covered in FHWA SA-96-069R - *Manual for Design & Construction Monitoring of Soil Nail Walls.*

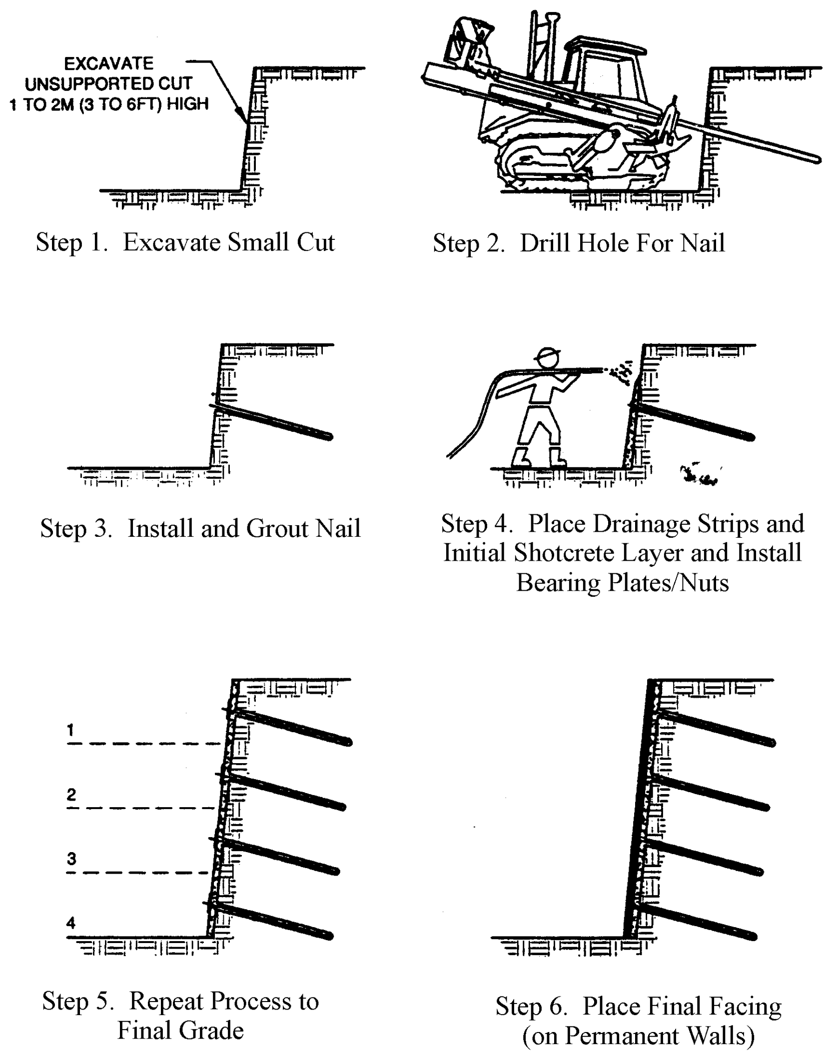


Figure 9-14 Construction Steps in Soil Nailing (Walkinshaw and Chassie, 1994)

9.6.6 Mechanically Stabilized Earth (MSE) Walls and Reinforced Soil Slopes

Mechanically stabilized earth walls have been used increasingly world wide since the 1960's. This type of wall consists of a select backfill material, reinforcing elements placed horizontally within the backfill, and wall facing elements (Figure 9-15). The reinforcing elements may be steel strips or bars, welded wire mats, or geosynthetics (geogrids and geotextiles). MSE walls are typically constructed in fill situations by placing alternating layers of soil and reinforcing elements. The resulting reinforced soil structure is flexible and can generally accommodate relatively large horizontal and vertical movements without excessive structural distress. This makes MSE walls ideal for stabilizing natural and man made slopes. This wall system, like gravity walls, increases the stability of a slope by increasing the resisting force at the toe of the slope.

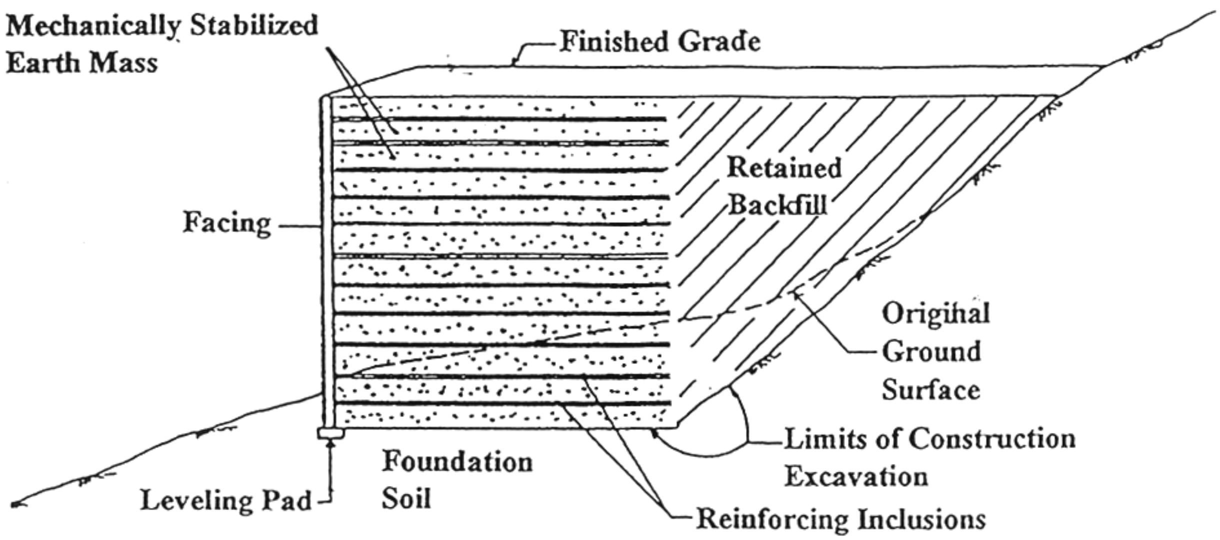


Figure 9-15 Principal Components of a Mechanically Stabilized Earth Wall. (Christopher, et al., 1990)

The design of MSE walls has been covered in great detail in several FHWA publications. The reader is referred to NHI 132036 – *Earth Retaining Structures,* FHWA-NHI-00-043 *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines* for specific design information on MSE walls.

Reinforced soil slopes (RSS) and embankments are fill slopes constructed at steeper face angles than the effective friction angle of the fill by the inclusion of soil reinforcement. The reinforcement for reinforced soil slopes is typically a geosynthetic (geogrid or geotextile), however, steel reinforcements have been used when select (free draining backfill) is used. Like MSE walls, RSS are constructed by placing alternating layers of fill and reinforcement. In new slope or embankment construction RSS is typically one of the most economical ways to increase the stability of the structure to acceptable levels. Figure 9-16 illustrates the typical application of RSS for soil slope. Figure 9-17 shows the design cross section of the reinforced embankment for Cannon Creek alternate embankment project.

The design of RSS takes into consideration the additional resisting moments developed about the center of the critical failure surfaces from the reinforcement layers that cross the failure surface. For a detailed discussion on the design of RSS the reader is referred to FHWA- NHI-00-043 *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines.*

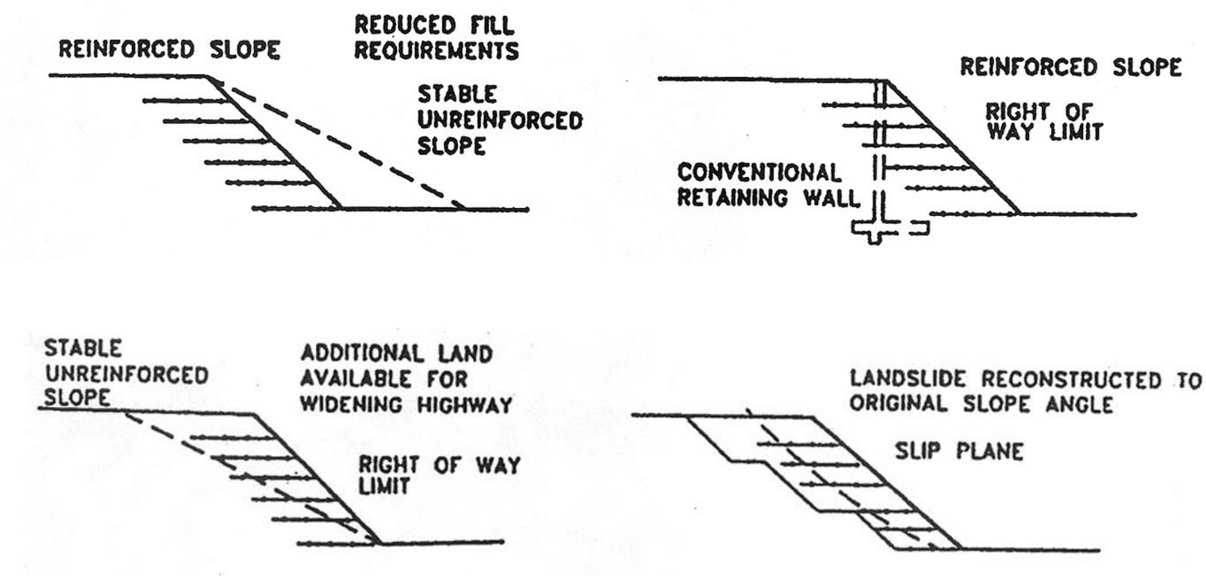


Figure 9-16 Application of Reinforced Soil Slopes (Christopher et al., 1990)

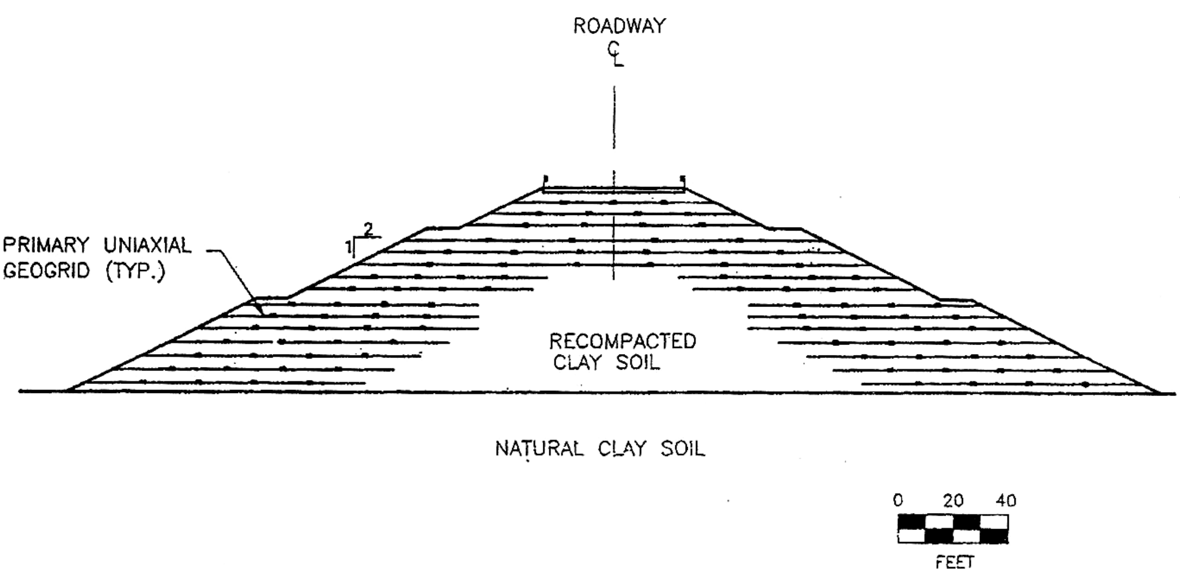


Figure 9-17 Design Cross-Section for Reinforced Highway Embankment for Cannon Creek Project (Christopher et al., 1990)

9.7 OTHER SOIL REINFORCEMENT APPLICATIONS

9.7.1 Stone Column

Stone column construction involves the partial replacement or displacement of unsuitable subsurface soils with a compacted vertical stone column. Stone column construction is accomplished by down-hole vibratory methods. The technique of creating a stone column involves introduction of backfill material into the soil so that dense and sometimes deep columns are formed that are tightly interlocked with the surrounding soil. Stone columns have been traditionally used to stabilize embankments, for bridge approach fill stabilization, bridge abutment support, and liquefaction mitigation (Figure 9-18).

A detailed description of the design of stone columns is provided in FHWA-SA-98-086 *Ground Improvement Technical Summaries*, FHWA-RD-83-02C *Design and Construction of Stone Columns*, and NHI 132034 *Ground Improvement*.

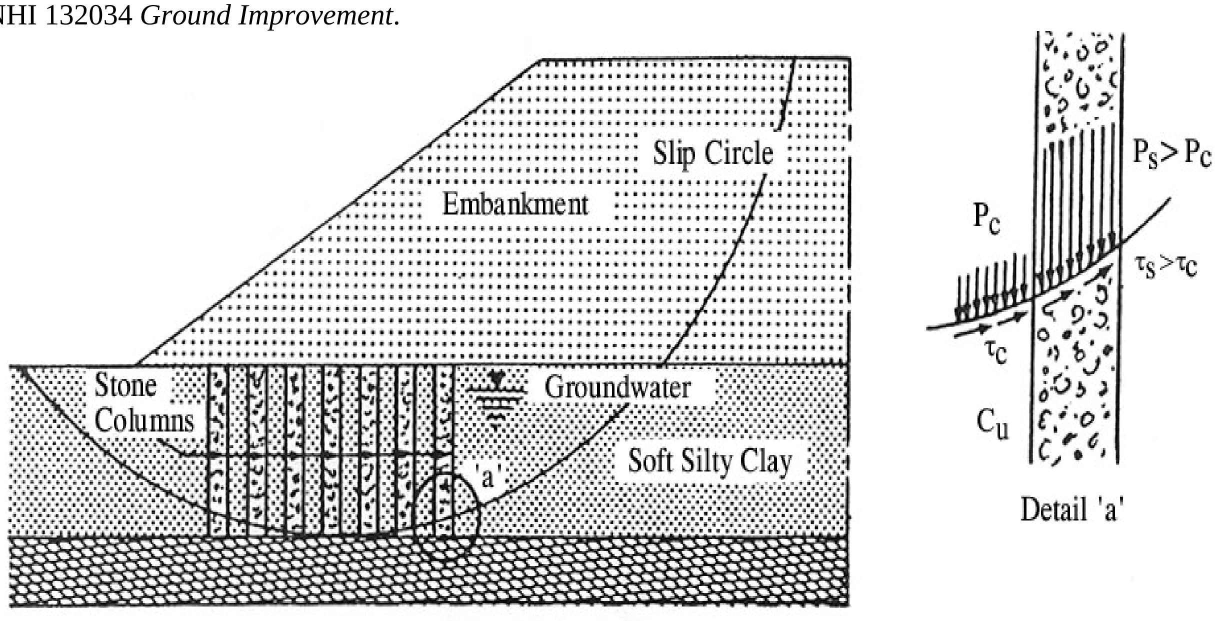


Figure 9-18 Stone Column for Embankment Support (Munfakh et al., 1987)

9.7.2 Micropile

Micropiles are small diameter (less than 300 mm (11.8 in)) drilled piles constructed with some form of steel reinforcement, and bonded to the ground with grout that may or may not be placed under pressure. Micropiles may be installed in clusters, usually at various angles to stabilize slopes. Figure 9-19 shows the use of micropiles to stabilize slopes. There are two micropile systems that are currently marketed for slope stabilization. They are the “reticulated micropile wall (RMP)” and the “type A insert wall”. These systems are designed to create in situ a coherent, composite, reinforced soil gravity structure. The design concept used for the “reticulated micropile wall” is similar to that used for gravity wall structures. “Type A insert walls” are not reticulated and depend less on the network “knot” effect between the soil and the piles. “Type A insert walls” do not behave as gravity walls. The design procedure for “type A insert walls” is, therefore, different than for RMP walls. For preliminary designs using these system the reader is referred to NHI 132036 – *Earth Retaining Structures* and FHWA-SA-97-070 *Micropile Design and Construction Guidelines.*

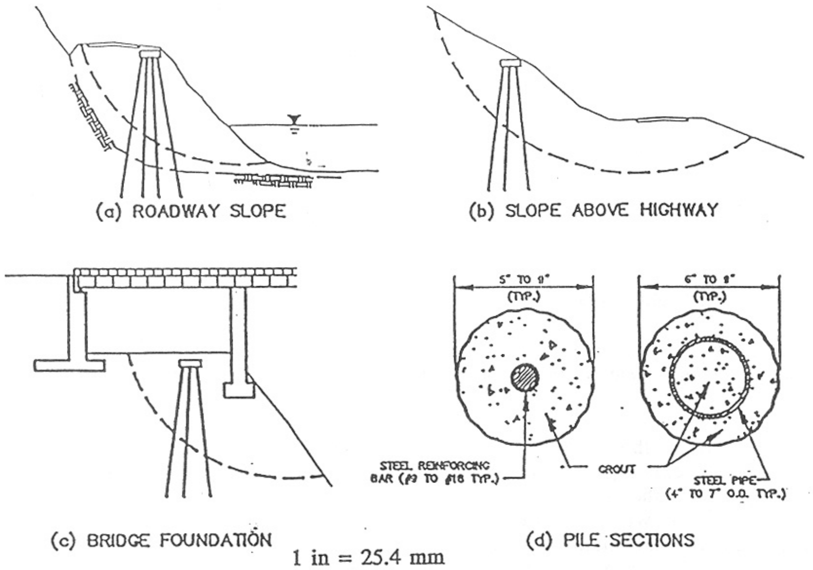


Figure 9-19 Example of Micro-Pile Walls; Reticulated Wall for Slope Stabilization (Pearlman et al., 1992).

9.8 PRECONSOLIDATION (PRELOADING & WICK DRAIN)

Another relatively economical method to improve weak cohesive foundation soils in advance of construction of a new slope or embankment is to preload the soil. The foundation soils may be preloaded by placing fill (soil or rock) across the site or by lowering the groundwater table. The objectives of preloading the foundation soil are:

- Reduce the settlement that would occur from construction of the new structure by temporarily surcharging the site.

- Improve the shear strength of the foundation soil and thus the stability of the slope or embankment by increasing the density, reducing the void ratio, and decreasing the water content of the foundation soil.

The amount and duration of the preload should be determined to reduce the post preload settlement of the structure to an acceptable amount. The amount of consolidation settlement of a soil is a function of the applied load, and the void ratio of the clay. The time that it takes to consolidate clay is a function of the permeability of the clay, and the length of the drainage path. Normally a preload is selected that is greater than the estimated weight of the proposed structure so that the majority of settlement occurs prior to building the structure. Figure 8-5 shows a typical settlement curve for an uniformly loaded normally consolidated clay. The figure also shows the settlement curve for the same clay preloaded to a higher vertical stress than expected from the structure. The preload is maintained until the settlement at the site reaches the settlement that would be caused by the weight of the structure. Because the preload was larger than the design load of the structure, the time required to preload the site is less than the time that it would take the soil to consolidate this amount under the structure load. The increase in shear strength may be determined in the laboratory by performing consolidated undrained triaxial tests with pore pressures measured. The stability of the slope may than be determined using the improved strength of the foundation soil.

Often, the effectiveness of the preload is limited by slope stability, or the effectiveness of the preload is limited based on the overall embankment height (i.e., a 5m (16.4 ft) high preload will not be very effective for a 25 m (82 ft) embankment) or the required time to achieve the consolidation is too long. Wick drains, prefabricated vertical drains, (see NHI 132034 *Ground Improvement* and FHWA-SA-98-086 for a detailed discussion of wick drains) may be used in conjunction with preloading to accelerate the consolidation process. Wick drains are band-shaped (rectangular cross-section) products consisting of a plastic core surrounded by a geotextile jacket (Figure 9-20). Wick drains function by allowing porewater in the soil to seep into the drain for collection and transmittal up and down the length of the core. The size of a wick drain is typically 100 mm (4 in) wide by 3 to 9 mm (0.1 to 0.3 in) thick. Figure 9-21 shows a typical wick drain installation.

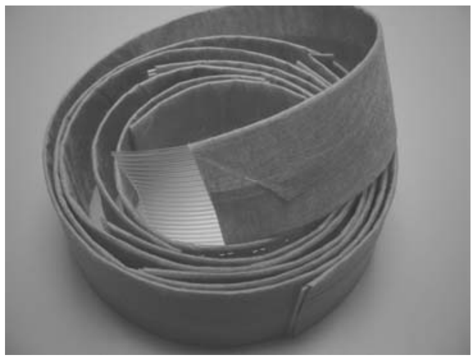


Figure 9-20 Cross-Section of Mandrel and Drain

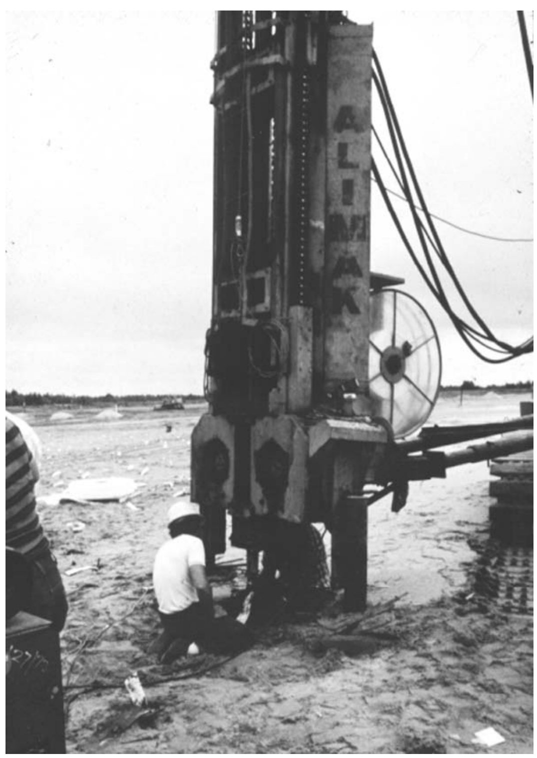


Figure 9-21 Wick drains Installation at Jourdan Road Terminal, New Orleans (Munfakh, 1999)

9.9 DENSIFICATION

9.9.1 Dynamic Compaction

The foundation of an embankment on loose cohesionless soil may be improved by in-situ densification of the soil. The two most commonly used methods of deep in-situ densification are dynamic compaction and vibro-compaction. Dynamic compaction is a method of ground improvement that results from the application of high levels of energy at the ground surface. The energy is applied by repeatedly raising and dropping a tamper with a mass ranging from 5 to 18 Mg (5.5 to 20 tons) at distances ranging from 9 to 30 m (30 to 100 feet). The tamper is typically lifted and dropped by a crane as shown in Figures 9-22 and 9-23. The energy from the impact of the tamper at the ground surface results in densification of the deposit to depths that are proportional to the energy applied. The depths of improvement generally range from about 3 to 11 m (10 to 36 feet) for light to heavy energy applications.

The primary use of dynamic compaction is to densify loose deposits so as to reduce the settlement and increase the shear strength under application of load. Densification results from the soil particles being compacted to reduce the void ratio. In partially saturated soils, densification is similar to laboratory impact compaction by the Proctor method. In saturated or nearly saturated fine sands and silts, excess pore water pressures develop on impact, which dislodge some of the point-to-point contacts between soil particles. Following dissipation of the pore water pressures, the soil particles restructure into a denser state of packing at a lower water content.

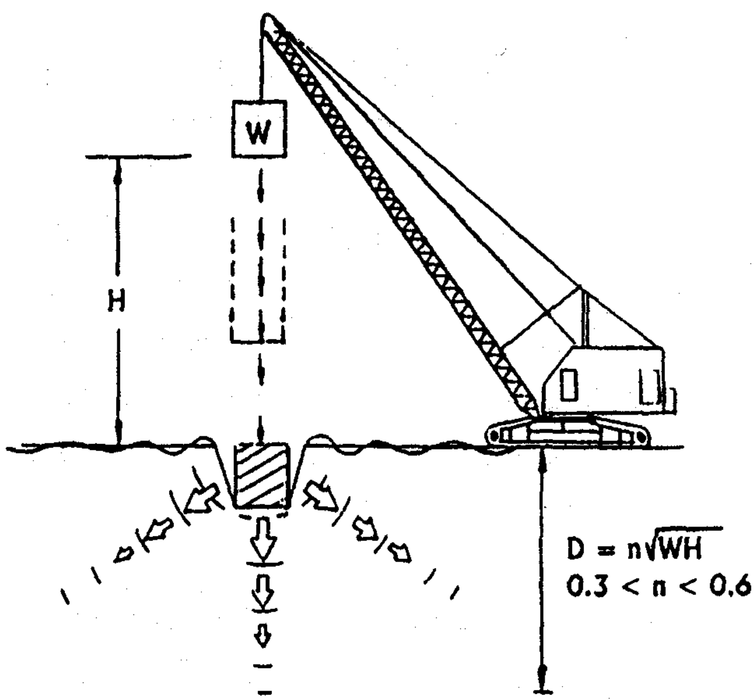


Figure 9-22 Schematic Illustration of Dynamic Compaction (NHI 132034)

Figure 9-23 Photo Dynamic Compaction at Port of Ningbo, China (Munfakh, 1999)

Dynamic compaction works best on deposits where the degree of saturation is low, the permeability of the soil mass is high and the drainage is good. Deposits considered most appropriate for dynamic compaction are pervious granular (cohesionless) soils. If these deposits are above the water table, densification is immediate as the soil particles are compacted into a denser state. If these deposits are situated below the water table, and the permeability of the soil is high, the excess pore water pressures generated by the impact of the tamper dissipate almost immediately and the improvement is correspondingly immediate.

The major disadvantage of dynamic compaction is that it produces ground vibrations that can travel significant distances from the point of impact. In urban areas, this may require the use of lightweight tampers and low drop heights and limited dynamic compaction to areas well within property lines. Evaluation of the potential for dynamic compaction to damage adjacent structures should be part of any dynamic compaction design. The design of a dynamic compaction soil improvement program is beyond the scope of this report. For a detailed design guide for dynamic compaction see NHI 132034 – *Ground Improvement* and FHWA-SA-95-037 *Dynamic Compaction*.

9.9.2 Vibro-Compaction

Vibro-compaction is a ground improvement technique which uses specifically designed probe-type, depth vibrators for in-situ densification of loose sands and gravel. The vibrator is inserted into the soil with water or air jetting and by using higher horsepower vibrators without jetting. The wet jetting process is the most typical method used in the U.S. Once the probe has been inserted into the ground to the desired depth, granular soil is dumped into the annulus around the probe and systematically densified by raising and lowering the probe. Figure 9-24 shows a typical vibro-compaction process.

The mechanism of densifying granular (cohesionless) soils with depth vibrators and water jetting can be briefly described as follows: the application of mechanical vibrations and simultaneous water injection nullifies the effective stresses between the soil particles, which are subsequently rearranged, unconstrained and unstressed under the action of gravity to a denser state, thus providing permanent compaction. In the immediate vicinity of the vibrator, the soil is saturated and liquefies locally and temporarily, under the influence of the vibrations (Moseley 1993).

The suitability of a soil for vibro-compaction methods depends mainly on its grain size distribution (Figure 9-25). Soils with their grain size distribution lying entirely on the course side of the #200 sieve are readily compacted with depth vibrators. For a detailed discussion on the design of either of these soil improvement techniques see NHI 132034 – *Ground Improvement* and FHWA-SA-98-086 *Ground Improvement Technical Summaries*.

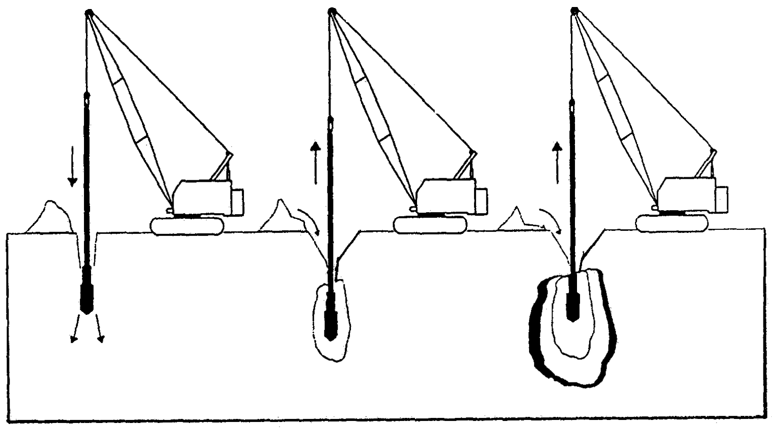


Figure 9-24 Schematic Illustration of Vibro-compaction Process (Elias, et al., 1999)

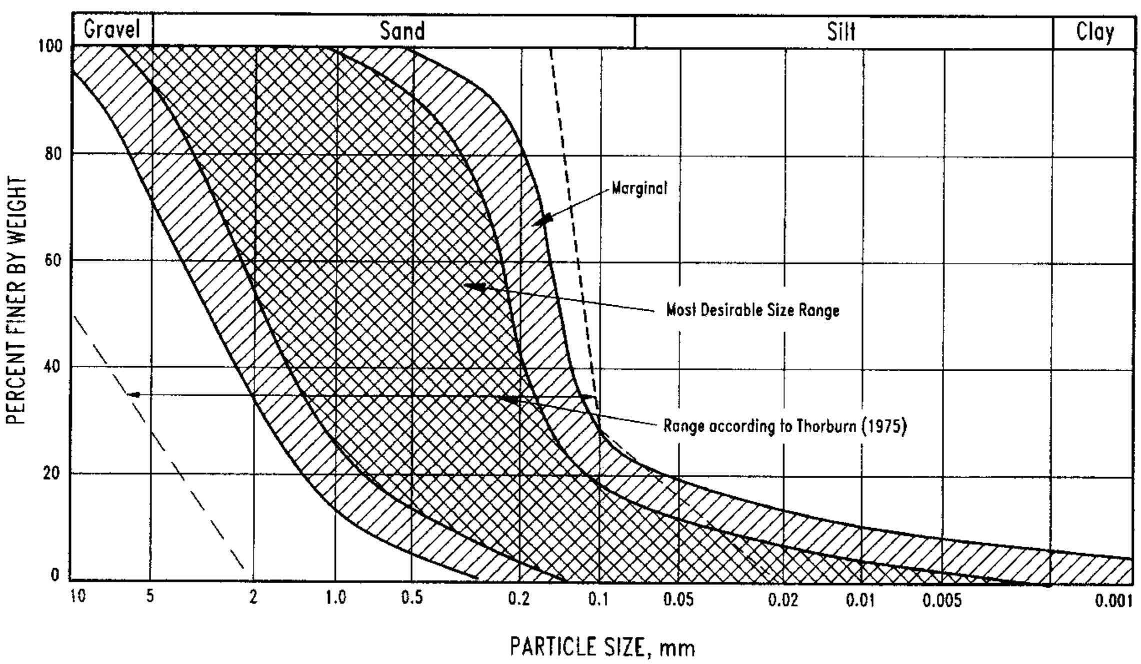


Figure 9-25 Range of Suitable Soil for Vibro-compaction (After Mitchell, 1981)

This chapter has provided only a brief overview of many of the stabilizing techniques that are available for the stabilization of soil slopes and embankments. The selection of the most appropriate stabilization method for a particular slope or embankment involves a preliminary feasibility design and cost estimate for the most viable methods. Only with a preliminary feasibility analysis can the list of options be refined. Once the list of options has been narrowed down to a few, the final selection may be based on bid prices.

CHAPTER 10
CONSTRUCTION AND PERFORMANCE MONITORING

A good geotechnical design along cannot ensure a soil slope or embankment to perform satisfactory within the intended life span. To ensure satisfactory construction and performance of a soil slope or embankment competent and thorough inspection and monitoring is required. The geotechnical designers must understand fully the construction issues and performance monitoring.

10.1 CONSTRUCTION ISSUES

Except for some minor roads in rural areas, most soil slopes, and earth fill or rock fill highway embankments are constructed in layers with certain specified in-place densities after compaction. This chapter will review the construction procedures, development of plans and specifications, construction monitoring necessary to assure that the soil slope or embankment is constructed in accordance with the contract documents, instrumentation and long-term monitoring for maintenance.

10.1.1 Foundation Preparation/Improvement

Surface Preparation

When satisfactory foundation conditions exist, relatively fewer surface preparations are required. They may consist of clearing and grubbing to remove the top soil and other organic materials such as logs and tree stumps and the removal of isolated areas of soft saturated soils by over-excavation and replacement. However, for embankments over soft soils it is recommended that tree stump and existing surface root mat remain in place as long as they are sufficiently far below the finished grade. These materials will act as reinforcement and assist the top layer of soil to support construction equipment. If the embankment height is more than 1.5 m (5 ft), the trees can be cut to 80 to 100 mm (3 to 4 inch)above existing ground and the stumps left in place. For very low embankments, it is common practice to undercut the embankment foundation to allow for a 1 m (3.3 ft) uniform subgrade for the pavement. The natural ground for these shallow embankments should be proof rolled. If soft spots are encountered, during proof rolling the soft in-situ materials should be replaced with granular materials.

Foundation Improvement

Soft foundation soils may include a variety of loose sands, swamp muck, peat, marl, and organic and inorganic silts and clays. In some locations a combination of more than one of these materials can be found. These soils will often require improvement or reinforcement to ensure embankment stability. In addition, soft cohesive soils can experience significant long-term settlements, and loose cohesionless soils, particularly loose fine sands can liquefy under a seismic event resulting in partial to complete failure of the embankment. Chapter 9 and NHI 132034 - Ground Improvement Methods present the various methods commonly used for improvement of weak ground.

Benching and Drainage

Construction of sidehill fills can be the most critical soil slope problem. In many sidehill locations, the natural bedding planes or boundaries of different soil/rock strata are oriented in a down slope direction. This sidehill condition presents two instability problems. First, the soil slope fills which are placed on these sidehill slopes will increase the slopes' tendency to slide along these bedding planes or boundaries. Second, the addition of the fill increases the potential for water to accumulate. This water accumulation increases the weight of the sliding mass while at the same time decreases the resistance of the soil/rock to sliding. Therefore, a major factor in side slope fill stability is control of groundwater.

To correct the first potential instability problem, a bench should be excavated into the sidehill not only to act as an anchor but also to establish a horizontal platform for the construction of the slope. Figure 10-1 shows the typical earth benching detail of many state DOT's. Benching is usually required when the natural hillside slopes exceed the range of 3H:1V. For example, on the Interstate 26 project in North Carolina, the potential unstable colluvium soils were completely removed and benches were cut into the underlying saprolite to provide stability for the embankment fills to be constructed (Figure 10-2). For the benches to be effective, they must extend into the firmer material within the sidehill. The second problem can be addressed by designing a variety of drainage control features which are addressed in Chapter 9.

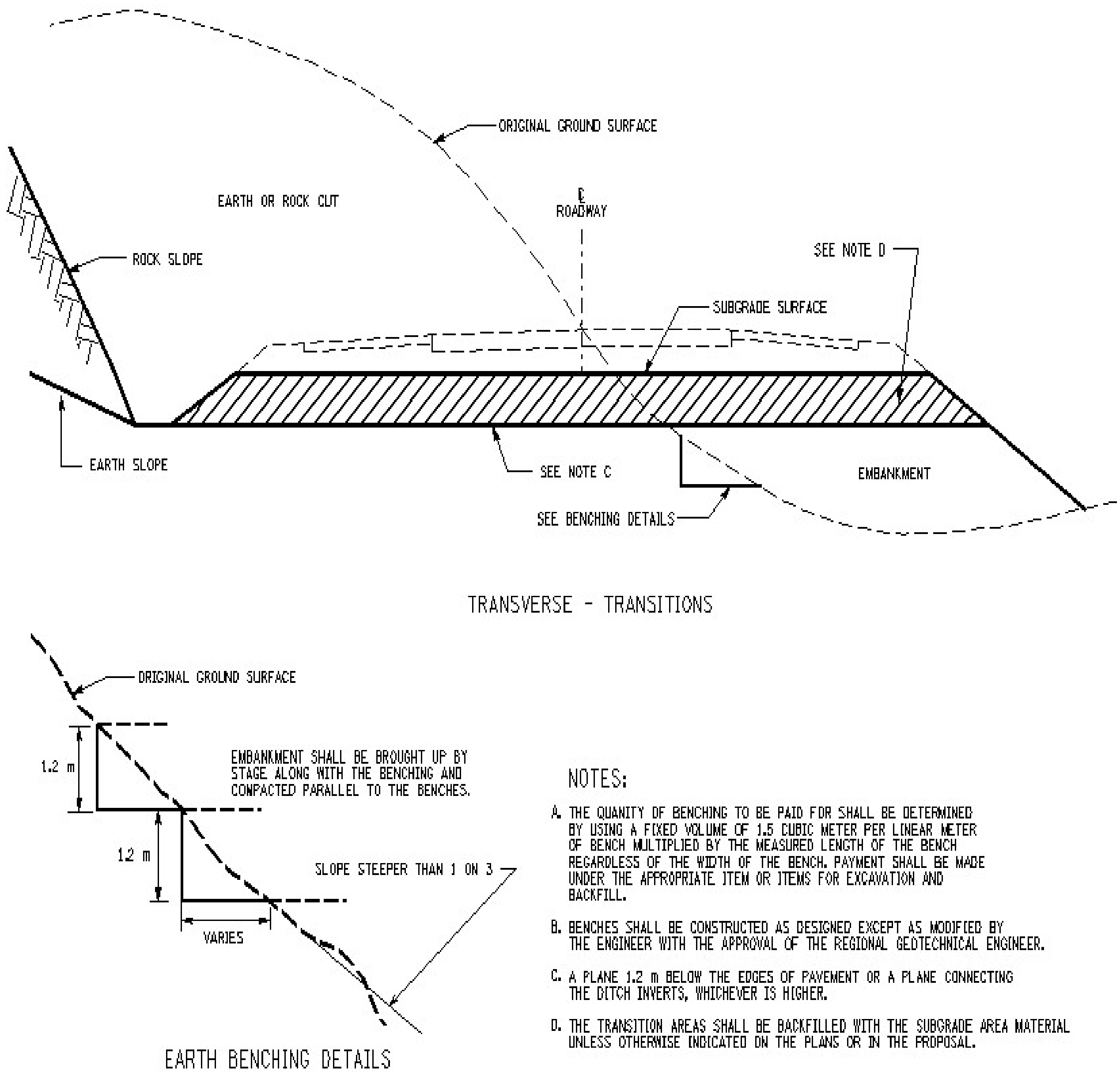


Figure 10-1: Typical Benching Details (New York State DOT Standard Sheet)

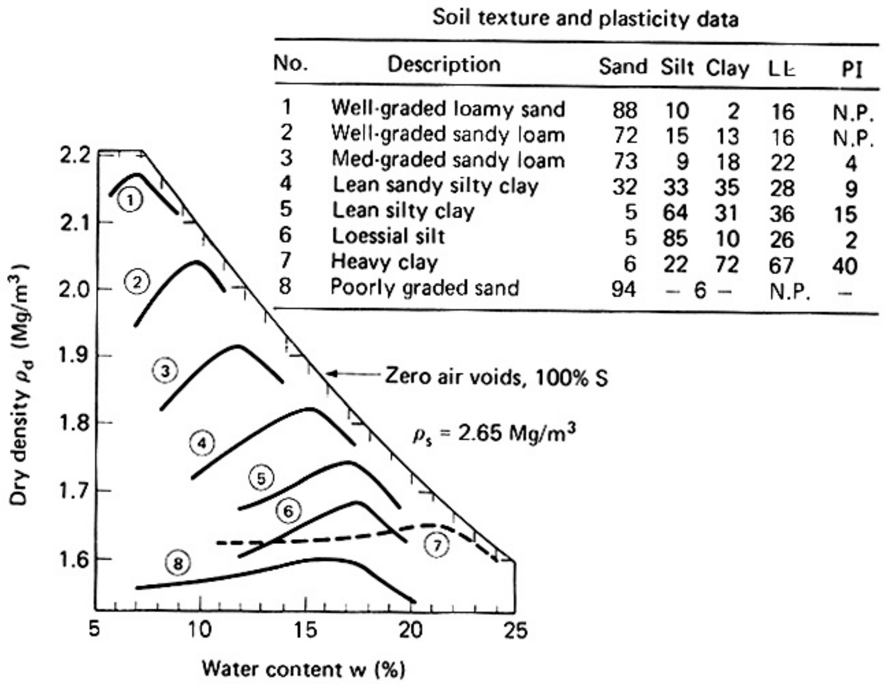
Figure 10-2: Bench Cuts for Interstate 26 Project in North Carolina

10.1.2 Compacted Fill Construction

Earthfill

Compaction of a soil increases its density and strength. The maximum density of a soil and the corresponding optimum moisture content which can be achieved by a given compaction effort can be determined from laboratory Proctor (modified or standard) moisture-density tests (AASHTO T99 – Standard Proctor and T180 – Modified Proctor). Figure 10-3 shows typical laboratory compaction curves for different soil types, according to the Standard Proctor method. Figure 10-4 shows the compaction curves using the standard and modified AASHTO laboratory tests. NHI 1320310 "Geotechnical Aspects of Pavements" provides details discussion of the fundamentals and mechanisms of soil compaction. The degree of field compaction effort will be specified as a percentage of these laboratory maximum values. In general, the unit weight specified is equal to 90 to 100 percent of the maximum density measured by the Modified Proctor depending on the importance of the embankment application.

The earthfill is specified to be placed in layers with a maximum loose lift thickness of 150 to 300 mm (6 to 12 inches) depending on the type of compacted soil and the type/size of the compaction equipment to be used. A thickness of 200 mm (8 inches) is commonly specified although thicker lifts can be allowed if the contractor can demonstrate that the specified density can be achieved through the depth of the lift. Cobbles larger than ¾ of the thickness of the specified layer should be discarded. Table 10-1 lists some of the commonly used compactors. A more detailed list of compaction equipment and their associated applications, recommended density requirements and layer thickness are presented in NAVFAC DM 7.2.



Soil texture and plasticity data

No.	Description	Sand	Silt	Clay	LL	PI
1	Well-graded loamy sand	88	10	2	16	N.P.
2	Well-graded sandy loam	72	15	13	16	N.P.
3	Med-graded sandy loam	73	9	18	22	4
4	Lean sandy silty clay	32	33	35	28	9
5	Lean silty clay	5	64	31	36	15
6	Loessial silt	5	85	10	26	2
7	Heavy clay	6	22	72	67	40
8	Poorly graded sand	94	– 6 –		N.P.	–

Figure 10-3 Water Content-dry Density Relationships for Eight Soils Compacted According to the Standard Proctor Method (Holtz, et al, 1981)

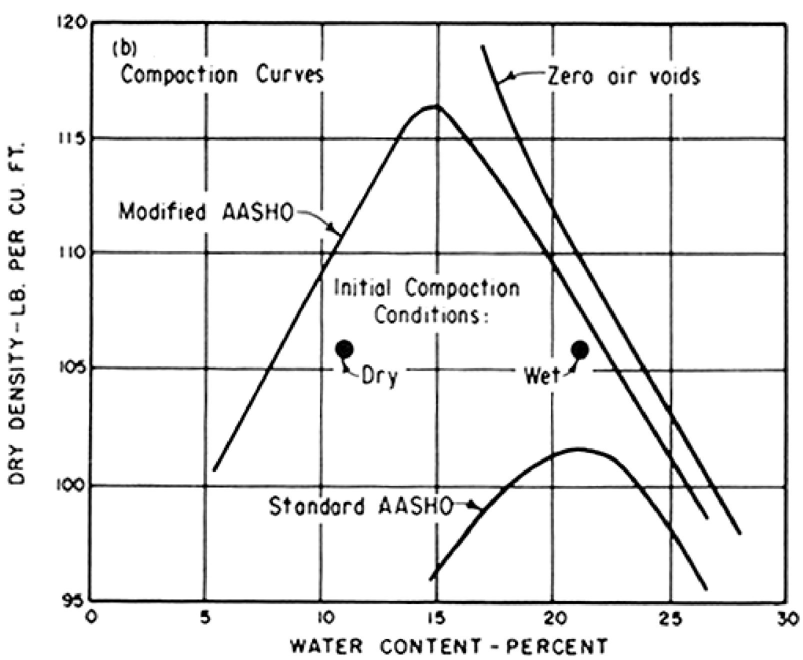


Figure 10-4: Compaction and Unconfined Compression Characteristics of Higgins Clay (Holtz, et al, 1981)

Typical scales for cross sections are 1 cm = 1 to 3 m (1 to 100 to 1 to 300). Often, specific details accompany the cross sections and are drawn to larger scales typically from 1 cm = 0.1 m to 1 m (1 to 10 to 1 to 100). Again, these scales are for guidance only. The engineer should select the appropriate scales to clearly show the details but also taking the size of the drawing into consideration. The horizontal and vertical scales should be the same and the cross section(s) plotted as shown in the plan.

When steel and concrete structures, such as retaining walls, drop inlets, and ditches are required, separate drawings should be used to depict layout, dimensions, and reinforcing.

10.2.2 Specifications

The specifications detail the materials to be used in the work, what submittals are required before the work starts and during the work, quality control requirements, what instrumentation will be required and protection of same during construction, how the work will be executed, how it will be measured for payment, and how it will be paid for. Most States have standard specifications and these should be used to the extent possible, particularly for concrete, earthwork, drainage, and other typical highway appurtenances. Many cities and counties also have their own or use FHWA’s, the states' and other standard specifications. The *Standard Specifications for Public Works Construction* (the Green Book), for example, are widely adopted in southern California. However, with some of the newer innovations in slope stabilization methods, such as soil nailing and mechanically stabilized embankments, it may be necessary to generate special provisions for a particular project. These can be based on FHWA generic specifications, FHWA design manual guidelines, and specifications used for similar projects. The U.S. Army Corps of Engineers also have developed several useful specifications for this type of work. Some States use what are called "Advisory Specifications" to elaborate further on the purposes and execution of the work.

10.3 CONSTRUCTION MONITORING

Quality Assurance/Quality Control (QA/QC) monitoring involves verifying that construction is being performed in accordance with the plans and specifications. In so doing, the field inspector should be knowledgeable about the intent of the design, as well as the contractual requirements. Sometimes field conditions will dictate that clarifications and modifications be made to the design. The inspectors should therefore inform and consult the designer on a regular basis regarding such matters.

The successful transfer of design objectives into construction is accomplished by consideration of construction operations during the design phase. In recent years the amount of coordination between design and construction has steadily decreased; primarily due to graduate engineers who specialize in design and who are never exposed to construction operations. In past years, engineers either began their careers in construction and advanced into design, or were assigned the design and construction responsibilities for projects. Present lack of coordination stemming from inexperience with field operations can result in a technically superior set of construction plans and specifications, which cannot be built. Rational construction control is vital to assure a safe, cost-effective foundation and to avoid unnecessary court of claims actions.

10.3.1 Quality Control/Quality Assurance

Quality control and quality assurance are defined as follows:

- Quality control (QC) formalizes traditional elements of design and construction, including checking, reviewing, examining, and supervising work performed. This is accomplished by

TABLE 10-1
COMPACTION EQUIPMENT AND APPLICATIONS (From NAVFAC DM 7.2)

Type of Field Compactor	Applications
Sheepsfoot Rollers	For cohesive soils or cohesionless soils with more than 20 percent passing No. 200 sieve. Not suitable for clean cohesionless soils.
Rubber Tire Roller	For clean cohesionless soils with 4 to 8 percent passing the No. 200 sieve. Also for cohesive soils or well graded, cohesionless soils with more than 8 percent passing the No. 200 sieve.
Smooth Wheel Rollers	Appropriate for subgrade or base course compaction of well-graded sand-gravel mixtures. Also may be used for cohesive soils other than in earth dams. Not suitable for clean well-graded sands or silty uniform sands.
Vibrating Sheepsfoot Rollers	For coarse-grained soils sand-gravel mixtures.
Vibrating Smooth Drum Rollers	For coarse-grained soils sand-gravel mixtures – rockfill
Vibrating Baseplate Compactors	For coarse-grained soils with less than about 12 percent passing No. 200 sieve. Best suited for materials with 4 to 8 percent passing No. 200 sieve, placed thoroughly wet.
Crawler Tractor	Best suited for coarse-grained soils with less than 4 to 8 percent passing No. 200 sieve, placed thoroughly wet.
Power Tamper or Rammer	For difficult access, trench backfill. Suitable for all inorganic soils.

The moisture content of the fill prior to compaction should be close to the optimum moisture content as determined in the laboratory Proctor moisture-density tests (AASHTO T99 – Standard Proctor and T180 – Modified Proctor). If the soil is too dry, water should be added. This is commonly done with water trucks equipped with splash bars or spray attachments. Disc hallows and various mixing equipment may also be used to spread the moisture more evenly throughout the more cohesive soils. If the soil is too wet, drying is required. Discing is frequently used for this purpose. It is recommended to blade a crown on the soil to reduce water penetration during and after a rainfall.

Test fill sections may be constructed to establish the loose lift thickness and the number of passes for soils for which there is no prior compaction experience or for soil and rockfill materials which have more than 30% by weight greater than 20 mm (0.8 inch) size particles. The test fills can also assist in selecting appropriate equipment and construction procedures for the project. If the test fill section is deemed satisfactory, it can be incorporated into the main slope or embankment.

Field density tests should be performed at regular intervals to ensure that proper compaction has been achieved. A systematic testing plan should be established prior to fill placement. The common range of testing frequency is one test for every 1,000 to 3,000 cubic meters (1,000 to 3,000 cubic yards) of material placed, depending on the importance of the embankment and the material type. The recommended testing frequency is to start with one test for every 1,000 cubic meters (1,000 cubic yards) for the first lift. As the height of the embankment increases and construction becomes routine, the test frequency can be relaxed. At least one test, however, should be performed for every full shift of compaction work. Additional tests should also be conducted in special areas where inadequate compaction is performed, different compaction procedures are used, or when compaction is performed adjacent to structures, instruments or utilities.

Field density can be measured by direct and/or indirect methods. The direct methods include sand cone (displacement), rubber balloon, drive cylinder, piston sampler and water displacement. The sand cone is the most reliable and most frequently used direct method and serves as the referee test for other tests. Methods for direct water content determination include conventional oven drying, hot plate or open flame drying, drying by forced air, and drying in a microwave oven. Indirect methods include nuclear density-moisture gauge and the Proctor penetrometer (often called the "Proctor needle"). Although they can often be used successfully to measure density and water content, they should be calibrated/correlated periodically against the results from the direct methods. The nuclear gauge by far is the most widely used method. Direct methods such as the sand cone test and conventional/microwave test are normally required for every 10 to 20 nuclear moisture-density tests. Laboratory moisture-density tests shall be performed at a regular interval to establish a base line for the field density tests.

Rockfill

Similar to the earthfill slopes and embankments, rockfill slopes and embankments are also constructed in layers. The recommended lift thickness is generally not more than 1 m. Steel vibratory rollers with static weights from 45 to 135 kN are commonly used for the compaction. The maximum rock size should not exceed 90 percent of the lift thickness. Generally, however, a test fill section is constructed to determine the most appropriate type of roller to be used, maximum rock size, lift thickness and number of passes. Sluicing can be used to improve stability and reduce potential settlement. It increases the point to point contact between the larger rocks and washes the fines into the voids. The quantity of water used is approximately two to four times the volume of rock. Rockfill with more fines will require more water. If sluicing is performed, care should be exercised to prevent mud forming at the bottom of the lift.

Because relatively few laboratory tests can be performed on a rockfill embankment, the construction of test fills in most cases are advantageous and in some cases necessary. The test fills can assist in defining the effects of variables that would otherwise remain unknown. A well designed and executed test slope or embankment can help in determining the appropriate equipment to be used, the lift thickness, the number of passes, maximum rock sizes, amount of degradation or segregation occurring during rolling, and physical properties of the in-place fill, such as density and grain size distribution. Similar to the earthfill test slopes or embankments, the rockfill test section can also be incorporated into the main slope or embankment.

Grain size distribution tests should be performed before and after compaction. This will give an indication of the breakage expected during handling and compaction. The in-place density is generally quite difficult to obtain using the large-scale conventional methods. An easier approach will be to observe the settlement which occurs and calculate the corresponding density. Care must be exercised that the settlement measured is from the layer in question and is not influenced by the foundation and the underlying layers. A few large-scale conventional density tests can be performed to verify the densities calculated using the settlement data.

10.2 CONSTRUCTION PLANS AND SPECIFICATIONS

After the completion of the slope or embankment designs, project documents are prepared to document the scope and content of the proposed work. The project documents for new construction and rehabilitation generally consist of plans, specifications, and geotechnical reports. These documents should fully depict and describe the work to be completed and the conditions present and to be expected.

During preparation of these documents, it should be recognized that contractors and construction personnel will not have the extensive knowledge of the site that the designer has. These documents should relate that knowledge as clearly as possible. Withholding or omitting pertinent information from the contractor increases the risk that the contractor is accepting and it will be evidenced by a higher bid price with greater built-in contingency costs.

Contractibility reviews are a valuable tool that can be used during the design document preparation process to assure that knowledgeable construction personnel outside the design team can readily understand the contract documents and what is shown can be built.

10.2.1 Contract Plans

The contract plans should include plan, elevation, cross section views of the proposed structure, and sufficient details to alert the contractor and construction personnel to such information as site conditions, access, adjacent utilities and facilities, topography, geologic contacts, excavation and fill requirements, slope angles, construction sequences, benches, shoring, reinforcement and stabilization details, borrow and disposal areas, geotechnical instrumentation, traffic control requirements, drainage plan and connections, architectural details, and structural details. Some of the construction or testing details (working drawings) may be furnished by the contractors and submitted to the Engineer for review/approval as required in the specifications. The plans should consist of a small-scale plan showing the site location and keyed to readily located features, such as highways, rivers, and cities, as well as a large-scale detailed plan that shows the site itself. Suitable scales are 1 cm = 10 to 30 m for the site plan and 1 cm = 2 to 5 m for the detailed plan. Note these scales are for guidelines only. The scales chosen should allow the appropriate details to be shown legibly on the respective plans without being cluttered. If the site is too large to fit on one sheet, multiple sheets with match lines should be used. Existing topography, as well as all existing features and facilities, should also be shown on the detailed site plan. Other plan sheets in the contract plans may include:

- Utility plans
- Final grading plans and sections
- Excavation plans and sections
- Drainage plans with details
- Traffic control and signing requirements
- Construction staging and sequence requirements
- Architectural (including landscape) plans and details.

It is helpful if these plans are drawn to the same scale so that different aspects of the work can easily be related from one plan to the other. A developed elevation should be prepared that depicts existing conditions, as well as proposed conditions. Stations, elevations, coordinates, and the like should be shown as accurately as possible. Different types of treatment can be shown on this elevation, for instance, a retaining wall next to a laid-back slope. Frequently the exact location of geologic contacts and therefore, proposed structures and stabilization elements may not be known at every point. However, it is the engineer's responsibility to make his/her best guess for the purposes of estimating quantities and costs. The exact locations will be determined in the field during construction and should be so noted on the drawing. Any proposed architectural finishes and landscaping should be shown on a separate elevation, in addition to the one presenting geometrics.

Cross sections should be drawn for typical and extreme conditions. Again, existing, as well as proposed, conditions should be shown. Geologic contacts, such as top of rock, also should be shown. Slopes, benches, reinforcement, drains, and other pertinent information should be presented in a clear manner.

having well-qualified professionals performing project tasks, the professionals understanding and carrying out their responsibilities, and documenting this process. Quality control for the construction of soil slopes and embankments involves the contractor taking responsibility for providing the Owner with a completed project in accordance with the plans and specifications.

- Quality assurance (QA) formalizes and documents the traditional overview role performed by supervisors and independent reviewers. Items evaluated as part of this process include technical adequacy, clarity, practicality, constructibility, conformance to professional standards and practices, and overall suitability. The owner is typically responsible for QA.

On a construction project, all quality control is the contractor's responsibility. Quality assurance is typically the responsibility of the owner.

10.3.2 Poor Construction Inspection Related Soil Slope and Embankment Problems

The following issues have been observed as typical problems when the construction is not in compliance with the project specifications:

- Placement of unsuitable materials in the slope or embankment
- Wet soft areas not treated or removed
- Poor grading related to drainage
- Vegetation not placed/replaced in time or not at all
- Toe of an excavation undermined due to lack of or inadequate shoring/bracing
- Modification of the natural conditions of seepage due to fills, ditches, or excavations
- Overloading of weak strata because of fill
- Exposure of hard fissured clays to air and water because of cuts
- Fill density below design requirements
- Fill material not meeting specified requirements.

10.3.3 Construction Monitoring Program

The proper construction of soil slopes and embankments requires a detailed construction monitoring program. The first requirement of the monitoring program is to assure that the contractor has an adequate quality control plan and is following it. The objectives of the construction monitoring program are to verify that:

The foundation soil stratum in the field is the same as that considered in the design
The foundation soil has adequate strength (bearing capacity) to meet the design requirements
The backfill material is placed in accordance with the specifications (e.g. lift thickness, moisture content, and density)
Benching of the existing slope on side hill fills is performed in accordance with the construction drawings
Drainage features are installed properly
The groundwater and surface water are controlled in a manner that will not affect the performance of the slope or embankment.

It is recommended that a full-time field inspector, under the direct supervision of a registered Professional Engineer, be present at the site during all earthwork operations. This will reduce the probability of having unacceptable soil slope or embankment performance due to non-compliance with project specifications.

In order to achieve the objectives listed above, it is important that the field inspector has a working knowledge of the project's plans and specifications and is able to correlate this knowledge to the field inspection. The field personnel should also be familiar with the geotechnical report prepared for the project. A review of the geotechnical report will provide the inspector with an understanding of the soils expected at the site and their anticipated behavior. Finally, the field inspector should be able to identify the different soil types encountered during excavation.

The elements of the inspection program include: the assessment of the foundation soils, compaction testing of engineered fill, review of the plans and specifications, inspection of the slope or embankment dimensions and slope face angles, and documentation of the inspection. The assessment of the foundation soil will typically include verification that the soil has adequate strength and that the foundation soil stratum in the field is the same as that considered in the design. This assessment will, therefore, typically consist of two parts. The verification that the soil has adequate strength will involve in-situ testing of the subgrade with either a pocket penetrometer (Clays) or a dynamic cone penetrometer (Sands and Silts) or some other in-situ testing apparatus. If engineered fill is required, then compaction tests will be required to verify that the design density of the fill is achieved. A visual inspection should also be performed to verify that the foundation soil stratum is the same as that considered in the design. This visual inspection should also note the presence of any standing water. Organic, and plastic (CH, MH) soils and debris (fill) should be identified and removed before the bearing surface is approved.

Identification of the foundation soils may be very difficult in the field. It is, therefore, important that the field inspector be familiar with the expected soil conditions at the site. Natural soils generally have an orderly appearance. There may be fine layering of soil types for depositional soils. For residual soils, the characteristics of the soil overlying the parent rock may be similar to those of the underlying material. Fill soils, on the other hand, may appear very different from the surrounding soils. The deposit may be very random in nature and may contain man-made debris and organic material.

10.4 INSTRUMENTATION MONITORING

The followings briefly describe the activities generally involved in geotechnical instrumentation monitoring. The instrumentation that is typically used in soil slope and embankment construction includes; inclinometers, extensometers, piezometers, observation wells, and settlement plates. Specific information on each of these instruments and their use may be obtained in 132041 "Geotechnical Instrumentation" (Dunnicliff 1998). A typical instrumentation plan for an embankment over soft foundation soils is shown in Figure 10-5. Table 10-2 (same as the Table 7-1 of 13241 Geotechnical Instrumentation (Dunnicliff 1998)) lists the suitable instruments for monitoring embankment on soft ground. Figure 10-6 shows a typical arrangement for instrumenting a cut slope in soils. Table 10-3 (same as Table 7-2 of 132041 Geotechnical Instrumentation (Dunnicliff 1998)) lists the suitable instruments for monitoring a cut slope.

The objectives of a monitoring program must be clearly defined prior to developing the plan. For example, for slide repairs horizontal and vertical movement of the slope both during and after remediation is a major concern (i.e., has the slope been stabilized?). In addition any changes in the groundwater table which will affect stability are also potentially major concerns. A monitoring system that consists of inclinometers, extensometers, and observation wells will provide the information desired for this remediation example.

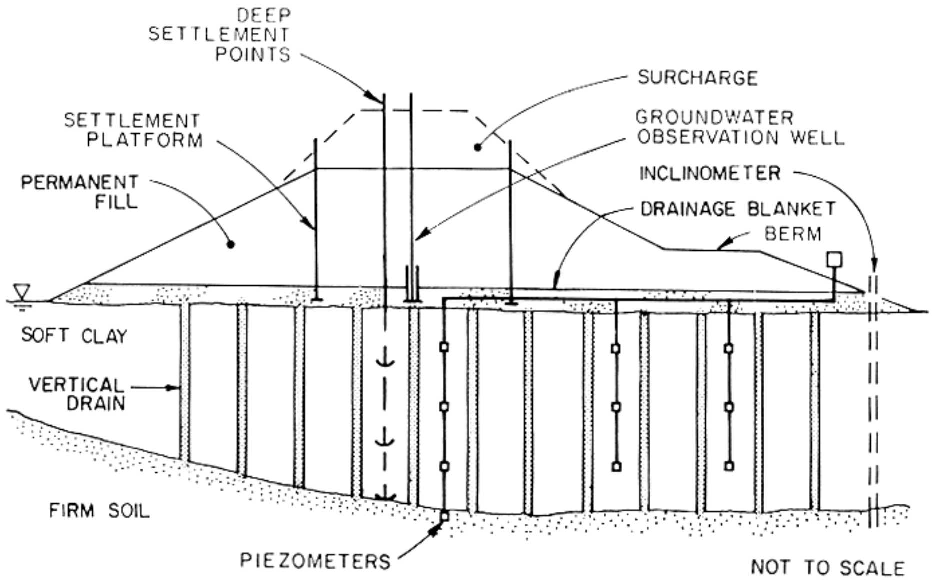


Figure 10-5: Schematic of a Typical Vertical Drain Installation for a Highway Embankment (Holtz, 1989)

TABLE 10-2
SUITABLE INSTRUMENTS FOR MONITORING EMBANKMENTS ON SOFT GROUNDS (NHI 132041A)

Geotechnical Questions	Measurement	Suitable Instruments
What are the initial site conditions?	Groundwater Pressure Vertical Deformation	Open standpipe, vibrating wire, or pneumatic piezometers Surveying methods
What information can be provided by instrumenting a test embankment?	As for "What is the progress of consolidation?	As for "What is the progress of consolidation?
What is the progress of consolidation?	Vertical deformation of embankment surface and ground surface at and beyond toe of embankment Vertical deformation of original ground surface below embankment Vertical deformation and compression of subsurface Groundwater Pressure	Surveying methods Settlement platforms Single-point and full profile liquid level gages Horizontal inclinometers Probe extensometers with induction coil or magnet/reed switch transducers Open standpipe, vibrating wire or pneumatic piezometers. Consider push-in type.
Is the embankment stable?	Horizontal deformation	Surveying methods inclinometers
What are the fill quantities?	Vertical deformation of original ground surface below embankment	Settlement platforms Full-profile liquid level gages Horizontal inclinometers

Whether as part of new construction or remediation, the instruments are typically monitored by the owner or owner's representative. The data collected should be processed and shared with the contractor in a timely manner. If data shows that the performance (settlement and/or stability) of the slope or embankment may be in jeopardy, all appropriate personnel shall be notified immediately.

Data reliability is usually more important than absolute precision (i.e., correct number) for most instrumentation applications. It is essential that instruments be calibrated not only prior to their installation but also from time to time during the life of the monitoring system. Where instruments are expected to remain in use for several years, the possibility of instrument drift can not be ignored, particularly when electrical instruments are used. The complete system, including the detecting device, associated electronics and recorder, should be calibrated as one unit. Instruments sensitive to weather and gravity variations should be calibrated on site. Probe-type instruments are easier to calibrate but are unsuitable for situations requiring continuous monitoring.

TABLE 10-3
SUITABLE INSTRUMENTS FOR MONITORING CUT SLOPES IN SOIL (NHI 132011A)

Geotechnical Questions	Measurement	Suitable Instruments
What are the initial site conditions?	Groundwater Pressure	Open standpipe, vibrating wire, or pneumatic piezometers
	Horizontal deformation	Surveying methods Inclinometers
Is the slope stable during excavation?	Surface deformation	Surveying methods Tiltmeters Metallic time domain reflectometry
	Subsurface deformation	Inclinometers Shear plane indications In-place inclinometers Metallic time domain reflectometry
	Groundwater pressure	Vibrating wire or pneumatic piezometers
Is the slope stable in the long term?	As for "Is the slope stable during excavation?"	As for "Is the slope stable during excavation?"
	Precipitation	Rain gages Snow stakes
	Load in tiebacks	Load cells

The instruments should be read systematically by a competent person who understands the purpose of each instrument. The frequency of reading will depend upon the situation and the nature of the changes that the instruments are monitoring. However, monitoring is usually conducted more frequently immediately after the completion of the project and becomes less frequent as time goes by. Readings are often most critical in inclement weather conditions. For example, piezometers may require reading several times a day during and after heavy rains. In the case of landslide investigations readings may be taken as often as several times a day. This may require that remote sensing instruments be used that can send data via satellite to the field or home office.

Readings should be recorded on standard field sheets that include details of the probable range of readings from the instruments being observed. Any readings that indicate a marked change in conditions should be checked immediately. The functioning of the instruments should be also checked when unexpected readings occur.

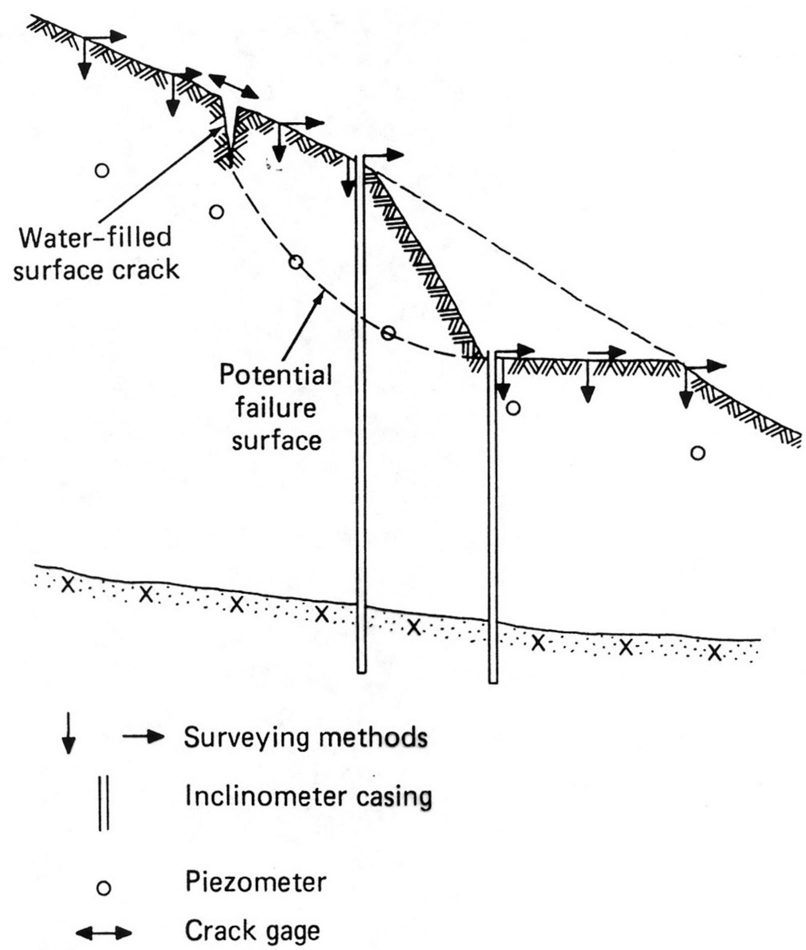


Figure 10-6: Typical Instrumentation Locations for Cut Slopes (After NHI 132041A Geotechnical Instrumentation)

10.4.1 Data Processing

The easiest way to process instrumentation data is with spreadsheets. However, if a huge amount of data is expected, a database program may be more appropriate. Raw data can be downloaded or entered by hand. All data require some sort of processing, for example, the processing could involve converting from length to strain or stress, or converting a reading into a measurement by comparing to the initial readings. Spreadsheets can be programmed to perform the calculations immediately. The processed data should then be plotted for visual inspection. It is very important that a hard copy of the field data should always be made. If data is lost in the computer, it can always be re-entered if a hard copy exists.

All instrument readings should be plotted on a time base so that the significance of the variation can be assessed more easily. Examples of typical plots are given in Figures 10-7, and 10-8.

Continuous-reading data loggers are now used routinely on large projects. These devices can be programmed to read a variety of instruments at preselected intervals and store the data for retrieval or even communicate to an office via telephone lines. Raw data can be collected or the device can be programmed to preprocess the data in many useful output forms and sound alarms when threshold values are exceeded.

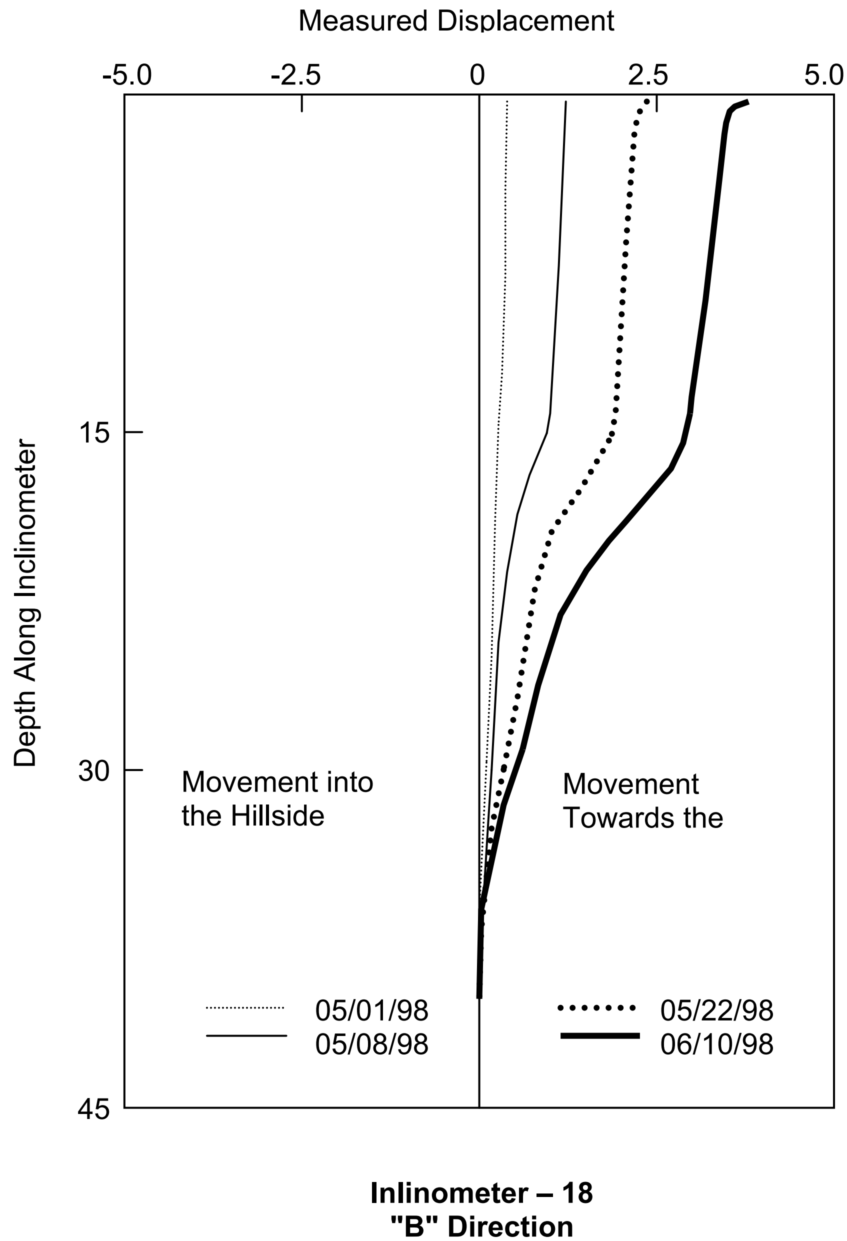


Figure 10-7: Typical Inclinometer Data Plot (After FHWA, 1994)

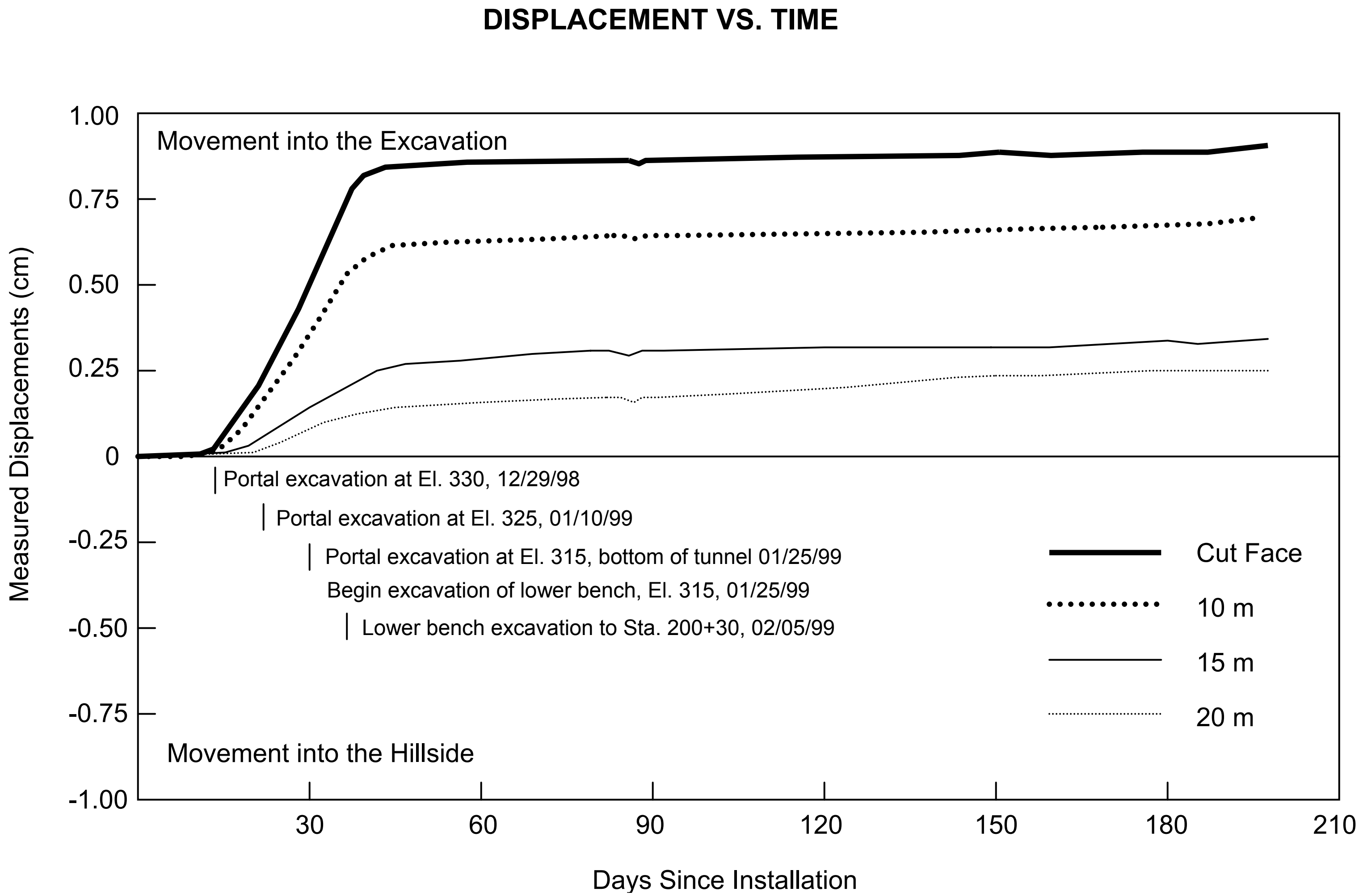


Figure 10-8: Typical Multiple-Point Borehole Extensometer (MPBX) Data Plot (After FHWA, 1994)

10.4.2 Interpretation

When the data have been collected and processed, the next step is to plot the information. Plots should show displacement versus time. Rates of movement can then be calculated from the slopes of the curves. It is advisable to keep the plots as current as possible. When displacements occur at an increasing rate, the interpretative plots are generally needed at a greater frequency.

Interpretative plots require showing all the information available, which might include the ground surface, crack locations, shear planes, groundwater table elevations, and pore pressure. Developing this type of plot is time-consuming but is well worth the effort. Interpretative plots should be to scale. All additional information should be clearly labeled.

For multiple-point borehole extensometers (MPBXs), the most easily visualized and informative plot is of anchor displacement with time. If the instrument head is considered to be stable, anchor displacement relative to the head should be plotted. For the slope stability applications, the deepest anchor is typically extended into stable ground. In this case, the plot should show anchor displacements and instrument head displacements relative to the deep anchor. For more information concerning instrumentation interpretation for not only the extensometer, but the inclinometer, piezometer, settlement plates, etc. see NHI 132041 – Geotechnical Instrumentation.

10.4.3 Response

Threshold values from the instrumentation program should be established prior to construction. Once the instrumentation data is collected and shown on plots or graphs, if the threshold values are exceeded remedial measures need to be implemented. Remedial measures are varied, but the corrective action chosen should be sufficient to eliminate the need for future corrective action, if at all possible. Various remedial measures have been outlined in Chapter 9.

10.5 LONG TERM MONITORING AND MAINTAINENCE

Regular inspections and maintenance are essential for the continued stability of highway slopes and embankments. Sometimes, recommendations for maintenance are not followed after the slope or embankment is constructed. Careful design and detailing can reduce both the maintenance required and the physical labor involved. Slopes and embankments should be designed maintenance free if possible. However, all slopes require some degree of periodic long term monitoring.

It is essential to keep a good construction database for the slope or embankment. This data would form the basis for future slope maintenance records. The designer should recommend a slope inspection and maintenance program after construction. Such a program should contain data regarding expected post-construction behavior of the slope, as well as guidance on maintaining and reading installed instruments, and should indicate the probable range of readings that will be obtained if the structure is functioning as designed. If, during the lifetime of the slope or embankment, the observed behavior and readings obtained from instruments indicate that conditions worse than that allowed for in the design are occurring, it may be necessary to undertake remedial measures to ensure slope or embankment performance. Instrument readings and inspection records should be kept on standard record sheets section 10.5.1). The maintenance engineer should use forms with appropriate observation items when performing regular inspections of the slopes. He or she should be responsible for initiating and scheduling future maintenance inspections and recommended remedial works, if necessary.

It is imperative to establish and maintain a precise record of all indications of slope instability and sections of highway that shows distress. Such a record should be reviewed periodically by the geotechnical engineering staff and consulting engineer, if appropriate. This review should be jointly undertaken by the maintenance engineers and the geotechnical engineers.

It should be noted that the maintenance engineers should consult the geotechnical engineers about the geotechnical assumptions used to design the slope and on matters concerning specialty structures, for example, ground anchors or mechanically stabilized embankment walls.

Chapter 4 of the FHWA "Highway Slope Maintenance and Slide Restoration Workshop" manual provides detailed guidances on the slope maintenance.

10.5.1 Inspections

Frequency

For slopes and embankments that have been in existence for many years, it may be necessary for maintenance engineers to screen all the slopes and embankments under their control to decide the frequency of inspections in view of the potential consequences of failure. As a general rule, slopes of high risk category (in terms of loss of human lives and economic loss or loss of highway service) should be more frequently inspected than those of low risk category. Inspections should be performed by the maintenance engineers and the geotechnical engineers at the recommended intervals shown in Table 10-4.

For new slopes, it may be necessary to undertake more frequent inspections for the first year, but inspections may be required less often thereafter if the slope performs as expected (Table 10-5). More frequent inspections in the first year are needed because signs of instability and settlement problems usually arise in the first year, especially after the first wet season. If drought conditions or low rainfall conditions exist during the first year, this "probation period" should be extended until the slope undergoes a substantial saturation and performs well as a result. If problems are detected, they should be repaired and remedied without delay. In addition, design assumptions should also be verified and experience gained from the actual performance of the slope so that future designs can be modified as necessary to improve performance (empirically based design).

Tables 10-4 and 10-5 are intended as a recommended guideline regarding the frequency for inspections. The frequency for inspections will vary depending on the geologic complexity, nature of the slope, and, most of all, available funding and resources.

TABLE 10-4
INTERVAL BETWEEN MAINTENANCE INSPECTIONS FOR EXISTING SLOPES

Inspector	Recommended Interval	
	High-risk slopes	Low-risk slopes
Maintenance Engineer	6 months	2 years
Geotechnical Engineer	2 years	5 years

TABLE 10-5
INTERVAL BETWEEN MAINTENANCE INSPECTIONS FOR NEW SLOPES

Inspector	Recommended Interval	
	High-risk slopes	Low-risk slopes
Maintenance Engineer	2 months for the first year and 6 months thereafter	6 months for the first year and 2 years thereafter
Geotechnical Engineer	6 months for the first year and 2 years thereafter	Once for the first year and 5 years thereafter

Technical/Maintenance Inspections

The purpose of technical inspections is to ensure that the slope is not deteriorating. These inspections also may be used to identify slopes that need to be upgraded into a higher risk category. It is important that guidance notes are issued to the maintenance inspector. These should highlight such points as the crest, all berms/benches, and toe of slope must be inspected. It is also important that worrisome observations should be brought to the attention of the maintenance supervisor and/or the geotechnical engineer. An action plan of additional repairs should be implemented without delay.

Engineering Inspections

Engineering inspections should be carried out on slopes that exhibit signs of instability or questionable behavior. The purposes of engineering inspections are as follows: (1) to verify the technical inspections, (2) to identify the cause of slope instability, and (3) to advise on the remedial work requisite for the unstable slope. The engineering inspections should be conducted by a geotechnical engineer who has a broad background in engineering geology and soil mechanics, as well as a working knowledge of construction and maintenance procedures.

Inspection Reports

It is of the utmost importance that all inspections should be properly recorded and that a system should exist to transfer inspection recommendations into remedial works, preventive works, or detailed investigation. The inspection reports should be properly filed and kept for later retrieval. Once the database is established, the reports should be added to produce a historical record of the slope. The maintenance engineer should always check previous inspection records and follow up to see if the last set of recommendations has been fully implemented. Where appropriate, photographs should supplement reports.
Figures 10-9 through 10-11 are examples of a 3-sheet-long slope maintenance inspection record. These are intended for general guidance for recording features that are often observed during inspections. These forms are not tailored to specific types of slope stabilization work, for example, ground anchors, soil nails, and mechanically stabilized earth walls.

To complete the inspection report, the following information (when applicable) should be included:

- Location (state, county, city, highway number, mile marker)
- Type of slope and general features (reinforcements, cut above road, highway embankment, downhill cut, remote/populated area)
- Geotechnical conditions (soil, rock, fill-hydraulic, semi compacted/dumped)
- Type of instability (mud flow, rotational, translational, wedge or composite failures)

- Contributing factors (surface drainage, subdrainage, debris material, rain conditions, blocked drains, man-made activities)

Inspection data (rate of movement, visual signs of distress, cracks, erosion, effect on roadway, utility locations, dimensions of slide, maintenance activity, property information, amounts of rainfall)
Sketches and photographs

SLOPE MAINTENANCE INSPECTION		
Slope Location:		Page 1 of 3
Slope Number:	Weather:	
Inspecting Officer	Position:	Date:

Risk Category Low/Medium/High
Last Inspection Date:
Is Interval Between Inspection OK? Yes/No
Previous Risk Category Low/Medium/High
Have Past Recommendations Been Carried Out? Yes/No

Summary:
Major Works Required? Yes/No
Minor Works Required? Yes/No
Investigation Needed? Yes/No
Slope Satisfactory? Yes/No

Access:
Is There Good Maintenance Access? Yes/No
Is It Difficult for the Public to Gain Access? Yes/No
Has the Inspecting Officer Gained Access to the Crest, the Toe, and All Berms? Yes/No

Comments:

Instrumentation:
Have All Instrumentation Systems Been Checked? Yes/No
Have All Instrumentation Results Been Plotted? Yes/No
Are All Readings Acceptable? Yes/No
Is There a Need for New Instrumentation? Yes/No

Comments:

Note: This inspection sheet is to be read together with the slope data sheet/file

Figure 10-9: Example Of A Maintenance Inspection Record For A Slope, Sheet 1 Of 3 (GCO, 1984)

SLOPE MAINTENANCE INSPECTION

Slope Location: Page 2 of 3

Condition of Slope:

	Status of Feature				
	None	Good	Satis-factory	Work Needed Minor	Work Needed Major
Condition of Impermeable Surface					
Extent of Impermeable Surface					
Condition of Weepholes					
Capacity of Weepholes					
Condition of Vegetated Surface					
Capacity of Surface Drainage					
Condition of U-Channels & Steep Channels					
Condition of Catchpits & Sandtraps					
Condition of Raking Drains					
Condition of Associated Culverts & nullahs					
Condition of Artificial Support					
Condition of Toe Fence/Toe Barrier					

Comments:

		Works Needed	
		Minor	Major
Has There Been a Recent Slope Failure?	Yes/No		
Has There Been Any Recent Erosion?	Yes/No		
Has There Been Any Recent Movement?	Yes/No		
Are There Any Tension Cracks at the Crest?	Yes/No		
Is There Adequate Protection Against Infiltration Above the Crest?	Yes/No		
Has There Been Any Recent Seepage?	Yes/No		

If Seepage Give Details:

If Movements Give Details:

Comments:

Figure 10-10: Example of A Maintenance Inspection Record For A Slope, Sheet 2 Of 3 (GCO, 1984)

SLOPE MAINTENANCE INSPECTION

Slope Location

Page 3 of 3

Associated Retaining Walls

		Works Needed	
		Minor	Major
Has there Been Wall movements?	Yes/No		
Has there Been Recent Wall Settlement?	Yes/No		
Has there Been Recent Wall Cracking?	Yes/No		
Has there Been Recent Wall Tilting?	Yes/No		
Has there Been Recent Wall Bulging?	Yes/No		
Has there Been Any Recent Movement?	Yes/No		
Is the Capacity of the Weepholes Adequate?	Yes/No		
Are the Weepholes Clear?	Yes/No		
Are the Mortar Joints/Pointing Satisfactory?	Yes/No		
Is Vegetation Adversely Affecting the Wall?	Yes/No		

Comments:

Services & Drainage:

Are Services Adversely Affecting the Slope? Yes/No
Do Any Services Need Testing? Yes/No
Has the Appropriate Authority Been Informed? Yes/No

General & Comments:

Does the Slope Need Upgrading to Meet the Present Risk Category? Yes/No

Recommendations:

Signature:

Figure 10-11 Example of A Maintenance Inspection Record For A Slope, Sheet 3 Of 3 (GCO, 1984)

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CHAPTER 11
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