GEOTECHNICAL PRINCIPLES FOR STABLE SLOPES

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF MAPS

GENERAL SLOPE STABILITY PRINCIPLES

PREFACE

This document was prepared by Terraprobe Limited on behalf of the Ontario Ministry of Natural Resources (MNR);

- Aggregate and Petroleum Resources Section (Pits and Quarries)
- Conservation Authorities Section, and
- Shoreline Section, Aquatic Ecosystem Branch.

MNR and the Conservation Authorities of Ontario carry out several natural resource and hazard management programs with slope stability components. These programs involve various site types including; aggregate resources (gravel pits), shorelines (bluffs), and riverine settings (valleys and gullies). Under existing provincial policies, rehabilitation or development approval may require a proponent to undertake geotechnical analysis to determine a stable slope for a specific design or site location.

The purpose of this study is to provide clear documentation of slope stability principles, to assist people in identifying and understanding important aspects of slope stability, and to require uniform consistency in site investigations and stabilization works. It should be noted, that this document will deal with slope stability issues from an applications standpoint. Its main use will be to assist agencies implementing provincial government policy during situations of land development. It will not focus in detail on the academic and theoretical principles. For those who require or are interested in more detail, the bibliography will provide direction to further reading on these issues. As well, the main focus will be on slope conditions and situations which arise most frequently in Ontario and this will not include detailed discussion of complex or rare sets of circumstances.

Finally, this document is intended as a general guideline to understanding slope stability. It is not intended to replace the judgement or experience of qualified professional engineers and scientists. The guidance of qualified professionals should be sought, particularly where the stability of a slope may affect public safety or may create large cost implications.

This document does not address rock slopes. Nor does it deal with sensitive Marine clays (Leda clays) in the Ottawa region ("retrogressive sliding"). This phenomena has been well documented previously and is too complex a subject to deal with in detail (see Bibliography for references).

The general structure of this document is as follows;

Sections 1 to 6 - General Slope Stability Principles

- soil types and properties, erosion and slope instability, types of slides/failures, common methods of analysis, comparisons
- main purpose is to familiarize laypersons with soil mechanics and engineering issues of slope stability

Sections 7 to 9 - Suggested Level of Geotechnical Investigation Required to Assess Slope Stability

- site conditions, level of investigation, design minimum Factor of Safety, field methods, laboratory methods
- main purpose is to suggest guidelines for investigations and design

Section 10 - Possible Responses to Unstable Slope Conditions

- stabilization methods, comparisons

- main purpose is to catalogue and describe common methods

Section 11 - Slope Rehabilitation for Pits

- site conditions, typical stable inclinations
- stabilization alternatives, submerged slopes
- main purpose is to describe slope rehabilitation for pits

1. INTRODUCTION

Slopes occur in many environments such as pits and quarries, shoreline bluffs, and river valleys. This document addresses only natural slopes; those comprised of naturally deposited materials and not man-made slopes such as dams or embankments. Therefore problems related to reservoirs, mine tailings, or impounded water will not be discussed.

Slope instability (also commonly referred to as a 'slide') can result in ground loss or ground movement, that could affect structures or natural features at the top or bottom of the slope (see Figure 1).

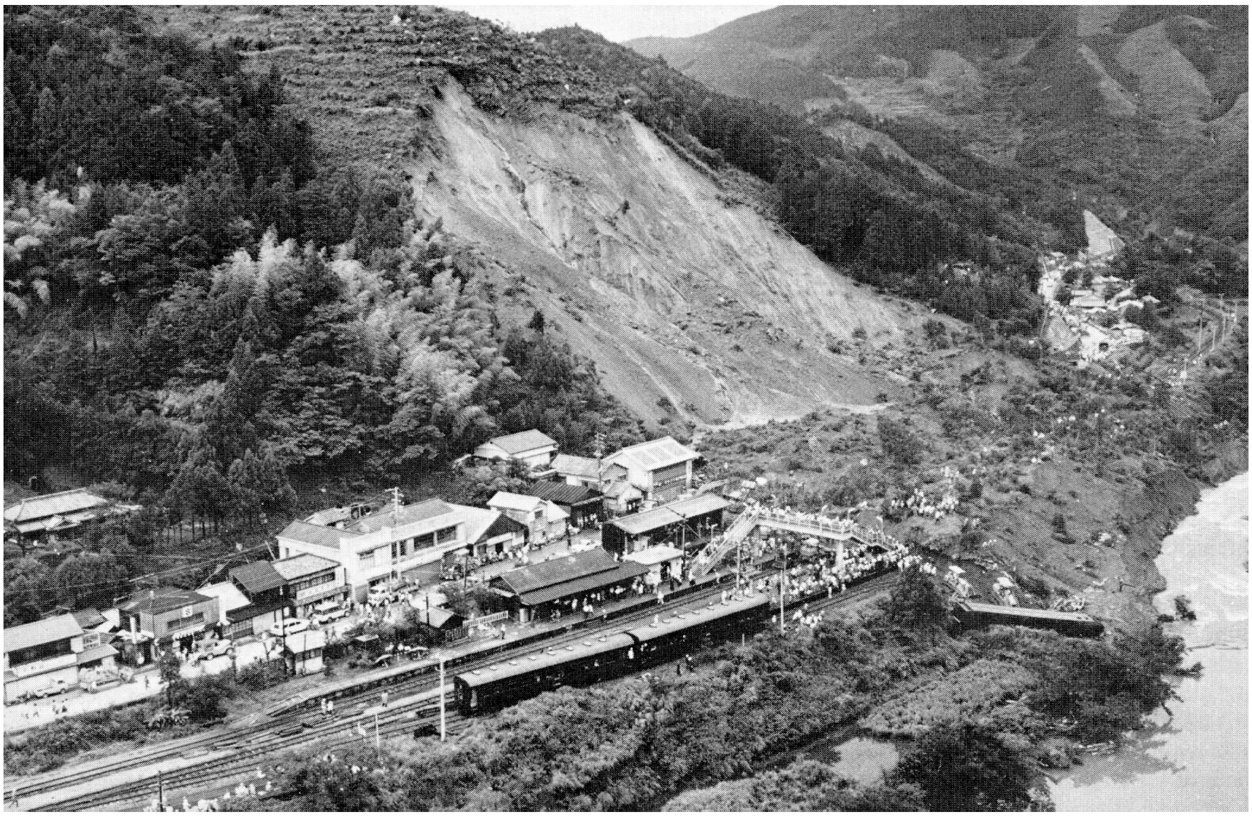


Figure 1 - Slope Failure Effects on Structures, Rivers

This movement could lead to loss of ground support and damage to buildings, roads, buried utilities, or to siltation or blockage of rivers (creeks or channels) and local flooding, damage to fish stocks, or even to personal injury. In extreme cases, lives have been lost due to sudden landslides in the "quick clays" (Leda) of Eastern Canada. In view of the safety and potential liability issues associated with slope movements, it is important that there be awareness and recognition of slope stability principles. This is reflected by requirements for geotechnical engineering reports on slope stability in various government regulations including the Aggregate Resources and Petroleum Act, the National Building Code (building departments), and by policies of local Conservation Authorities and municipal planning authorities.

2. SOIL TYPES AND PROPERTIES IN ONTARIO

The following sections describe the composition and physical properties of natural soils in Ontario, as they pertain to slope stability. Comparison summaries are provided and soil mechanics terminology is introduced.

Unlike manufactured materials such as steel and concrete, soil has properties that can be unique to each site and quite variable. For this reason a subsurface investigation of soil stratigraphy and strength properties should be considered at every site where the slopes are high and steep (ie. higher than 3 m and steeper than 3 to 1, horiz. to vert.) or where there are issues of public safety or property value.

Soil slopes can be composed of a single soil type (referred to as 'massive' or homogeneous) or of many soil layers (stratified) with different strength properties (see Figures 2 and 3). Similarly a wide variety of groundwater and seepage conditions may occur, ranging from dry conditions to several distinct groundwater flow systems within the same slope.

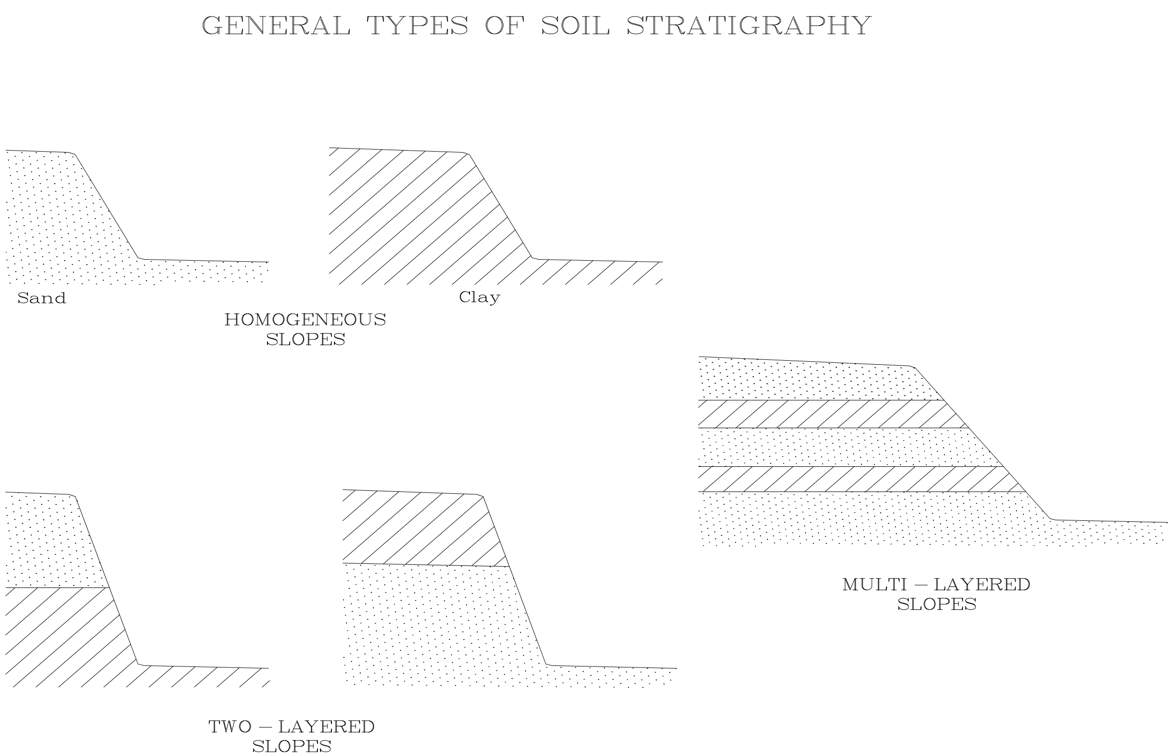


Figure 2 - Stratigraphy of Soil Slopes

Figure 3 - Photo, Slope Soil Stratigraphy

2.1 Geological History

Following is a brief outline of the common soil and bedrock types in Ontario. It describes the soil structure, behaviour, and typical index properties pertaining to slope stability.

Much of Ontario has been subjected to the Wisconsinan glaciation (about 12,000 years ago) when thick glaciers advanced and retreated across the land. This period created the soil deposits , surface features, and drainage patterns of much of southern Ontario (refer to Chapman and Putnam, 1984; Physiography of Southern Ontario). The most common glacial deposits found in Ontario are noted below;

- glacial till is a heterogeneous mixture of many particle sizes ranging from clay to boulders (clayey silt, sandy silt, silt and sand); transported and laid down near the base of a glacier; typically consolidated and competent; non-sorted and non-stratified; 'till' is a Scottish word describing a stony clay; sometimes referred to as 'boulder clay'

- glaciofluvial outwash sands and gravels (alluvium) were deposited by drainage of ice meltwater, often well-sorted and stratified.

- glaciolacustrine clays and silts (laminated or varved) deposited in bottoms of glacier lakes and ponds; fine-grained.

- glacial marine sediment (Leda clays) clay-rich, flocculated structure, sensitive,

- ice-contact stratified drift (kames, eskers, kettles) modified by meltwater during or after deposition; may have considerable sorting and stratification, as well as large range of sizes, chaotic structure, and inclusions of till.

- eolian deposits of sand dunes, sand sheets, and loess.

After the glacier retreat, the Ontario landscape was subjected to erosion and the cutting of river valleys as the result of surface run-off and drainage. Locally the glacial deposits have been modified by this erosion. Land development activities such as cutting and filling (earth-moving) and drainage may also result in significant changes to soil and groundwater conditions.

Soil materials are often referred to as 'overburden' or unconsolidated materials. Bedrock is encountered beneath the soil materials, and is generally strong and consolidated. Most of southern Ontario is underlain by sedimentary rocks such as shale and limestone. Central and northern Ontario is predominantly underlain by igneous and metamorphic rocks such as granite and gneiss (see Figures 4 to 7).

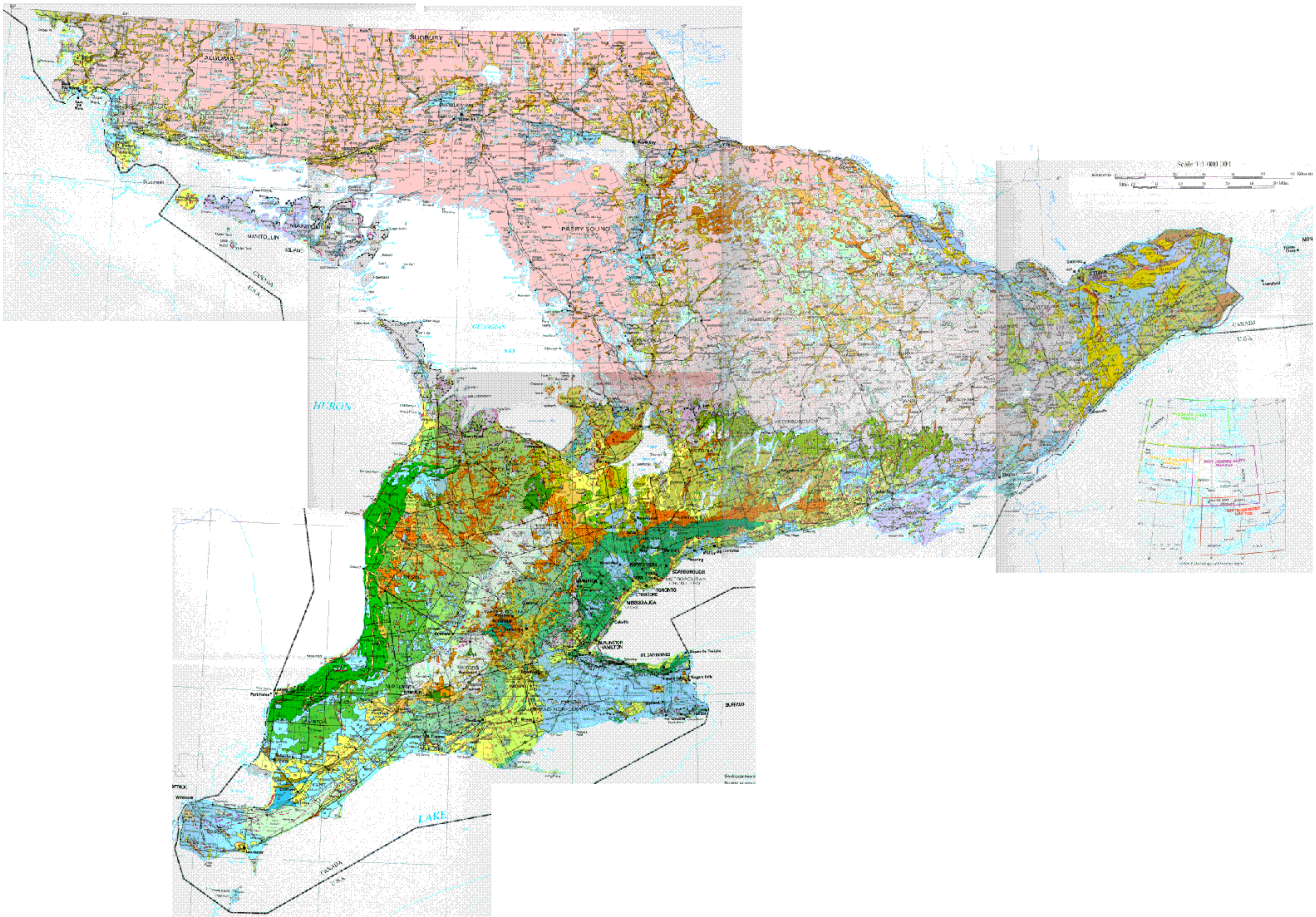
Figure 4 - Photo, Rock Slope Shoreline

Figure 5 - Photo, Stratified Sand Slope

Figure 6 - Photo, Silt Slope

Figure 7 - Photo, Gravel Slope

There is geologic mapping available for most of Ontario from MNR. This mapping can provide useful general information regarding soil and bedrock types for specific areas. Information can also be obtained from water well records which are filed with the Ministry of Environment. Map 1 follows and shows the Quaternary Geology for Southern Ontario and is followed by Maps 2A and 2B showing the Bedrock Geology of Ontario. Further details can be found in the Map Appendix.


SUDBURY
ALGOMA
PARRY SOUND
MANITOULIN
GEORGIAN BAY
HURON
LAKE
CANADA
U.S.A.
TORONTO
MISSISSAUGA
SCARBOROUGH
HAMILTON
BURLINGTON
ST. CATHARINES
BUFFALO

SYMBOLS
Geological boundary
Geological province boundary
Geological subprovince boundary (defined, approximate)
Fault
Structural arch
The positions of all boundaries and surveyed lines are approximate.
Scale 1:5 000 000
Kilometres 50 0 50 100 150 200 Kilometres
Miles 50 0 50 100 150 Miles
MOOSE RIVER BASIN
Fort Albany
Moosonee
Albany R.
Attawapiskat R.
Wabigoon
Subprovince
Geraldton
Quetico Subprovince
Kapuskasing
Opatica Subprovince
QUEBEC
Fraserdale Arch
Wawa Subprovince
Kapuskasing Structural Zone
Abitibi Subprovince
Timmins
Kirkland Lake
Cobalt
Cobalt Embayment
Grenville Front Tectonic Zone
Superior
Sault Ste. Marie
Penokean Fold Belt
Sudbury
GRENVILLE PROVINCE
Ottawa River
Ottawa-Bonnechere Graben
Central Gneiss Belt
Central Metasedimentary Belt
Central Metasedimentary Belt Boundary Zone
Ottawa Embayment
Ottawa
Frontenac Arch
St. Lawrence River
Lake Michigan
U.S.A.
Lake Huron
Georgian Bay
Lake Simcoe
Toronto
Lake Ontario
Algonquin Arch
MICHIGAN BASIN
APPALACHIAN BASIN
London
Chatham Sag
Lake St. Clair
Windsor
Lake Erie
Findlay Arc

Generalized distribution of major lithographic units, structures and subprovince boundaries. The subprovince boundaries within the Superior Province are after Card and Ciesielski (1986) with minor revisions. Compilation by I.A. Osmani.

2.2 Soil Structure

This section describes the various distinguishing properties of soils and the classifications or descriptions commonly used.

A soil mass is composed of individual soil particles in which there are void spaces between the particles, filled by either air or water. In engineering terms, soil is defined as unconsolidated material composed of discrete solid particles. There is a large variety in distribution of the particle sizes and shapes in soils, ranging from granular soils such as gravel or sand, to silt or clay soils or mixtures thereof.

Soil descriptions are based on the 'gradation' or distribution of particle sizes (by weight) for the following general types;

TABLE 2.1 - Soil Particle Sizes

Soil Type	**Range of Particle Size (equivalent diameter) (see Figure 8)**	**Texture**	**Notes**
gravel	2 to 60 mm	coarse	grape-, pea-like; cobbles 60 to 200 mm, grapefruit-like
sand	0.06 to 2.0 mm	gritty	particles visible by eye, salt-, sugar-like
silt	0.002 to 0.06 mm	smooth	powder-like, grains not visible to eye, cannot roll thread
clay	< 0.002 mm	silky	smears, can roll thread, like play-dough

Table 2.1 shows the large range in soil particle size,

from less than	0.002 mm	($2x10^{-3}$)
to more than	200 mm	($2x10^{2}$),

or over 5 orders of magnitude (10^5 or 100,000).

Figure 8 - Photo, Soil Particle Sizes

The grain size distribution is usually measured in a laboratory by mechanical sieving or through hydrometer analysis (sedimentation) and the test results are provided on a grain size distribution graph. These are often referred to as 'gradation curves'. Ranges of soil particle sizes are measured and the measurements are connected by a line that forms a curve.

Each soil specimen is represented as a single (generally curved) line on a semi-log graph. The grain size curve indicates the percentage of a sample that is finer than a given diameter or particle size. The knowledge of grain size distributions is valuable in designing for filtering of granular materials and engineering works, as well as erosion potential from flowing water. The two most used classification systems are the Unified System (from airfield pavement design) and the M.I.T. System (engineering and scientific applications).

Figure 9 is an example gradation graph with four soil types.

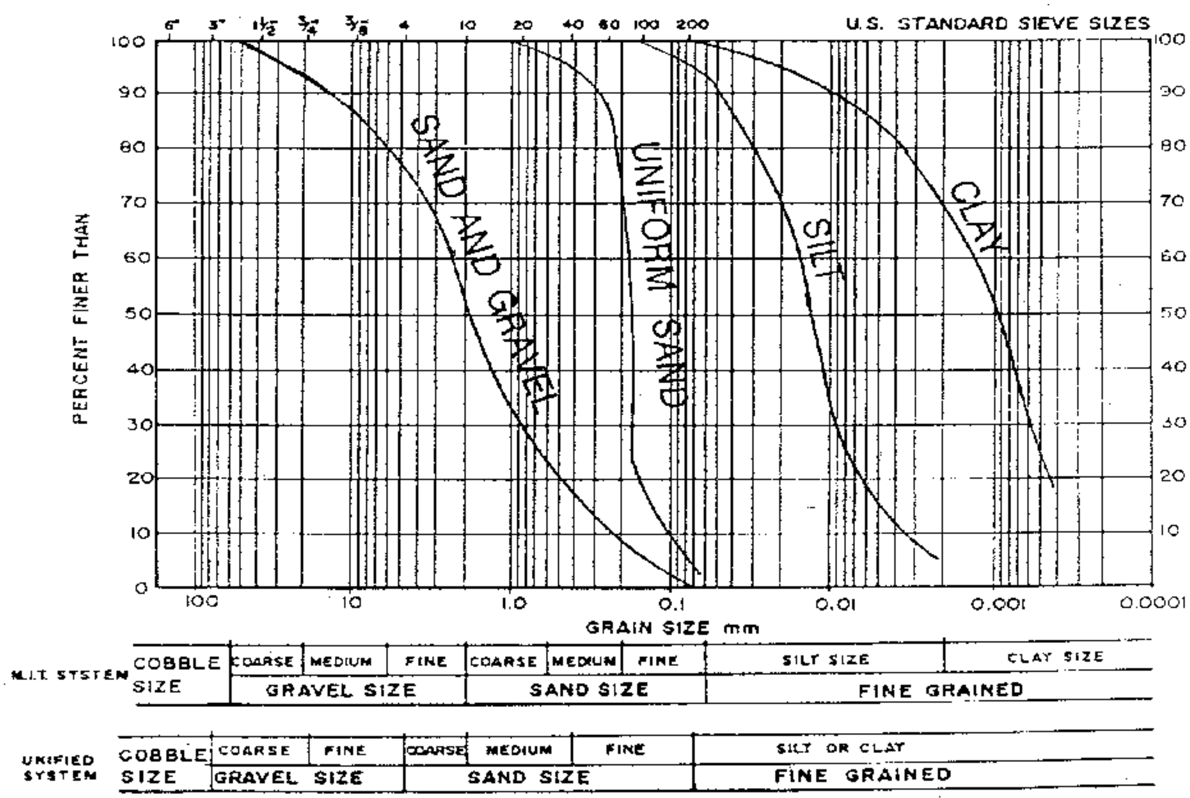


Figure 9 - Grain Size Distribution

A soil identification is based on the main particle size component (by weight), and the adjectives as follows on Table 2.2.

TABLE 2.2 - Soil Classification by Composition

Composition By Weight	**Description**	**Example**
1 - 10 %	"trace"	trace silt, trace gravel
10 - 20 %	"some"	some silt, some sand
20 - 30 %	adjective	silty, clayey, sandy
30 - 35 %	"and"	silt and sand, sand and gravel.

Following are examples of some soil descriptions;

a) silty <u>fine sand</u> with some gravel	b) <u>sand and silt</u> trace clay	c) <u>clayey silt till</u> some sand, trace gravel.

In the following gradation graph (Figure 10) the silty fine sand sample (A), using the M.I.T. system the sample was found to be about 80 percent by weight finer than gravel size (finer than 0.2 mm). Conversely, the sample has about 20 percent gravel sizes. The same sample had about 20 percent by weight finer than sand sizes (silt). With 20 percent gravel size and 20 percent silt size, there is about 60 percent sand size in the sample. Similarly for samples B) and C),

	B)	C)
Gravel sizes	0	2 %
Sand Sizes	50 %	19 %
Silt Sizes	45 %	47 %
Clay Sizes	5 %	32 %

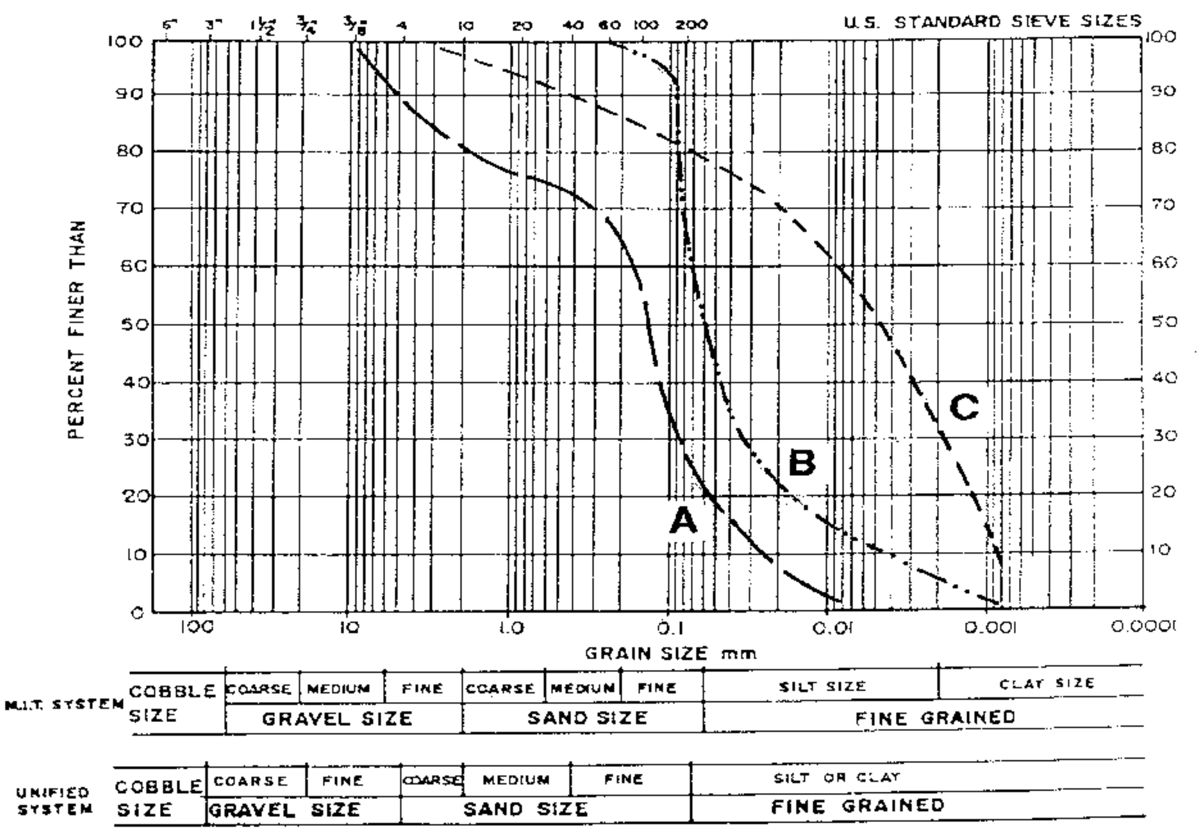


Figure 10 - Grain Size Distributions

In addition to the grain size distribution, soils are also identified or classified on the basis of various other index properties such as density, water content and plasticity (see Appendix on Laboratory Testing). Further, some slang terms or labels for certain soil types have developed as follows;

Gumbo	highly plastic clay, having a very sticky, greasy feel when wet; develops large shrinkage cracks upon drying
Hardpan	soil which has become rock-like due to accumulation of cementing minerals; sometimes used to refer to very hard till
Marl	water deposited sand, silt, or clay containing calcium carbonate; white to grey appearance; very wet and soft
Peat	fibrous, organic matter sometimes with intermixed soil; dark brown or black and very compressible
Till	mixture of sand, gravel, silt, clay produced by ploughing action of glaciers; also called 'boulder clay'
Varved Clay	sedimented deposit of alternating thin layers of silt and clay; generally 2 to 15 mm thick layers

2.3 Cohesionless Soils; Gravels, Sands, Silts

Soils such as gravels, sands and silts are often termed "granular" or "cohesionless" soils. The soil particles do not stick together and there is no strength or adherence between individual particles. They are usually composed of non-clay minerals. The soil particles have a bulky shape and are rounded to angular (see Figure 11).

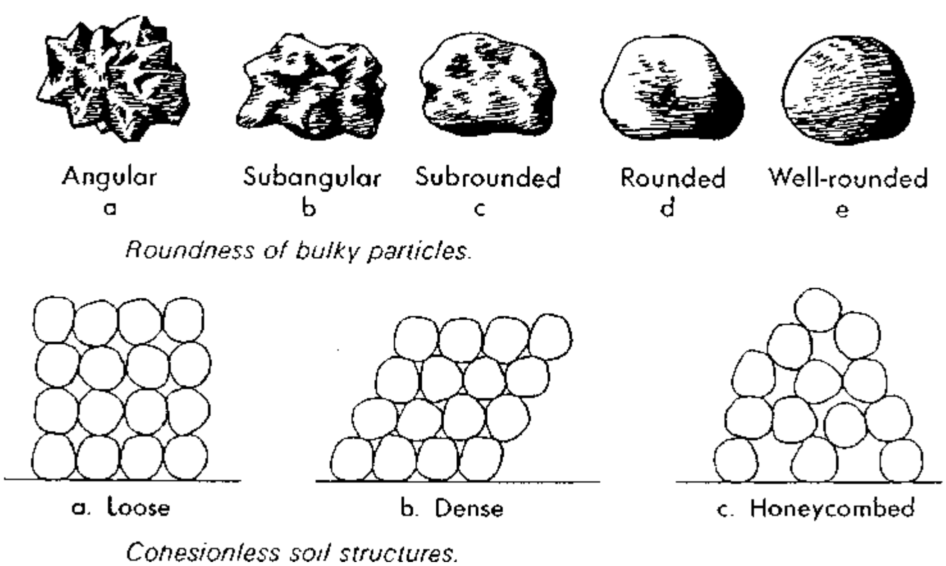


Figure 11 - Particle Arrangement, Cohesionless Soils

2.4 Cohesive Soils; Clays, Clayey Silts

Soils containing significant amounts of clay and silt are termed "cohesive". These soils can stick together in a cohesive mass or clump. The individual particles are generally not distinguishable (too fine) by the unaided eye (see Figure 12). These soils are termed "plastic" as they can be moulded in a cohesive mass or shape when moist. A measure of the 'plasticity' can be obtained with a laboratory Atterberg Limits test.

Figure 12 - Photo, Fissured Clay

The very fine individual clay particles are plate-shaped or flake-shaped and can be arranged in a "flocculated" condition similar to a house of cards or, in a dispersed condition with orientation of the particles (see Figure 13).

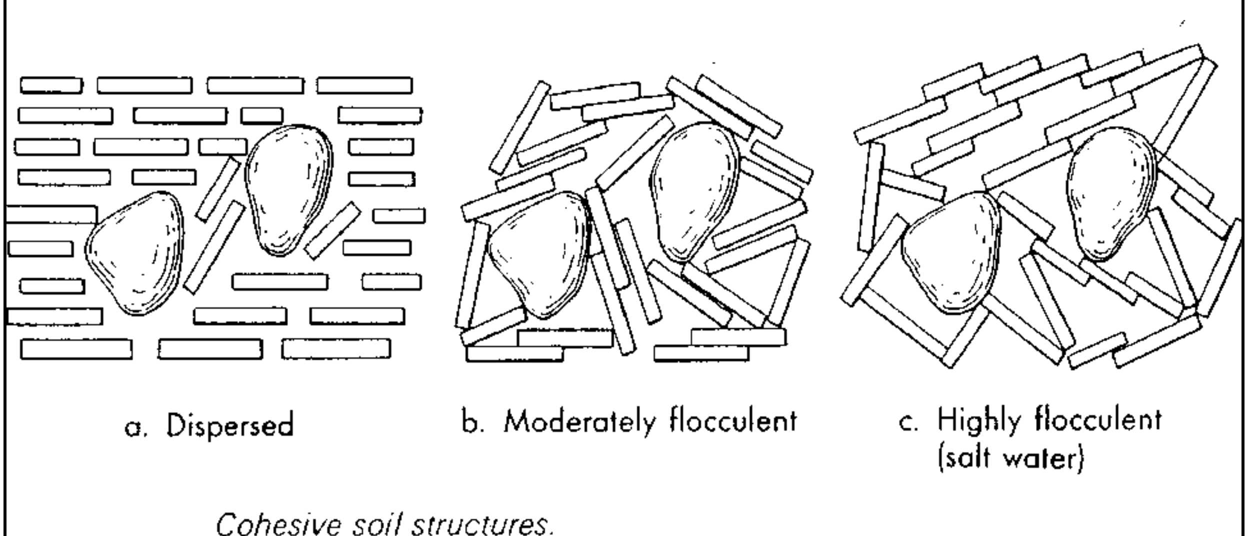


Figure 13 - Flocculated Particle Arrangement

2.5 Groundwater

The pore spaces or voids between soil particles may fill with water. When all of the voids are filled, the soil is termed 'saturated'. Groundwater is the subsurface water that occurs in saturated soils.

The flow of groundwater is governed by gravity. Groundwater seepage or 'springs' may occur on the face of a slope where the groundwater outlets or discharges onto the ground surface. The groundwater levels and flow directions can be determined by measuring water levels in observation wells, standpipes, or piezometers. Groundwater levels and flows can have a large influence on soil strength in slopes.

2.6 Soil Drainage and Pore Water

Water in the pores between soil grains significantly affects the properties and behaviour of a soil mass. In general, water tends to have a weakening affect on soil strength. Water drains or moves through the soil pores in response to both natural flow conditions and in response to loads placed on the soil (see Figure 14).

Figure 14 - Slope Drainage

Drainage characteristics have been found to be strongly affected by grain size distribution; fine soils drain slowly, coarse soils drain quickly. Coarse-grained or granular soils (sand, gravel) are generally well drained and water passes through the voids easily. Fine grained soils (silt and clay) and even some cohesionless soils that are very silty, have poor drainage and water passes through the voids slowly. The rate of drainage or flow of water through a soil is governed by the 'permeability' of a soil. The following table shows the range of permeability (measure of drainage capability).

The permeability of different soil types can vary by over 8 orders of magnitude (10^8, a factor of over 100 million) as noted in Table 2.3.

TABLE 2.3 - Relative Values Of Permeability, k (After Terzaghi and Peck)

Relative Permeability	Values of k (cm/sec) *	Typical Soil
Very permeable	Over 10^{-1}	Coarse gravel
Medium permeability	10^{-1} to 10^{-3}	Sand, fine sand
Low permeability	10^{-3} to 10^{-5}	Silty sand
Very low permeability	10^{-5} to 10^{-7}	Silt
Impervious	Less than 10^{-7}	Clay

* actually cm^3 / cm^2 / sec; volume flow / area soil / time

2.7 Drained and Undrained Behaviour

In a slope slide, the mobilization of soil resistance to sliding along the failure plane during shearing (movement), is termed "drained" or "undrained". These terms do not refer to the amount of water in the soil but rather to the stress response of the soil mass when subjected to a load or pressure, or to whether pore water pressures also contribute to the soil resistance or strength.

The two extreme possibilities consist of

drained conditions	where stress changes are applied so slowly that the soil can drain pore water without developing excess pore pressures
undrained conditions	where stress changes are applied so quickly that the soil cannot drain without developing excess pore pressures.

The drainage rate of a soil is controlled by the soil 'permeability'. High permeability soils have quick drainage (sands and gravels) while low permeability soils have slow drainage (clays and silts).

"Undrained conditions" can occur in fine-grained soils which drain very slowly. Coarse-grained soils usually have a "drained" behaviour due to their high permeability. A "drained condition" is considered to be present if the pore pressures are able to dissipate almost immediately after the load is applied. The ability of the soil to drain during loading or movement, controls the available soil strength or shearing resistance.

"Undrained" conditions are typically representative of "short term" conditions in slow draining soils such as immediately after a change in loading where the pore pressures have not had time to dissipate. Such conditions may be after excavations, or after the application of new loads on slopes (filling, or building, or earthquakes).

The length of time required for pore pressure dissipation is dependent on the soil permeability and may vary from almost instantaneous (in sands and gravels), to several months or more (in clays). Therefore, a "short term" condition for a poorly drained soil (clay) may extend for several days or weeks or months.

An engineering analysis method which takes into consideration the groundwater pressures is referred to as an "effective stress" analysis (ESA). The effective stress is a measure of the applied load or stress minus the pore pressure in the soil. Thus a low effective stress can develop if higher pore pressures occur and decreased soil strength can result.

For most stable natural slopes pore pressures in all soils (fine and coarse grained) are usually in a "drained" condition. Temporary or short-term undrained conditions can develop if toe erosion takes place (similar to excavation), or if filling or loading is applied to the slope, or if natural drainage is blocked or if the soils are flooded (i.e. watermain leak, swimming pool leak).

2.8 Soil Strengths

In soil mechanics, three basic soil properties define the soil strength and the available resistance to sliding (see Figures 15 and 16).

- soil unit weight, gamma (kN/m^3)
  γ

- angle of internal friction, phi,
  ϕ'

- cohesion (kN/m^2)
  c'

Soil strength is defined as resistance to shearing of a soil mass along a defined surface. If shearing occurs it typically results in ground movement.

The shearing resistance or 'strength' of a soil is defined as the sum of the cohesive resistance and the frictional resistance. Only fine-grained soils (clays, silts) have cohesive resistance which is the result of attraction or adhesion between soil particles. Both fine-grained and coarse-grained soils have frictional resistance, caused by individual grains rubbing at their contact points.

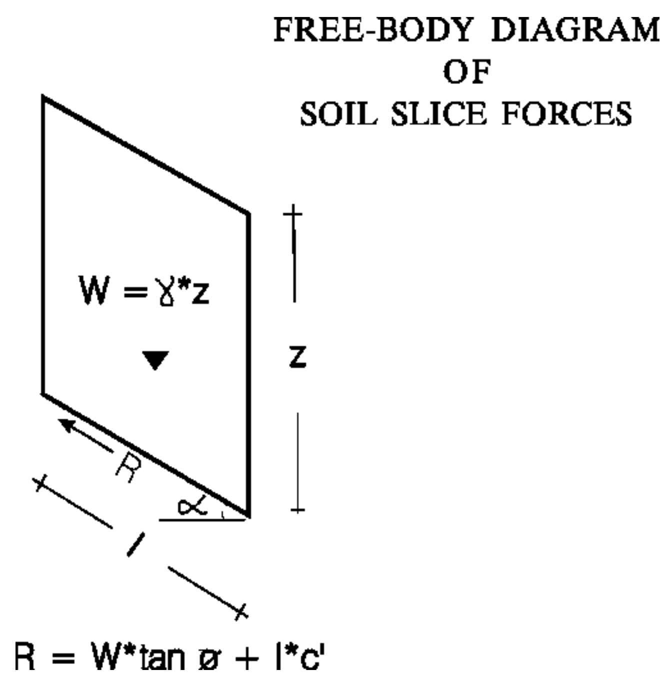


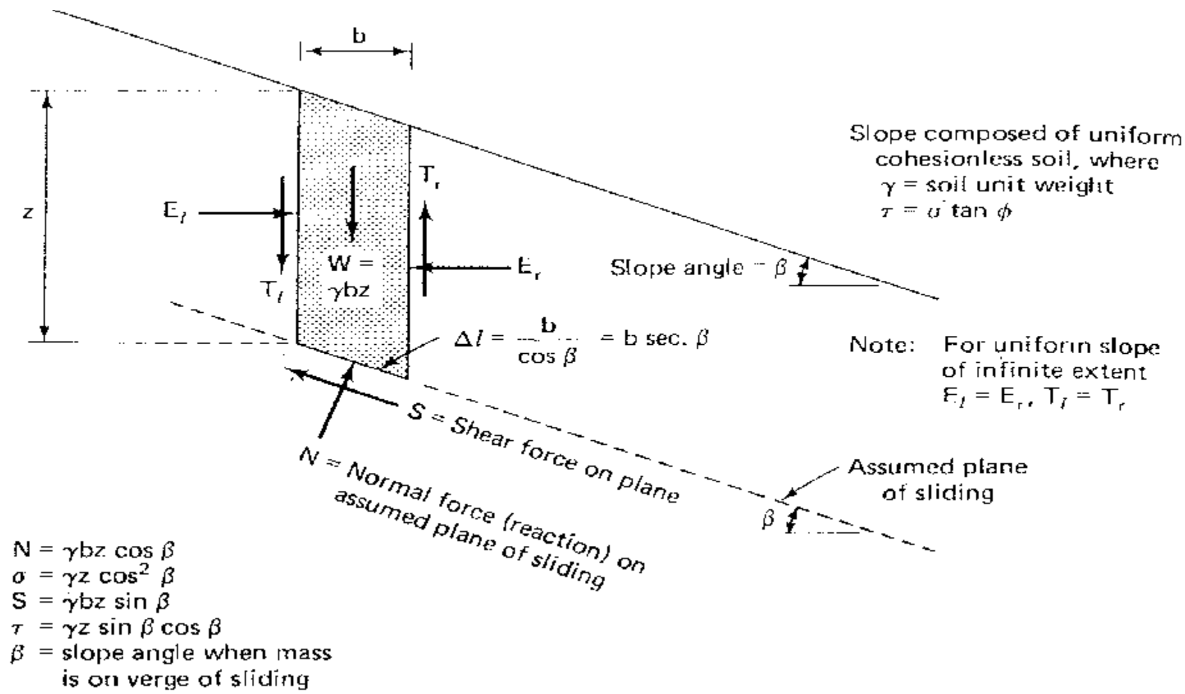
Figure 15 - Soil Resistance Properties

The frictional resistance is directly related to the contact pressure between the grains (i.e. a high contact pressure results in a high frictional resistance). The shear strength of a soil is defined by the following formula;

$$\tau = c' + (\sigma - \mu)\tan\phi'$$

where τ = soil shear resistance
c' = soil cohesion
σ = normal stress (perpendicular to shear surface)
μ = soil pore pressure
ϕ' = soil angle of internal friction

The normal stress on the shear surface is caused by the weight and applied load of the overlying soil. The pore pressure is subtracted from this load to obtain the effective load.
An analysis which takes into account the pore pressure and effective load, is termed an 'effective stress' analysis (ESA). An analysis which neglects the pore pressures is called a "total stress" analysis. Effective stress analysis can be used for both fine-grained and coarse-grained soils but it requires knowledge of the pore pressures.



Description of forces acting on representative slice of cohesionless soil in uniform slope of infinite extent.

Figure 16 - Forces on Soil Slices

A "total stress" analysis is used only for fine-grained cohesive soils. In this case the shear strength is defined by another parameter termed the 'undrained shear strength' or c_u (not effective shear strength, c'). The undrained shear strength includes the influence of frictional strength and pore pressure in one parameter. The total stress analysis does not permit evaluation of the effect of varying groundwater levels on slope stability. For short-term loading, this is equivalent to the shear strength measured by effective stress methods. However for long-term loading, total stress analysis is generally not appropriate for slopes as it tends to over-estimate the Factor of Safety.

For most "cohesionless" soils in Ontario such as sands (gravelly to silty) the following ranges of soil properties are considered typical (see Table 2.4).

TABLE 2.4 - Cohesionless Soil, Typical Properties

$c' = 0$
$\phi' = 26°$ to $42°$
$\gamma = 17$ to $19\ kN/m^3$.

For most "<u>cohesive</u>" soils in Ontario such as <u>clays</u> (clayey silt, tills), following are typical ranges (see Table 2.5).

TABLE 2.5 - Cohesive Soil, Typical Properties

$c' = 10 - 60$ kPa (kPa = kN/m^2)
$\phi' = 26°$ to $38°$
$\gamma = 17$ to $21\ kN/m^3$.

Direct measurements of the angle of internal friction can be carried out in the laboratory using the Direct Shear Test or Triaxial Compression Test (see Appendix).

However, a good estimate of ϕ' for 'cohesionless soils' (sands) can be obtained on the basis of 'N' Value (<u>Standard Penetration Test</u>), routinely measured in boreholes (see Figure 17).

The effective angle of internal friction is dependent on the relative density of the "cohesionless" soils and typically increases with density as shown on the adjacent graph, Figure 17 and Table 2.6.

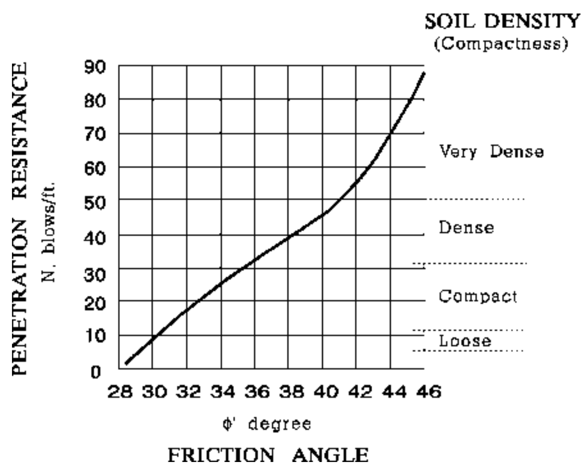


Figure 17 - 'N' vs ϕ' Relationship

TABLE 2.6 - Relative Density of Cohesionless Soils

Compactness	Very Loose	Loose	Compact	Dense	Very Dense	
Relative Density, Dr	0	15%	35%	65%	85%	100%
'N' Value, Std. Pen. Resistance	1	4	10	30	50	
ϕ' (°), angle of internal friction		28	30	36	41	
Unit weight, γ moist, kN/m^3	15		16 - 19.5	17 - 20.5	17 - 22	20.5
submerged	9.5		8.5 - 10	9.5 - 11	10 - 13.5	12
Identification in the field	reinforcing rod can be hand pushed into soil 1 m				difficult to drive 2" by 4" stake with sledge hammer	

The value of ϕ' is also dependent on the particle shape; more-angular particle shapes have higher ϕ's than more-rounded particle shapes (better inter-locking between particles). The value of ϕ' also depends on the method of testing, the sample size and condition. Direct shear tests provide the most representative results. Triaxial tests tend to under-estimate ϕ' due to the small sample size and disturbance to the sample.
The following summary chart of detailed testing and measurement of ϕ' (Table 2.7), shows the relative effect of density and particle shape on the value of ϕ';

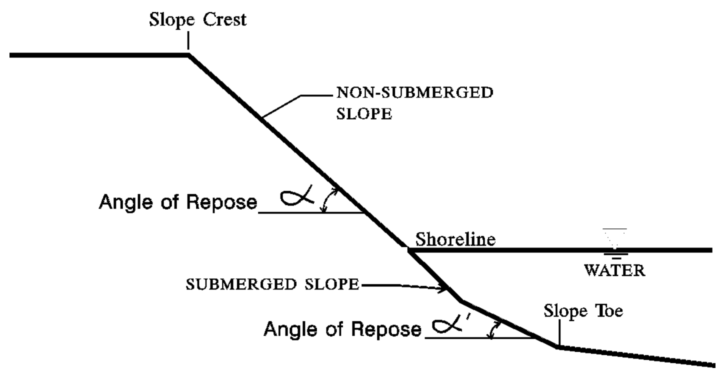


Figure 18 - Angle of Repose

TABLE 2.7 - Angle of Internal Friction of Cohesionless Soils, After A. Casagrande

#	General Description	Grain Shape	Loose ϕ'°	Dense ϕ'°
1	Ottawa standard sand	Well rounded	28	35
2	Sand	Rounded	31	37
3	Beach sand	Subangular	35	46
4	Silty sand	Subrounded	33	37
5	Silty sand	Subangular	36	40
6	Fill-sand	Angular	38	47
7	Well graded gravel with sand	Subrounded	42	57
8	Well graded, crushed rock	Angular	-	60

For dense specimens, ϕ' based on small normal pressures.

Empirical charts have been developed over the years, of the "angle of repose" (see Figure 18 and Table 2.8). This is defined as the slope inclination achieved by a soil dumped in a loose condition. This represents the 'angle of internal friction' (ϕ') of a cohesionless soil in a loose or disturbed condition. The angle of repose tends to be less for submerged slopes than non-submerged slopes.

TABLE 2.8 - Angles of Repose, Submerged and Non-submerged

SOIL TYPE	ANGLES OF REPOSE, H:V			
	non-submerged		submerged	
Sand, clean	1½ on 1	34°	3 on 1	18½°
Gravel, clean	11/3 on 1	37°	2 on 1	26½°
Hard rock, rip rap	1 on 1	45°	1 on 1	45°

Fine-grained soils (clays and clayey silts) have cohesive strength (c') as well as a frictional strength. The "drained" properties of cohesive soils can be determined in the laboratory by "triaxial tests" (see Appendix) on relatively undisturbed samples obtained by thin-walled Shelby tubes (75 to 100 mm diameter).

Approximate estimates of 'undrained shear strength' (c_u) can be made on the basis of Standard Penetration Tests ('N' Values) as shown on Table 2.9.

TABLE 2.9 - Consistency / Shear Strength, Cohesive Soil

Consistency Term	**Approximate Undrained Shear Strength, c_u** kPa	psf	**'N' Value**	**Field Description**
Very Soft	< 12	< 250	< 2	easily penetrated several cm by fist
Soft	12-25	250-500	2 - 4	easily penetrated several cm by thumb
Firm	25-50	500-1000	4 - 8	moderate effort to penetrate by thumb
Stiff	50-100	1000-2000	8 - 16	readily indented by thumb, great effort
Very Stiff	100-200	2000-4000	16 - 32	readily indented by thumbnail
Hard	> 200	> 4000	> 32	very difficult to indent with thumbnail

It should be noted that the drained cohesive resistance (c') of fine-grained soils is always less than the undrained cohesive resistance (c_u). The cohesive resistance (shear strength) of 'heavily consolidated' clays (very stiff to hard consistency) can be affected by weathering and shrinkage. This may cause cracks or fissures which may weaken the soil mass.

For fine-grained soils, the soil strength properties are governed, to a very large degree, by the soil composition (grain size distribution) and by the amount of consolidation (relative density or consistency) the soil mass has previously experienced. The following Table 2.10 presents a comparison of typical soil shear strength parameters (ϕ' and c') for fine-grained soils.

TABLE 2.10 - Approximate Soil Strength Parameters, After Terzaghi and Peck

Soil Type	ϕ'	c'
1. Silt Soil	30° - 32°	Often close to 0. Determine by test.
2. Clay Soil		
a. Very Stiff ('N' > 16)	28° - 32°	30 - 60 kPa
b. Stiff ('N' = 8–16)	28° - 32°	15 - 30 kPa
c. Medium ('N' = 4–8)	28° - 32°	5 - 15 kPa
d. Soft ('N' = 2–4)	26° - 32°	2 - 5 kPa
e. Very soft ('N' < 2)	26° - 32°	Determine by test.

2.10 Bedrock (Shale, Limestone)

Generally there are few stability problems with bedrock slopes in natural environments due to the high relative strength in comparison to soil. Shales however can be relatively weak in comparison to other rock types (limestones, granites) and can weather or deteriorate to a degree where their strength approaches that of strong soils.

The properties of shale can vary greatly depending on its durability. Long-term stable slope angles of 8° to near vertical have been observed. Most natural slopes of shale have weathered to a stable inclination (usually not steep unless interbedded with more resistant rocks) and stability becomes an issue only for excavations or highway cuts. For highway cuts in shale slopes, it is quite common to see design slope inclinations of about 1 to 1 (horiz. to vert.) to about 1.5 to 1 (horiz. to vert.).

3. EROSION AND SLOPE INSTABILITY PROCESSES

Slope movement or instability can occur in many ways (see Figure 19) but is generally the result of;

- changes in slope configuration, such as steepness or inclination
- increases in loading on a slope, such as structures or filling near the crest
- changes in drainage of the soil which create higher water levels or water pressures, such as heavy rainfall, blocked drainage, broken watermains etc.
- loss of vegetation.

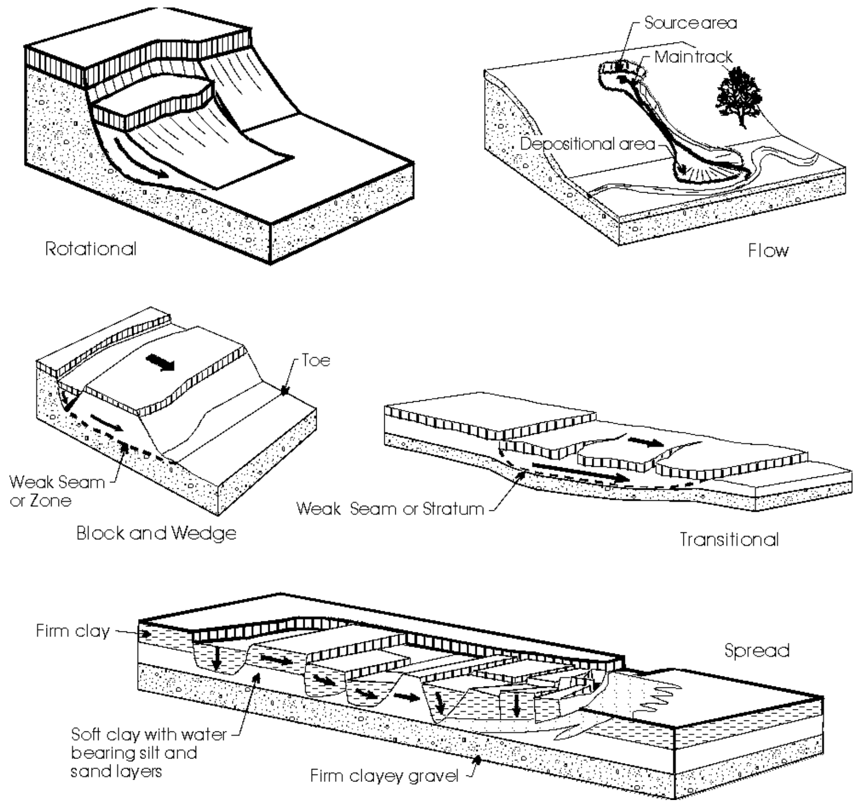


Figure 19 - Slope Movements

Slope instability and erosion are two different processes which are often associated together. Erosion is the loss of soil at the ground surface, while slope failures consist of a large mass of soil sliding along a planar surface. One very common event is for 'toe erosion' to trigger slope instability, due to steepening or undercutting of the slope. These are explained below, along with the natural forces which often contribute.

Erosion is a process of gradual washing away of soil by water movement or seepage (at the ground surface, see Figures 20 and 21), commonly occurring in one of the following manners;

a) rainfall or snowmelt and surface run-off (sheet or rill or gully erosion),

b) internal seepage (springs) and piping,

c) water flow (banks or base of river, creek, channel),

d) wave action (shorelines of ponds, lakes, bays).

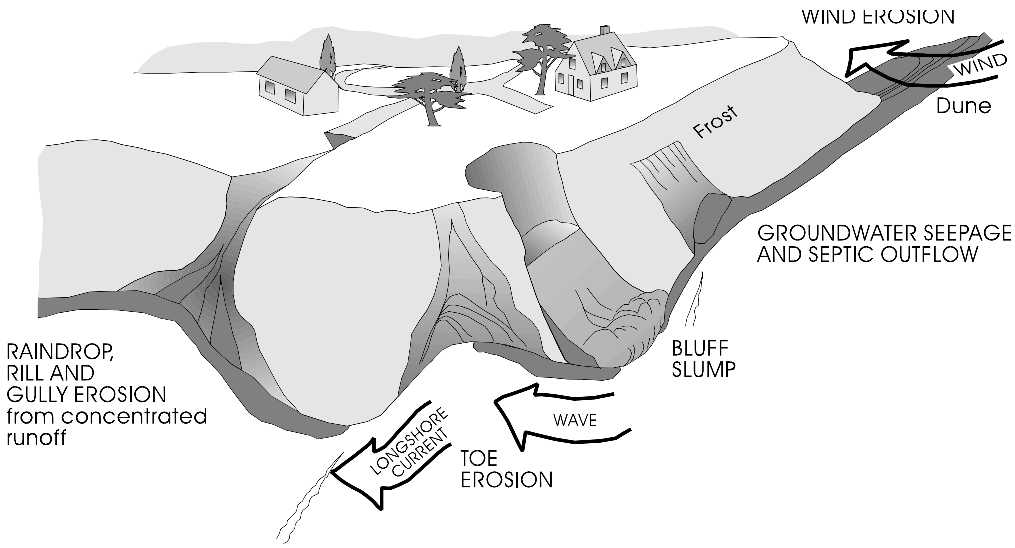


Figure 20 - Erosion Process

The erosion process affects the soil at the particle level, by dislodging and removing (transporting) the soil particles from the parent mass (with water movement as the agent, see Figure 21). Other processes such as wind and frost may assist in the weathering or dislodging and transport of soil particles.

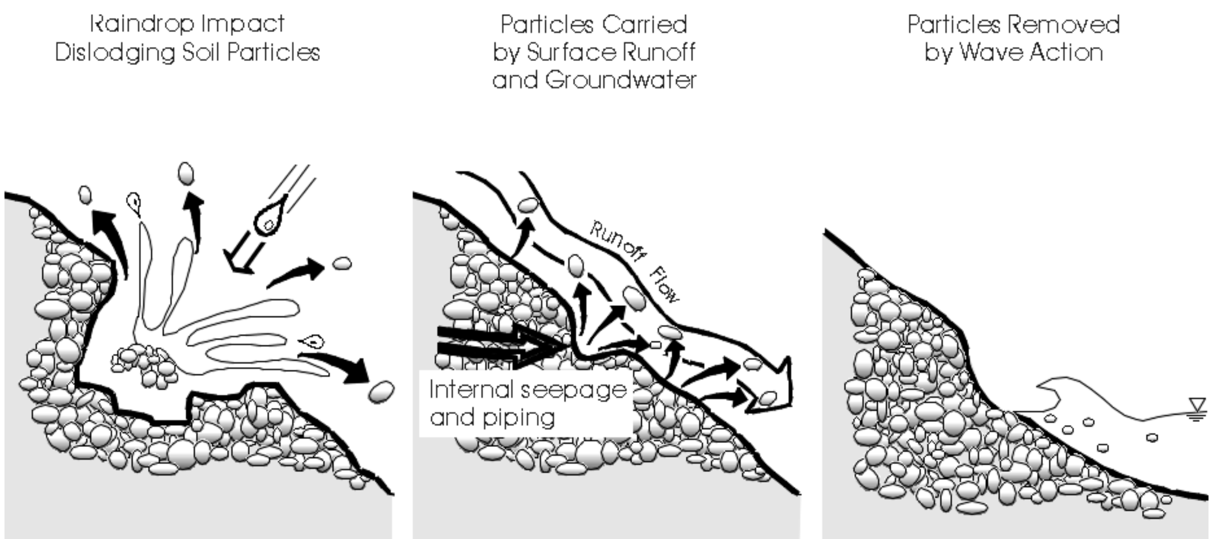


Figure 21 - Water Action on Soil

Water action and erosion (by flowing water or waves) are integral to slope stability. Slides may be caused by undercutting or steepening of the slope toe (removing support for the slope). Water seepage or groundwater levels can also affect slope stability since they affect the slope strength. 'Piping' on a slope face (see Figure 23) can be related to 'springs' or seepage, where soil erosion occurs in water bearing sands on slopes. As water drains from a sand layer on the slope face, the flow sometimes dislodges and transports (erodes) the sand particles away from the parent soil mass, leaving voids termed 'pipes'. The most susceptible location for piping to occur is near the bottom of a sand layer where the underlying soil is much less permeable than the sand (silt, clay, till, rock).

Figure 22 -Photo, Piping Erosion

Initial formation of valley and coastal slopes takes place through cycles of water erosion, followed by stabilization and re-vegetation. Stabilization occurs when the slope reaches a stable angle, and soil movement stops. Vegetation then becomes established on the stable slope mass, which provides protection against surface erosion. Sloping surfaces are prone to increased erosion due to the increased flow velocities and to the increased concentration of flow quantity, or duration.

Environmental influences (climate and heavy rainfall) may interrupt stabilization by causing new erosion that can trigger or re-initiate slope movements (see Figure 24). Studies have found that along river valley slopes, low intensity but long duration storms seem to produce more slope failures related to toe erosion (i.e. water flow along toe). Comparatively along lake shoreline slopes, sustained storms or high lake levels seem to produce more slope failures influenced by toe erosion (i.e. wave attack on toe).

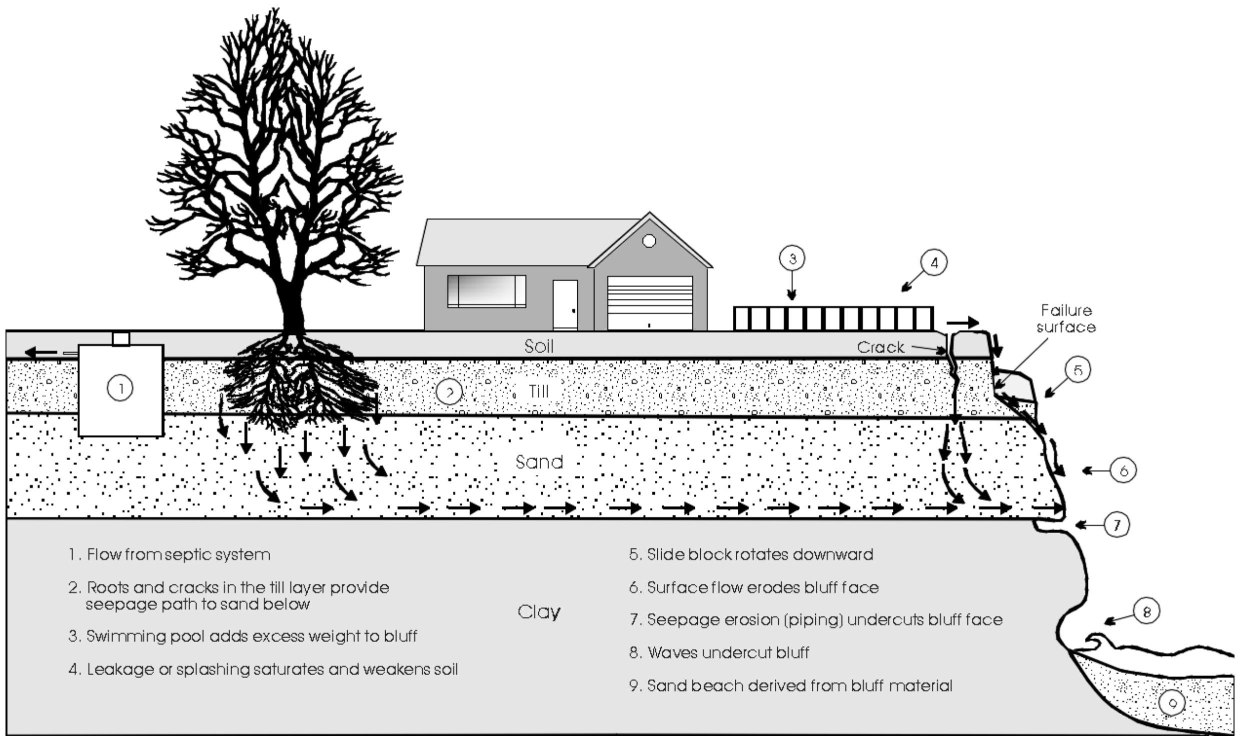


Figure 23 - Toe Erosion by Waves and Water Flow

3.1 Gully Erosion

Gully development is common on high bluff shorelines along the Great Lakes, and along river valleys where surface drainage may become concentrated and where erosion goes unchecked (see Figures 25 and 26). The process begins with the accumulation or concentration of surface run-off in narrow channels, which then experience progressive erosion and the formation of larger channels or gullies. The gully erosion process is attributed to 2 actions;

a) downcutting of the gully base by swiftly flowing water,

b) slumping or failure of the gully banks (this causes the gully to become wider).

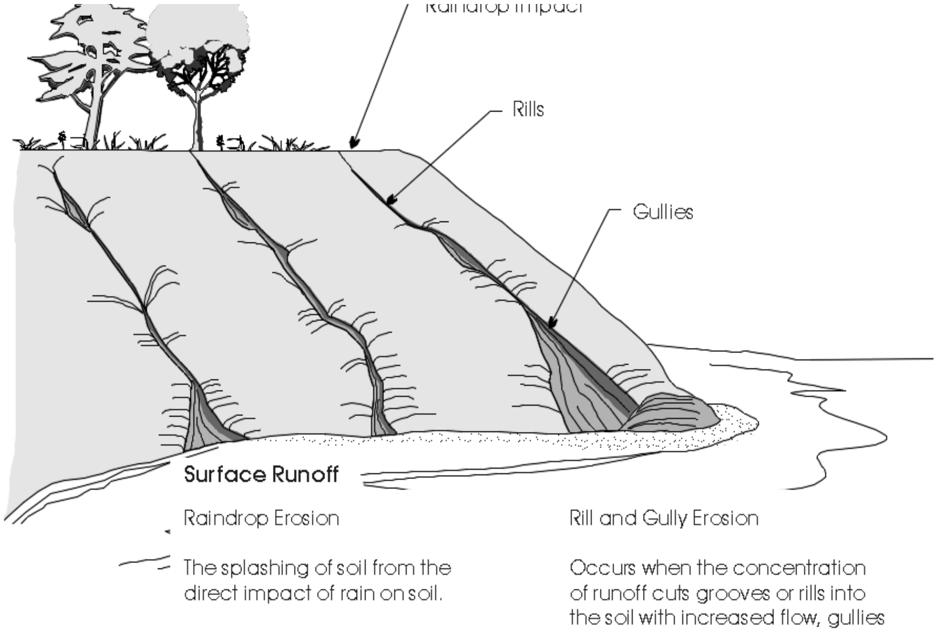


Figure 24 - Gully Erosion

Figure 25 - Photo, Rill Erosion

Gully development typically starts at the slope toe and progresses up the slope face to the slope crest and into the table land (see Figure 26). It also can be initiated inland by natural drainage processes or by man-made drainage features such as storm sewer outfalls, ditches, farm field tiles, and the like.

Figure 26 - Photo, Gully Erosion

The typical gully erosion process is summarized as follows;

1) sufficient run-off drainage to disrupt natural vegetation cover,
2) establishment of a drainage channel and start of downcutting,
3) channel banks steepen by continuing base erosion, until slope failure
4) gully widens with slope slides, and debris interrupts downcutting,
5) cycle of downcutting and slumping is repeated after debris is washed away and downcutting resumes,
6) gully can mature once stable gradient is achieved by drainage flows.

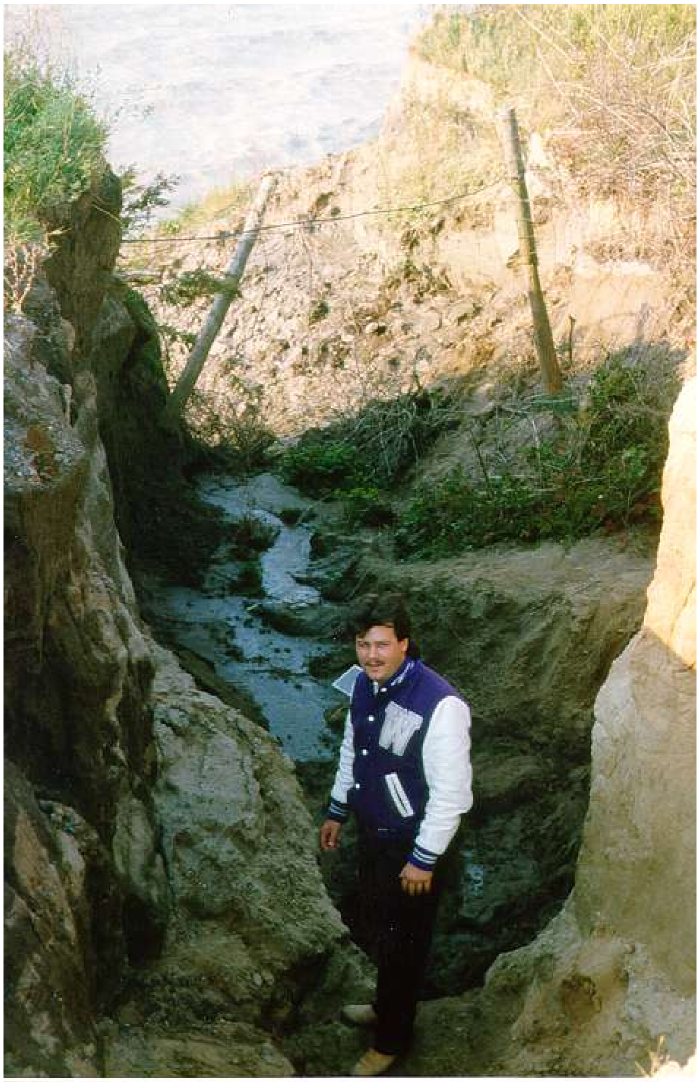


Figure 27 - Photo, Gully Erosion

Erosion of the gully base followed by slumping of the side-slopes, will result in the gully slope crest receding and the loss of table land (see Figure 28). The erodibility is influenced predominantly by the nature of the soil, and by the slope gradient (steepness). Strongly bonded 'cohesive' soils (clays, clayey silts, tills) are generally less erodible than 'cohesionless' soils (sands, silts).

3.2 Shoreline Bluffs

Shoreline bluffs subject to wave action at the slope toe, commonly experience cycles of erosion and slope instability (see Figure 28), leading to crest recession (loss of table land). Erosion may start when lake levels rise and cover previous beach areas along the bluff toe. This allows wave action to undercut and locally over-steepen the slope toe. Similar to gully and river erosion, this toe undercutting initially triggers the loss of vegetation cover near the slope toe, which progressively spreads up the slope face.

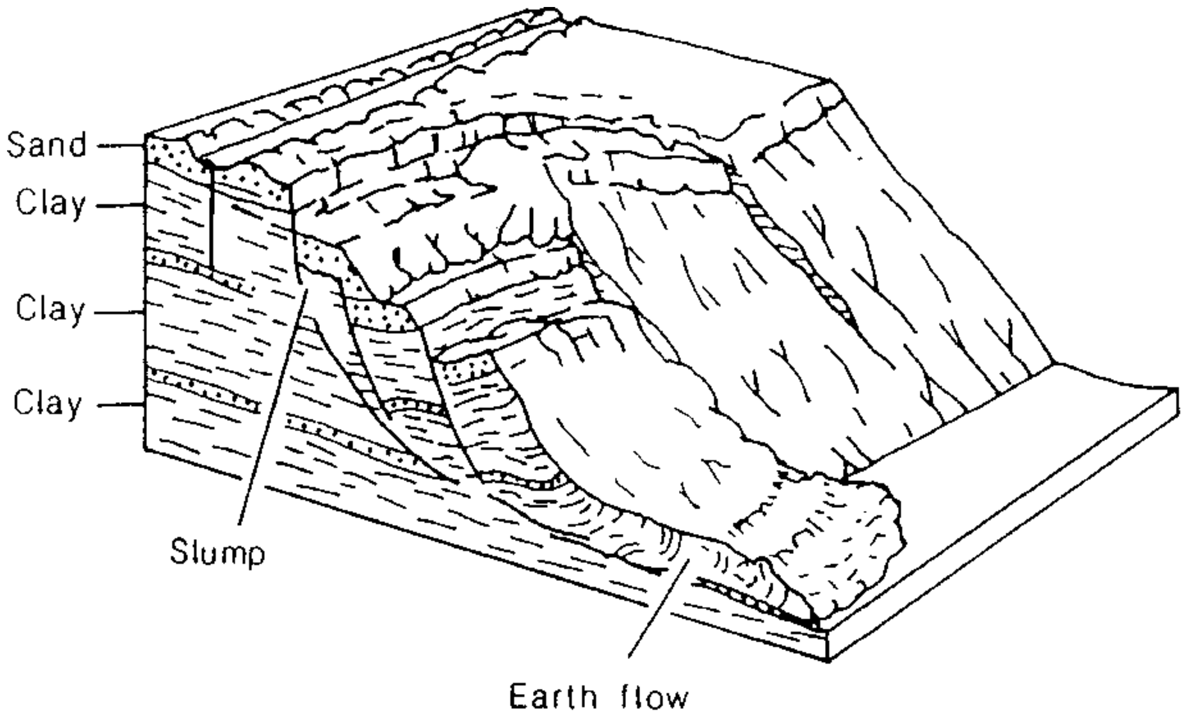


Figure 28 - Slope Failure Mode

Once over-steepened, the bluffs face will experience slumping or sloughing to attain a flatter and more stable slope inclination. The slumping can start locally and extend back to the crest. The most important initial step in stabilization of bluff erosion is to ensure that the slope toe is protected from wave action (where possible), prior to undertaking slope works. Any shore protection works should consider possible effects on the littoral system and sediment transport.

3.3 River Bank Erosion

Flowing water in rivers, creeks, and streams can cause surface erosion of the bank or channel. This erosion can affect the toe of a larger slope thereby causing steepening (undercutting) and likely slope instability (see Figure 29). The erosion is usually due to increased flow velocities from climatic events such as heavy rains or snowmelt. Locations which are particularly susceptible to river bank erosion, are where the river changes flow direction such as the outside of 'meanders' or bends in the river alignment.

Figure 29 - River Bank Erosion

The most important initial step in stabilization of river erosion is to ensure that the slope toe is suitably protected from the water flow velocity, prior to undertaking slope works.

3.4 Slope Failure or Instability

Through prolonged natural weathering, most slopes tend to achieve a stable inclination and vegetation cover. Changes or disturbances to the slope conditions can result in slope slides whereby the slope is attempting to assume a more stable and flatter inclination (see Figure 30). Slope failure or instability involves the sudden movement or sliding of a large mass of soil over a failure plane (also called slip plane). Slope movements or failures tend to occur rapidly, when compared to erosion processes. The movement often leaves a 'scarp' at the top of the slope and slumped ground below.

The principle driving force in slope instability, is gravity. Therefore, the slope inclination or steepness, has the greatest effect on stability. Steep slopes are most susceptible or vulnerable to failure, if there are minor changes in the other variables (loading, undercutting, wet weather). Flatter slopes tend to be affected less by changes in these other variables.

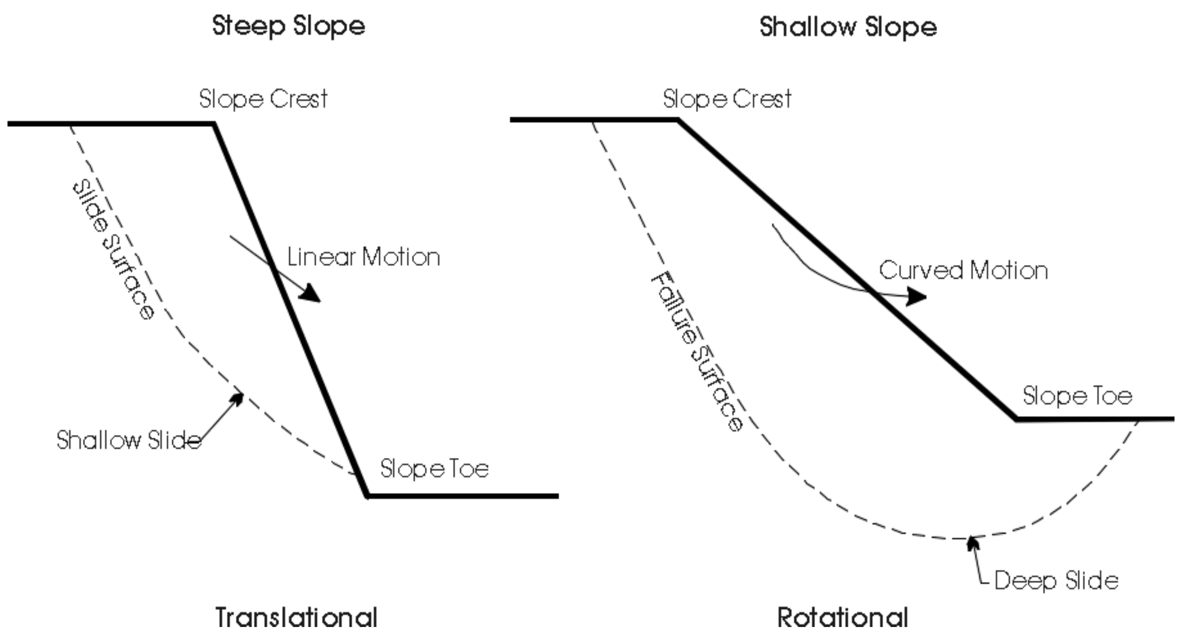


Figure 30 - Slope Steepness and Failures

Decreases in soil strength caused by increases in groundwater levels, weathering, shocks, and vibrations can also trigger instability.

Table 3.1 provides a summary of factors leading to slope instability;

TABLE 3.1 - Factors Contributing to Slope Instability

	Natural	**Man-made**
1. Increased Slope Steepness or Inclination	toe erosion by water	excavation, retaining walls
2. Additional Loading (weight, load)	trees, snow & ice	filling, structures, pools
3. Reduced Soil Strength	increased groundwater levels, flooding, drying, frost	changes in drainage, leakage of buried pipes/tanks/pools,

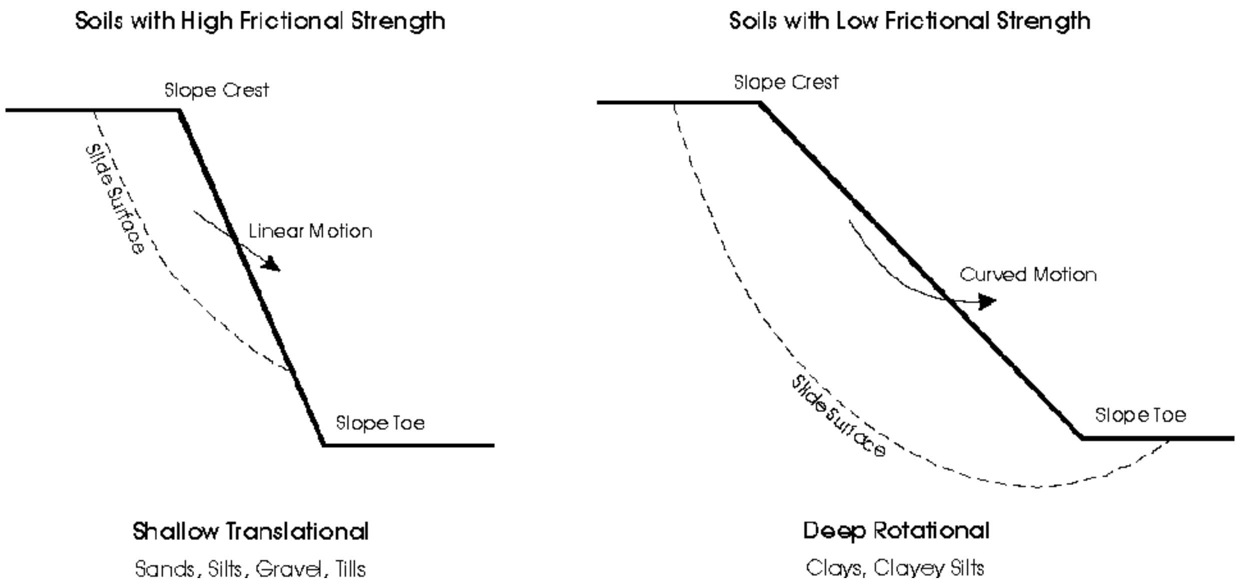


Figure 31 - Failures and Soil Types

Slopes composed of granular (cohesionless) soils or competent cohesive soils (i.e. soils with a high friction angle), usually experience shallow slides. Failures through cohesive soils (such as clays, with a low friction angle), tend to be deep slides involving much larger masses of soil (see Figure 32).

With the activities of urbanization and land development, fill placement near slope crests and excavations into slopes (or retaining walls) may alter the stability of shorelines, valleys, and sloping ground. Filling is a common practice in most urban areas as people try to reclaim more usable flat tableland along existing slope crests. Fill placement often occurs in an uncontrolled manner (sometimes over an extended period of time) and may result in an unstable fill mass which eventually experiences movements. Slides within fill materials (placed randomly and not engineered) can be quite unpredictable and extensive. The resulting instability may occur through the fill materials only or, through both fill and underlying native soil (see Figure 32).

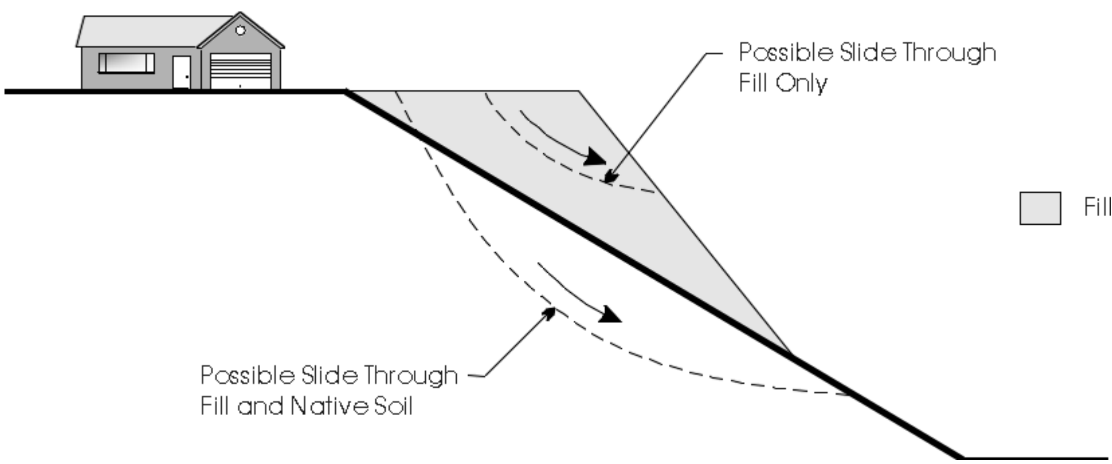


Figure 32 - Filling On Slopes

Filling on slopes can be carried out in a safe and stable manner with suitable control and precautions, and under the responsibility of a qualified geotechnical engineer.

3.5 Vegetation

A vegetation cover on a slope is the primary defence against soil erosion and is very important to long term erosion protection. As indicated on Figures 33 to 37, vegetation protects against surface erosion and shallow translational slope slides by;

a) by holding, binding, or reinforcing the soil with a root system,

b) removing water from the soil by uptake and transpiration,

c) reducing run-off flow velocity,

d) by reducing frost penetration,

e) by the buttressing or reinforcing action of large tree roots

Figure 33 - Photo, Tree Roots

By reducing surface erosion, the likelihood of shallow instability is also decreased.

Figure 34 - Photo, Surface Erosion

Vegetation also improves the visual aesthetics of a slope (shoreline, valley, or gravel pit) and is a vital part of the ecosystem.

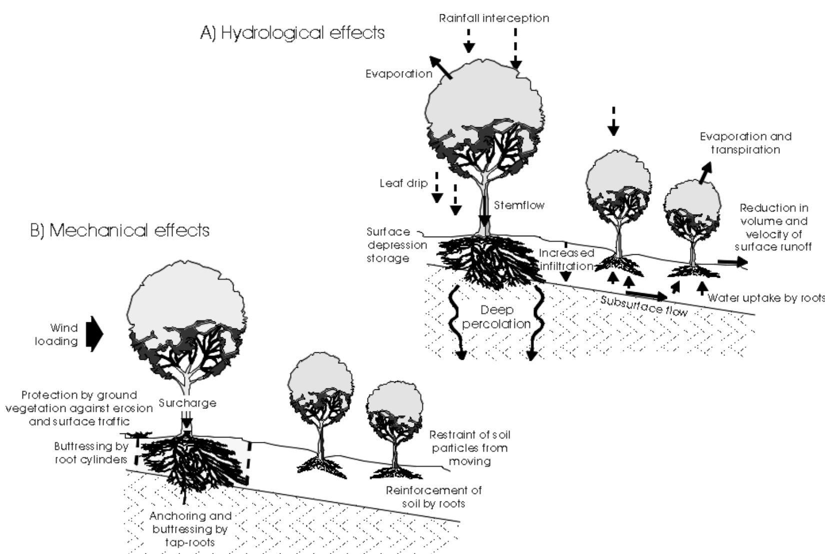


Figure 35 - Effects of Vegetation

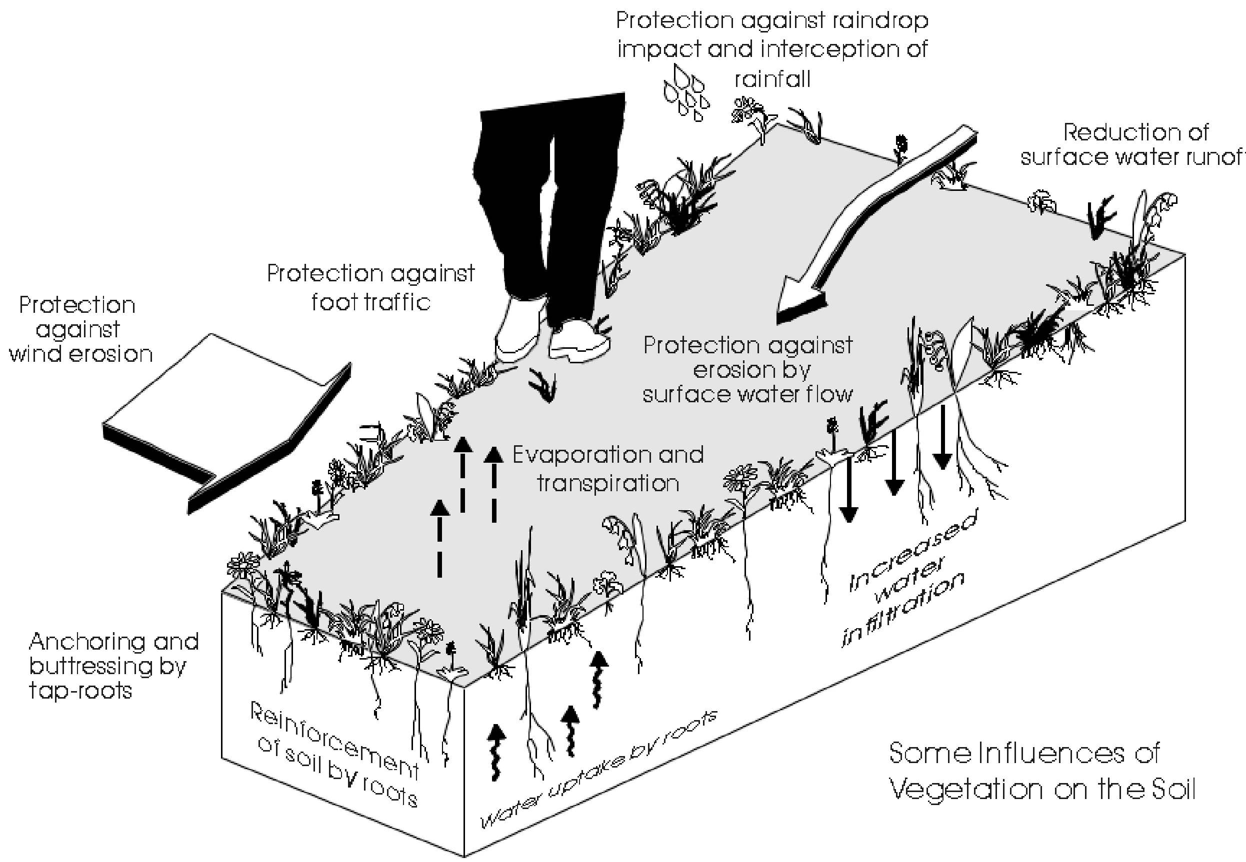


Figure 36 - Effects of Vegetation

4. CLASSIFICATION OF SLOPE SLIDES IN ONTARIO

There are many descriptive terms for slope slides, partly due to the many types of movements that are possible, and the many disciplines involved in studying slope movements including geology, geomorphology, and engineering. Enclosed is Table 4.1 prepared by Varnes (1958) showing a classification system for the various types of slope movements. This chart is summarized as follows;

TABLE 4.1 - Slope Slide Classification

TYPE OF MOVEMENT		TYPE OF MATERIAL		
		BEDROCK	ENGINEERING SOILS	
			COARSE	FINE
FALLS		Rock Fall	Debris Fall	Earth Fall
TOPPLES		Rock Topple	Debris Topple	Earth Topple
SLIDES	ROTATIONAL	Rock Slump	Debris Slump	Earth Slump
	TRANSLATIONAL	Rock Block Slide Rock Slide	Debris Block Slide Debris Slide	Earth Block Slide Earth Slide
LATERAL SPREADS		Rock Spread	Debris Spread	Earth Spread
FLOWS		Rock Flow (deep creep)	Debris Flow (soil	Earth Flow creep)
COMPLEX		Combination of two or more principal types of movement		
			Sands, Gravels, Tills	Clays, Clayey Silts,

One simple slope failure classification criteria originated by Skempton (1953) is the "D/L Ratio", termed the maximum thickness of the slide divided by the length of the downslope (see Figure 37 and Table 4.2); as follows,

TABLE 4.2 - D/L Ratio

Landslide Type	D/L Ratio (percent)
Flows	0.5 to 3 %
Translational slides	5 to 10 %
Rotational slides	15 to 30 %

(Crozier, 1986)

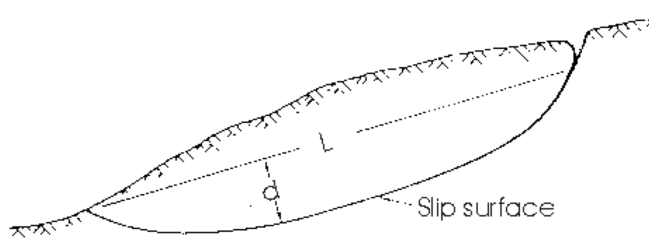


Figure 37 - D/L Ratio

Following is an outline of the slope movements mostly likely to occur in natural soil slopes in Ontario.

4.1 Slope Features and Description

In referring to physical features of a slope (see Figure 38), the top of a slope is considered to be the "crest" or "top of bank" and the bottom the "toe". The inclined slope surface is called the "face". In a slope slide, a large soil mass generally slides downward and outward over a discrete surface (failure plane), under the force of gravity. The soil mass may remain intact and movement occurs mostly along this "failure plane" (often curved).

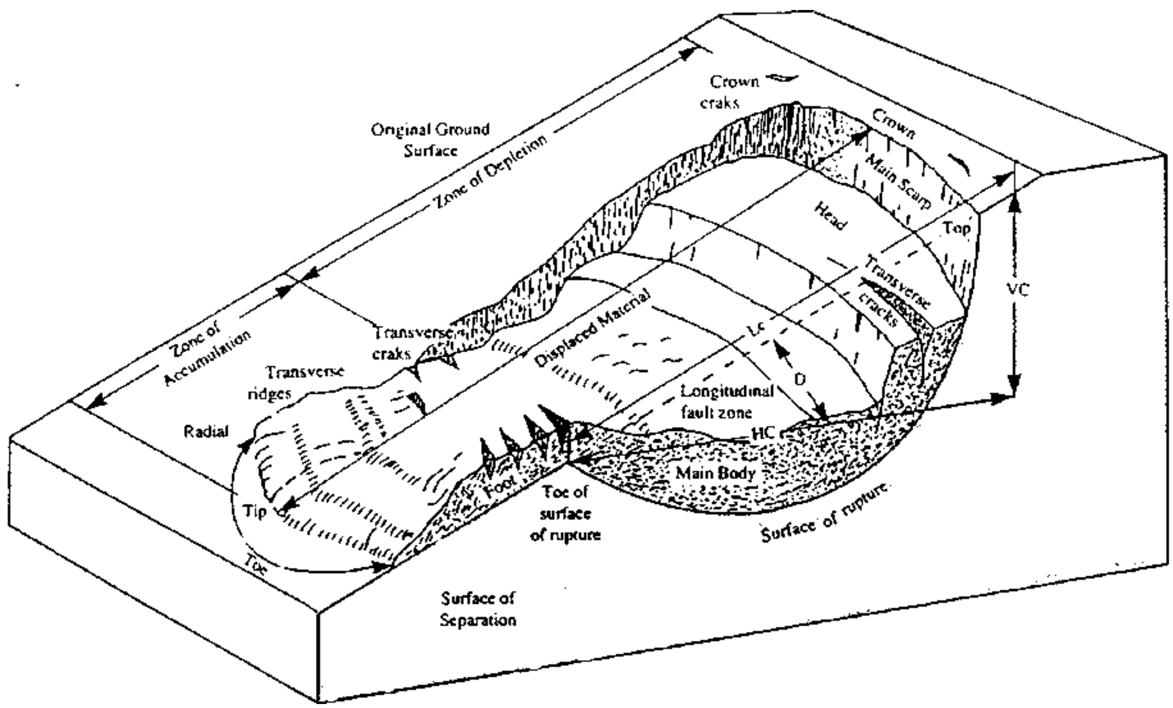


Figure 38 - Slope Anatomy

4.2 Flows

"Flows" in soil slopes occur when the soil is of low strength and behaves as a fluid (see Figures 39 and 41). Often flow slides are assisted by water saturation. Flow slides involve movement with greater internal soil deformation than a "slide" and movement is not limited to a discrete failure plane. Flows slides may occur in cohesive soils of low strength (such as Leda Clays).

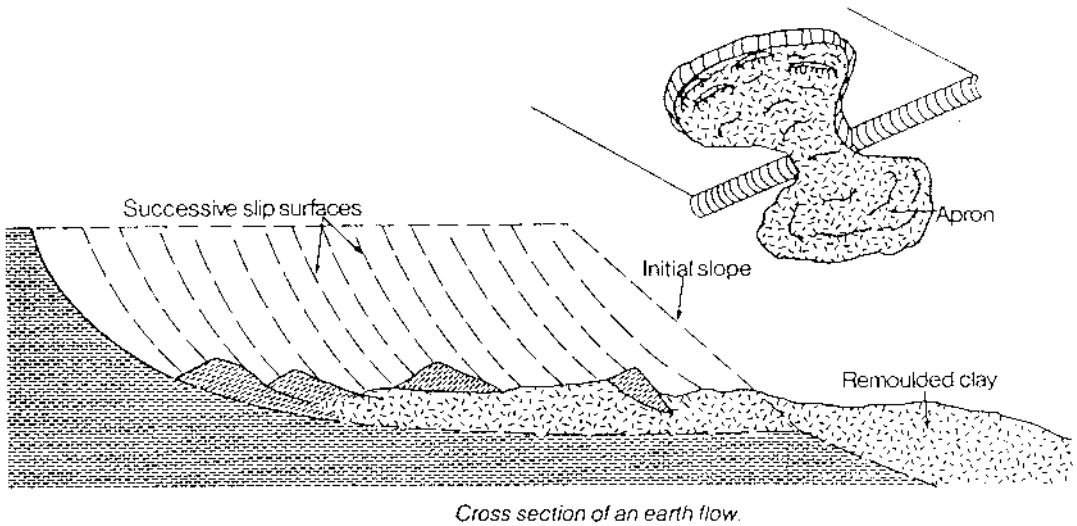


Figure 39 - Flow Slides

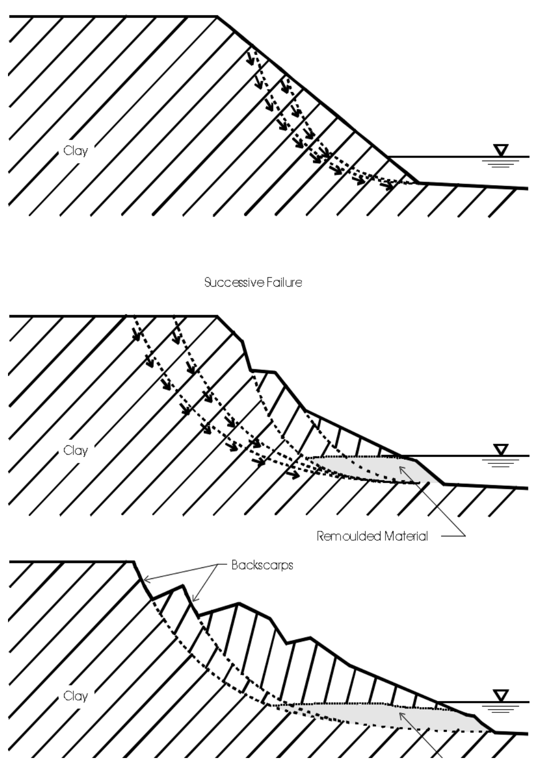
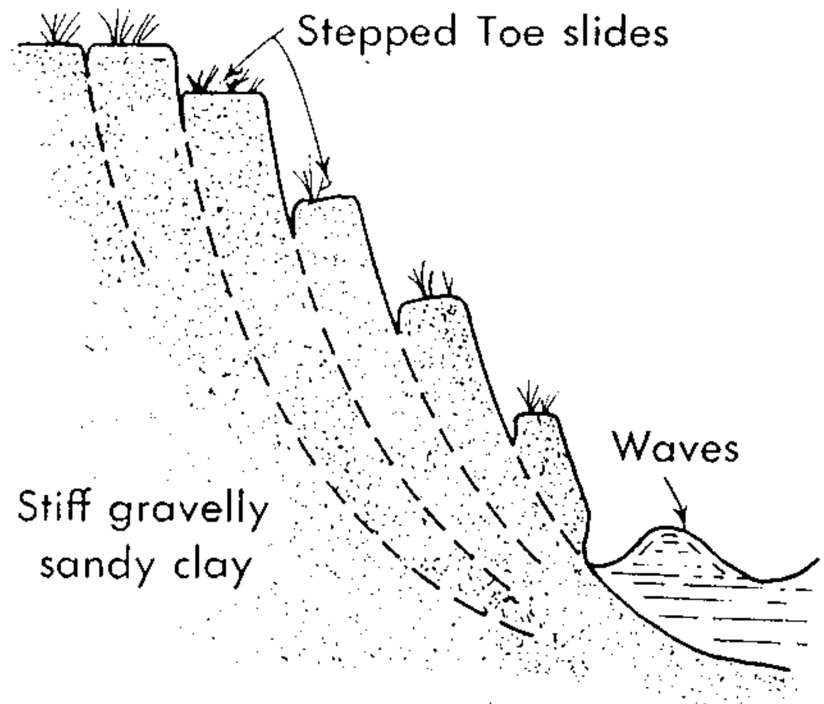


Figure 40 - Slope Failure Mechanism

Flow slides may also occur in fine sand or silt soils, where there is heavy seepage or high pore water pressures. This may happen as the result of sudden soil saturation caused by flooding, watermain leaks, heavy rain, and the like. It may also occur during earthquake or shock loadings, which may cause high pore water pressures. This type of failure is generally limited to soft clays, fine sands and silts, or loose earth fill.

4.3 Progressive Failures (Quick Clays / Leda Clays)

The "Leda Clays" common to the Ottawa Valley area are also termed "quick clays" because of the rapid loss of strength which results when they are disturbed. This loss of strength is related to a flocculated structure (similar to a house of cards). The structure collapses upon disturbance and causes a liquid consistency. The disturbance might be due to minor soil movement or changes in slope inclination or loading.



a. Toe slides, retrogressing and forming steps in steep cliff undercut by waves

Figure 41 - Retrogressive Rotational Failures

The strength loss may lead to progressive failures. Progressive failures are also called "retrogressive rotational failures" (see Figure 41 and 42) and are typically initiated by a small slip which causes subsequent successive segments of slope to continue to fail (can also generate "flow slides"). This type of movement may also occur in other soft or loose soils, including fill materials.

Figure 42 - Photo, Retrogressive Slides

Detailed information on slope slides in Leda Clays is available from,

a) Hazardous Sites Technical Guide
Ministry of Natural Resources, Version 1, December 1996

b) 'Earthflow Terrain Evaluation in Ontario'
Ministry of Transportation & Communications
Research & Development Division, RR 213, January 1978

c) 'Slope Stability Study of the South Nation River and Ottawa River'
Ministry of Natural Resources, OGS MP 112, 1983

4.4 Translational Slides

"Translational slides" tend to be shallow sheet-like slides (see Figure 43), generally exhibiting a linear failure plane. They occur mostly in cohesionless soils (sands). Vegetation root systems play an important role in reinforcing the shallow-depth soil to resist these slides. Translational slides are common on steep slopes composed of competent soil (ie. sandy soils and dense glacial tills with a high friction angle).

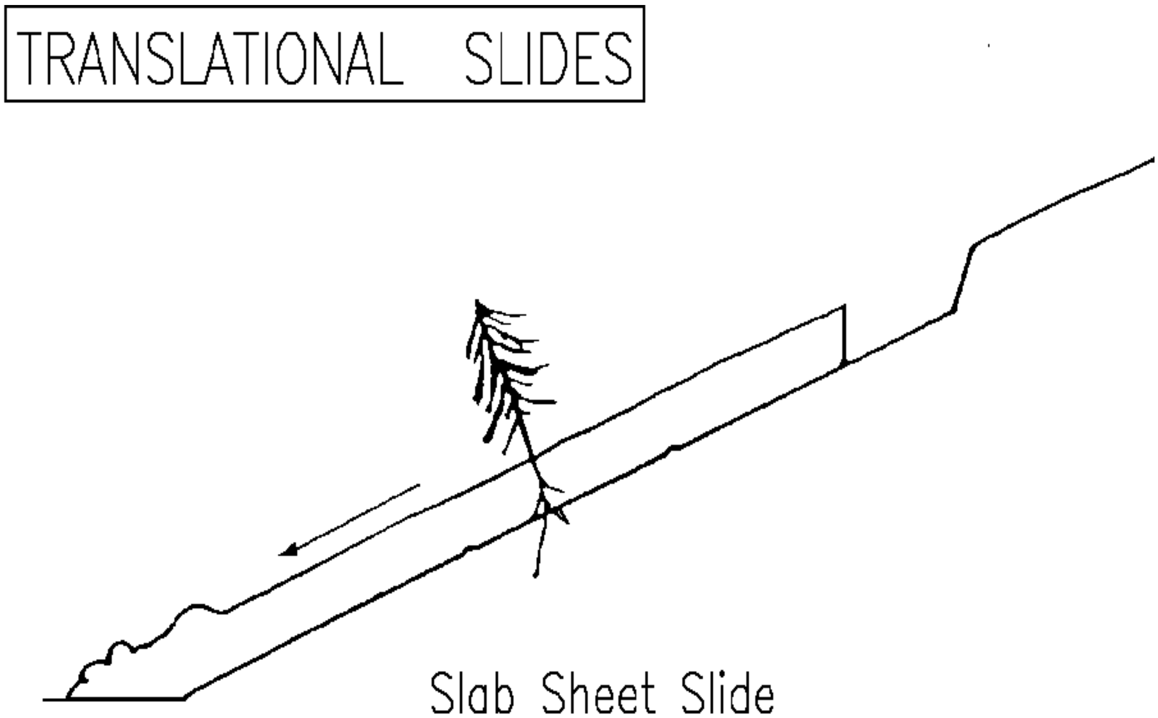


Figure 43 - Translational Failures

4.5 Rotational Slides

"Rotational slides" (see Figure 44) occur usually within homogeneous soils and have a curved failure plane. They tend to be deeper than translational slides and occur mostly weaker or softer soils such as clays, clayey silts, and earth fills.

"Base failures" usually occur within soft clays or slopes containing a soft seam (thin weak layer); "toe failures" generally take place on relatively steep slopes; and "face failures" are often the cause of stratigraphic boundaries or undercutting (see Figure 46).

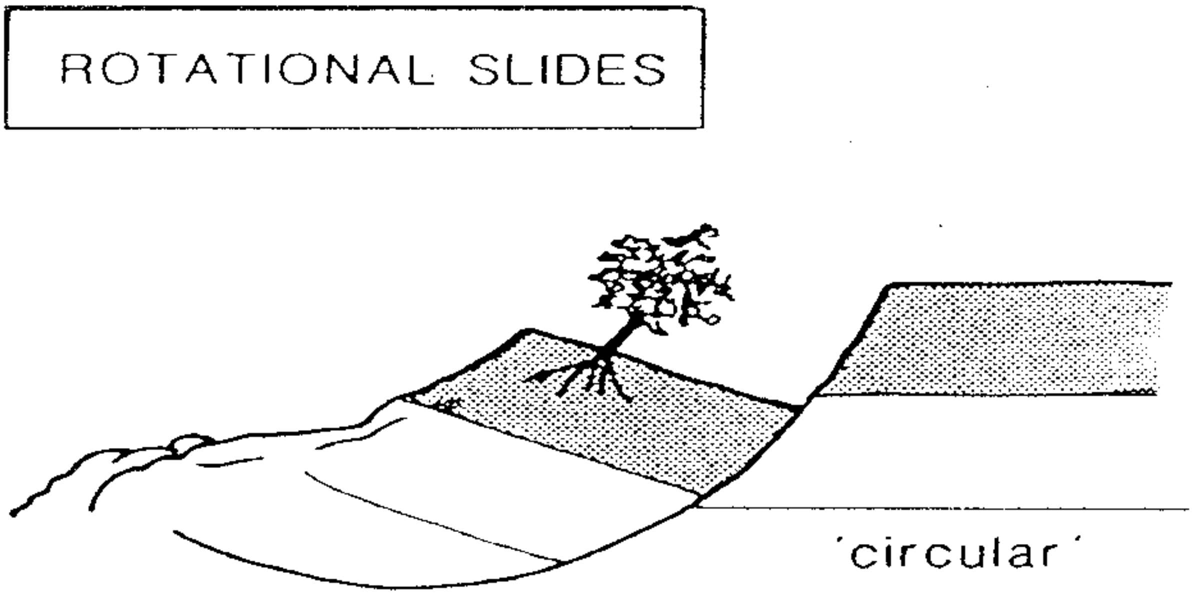


Figure 44 - Rotational Failures

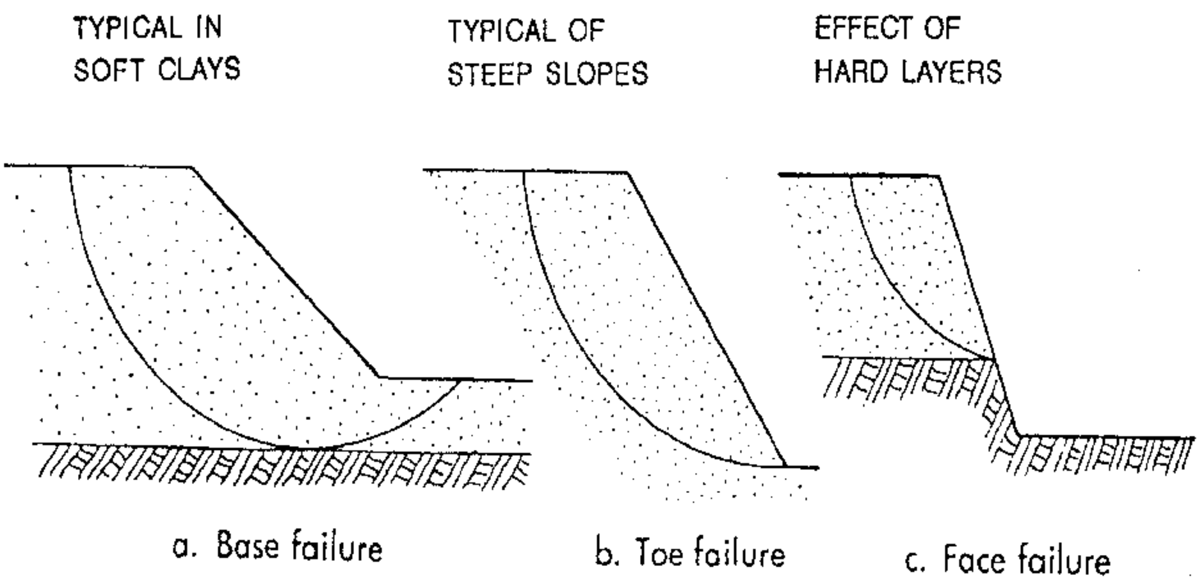


Figure 45 - Face Failures

4.6 Stratified Soil Slopes

The above sections have discussed the properties of individual soil types which influence slope stability. Many natural slopes consist of more than one soil type or layer, and the layers can be of varying thicknesses; many combinations are possible. The behaviour of the slope and its stability will therefore be dependent on the combined interaction of the soil layering or stratification, and the individual properties of each layer (see Figures 46 and 48).

In some cases, the soil strength throughout a slope may be relatively uniform in spite of stratification (layering). The controlling factors will be the combined soil resistance or strength contributed by the soil layers which are intersected by the potential failure surface or slip plane.

Figure 46 - Soil Stratification

In some slopes with many soil layers, the global stability may be governed by a single thin weak layer if it is critically located (see Figure 49).

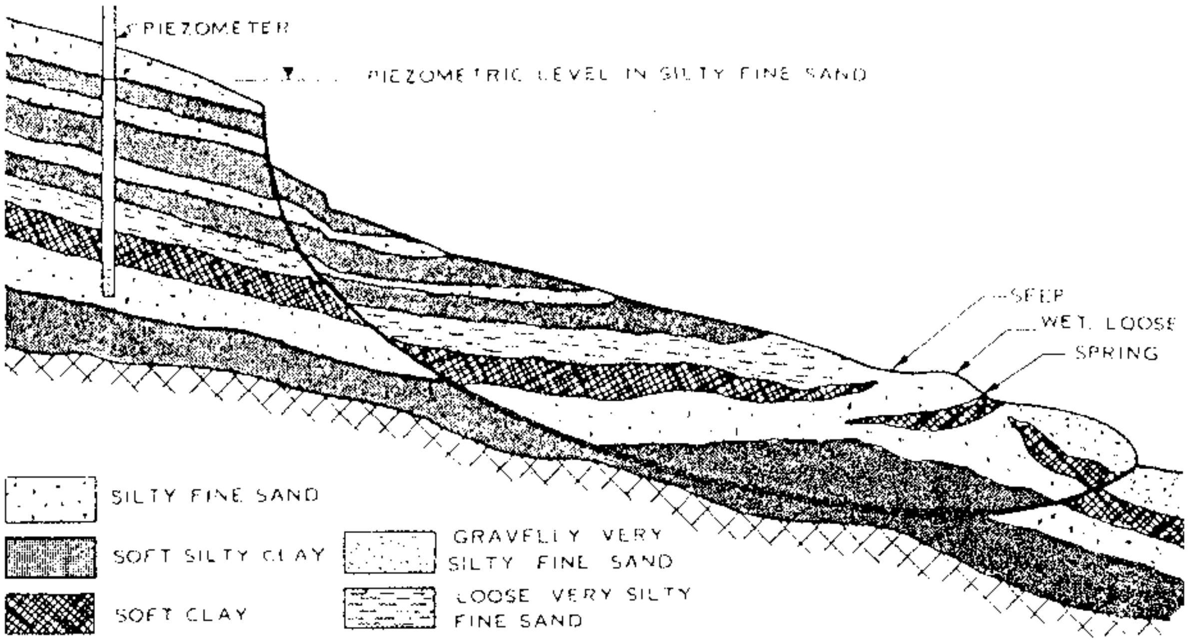
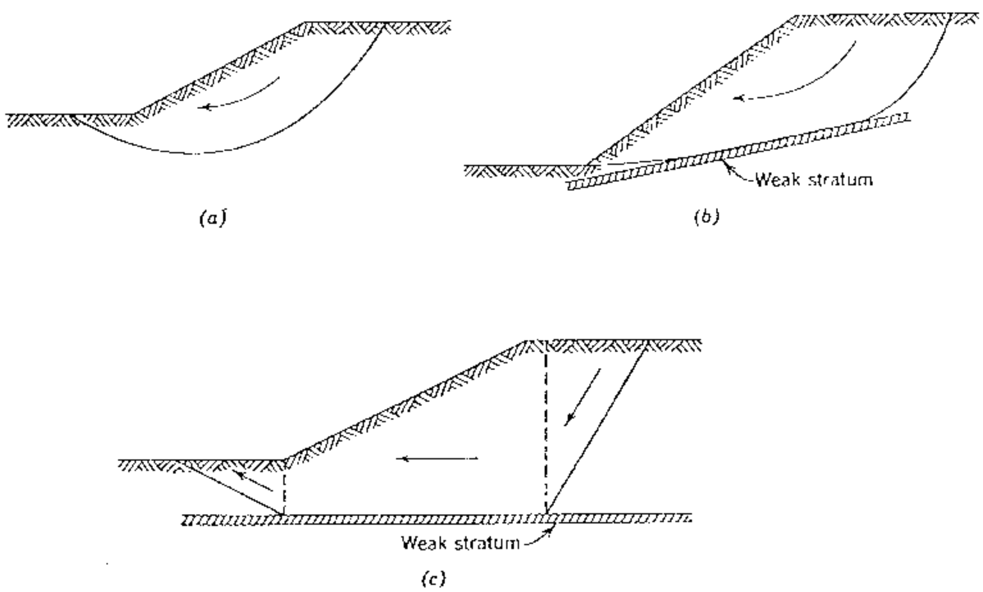


Figure 47 - Soil Layers in Slopes



Types of failure surfaces. (*a*) Circular failure surface. (*b*) Noncircular failure surface. (*c*) Sliding block failure.

Figure 48 - Thin Weak Layers

4.7 Indicators of Instability

There are several general indicators of slope stability, including slope inclination, soil types, groundwater levels, and other slope features such as tension cracks.

Immediately before a translational or rotational slide occurs, "tension cracks" may develop parallel and close to the slope crest. The slope surface after a slide (see Figure 49), often displays these "tension cracks" above the slide and, a distinct "scarp" at the "head" or "crown" where the sliding mass has separated from the slope. A bulging soil mass is often found at the "toe" of the slide.

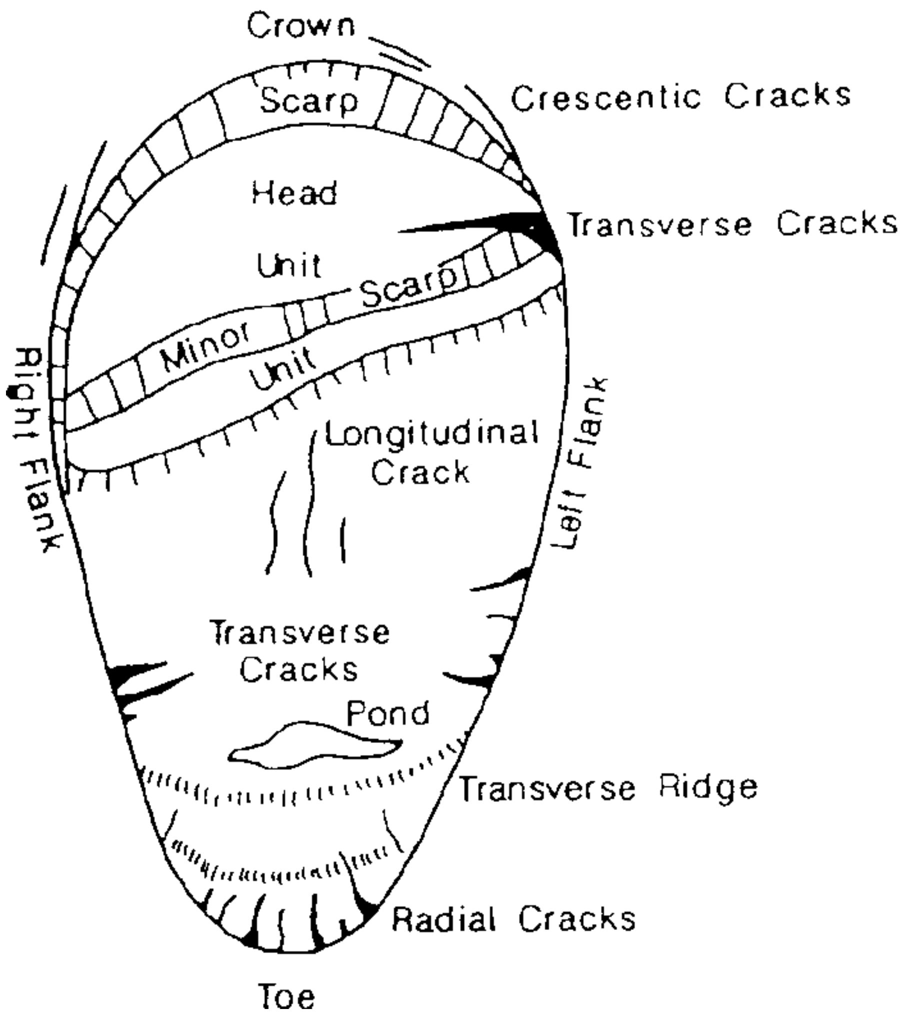


Figure 49 - Scarps, Slumps

Slope failures tend to be self-stabilizing in that the slope configuration becomes flatter and more stable. This assumes that the slumped soil is not removed by toe erosion.

Evidence of past slope slides may include;

a) bare slope areas (no vegetation),
b) tree trunks which are bent or bowed, or dead trees whose roots may have been damaged by slope movements,
c) scarps and tension cracks,
d) irregular slope surface such as slumped soil masses, or humps, displaced posts, poles, monuments, guard rails (see Figure 52),
e) see list on Figure 51.

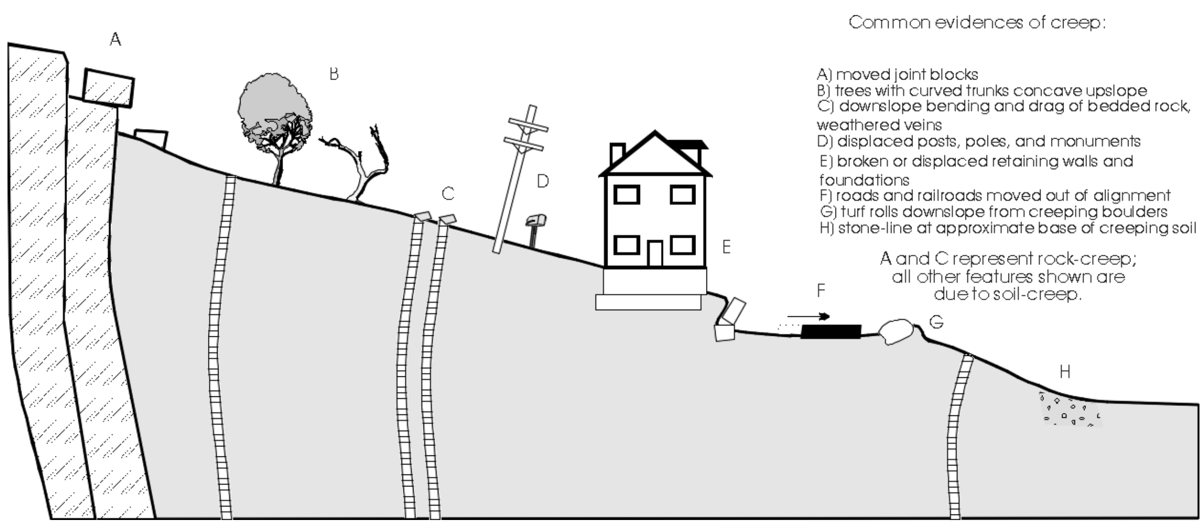


Figure 50 - Indicators of Instability

Figure 51 - Photo, Leaning Guard Rail

It is important to note that in some steep slope areas, bent or bowed tree trunks are not necessarily caused by slope movements (though they often do). The curved tree trunks may be due to initial root development and twisting or bowing growth in response to reaching for moving sunlight and adjacent tree canopies.

Examination of historical air photographs, surveys, and discussions with local residents can be important in detecting the extent or presence of past slides.

4.8 Stable Inclinations

Past studies have also investigated the concept of the "lower limits of slope inclinations" ("thresholds"); slope inclinations below which landslides are not likely to occur. For Ontario glacial till in the undisturbed state, the threshold limit has been previously documented as 20° or a slope inclination of 2¾ to 1 (horizontal to vertical). This limit suggests that if a natural till slope is at an inclination of 2¾ to 1 (horiz. to vert.) or flatter, then landslides are not likely to occur. The above guideline could also be used for fill slopes constructed with till soil, if it is constructed (placed in thin layers and heavily compacted) under the direction of an experienced and qualified geotechnical engineer.

Generally, natural vegetated slopes have been observed to be stable over the long-term as shown on Table 4.3, at the following inclinations,

TABLE 4.3 - Observed Stable Slope Inclinations

Natural Slope Material	**Steepest Stable Inclination Observed** (horiz. to vert.)
Shale, Limestone	near vertical to 1 : 1 90° to 45°
Glacial Tills	1½ : 1 to 2 : 1 34½° to 26½°
Cohesionless Sands, Gravels	1½ : 1 to 2 : 1 34½° to 26½°
Glaciolacustrine Clays and Silts Marine Clays (Leda Clays)	2 : 1 to 4 : 1 26½° to 14°

The natural soil slope inclinations on the above table are the result of long-term weathering of the slope surface and slope stability. Competent soils with high frictional resistance (sands, glacial tills), have relatively steep natural slope inclinations. The slope might stand at a steeper inclination except that natural surface erosion (run-off, frost, drying, wind) also acts on the slope. The surface erosion and weathering, alters the slope face by loosening and dislodging soil particles near the ground surface. The shallow loosened or disturbed zone is not as competent or consolidated as the underlying parent soil and tends to have a flatter natural inclination. The slope inclination gradually flattens through surface erosion until vegetation cover can develop and provide protection from continued erosion. An inclination of between 1½ to 1 (34½°) and 2 to 1 (26½°) is the steepest on which vegetation cover can establish or be maintained.

Weaker soils with low frictional resistance (glaciolacustrine clays, marine clays) typically have relatively flatter natural slope inclinations. This can be due to slope stability and not to surface erosion. Stable slope inclinations may be 2 to 1 or flatter and therefore surface erosion may be less and vegetation cover may not be an issue.

Slope inclination and soil type can be used to provide a general indication of slope stability. Many Conservation Authorities suggest a 3 to 1 inclination or flatter for slopes where there is little or no information on the subsurface conditions. In rehabilitation of pit slopes, a 3 to 1 is the steepest inclination recommended. Almost all slopes will be stable at this inclination, except for soft or sensitive soils (Leda Clays or earth fill), or areas of heavy groundwater seepage. In these cases (Leda Clays, earth fill, heavy seepage), the stable slope inclination may be as flat as 4 to 1 or 5 to 1.

5. HISTORY OF SLOPE STABILITY CONCEPTS AND ANALYSIS

This section describes the historical development of the analytical processes which form the basis of current modern analysis methods.

Slope stability concepts began with the construction of fortifications by French military engineers in the 1600's and 1700's and produced earth pressure theories by Culman and Coulomb. The French canal engineer Alexander Collin made some advances in slope stability analysis in the 1800's but it was not until 1916 that studies of the Gothenburg Harbour sea wall failure, evolved with the 'Swedish Slip Circle Method' or 'Method of Slices' (Petterson and Fellenius) which forms the basis of most popular methods.

Modern soil mechanics generally began with Terzaghi's work in Britain in the 1930's and 1940's and was followed by many developments in the 1950's and 1960's at Imperial College with Skempton, Bishop, Kenney, Morgenstern-Price, and Spencer; and at the Geotechnical Institutes of Norway and Sweden with Janbu. Sarma made further advances in the 1970's.

The more modern analytical methods are relatively complicated and the solutions are based on modelling or simulating the site conditions and then carrying out numerous repetitive iterations, with the solution converging. Many of the stability analysis methods have been adapted to computer applications, thereby allowing quick analysis, interactive graphics, and printing capability of results. The computer applications still require a accurate assessment of the site conditions and of the mode of failure in order to obtain the correct solution.

The following summary table, Figure 53 presents a brief list of several computer applications which have been developed for slope stability analysis. The different software have varying capabilities in the methods used to solve the problem, and the manner of inputting and outputting the information.

SUMMARY TABLE OF PC COMPUTER SOFTWARE

GEOTECHNICAL ENGINEERING SLOPE STABILITY ANALYSIS

SOFTWARE NAME	METHOD(S) OF ANALYSIS	SPECIAL FEATURES	RETAIL PRICE	INQUIRIES AND ORDERING	SYSTEM REQUIREMENTS
SLOPE/W	1. Fellenius 2. Bishop's Simplified 3. Janbu's Simplified 4. Spencer 5. Morgenstern-Price 6. U.S. Corps of Engineers 7. Lowe-Karaflath 8. GLE (General Limit Equilibrium)	• CAD-like screen and menus • Windows compatible • graphical input of geometry • labelling available • forces along slip surface • graphical output	$ 3,495 CDN includes 1 yr. support	GEO-SLOPE International Ltd. #830, 633-6th Ave. SW Calgary, Alberta Canada T2P 2Y5 Tel. (403) 269-2002 Fax (403) 266-4851	Intel 286 or higher 386 recommended Math Co-processor recommended but not required Windows 3.0 or higher Mouse min. 1 MB RAM, 4 MB RAM recommended EGA / VGA
GSLOPE	1. Bishop's Modified 2. Janbu's Simplified	• user friendly menus • graphical input of geometry • automatic search capability	$ 1,025 CDN $ 895 US includes 1 yr. support	Mitre Software Corporation #200, 9639-51st Ave. Edmonton, Alberta Canada T6E 6A5 Tel. (403) 434-4452 Fax (403) 437-7125	XT, 286 or higher Math Co-processor recommended but not required min. 512 K RAM DOS 2.0 or higher Herc./ CGA / EGA / VGA
CLARA 3D	1. Bishop's Simplified 2. Janbu's Simplified 3. Morgenstern-Price 4. Spencer	• 3D versions of Bishop' and Janbu's Simplified • user friendly menus • can output graphic images for Autocad files	$2,400 CDN includes 1 yr. support	O. Hungr Geotechnical Research Inc. 4195 Almondel Rd. West Vancouver, B.C. Canada V7V 3L6 Tel. (604) 926-9129	XT, 286 or higher Math Co-processor recommended but not required min. 640 K RAM Herc./ CGA / EGA / VGA
GALENA	1. Bishop's Simplified 2. Spencer-Wright 3. Sarma	• user friendly menus • 'back analysis' • can use 'constraints' to aid in search • text and graphical output	$ 795 US	BHP Engineering (Americas) Inc. 200 Fairbrook Dr., Suite 204 Herndon, Virginia U.S.A. 22070-5200 Tel. (703) 478-0975 Fax (703) 478-2673	XT, 286 or higher, 386 recommended Math Co-processor required min. 640 K RAM Herc./ CGA / EGA / VGA
XSTABL	1. Bishop's Modified 2. Janbu's Modified	• user friendly menus • automatic search capability • text and graphical output • can output CGM and HPGL graphic image files	$ 395 US	Interactive Software Designs Inc. 953 North Cleveland Moscow, Indiana U.S.A. 83843 Tel. (208) 885-6403	XT, 286 or higher Math Co-processor recommended but not required min. 512 K RAM DOS 2.1 or higher CGA / EGA / VGA
SLIDE	1. Bishop's Simplified	• user friendly menus • automatic search capability • graphical input of geometry • text and graphical output	available through MNR	Brent Corkum University of Toronto Dept. of Civil Engineering Toronto, Ontario M5S 1A4 Tel. (416) 978-4576	XT, 286 or higher Math Co-processor recommended but not required min. 512 K RAM Herc./ CGA / EGA / VGA

Figure 52 - Summary of Computer Software

6. METHODS OF SLOPE STABILITY ANALYSIS

The following section summarizes the common methods for engineering analysis of slope stability, describing their basis and assumptions, providing comparisons of their results, and commenting on advantages or disadvantages (or limitations).

Engineering analytical methods are generally used to determine slope stability. The analysis methods are based on a mathematical solution of the various forces acting on the slope. The various available methods can be summarized as follows;

a) limiting equilibrium (most commonly used methods),

b) deterministic methods other than limiting equilibrium (finite element etc.),

c) probabilistic methods,

d) stability charts.

a) Limiting Equilibrium Methods

The most common methods of slope stability analysis are based on "limiting equilibrium" (to be discussed in detail in following sections). These are 'deterministic' methods since they assume uniform or average soil properties. The methods use physics (statics) and free-body diagrams to model forces, pressures, and moments (see Figure 54). Hypothetical failure planes are considered through the slope and the sliding soil mass is analyzed by comparing gravitational forces and soil resistance or strength. These methods can be developed from simplistic solutions, or from complex solutions that model the subsurface conditions and loadings.

Limiting equilibrium methods examine an assumed distinct or defined failure plane through the slope. They compare

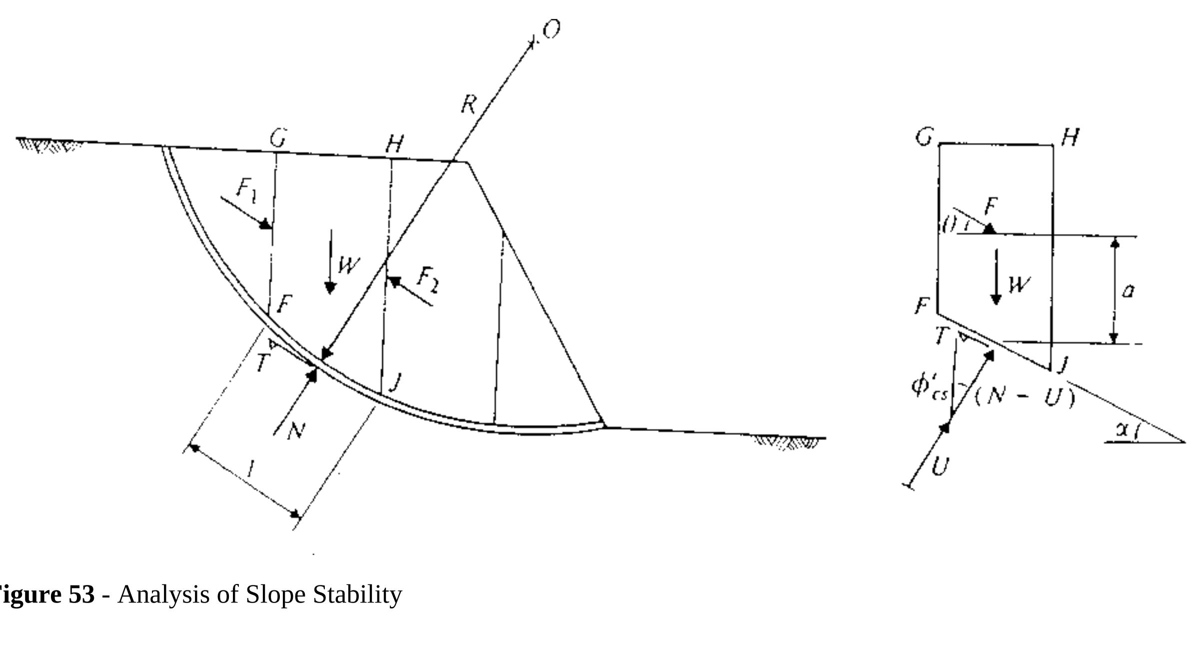


Figure 53 - Analysis of Slope Stability

the forces which tend to cause slope movement (soil weight, slope inclination, groundwater levels), to the forces resist slope movement (soil strength). Based on the forces along the assumed failure plane, a 'Factor of Safety' (FS) is calculated and defined as the ratio of available soil shear resistance to the gravitational loads along an assumed failure surface or plane.

$$\text{Factor of Safety} = \frac{\text{Soil Resistance, Preventing Movement}}{\text{Gravity Forces, Tending to Cause Movement}}$$

The Factor of Safety represents a measure of risk of failure or movement. These methods do not result in calculation of expected deformation (amount of movement), or rate of movement.

A F.S. of 1.0 suggests the soil resistance is equally balancing or resisting the gravitational pull on the soil mass, and it is at a point of 'limiting equilibrium' prior to movement. If the resistance is overcome (ie. F.S. < 1.0), some movement will take place as the slope tries to achieve a more stable condition.

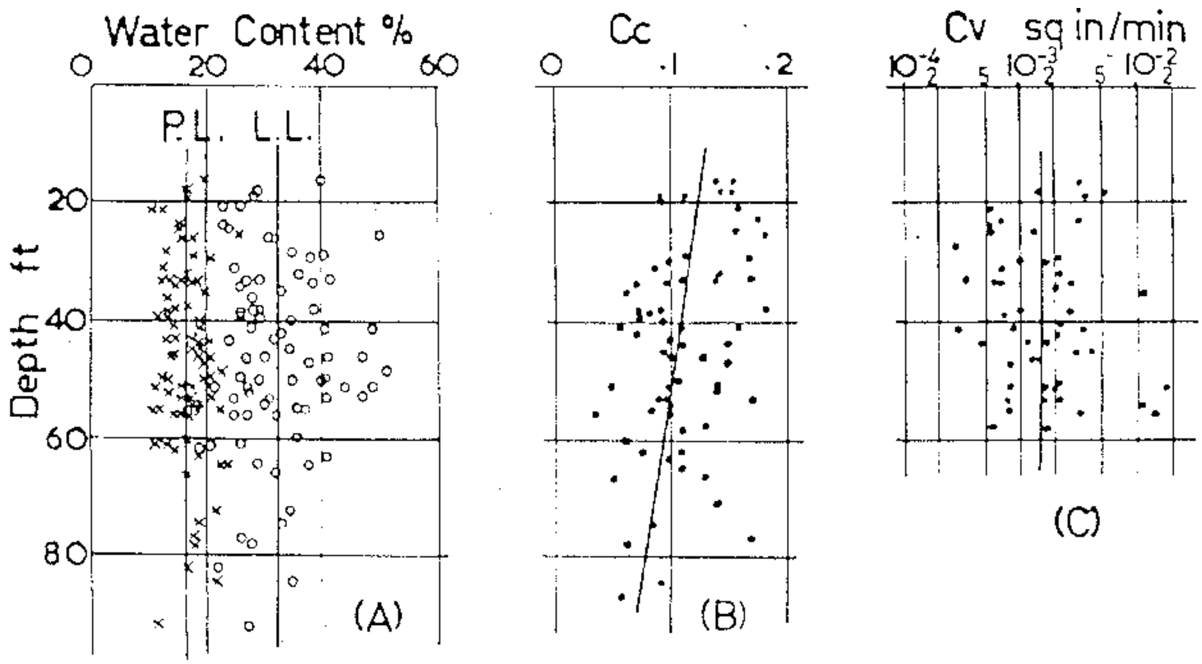
It is important to accurately determine the subsurface conditions which will be used in the analysis. A Factor of Safety greater than 1.0 is often used in design, to provide some margin of error in determining the soil properties. <u>For engineering design of slopes, a F.S. of 1.2 to 1.5 is commonly used</u>. The analytical methods have been calibrated (historical experience) to these values which represent the level of risk for failure to occur, based on various combinations of loading or groundwater conditions. There is additional detailed discussion of the Factor of Safety, in the following sections of this document.

b) Deterministic Methods

<u>Deterministic methods</u> other than limit equilibrium include plasticity theory, finite element solutions, and finite difference solutions. They assume soil properties to be fixed or uniform. The methods are generally very complex and intensive. While the focus of continuing research work, deterministic methods are not commonly utilized in land development situations.

c) Probabilistic Methods

<u>Probabilistic methods</u> are based on probability theory and recognize the inherent variability of soil properties. Soil strength properties can be analyzed to obtain statistical distributions; range of values, mean or average, median, standard deviation, variance. Probability theory is used to calculate the likelihood that the answer or solution to a slope stability problem is within a specified range.



(*a*) Atterberg limits against depth for sandy clay. (*b*) Compression index against depth for sandy clay. (*c*) Coefficient of consolidation against depth for sandy clay

Figure 54 - Variability of Soil Properties

Probabilistic methods of analysis should in theory be the most correct approach to assessing slope stability, since soil properties have a natural variability. However a large volume of test data (soil properties) are required to accurately model the natural variability (see Figure 55). There has been considerable research on probabilistic methods but not a great deal experience with practical applications.

d) Stability Charts

Most stability charts were originally developed from deterministic methods for uniform simple slopes only, using undrained (total stress) analysis for cohesive soils. These were prepared on the basis of many calculations for various assortments or combinations of the key parameters (i.e. slope height, inclination, soil properties, groundwater conditions). Prior to the use of computers, the stability charts were used to obtain quick approximate solutions to the many computations normally required in slope stability analysis.

6.1 Important Slope Stability Factors

The key variables which become important in slope stability analysis are indicated on Table 6.1.

TABLE 6.1 - Important Slope Stability Factors

- slope height
- slope inclination (steepness)
- soil stratification or layering
- soil type (composition)
- soil density and strength
- groundwater pressures
- external loads (structures, trees, fills)
- toe erosion (rivers, wave action, excavation)
- surface cover (vegetation).

The following sections will include an overview of the important analytical factors influencing slope stability;

- total stress analysis (undrained loading) and effective stress analysis (drained loading,)
- types of failure (translational and rotational),
- types of analysis (Ordinary, Bishop's, Janbu, Morgenstern-Price),
- Factors of Safety.

6.2 Total and Effective Stress Analysis

In applying the limiting equilibrium solutions, for fine grained soils there are 2 types of soil properties and loading conditions that can be considered;

a) analysis for short-term conditions where pore water does not have time to drain (pore water pressures not used in analysis); termed 'undrained' conditions or, 'total stress analysis',

b) analysis for long-term conditions where pore water can drain (or the pore water pressures are known or can be measured); called 'drained' conditions or 'effective stress analysis'.

Generally, a total stress analysis is valid only for slopes in poorly drained soils (clays or clayey silts) which are loaded rapidly. This may occur if fill is quickly placed near a slope crest or, where a new slope is created by excavation. The total stress analysis is used to assess short-term stability only.

An effective stress analysis is valid to all slope conditions (short term and long term) however, it is necessary to know or measure the groundwater levels or pore water pressures within the slope (see Table 6.2).

TABLE 6.2 - Total and Effective Stress

Rate of Change Loading Conditions	Type of Shear Strength	Appropriate Analysis	
Relatively Fast ie. after excavation or after construction short term conditions	Undrained ϕ_u, c_u	Total Stress does not require knowledge of pore pressures	clays, clayey silts
Relatively Slow long term conditions ie. natural slopes	Drained ϕ', c'	Effective Stress requires knowledge of pore pressures	all soils

6.3 Translational Slides, Infinite Slope Analysis

Most limiting equilibrium methods require an iterative process (trial and error) where an estimate of forces on a soil mass is made and a solution converges after successive re-calculation. For a specific set of slope conditions however, the Factor of Safety can be calculated using a very simple formula which is not iterative. Such conditions are uniform slopes (simple geometry, single inclination) in homogeneous (not layered or stratified with different soil properties) cohesionless soils (gravels, sands, silts) which are susceptible to shallow translational slides (planar), parallel to the slope surface (see Figure 56).

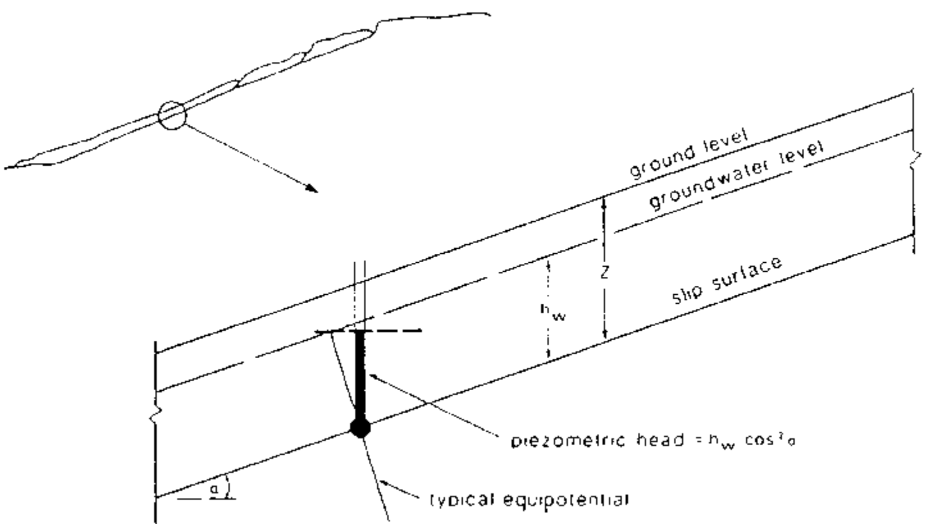


Figure 55 - Analysis of Translational Slides

The Factor of Safety (F.S.) for translational slides in cohesionless soils can be defined as;

$$F.S. = (1 - r_u)\frac{\tan\phi'}{\tan\beta}$$

where;
- ϕ' is the soil angle of internal friction (degrees),
- ß is the slope inclination (degrees) from the horizontal,
- r_u is defined as the "pore pressure ratio" at the slide base; the piezometric head divided by the soil weight

$$r_u = \frac{u}{\gamma h}$$

where;
- u is the pore water pressure, kN/m^2
- γ is the soil unit weight, kN/m^3
- h is the height of soil above the slide base, m.

For a dry slope (groundwater level below depth of failure plane), the first term in the equation equals zero and the Factor of Safety is simplified to

$$F.S. = \frac{\tan\phi'}{\tan\beta}$$

For a natural dry slope (ie. groundwater level deeper than potential failure surfaces) of cohesionless material, the slope will remain stable if the inclination angle (ß) equals (or is less than) the angle of internal friction (ϕ') as shown on Figure 57. The slope is at "limit equilibrium" and is said to have a Factor of Safety (F.S.) equal to one when tan ϕ' = tan ß (when resisting forces equal disturbing forces). When ß is < ϕ', the slope is stable with F.S. > 1, and when ß is > ϕ' the slope is unstable with a F.S. < 1.

It should be noted that the F.S. is the same for a dry or submerged slope assuming that the angle of friction ϕ' remains unchanged. Submerged slopes may tend to assume flatter inclinations than non-submerged slopes, in response to additional external forces such as wave action, currents, frost and ice forces. These forces cause loosening of the soil and reduction of the angle of internal friction (ϕ'). Current design methods rarely incorporate these factors.

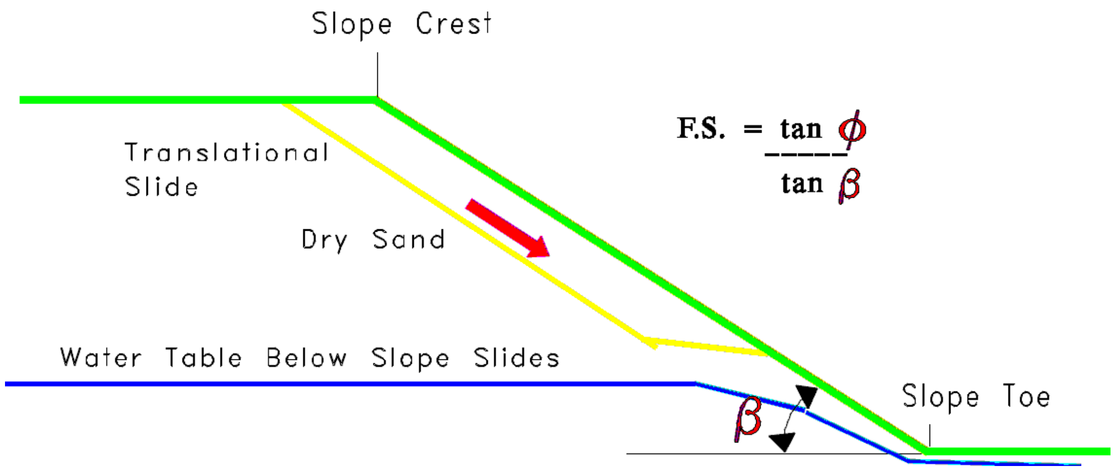


Figure 56 - Stable Inclinations in Dry Cohesionless Soil Slopes

Due to the surficial loosening of cohesionless soils, most stable natural slopes have an inclination of about 2 to 1 (26½°). Surficial erosion and sloughing due to rainfall and runoff are the most important factors in determining the stable slope inclination in cohesionless soils.

6.4 Taylor's Charts; Uniform Slopes, Total Stress Analysis

Taylor's curves (1937, 1948) were the first attempt at reducing the problem of slope stability analysis (many calculations, many variables) to a more simple process for practical applications. For a simple uniform slope profile (no complicated geometry) and homogeneous soil conditions (single soil type), Taylor conducted extensive calculations to produce a series of curves or graphs, depicting the relationship between the soil properties, soil inclination, and slope height etc., from which design could be undertaken for stable slopes.

These results are <u>applicable only to "total stress" analysis (undrained) or short term conditions</u>, not usually suitable for analysis of long-term stability of natural slopes. They are suitable only for soils that exhibit some cohesion (i.e. clays, glacial tills). The basic parameters consist of;

a) soil properties;

- γ unit weight, kN/m^3
- c_u shear strength (undrained), kN/m^2

b) slope geometry;

- β inclination
- H height, m

The results were simplified for the graph (see Figure 57) by the use of a "stability number N" which is a dimensionless term as follows

$$N=\frac{c_u}{F.S.\gamma H}$$

Taylor found that critical failure surfaces tended to be deep for predominantly cohesive soils (clays, clayey silts) and shallow for soils with high frictional strength (tills).

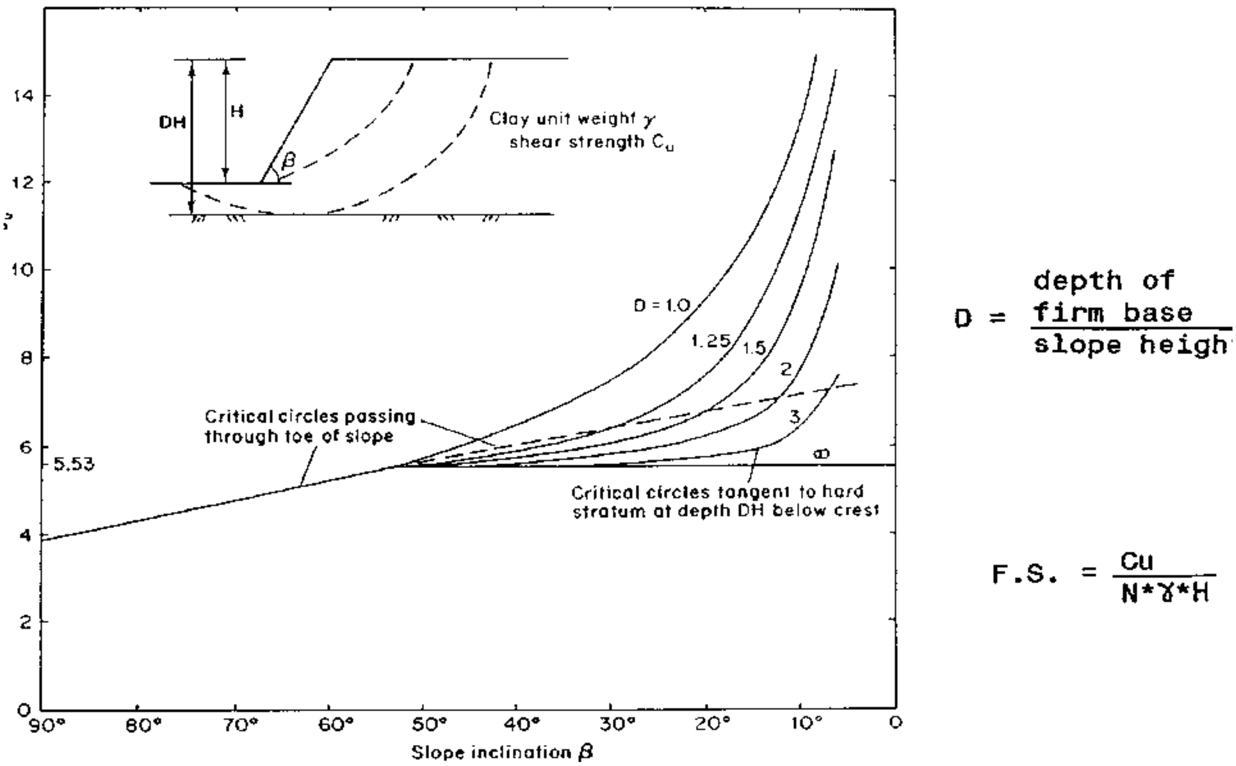


Figure 57 - Taylor's Chart

Bishop and Morgenstern prepared charts based on Effective Stress Analysis and pore water pressures were included in the stability charts, through the use of the pore water pressure ratio (r_u) which represents the ratio of pore water pressure to soil pressure at any point in the slope. Values of r_u typically range from about 0.2 to about 0.5 (water table at ground surface), but can be slightly higher with seepage or flow.

6.5 Fellenius (Method of Slices)

Many methods of analysis have been based on the 'method of slices' as originally developed by Fellenius (1927, 1936) also called the "Swedish", "Ordinary", or "USBR" method. This method can be applied to slopes with non-uniform inclinations (complex geometry, or composite slope angles) and with complex stratigraphy or layering.

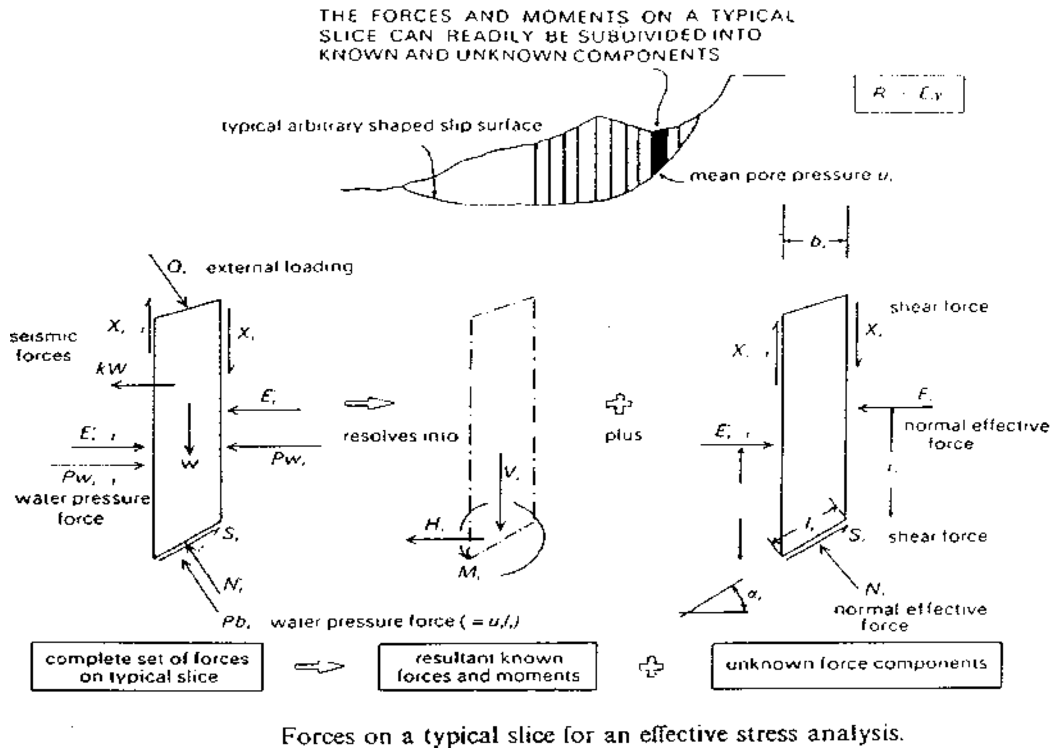


Figure 58 - Method of Slices, Fellenius

The method assumes a circular failure surface through the slope. The sliding soil mass above the failure surface, is divided into many vertical slices (see Figure 58) which are analyzed individually and as a whole, for moment equilibrium (balancing of forces causing and resisting movement). For simplification of calculation, inter-slice forces were ignored. The method permits either total stress or effective stress analysis. The inclination of the slice base, the material properties and water pressures at the base of the slice are determined and a stability analysis is carried out. The method is based on consideration of each slice as a "free body" where the forces on the slice base and slice sides are balanced.

The original Fellenius method has been found to over-conservatively under-estimate the Factor of Safety (F.S.) by 10 to 15 percent. The error was due to the simplifying assumption that there were no inter-slice forces. The analysis results for this method indicate a slightly less safe condition than really exists. The inaccuracy can be large for deep failure circles (where the slice base inclination at the toe becomes negative), and with increasing groundwater pressures.

6.6 Bishop

Bishop (1955) extended the solution method of Fellenius by solving the problem rigorously and including inter-slice forces (see Figure 59) in the equations of equilibrium.

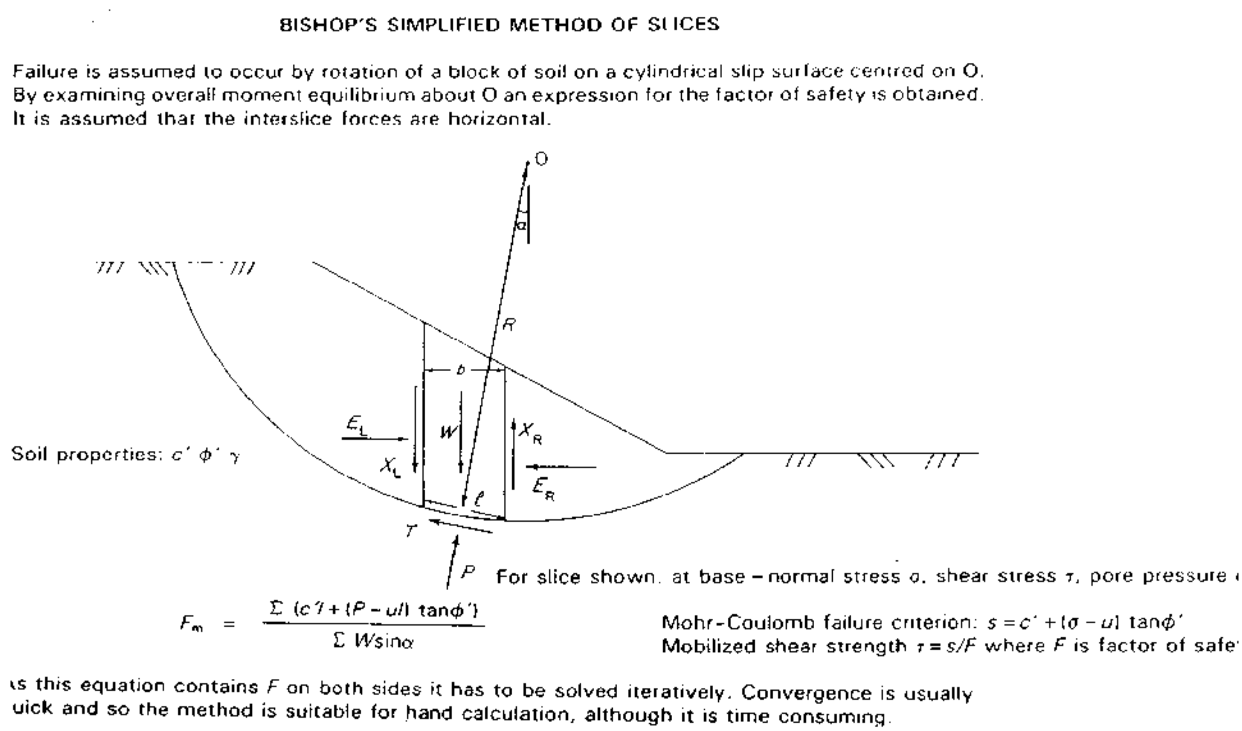


Figure 59 - Bishop's Method

A <u>rigorous solution</u> requires that;

a) the total forces on each slice satisfy conditions of equilibrium, and

b) the total forces acting on the entire sliding mass satisfy conditions of equilibrium.

The problem becomes a process of <u>iterative or repetitive calculations</u> in which assumed values of the inter-slice forces are improved with successive approximations. The solution converges quickly to the final answer.

Bishop also found that a "simplified" version (where inter-slice shear forces are neglected) of the rigorous solution, produced relatively accurate results (results from simplified version within 2 percent of results from rigorous version) with a significantly less complex calculation process. This method is used widely and is suitable for most situations where circular failure planes (uniform) are appropriate. However it is less accurate for composite failure planes.

Stability charts (see Figure 61) based on the Bishop's method were prepared by Bishop and Morgenstern (1960) to permit 'effective stress' analysis using the pore pressure parameter r_u. The Factor of Safety was calculated as;

$$F.S. = m - n \cdot r_u$$

where m and n are dimensionless parameters dependent on

β = slope inclination H = slope height D = depth factor

c' = soil cohesion **γ** = unit weight **ϕ**' = soil friction

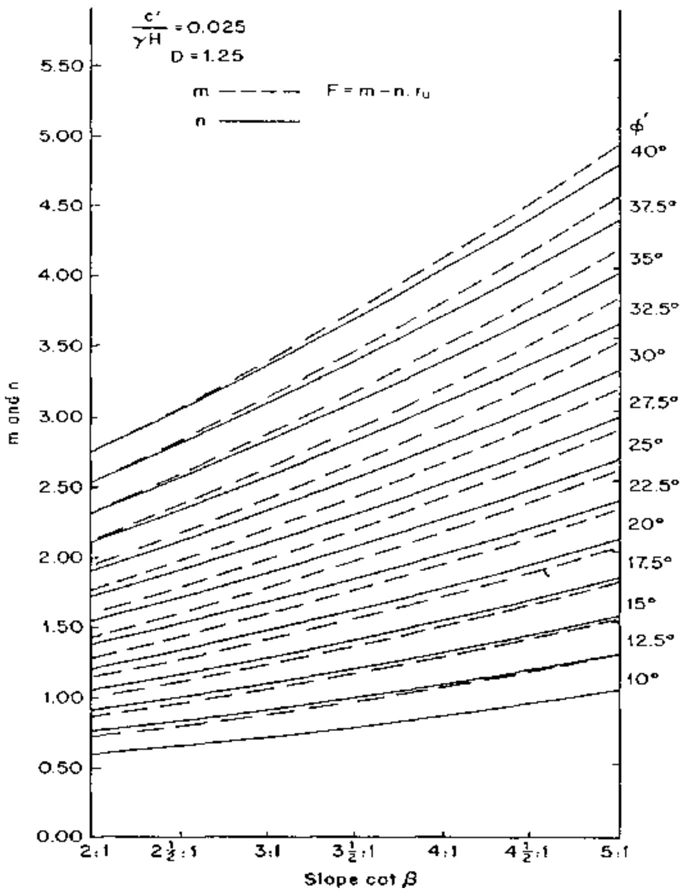


Figure 60 - Bishop and Morgenstern Stability Charts

6.7 Janbu

Janbu (1954, 1973) improved on the Bishop approach by developing a similar method applicable to slope slides with any shape of failure surface (not limited to circular) and any stratigraphy. Janbu's method considered the force and moment equilibrium for the typical individual slice, as well as the force equilibrium of the entire sliding mass (not moment equilibrium of entire mass). The side forces and side shear force on each slice are determined, along with the position of a "line of thrust" (see Figure 62).

As with the Bishop method, an iteration process is undertaken, with rapid convergence on the correct answer. The estimates of the line of thrust position must be checked to confirm that tension is not implied in the sliding mass. Janbu also developed a "simplified" method which incorporated an empirical correction factor (see Figure 63) to estimate the effects of inter-slice shear forces (increasing the calculated F.S.) on the basis of the D/L Ratio previously mentioned, and on the nature of the soil properties. The maximum correction is about 13 percent.

JANBU'S RIGOROUS ANALYSIS

Failure is assumed to occur by sliding of a block of soil on a non-circular slip surface. By examining overall force equilibrium an expression for factor of safety is obtained. The interslice forces are evaluated by considering the moment equilibrium of each slice. For this it is necessary to assume a position of the line of thrust of the interslice forces. Overall moment equilibrium is satisfied implicitly.

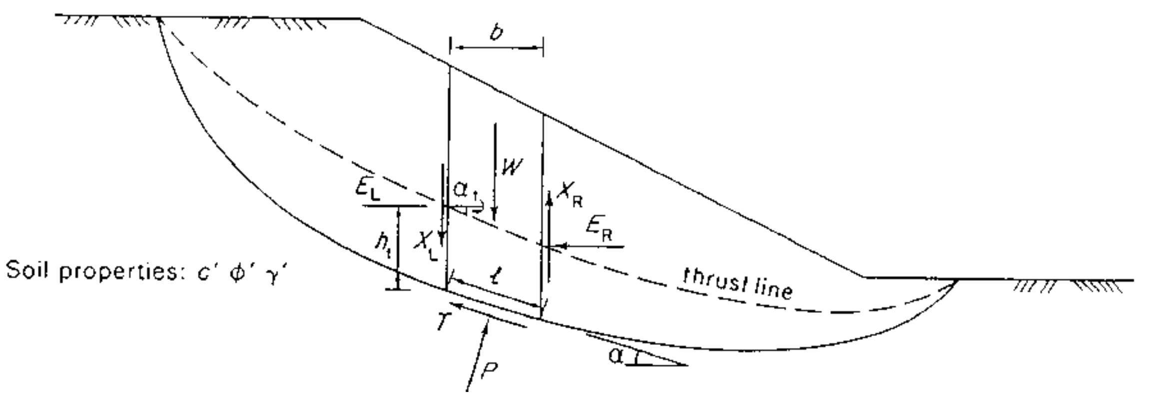


Figure 61 - Janbu's Method

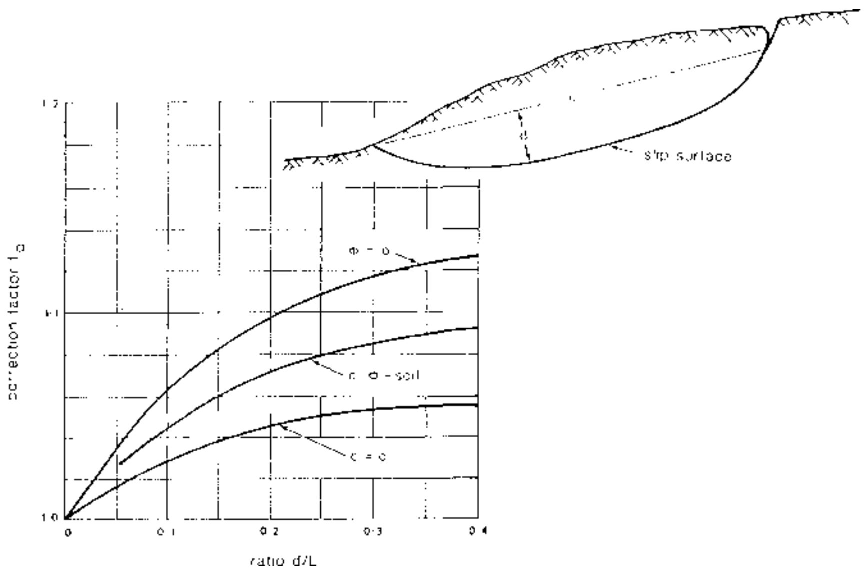


Figure 62 - Correction Factor, Janbu's Simplified Method

6.8 Morgenstern and Price, Spencer

Morgenstern and Price (1965, 1968) developed a general approach for limit equilibrium analysis for slip surfaces of any shape. Their <u>very complex</u> method is based on solving a pair of simultaneous partial differential equations, and is therefore ideal for computer application.

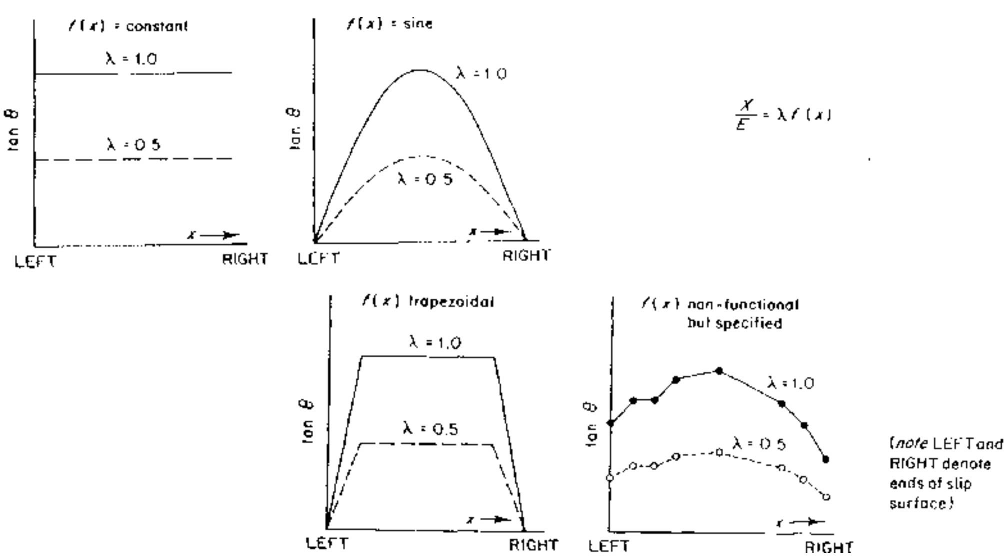


Figure 63 - Morgenstern and Price Method

The Morgenstern-Price approach satisfies all static equilibrium requirements by assuming a mathematical relationship (see Figure 64) between the inter-slice shear and normal forces (interslice forces vary along slide surface). Some limitations are difficulties in accounting for concentrated surface loads, and where there are near-vertical slope sections. This method may yield F.S. which are 25 to 30 percent larger than those obtained by other methods based on force equilibrium only.

Spencer's method (1967) is very similar to Morgenstern-Price except that it assumes interslice forces at a constant inclination.

6.9 Sarma

In the 1973 publication by Sarma (expanded in 1976), a very different approach was presented for the calculation of slope stability. The computations find a critical value for uniform horizontal acceleration that will just cause failure of the slope (ie. limit equilibrium). The analysis can also calculate the conventional Factor of Safety for the case of zero horizontal acceleration. Hoek (1983) developed a simplified version for computer, of the Sarma method.

The Sarma method is considered to be a rigorous method as both force and moment equilibrium are satisfied. This method provides a complex function to determine the inter-slice forces based on lateral earth pressures in the soil. A computer is necessary to use this method, as in the case with the Morgenstern-Price method. The local F.S. is also calculated.

6.10 Comparison of Different Limit Equilibrium Methods

The differences of accuracy of Factor of Safety (F.S.) depend mainly on the type of problem (site conditions) rather than on the analysis method used. In some problems (site conditions), simple analytical methods may not differ much from more accurate or complex methods of analysis. Accordingly, it is more important to accurately define the site conditions than to use a very sophisticated method of analysis.

The distinction of being a "rigorous" method or a "simplified" method, is determined by whether both force equilibrium and moment equilibrium are satisfied in the solution. Methods based on only force equilibrium being satisfied, find the F.S. is quite sensitive to the inter-slice forces. Conversely methods which satisfy only moment equilibrium, find that the F.S. is relatively independent of the inter-slice forces.

The "simplified" versions are usually solved quickly (require much less computation) and the error in the results is generally less than 7 to 10 percent when compared to the rigorous versions. This level of accuracy is considered acceptable for practical applications. Most soil properties have a natural variability which is seldom measured to a comparable accuracy, by geotechnical investigations for land development purposes. Thus it may be more important to accurately measure soil properties than to utilize a more accurate method of analysis.

It is clear that the Fellenius method under-estimates the F.S. and therefore is conservative and safe. As a general rule, deep failure surfaces with high pore water pressures, should be analyzed by relatively rigorous procedures.

For all computerized methods, it has been suggested that the slope surface should be divided into 20 to 30 slices or more, to adequately represent the failure surface shape and the force distributions(see Figure 65). All of the commonly used methods, except that of Sarma, are essentially the same and differ only on the basis of the assumptions made for either;

a) the location of the inter-slice horizontal force (line of thrust), or

b) the magnitude of the inter-slice shear force.

The following Table 6.3 is a comparison of the basic mathematical formulae for the various methods,

TABLE 6.3 - Slope Stability Analysis Formulae

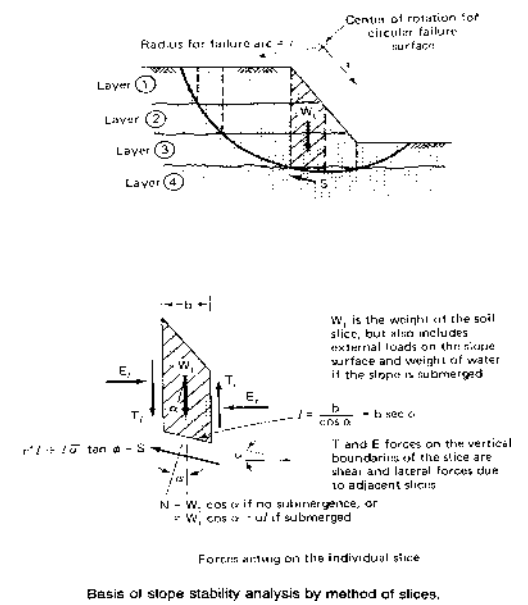


Figure 64 - Soil Slice Forces

METHOD	FACTOR OF SAFETY
ϕ' = 0	
(1)	$F.S.=\dfrac{\Sigma(c_u \cdot b)}{\Sigma W \cdot \sin\alpha}$
Translational Slide	
(2)	$F.S.=(1-r_u)\cdot\dfrac{\tan\phi'}{\tan\beta}$
Fellenius	
(3)	$F.S.=\dfrac{\Sigma(c'\cdot b+(W\cdot\cos\alpha-u\cdot b))}{\Sigma(W\cdot\sin\alpha)}$
Bishop's	
(4)	$F.S.=\dfrac{\Sigma(c'\cdot b+(W-u\cdot b)\tan\phi')\dfrac{\sec\alpha}{1+\tan\alpha\tan\phi'/F.S._a}}{\Sigma W\cdot\tan\alpha}$
Janbu's	
(5)	$F.S.=\dfrac{\Sigma(c'\cdot b+(W-u\cdot b)\tan\phi')\dfrac{\sec^2\alpha}{1+\tan\alpha\tan\phi'/F.S._a}}{\Sigma(W\cdot\tan\alpha)}$

The comparison Table 6.4 of analysis methods can be further simplified;

TABLE 6.4 - Comparison of Stability Analysis Methods

METHOD	FAILURE SURFACE	INTERSLICE FORCES	ADVANTAGES, SUITABILITY
Translational, Infinite Slope	Planar	Parallel to slope.	- simple calculation - cohesionless soils (sands, c'=0) only
Fellenius	Circular only	Ignored. Resultant parallel to slice base.	- calculation simple - no iteration needed - under-estimates F.S. - large error possible for deep slides and for high pore pressures - <u>not always suitable for effective stress analysis</u> - moment equilibrium
Bishop	Circular only	Assumed horizontal. Shear forces neglected.	- iteration required - error possible near slope toe if slice base has negative angle - <u>suitable for both total and effective stress analysis (E.S.A.)</u> - can calculate normal forces on failure surface - moment equilibrium
Janbu	Any shape	Forces assumed. Defined by thrust line. Simplified version utilizes a correction factor.	- iteration needed - solution must be checked for validity; line of thrust, tension - <u>suitable for both total and E.S.A.</u> - force equilibrium
Morgenstern-Price	Any shape	Assumes mathematical variance between normal and shear forces. Assumes side force inclinations to vary linearly across slice base.	- computer required - versatile method - much numerical modelling judgement needed - solution must be checked for validity - <u>suitable for both total and E.S.A.</u> - moment and force equilibrium
Sarma	Any shape	Calculated by complex function.	- computer required - readily includes seismic loads - <u>suitable for both total and E.S.A.</u> - moment and force equilibrium

To provide an appreciation of the calculated answers of Factor of Safety (F.S.) for different slope conditions (different sites), the following Table 6.5 presents a comparison of F.S. as calculated by various analysis methods. These results are for three different hypothetical slopes for example; uniform soil conditions, layered soil conditions, fill conditions.

TABLE 6.5 **-** Comparison of F.S. Obtained for Different Slopes and Different Methods

	Example Slope Conditions	**FACTORS OF SAFETY** Fellenius	Simplified Bishop	Morgenstern-Price
1.	Uniform slope, homogeneous soil, no pore pressure.	1.49	1.61	1.58 - 1.62
2.	Slope on organic silt, 3 different soils layers, long term stability	1.09	1.33	1.24 - 1.26
3.	Submerged rockfill slope resting on inclined cohesive soil stratum.	1.14 1.84*	2.00	2.01 - 2.03

* The first value corresponds to total unit weight and pore pressure, the second to submerged unit weight; a special difficulty of the ordinary method of slices.

Note: The range of results for Morgenstern-Price is based on choice of assumption concerning inter-slice forces. The two values for case 2 by Bishop were due to difficulty in determining the shear force at the slice base near the toe.

From the above table it can be seen that using either basic or sophisticated methods of analysis, the calculated Factors of Safety can be quite similar (ie. 1.5 vs 1.6 or, 1.1 vs 1.3) for many types of slope conditions. More complex site conditions may result in greater differences in calculated FS between types of methods. It is more critical that the evaluation of the site conditions be accurate than the choice of analysis method.

Following is another comparison of F.S. results (calculated by different methods) for a hypothetical set of site conditions (see Table 6.6). This analysis examined the effects on the F.S. where there is one slope type under different groundwater conditions and with thin weak soil layers,

TABLE 6.6 - Comparison of Factors of Safety Obtained by Different Methods

Examples	**FACTORS OF SAFETY**				
Cases	**Fellenius**	**Bishop Simplified**	**Janbu Simplified**	**Janbu Rigorous**	**Morgenstern -Price**
1. Simple 2:1 slope, 12 m high $\phi'=20°$, $c'=28.75$ kPa	1.93	2.08	2.04	2.01	2.08
2. Same slope with weak thin layer, $\phi'=10°$, $c'=0$	1.29	1.38	1.45	1.43	1.38
3. Same slope, No weak layer $r_u=0.25$	1.61	1.77	1.74	1.71	1.77
4. Same slope with weak layer $r_u=0.25$	1.03	1.12	1.19	1.16	1.12
5. Simple slope with piezometric line	1.69	1.83	1.83	1.78	1.83
6. With weak layer and piezometric line	1.17	1.25	1.33	1.30	1.25

Again the results show small differences in calculated F.S. between the various methods, and confirm that the specific method of analysis is less important than the correct interpretation of the site conditions and the appropriate modelling for the analysis.

6.10.1 Summary of Comparison

The variables which affect the F.S. the most are,

a) slope geometry

b) groundwater conditions

c) soil cohesion.

In summary,

a) all the methods produce similar answers for c'=0 soil slopes (ie. sand)

b) all the methods based on moment equilibrium being satisfied, produce the same answers for ϕ'=0 soils (undrained conditions)

c) all the methods based on force equilibrium being satisfied, show differences of up to 20 percent in the F.S. in c'- ϕ' soils (ie. soils with some cohesion),

d) only the Morgenstern-Price and Sarma methods do not assume the normal force on the slice base acts at the slice centre,

e) the various "rigorous" methods (Janbu, Morgenstern-Price, Sarma) have been found to produce similar F.S. with the typical variation being about 2 percent.

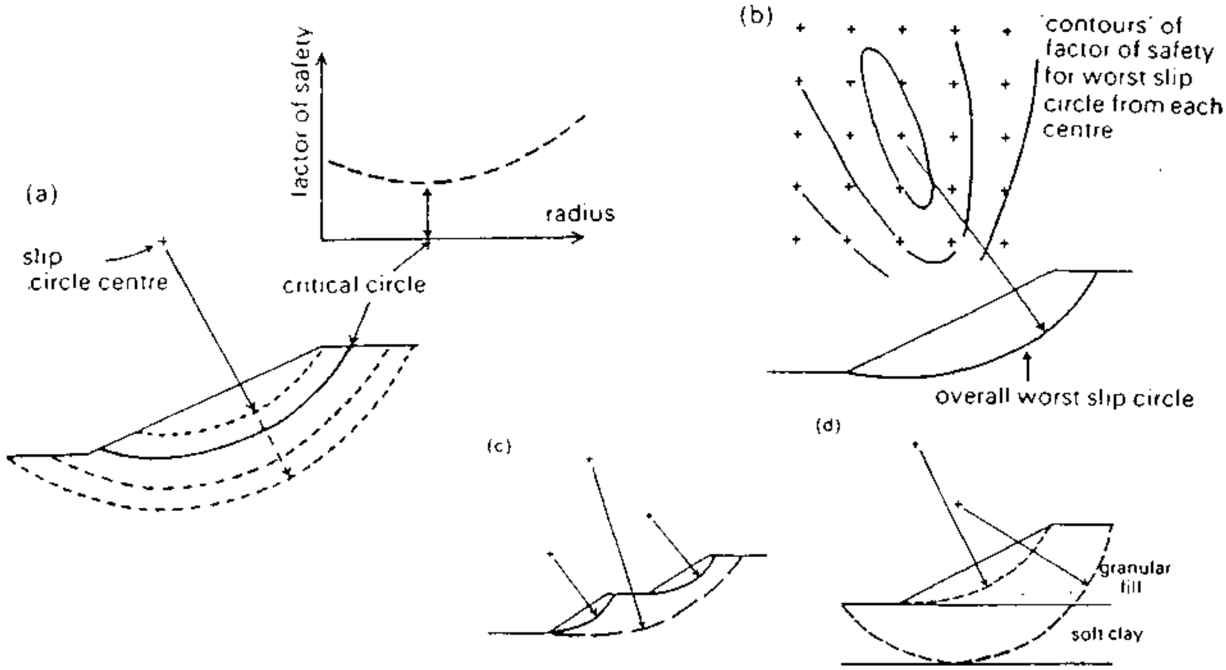
It should be noted that all of these methods are based on 2-dimensional analysis and that 3-dimensional analysis may increase the F.S. between 5 and 20 percent depending on the L/D ratio (4 to 2 respectively). Regardless, the generally accepted design criteria and the use of F.S. have been historically developed and calibrated on the basis of 2-dimensional analysis.

6.11 Technique of Analysis - Contours of Minimum F.S.

The following section briefly describes the typical techniques used in analysis, to model specific conditions or to help in searching for the critical Factor of Safety.

Analysis of slope stability is based on an arbitrarily assumed failure surface. The assumed failure surface may not be the most critical failure surface (i.e. the surface with the lowest F.S.). Therefore, a strategic search can be made for the critical or minimum Factor of Safety.

The search for the most critical failure surface in a slope can be simplified if slip circles are used (rather than composite slip planes). By considering a series of slip circles of different radii but each with the same centre of rotation, the 'minimum F.S.' from these successive radii is chosen for that centre. Then a different centre of rotation is assumed on a grid pattern and repeated until many points are obtained (see Figure 65) and the results are contoured through values of similar F.S. increments. It is important to analyze a large number of failure surfaces with a large spread of radii and a large grid of individual centres of rotation.



Searching for a critical slip surface. Taking first a variety of circles from the same centre, a minimum is found. This is then compared to the minimum factors of safety for other centres to find the most critical one.

Figure 65 - Contours of Minimum F.S.

The final results of the minimum F.S. for each grid location are plotted and contours of equal F.S. drawn. Often a coarse grid of centres is first chosen and a finer grid may be undertaken to better define the critical minimum F.S. and associated failure surface. A closed minimum value contour likely indicates the location of the worst slip circle. The following figure illustrates the various features of minimum F.S. searching.

The results of a search analysis with a grid of slip circles can determine if more than one local minimum F.S. is present. Sometimes two local minima for 2 different modes of failure can exist. In addition, some modes of failure are inconsequential and a low F.S. can be excepted in design (ie. very shallow slides).

6.11.1 Sensitivity Analysis

A sensitivity analysis is usually carried out to examine the relative effect on the calculated F.S., of minor changes in the key important variables (slope inclination, soil strength, groundwater levels). The procedure involves numerous re-calculations to obtain a general understanding of the relative effects of these changes.

6.12 Other Complicating Factors

Several other phenomena can complicate the modelling of a site, such as submerged slopes, very thin weak layers of soil, external loading, and tension cracks.

6.12.1 Tension Cracks

Tension cracks can form near the slope crest (see Figures 66 to 69) and the addition of tension cracks near the slope crest in an analysis can reduce the Factor of Safety (F.S.) since it reduces the length of the failure surface along which soil shear strength can be mobilized. As well, a crack may fill with water and additional force due to water pressure, adds to the instability (some tension cracks can extend to several metres depth).

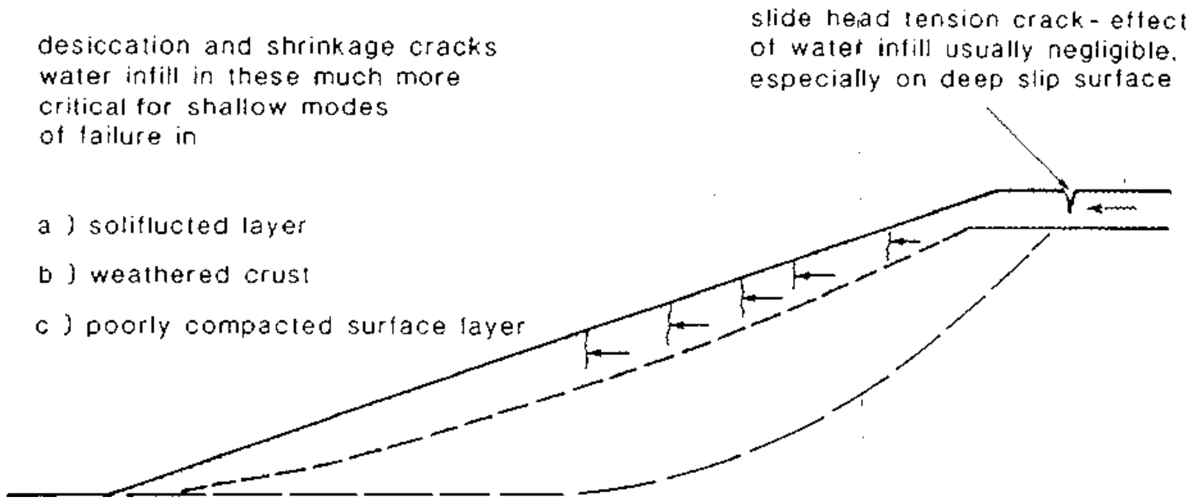


Figure 66 - Tension Cracks

However, some research has concluded that the effect of tension cracks on the F.S. is negligible (Spencer; 1968, 1973). Such conclusions were based on the small depth of tension crack predicted, in comparison with the total length of the failure surfaces. Nonetheless, tension cracks at the head of slide can be an important indicator of the onset of instability.

Figure 67 - Photo, Tension Cracks

Figure 68 - Photo, Failure Scarp

6.12.2 Partly Submerged Slopes

Slopes that are submerged present additional problems in the treatment (by the analysis method) of the applied loading (water against the lower slope). The vertical components of the water load can simply be added to the appropriate slice weights (see Figure 69). For most situations the water table is horizontal and an 'effective stress' analysis using 'drained' parameters (long-term conditions) can be undertaken with one of the following methods of modelling the free water against the slope face;

a) specify a horizontal water table (piezometric surface) within the soil mass and, apply the weight of water against the slope face as an equivalent force,

b) specify a horizontal water table within the soil and treat the free water as a soil layer with unit weight equal to water but with zero shear strength,

c) for the soil above the water table use the bulk unit weight, for the soil below the water table use the buoyant soil weight; and the water table need not be specified.

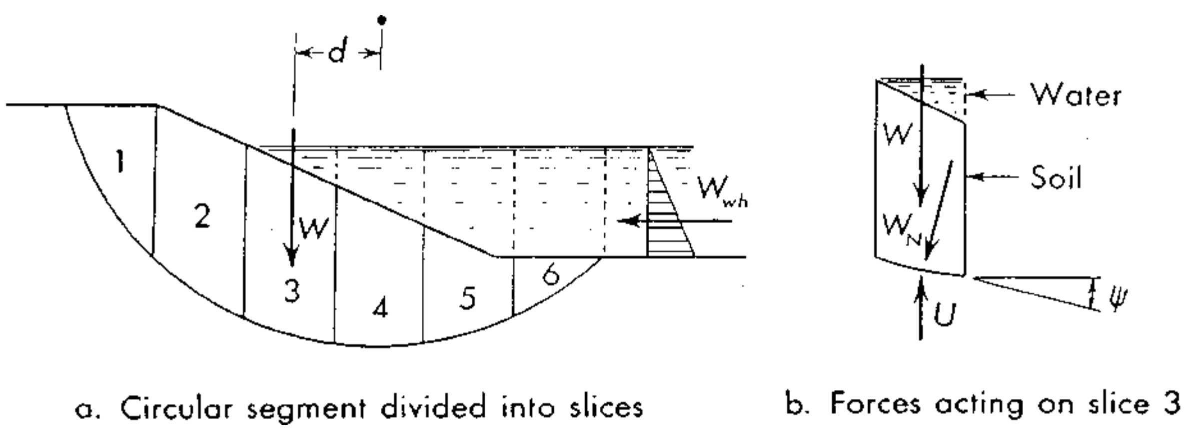
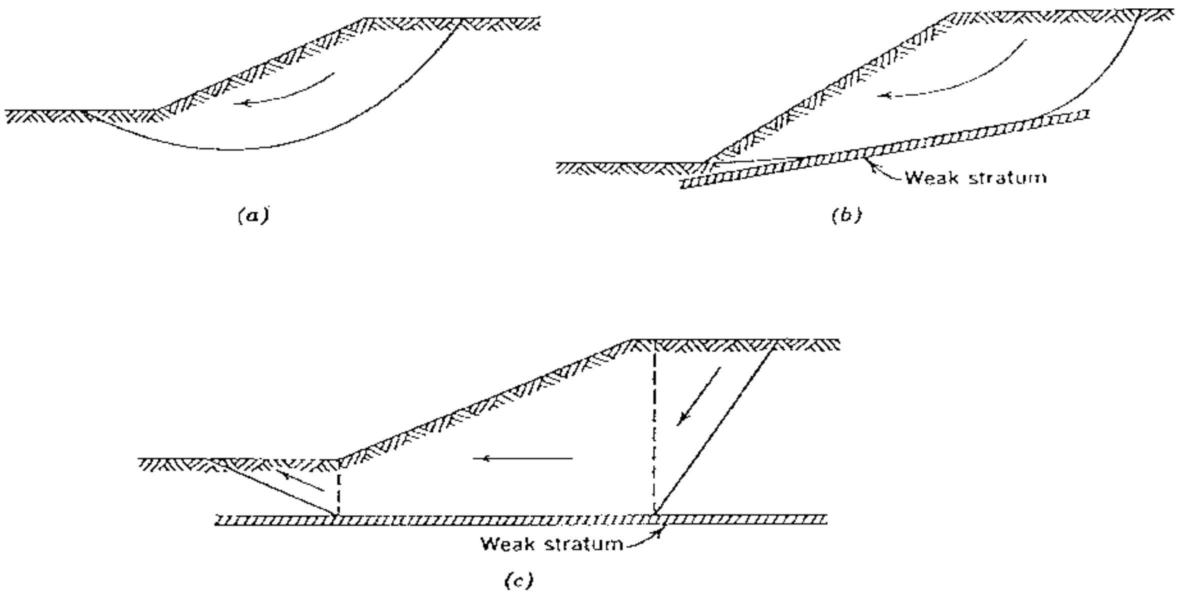


Figure 69 - Partly Submerged Slopes

Provided the pore water pressures in the slope remain the same (as they do in most natural slopes), the external water load applied to the slope (due to submergence) can have balancing effects on the stability and increase the Factor of Safety (F.S.). Despite the weight of water above the sloping surface increasing the overturning moment to reduce the stability, this increase is more than offset by the resisting moment from the horizontal water pressure on the slope face. The net result is that the submerged slope usually has a larger F.S. than the same slope without submergence (ie. submerged slope is more safe).

6.12.3 Thin Soil Layers

Soil stratigraphy or layering can influence the Factor of Safety greatly (see Figure 70). Stability can often be dependent on thin layers of weak soil with the critical failure surface running along the weak layer (composite failure planes). The best solution is the use of an analysis method that allows modelling of thin layers and non-circular failure surfaces. Some computerized analysis methods may not permit enough definition of the failure surface to ensure it fully intersects the weak layer. Another approach to overcoming this limitation is to increase the thickness of the weak layer, thereby ensuring more of the failure surface intersecting the weak soil. This will give a good preliminary result which can be further investigated in detail.



Types of failure surfaces. (*a*) Circular failure surface. (*b*) Noncircular failure surface. (*c*) Sliding block failure.

Figure 70 - Effect of Thin Layers

6.12.4 Surface Loads

External vertical loads can sometimes be present on the slope surface or near the crest, such as buildings, other structures, or fill. These loads are normally applied to the slope surface and added to each analyzed slice. Often the loads are modelled as additional soil layers with sufficient unit weight to produce the same slope loading conditions.

6.12.5 Pore Pressures

For 'effective stress' analysis, pore water pressures are determined for the base of each slice analyzed. The most common method used is to define a water table or piezometric surface. When there is a significant slope or gradient to the piezometric (groundwater) surface, this method may result in errors since the pore pressures can be quite variable. To overcome this problem, either several piezometric surfaces can be used or a pore pressure grid can be prepared and individual values can be applied to each slice. This can be a very complex problem in earth dams which retain or impound water (see Figure 71) and, which incorporate seepage boundaries such as grout curtains and impermeable cores. This document does not address such conditions.

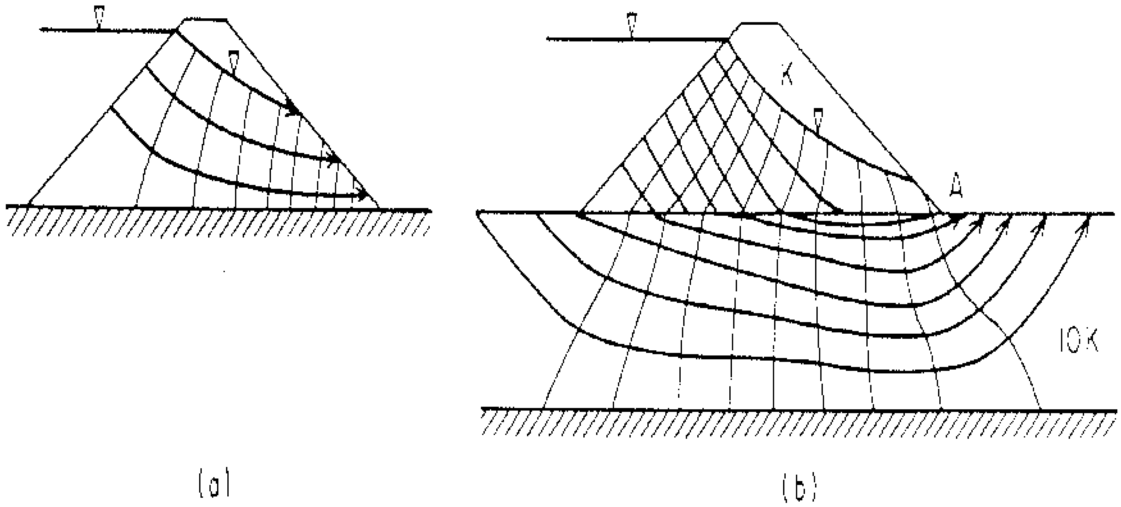


Figure 71 - Seepage Through Dams

It is important to measure groundwater levels but with some caution since the worst conditions may not occur during the monitoring period. As well, river or lake flooding which results in water rising up the slope and followed by rapid drawdown, can result in stability problems with sand soils or sand layers. This is because the high pore pressures may take an extended period of time to drain (i.e. they may not drain as fast as the impounded water is drained).

Use of the r_u value (pore pressure ratio) permits application of pore pressure proportional to the soil weight but incorrect results might be obtained where there is a natural variation in the r_u value. In most natural slopes (where water is not impounded as in dams) the hydraulic gradient often does not exceed unity and the use of a piezometric surface is usually sufficient.

6.12.6 Seismic Loading

A Seismic Zoning Map of Canada is available from the Dept. of Energy, Mines, and Resources (Seismology Division, Earth Physics Branch) which shows that most of Ontario has a relatively low rating for ground acceleration from earthquakes, except for the St.Lawrence River / Ottawa River area, and a small area south-east of Hamilton (see Figure 73). As such, for most sites in low rating areas the seismic affects are ignored. For other sites they may also be ignored unless high slopes are involved, or the calculated Factor of Safety is close to 1.0, or Leda Clays are involved, or there are high pore pressures.

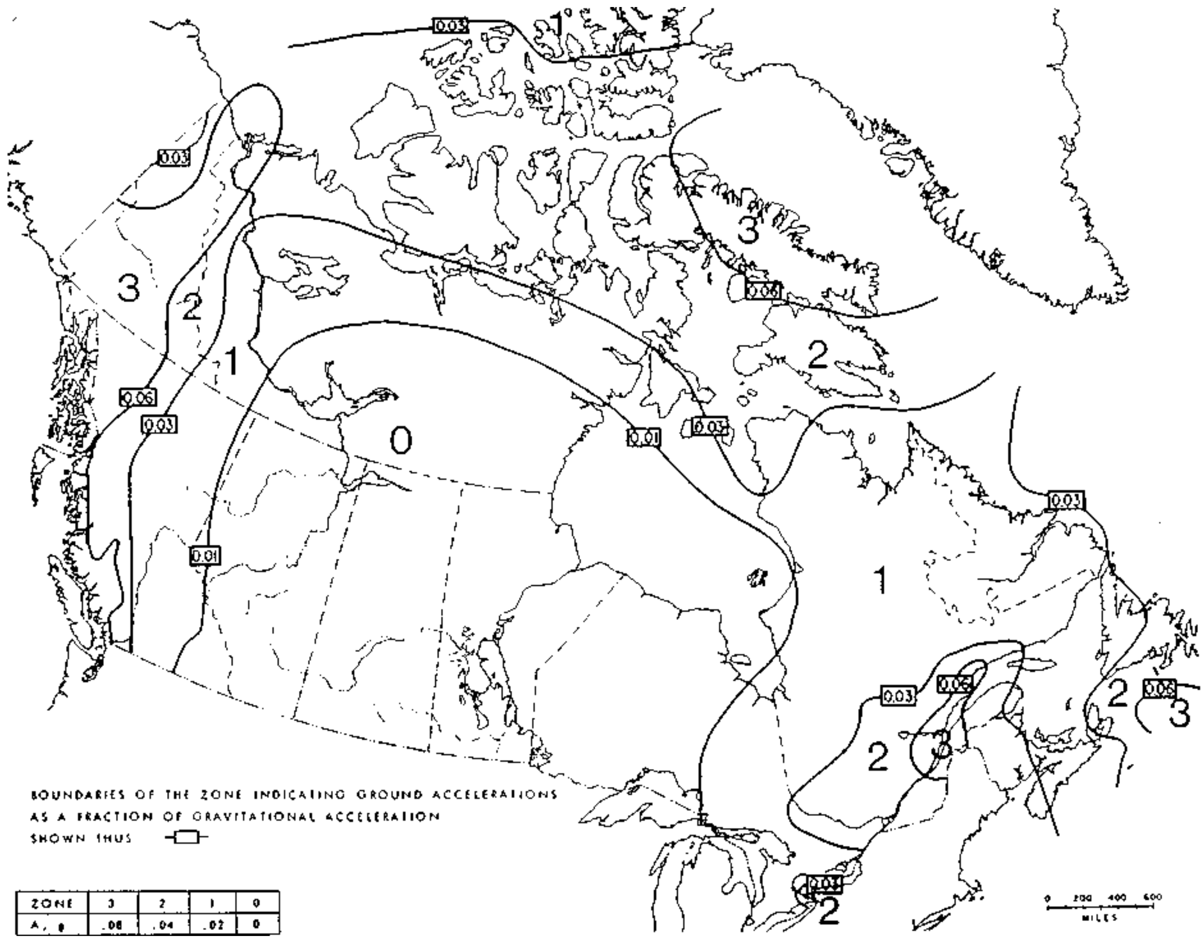


Figure 72 - Seismic Zones

7.0 DESIGN FACTORS OF SAFETY

The previous section introduced the concept of Factor of Safety and methods of calculation. This section discusses the appropriate selection of Factors of Safety as they relate to various site conditions and applications. This is referred to as the 'Design Factor of Safety'.

There is no widely accepted single minimum Factor of Safety (F.S.) for slope stability design. Rather there is a range of acceptable minimum F.S. into which various other criteria have been factored such as,

a) possible consequences of instability (level of damage which may occur), and

b) the degree of confidence that can be placed in knowledge of the soil strength characteristics and groundwater conditions.

As an example, the "Pit Slope Manual" produced by Energy Mines and Resources Canada (Can Met Reports 77-01) for use in the design of dams and mines, suggests the choice of F.S. should be based on two levels of consequences (in the event of slope failure);

1) that where severe damage would be anticipated as a result of the slope failure,

2) that where severe damage would not occur as a result of the slope failure.

The "Pit Slope Manual" also suggests the F.S. (See Table 6.7) be based on the various possible combinations soils strengths (peak and residual), and severity of loading conditions (earthquake loadings). The minimum design F.S. ranges from 1.1 to 1.5, with 1.3 being the most commonly occurring minimum F.S.

TABLE 6.7 - Suggested Minimum Factors of Safety for Design of Dam Slope

Assumptions		I *	II **
1.	Using peak shear strength parameters	1.5	1.3
2.	Using residual shear strength parameters	1.3	1.2
3.	Including loading for 100 yr earthquake	1.2	1.1

* I - where severe damage is anticipated as a result of slope failure
** II - where severe damage is not anticipated as a result of slope failure.

As another example, the "Geotechnical Control Office in Hong Kong" (where most of the terrain is slope) regulates many land development projects (not dams) and has adopted a policy based on a minimum F.S. for slopes near structures of

a) 1.4 for permanent developments in residential areas, and

b) 1.2 for temporary structures.

Also in Hong Kong, Table 6.8 shows the minimum F.S. to be used for design, based on risk to life and property, and on level of investigation, with a range of 1.2 to 1.4 for the minimum F.S. as follows,

TABLE 6.8 - Design Factors of Safety, Hong Kong

		FACTOR OF SAFETY	
Class	**Type**	**Comprehensive * Site Investigation**	**Cursory ** Site Investigation**
1	remote area	1.1	1.2
2	arterial route, no buildings	1.2	1.3
3	near buildings	1.2	1.4
4	beside buildings	1.4	n/a

* boreholes, soil testing, surveying ** site inspection; assumed information

In most of the historical technical literature (Terzaghi, Bjerrum, Bowles, and U.S. Navy), the most commonly recommended minimum F.S. for design was (mostly for dams);

a) about 1.25 for extra-ordinary loading conditions and,

b) about 1.5 for ordinary loading conditions.

Ordinary loading conditions are those which the slope would experience for most of the time. Extra-ordinary loading conditions are those which are of short duration or infrequent occurrence.

Computerized calculations of Factor Safety should always be cross-checked with simple manual calculations, to protect against inappropriate modelling or numerical difficulties which are sometimes present in computer applications. As well, the manual calculations often provide a better feel or understanding of the problem. An added precaution would be to also analyze the slope using a different method of analysis.

As shown above, the Design Factor of Safety has traditionally been selected in the range of about 1.2 to 1.5 using a variety of conditions based on consequences of failure and on accuracy of information on site conditions.

7.1 Probability and Failure Risk

As discussed in the previous section, the Factor of Safety (F.S.) should be related to reliability (probability) and consequences of failure yet a constant F.S. (same F.S.) for different designs could result in different reliabilities. The following Table 7.1 shows a general comparison of probabilities for failure in engineering design;

TABLE 7.1 **-** Failure Probability in Engineering Design

Design Area	**Failure Probability**		**Design FS**
Reinforced Concrete	0.01 %	1 in 10,000	
Slopes	1.0 %	1 in 100	1.2 - 1.5
Retaining Walls	0.1 %	1 in 1,000	1.5 - 2
Foundation Soil for Structures	0.01 %	1 in 10,000	2 - 3

The above comparison recognizes that engineering design is not based on absolute terms and that many variables (including human error) can influence design and construction. This indicates the current system and design standards risk a failure of 1 in every 100 slopes, which is much more frequent or likely than designing reinforced concrete which may experience a failure of only 1 in 10,000 designs.

The structural engineer generally has good confidence in the material properties but finds difficulty in assessing the loads which the structure will actually carry in service. Tabulated loadings, often in excess of probable loadings, are commonly used for design. The uncertain unloading conditions produce the use of a load factor.

In contrast the geotechnical engineer has much more confidence in the loading conditions (in most cases the self weight of the soil), but has much more doubt in the soil shear strength. A study by Alonso (1976) examined the effects of variability of the input parameters on the variability of the resulting F.S. (see Figure 73) and found that the variability in cohesion (c') and pore water pressure (u) had the largest effect on the F.S. variability.

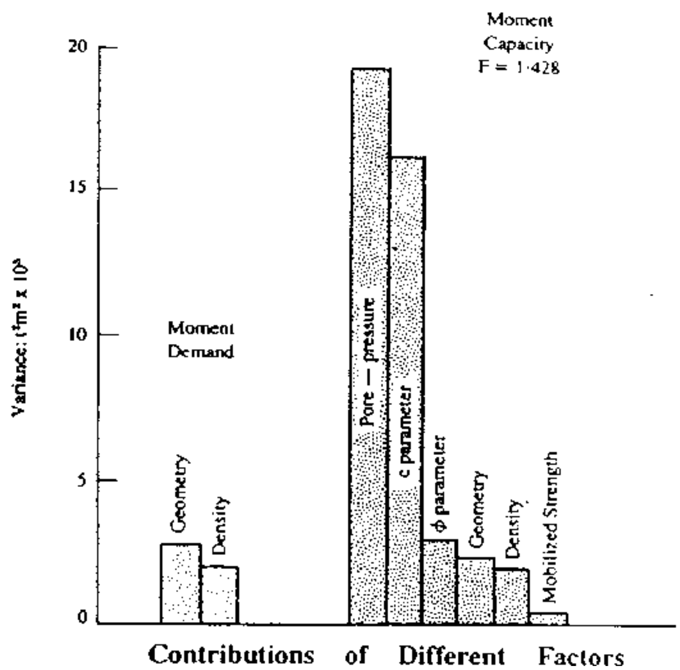


Figure 73 - F.S. Sensitivity

Further, it has been reported that considerable scatter in shear strength parameters may also occur for the same soil tested by different laboratories; ±25 % for ϕ' and ±50 % for c' so that the shear strength may be on the order of ±30 %. This scatter is attributed to a combination of natural variability in the soil and sample size, as well as variability in the testing procedure by different laboratories.

It should be noted, that calculated values of F.S. are generally most meaningful for values close to 1.0 and, that different values are only a relative index of stability. The values are not linearly related to each other in that a F.S.=4 is not twice as safe as a F.S.=2. They only exist to provide general comparison when considering effectiveness of proposed remedial measures (ie. one method of stabilization as compared to another, and only in relative terms).

If one were to design a slope assuming the combination of all worst possible affects on the slope (highest conceivable pore water pressure, undermining of the toe, surcharge loading at the crest, soil strength reductions and seismic accelerations), a F.S.= 1 might be perfectly acceptable because the safety margins are already built into the analysis. It is more common to analyze the most likely situations which may actually occur and to require the conventional Factors of Safety in the range of 1.2 to 1.5 .

7.2 Using Factor of Safety to Determine a Stable Slope Inclination

The following section will outline the procedure for using a design minimum Factor of Safety to determine a stable slope inclination. It should be again noted that a slope will achieve a stable angle only when toe erosion has been arrested.

- The first step is to select an acceptable design minimum Factor of Safety and the appropriate stable slope analytical model(s).

- The second step is to input and analyze the existing slope conditions using the appropriate models. If the model produces a slip plane with a value for the Factor of Safety equal to or greater than the design minimum Factor of Safety, the original slope is considered stable (see Figure 74).

- If the analysis produces a slip plane with a Factor of Safety that is lower than the design minimum Factor of Safety, the original slope is considered unstable (see Figure 75).

  Where the analysis of original slope conditions reveals an unstable situation, the original slope entered in to the model is modified by flattening the slope (i.e. reducing the slope angle, see Figure 76). Other stabilization measures such as berms, retaining walls, or improved drainage can also be evaluated.

- A flatter slope inclination is assumed in the model and, calculations are carried out on this assumed flatter slope. A new slip plane may result with a new Factor of Safety. This new Factor of Safety is again compared with the design minimum Factor of Safety (see Figure 76). This process is repeated until a Factor of Safety either equals or exceeds the design minimum Factor of Safety.

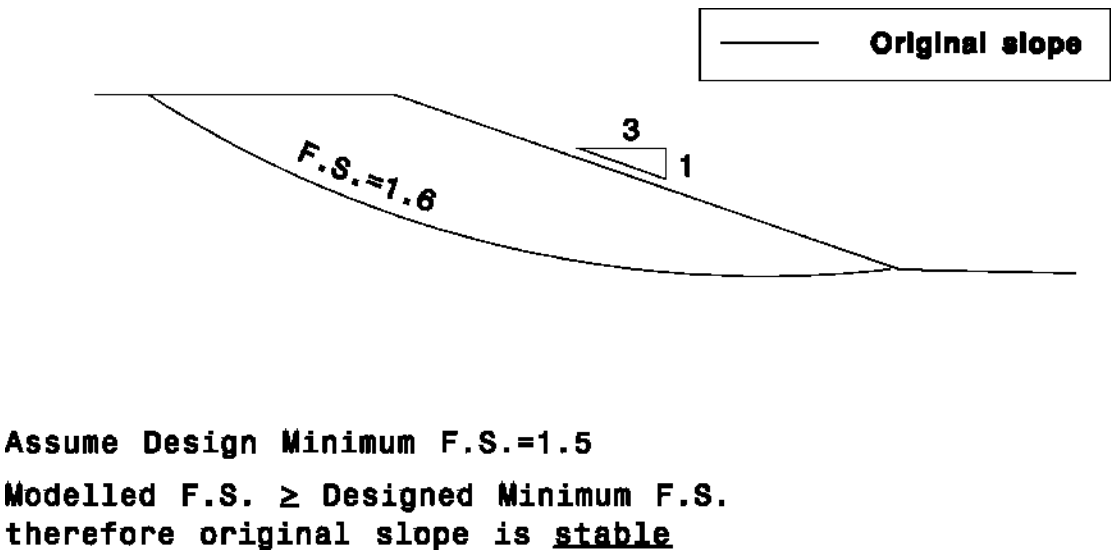


Figure 74 - Analysis of Original Slope Conditions: Stable

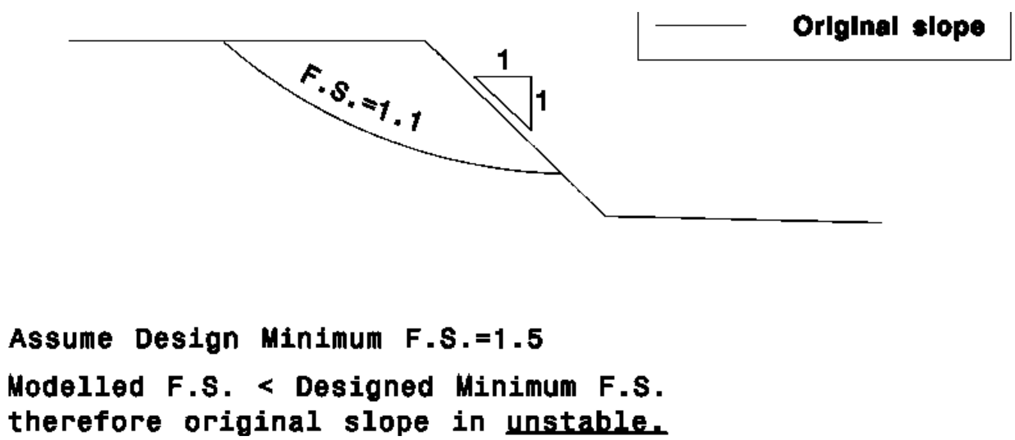
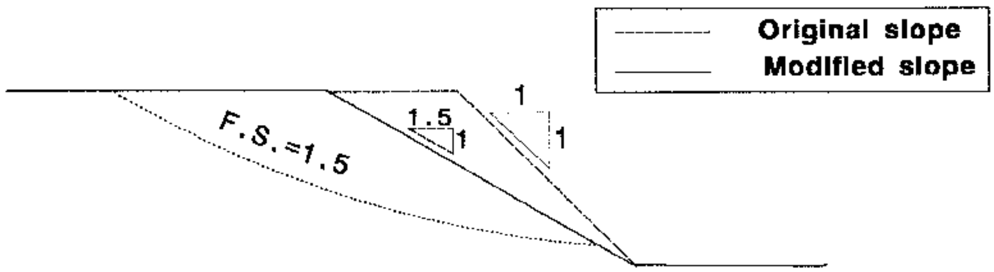


Figure 75 - Analysis of Original Slope Conditions: Unstable



Assume Design Minimum F.S.=1.5

Modelled F.S. for modified slope is ≥ Designed Minimum F.S., therefore the 1:1 slope is considered stable at a ratio of 1.5:1.

Figure 76 - Analysis of Modified Slope Conditions

The recommended stable slope inclination (possibly modified from existing slope) is the final slope inclination used to produce the slip plane with the design minimum Factor of Safety. It should be noted that the slip plane that produces a F.S. equal to or greater than the design minimum F.S., may daylight (or exit) on the face of the slope, through the toe, or even below the slope face (Figure 77).

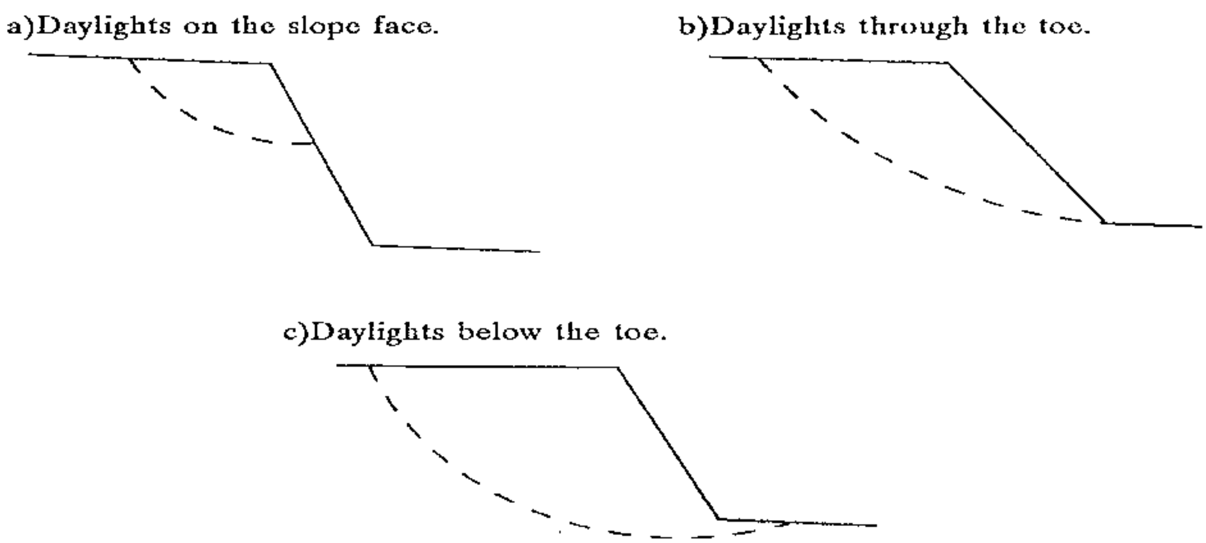


Figure 77 - Slip Face Daylight Locations

7.3 Design Minimum Factor of Safety

All detailed geotechnical slope stability analyses require the selection of a design minimum Factor of Safety (F.S.). The selection is based on the physical and hydrological characteristics of the slope, as well as on the proposed activity and consequences of slope failure. The intent of this section is to provide the reader with some direction in the selection of the design minimum Factor of Safety and indicate typical stable slope inclinations for different soil conditions. It should be noted that the numbers contained in this section should be used for guidance only and do not replace the judgement and experience of a qualified professional.

As discussed in Section 6, the Factor of Safety has traditionally been applied to account for uncertainty in soil properties (natural variability) and in measurement methods. The design minimum Factor of Safety has commonly been chosen in the range of 1.2 to 1.5. The actual value selected was also based on the consequences of slope failure.

The selection of a design minimum Factor of Safety can also based on the reliability of the basic information used in the slope stability analysis (methods of data acquisition, amount and variability of data), or on the site conditions (soil types, slope height and steepness). However, the number of factors involved and the wide variety of slope conditions that occur, prohibit the development of an analytical procedure for the selection of the design minimum Factor of Safety. Rather, the selection of the design minimum Factor of Safety is most often based on the consequences of slope failure, particularly on the land-use. Subsequently, Table D.2 has been created, based on sound geotechnical principles, which identifies four general classes of land-use (passive, light, active, and infra-structure or public use) and their associated design minimum Factors of Safety.

The land-use classes attempt to group activities associated with a commonly accepted level of risk in engineering design. <u>Passive</u> land-use is primarily that which involves a minimum level of investment, little regular use by people, and does not include buildings or structures of any kind. <u>Light</u> land-use is activity involving a larger capital investment, occasional or seasonal utilization by people, or secondary non-habitable buildings or structures. <u>Active</u> land-use includes habitable or regularly occupied buildings or structures, and significant capital investment. <u>Infra-structure or public use</u> activity includes buildings or structures which are constructed or maintained with public funds and, may include use by large numbers of people.

A range of design minimum Factors of Safety is given for all of the land-use classes except passive, because in reality there are a number of other factors that also need to be considered including, type and location of possible failure surfaces. Slope failures tend to be relatively deep (rotational or block) in 'cohesive' soils and can extend back beyond the slope crest. Slope failures tend to be relatively shallow (translational) and often limited to the slope face in 'cohesionless' soils. The danger to structures along the slope crest may therefore more if the slope is comprised of cohesive soils rather than cohesionless soils.

There are several other considerations that should be noted before using the chart, including;

- the detailed engineering analysis should only consider slides which are at least 2 m deep, for the design minimum Factor of Safety. In other words, very shallow slides are inconsequential and can be prevented by surficial stabilization techniques such as planting or reinforcing with various media (i.e. timber cribs, geogrids, cellular media and the like)

- the values for design minimum Factor of Safety should be used to determine the 'long-term stable slope inclination' or slope configuration rather than for short-term changes in slope or site conditions created for instance during construction projects

- for the purpose of assessing appropriate set-backs for development, the 'long-term stable slope inclination' will be defined as the slope inclination or configuration which satisfies the design minimum Factor of Safety. Accordingly the 'long-term stable slope crest' will be defined as the slope crest position of the slope inclination or configuration which satisfies the design minimum Factor of Safety.

In summary, the most practical basis to use for selection of the design minimum Factor of Safety is the type of land-use (people and property) associated with the slope area (see Table 7.2). This approach will be suitable in most cases, but some exceptions may apply if,

- the subsurface conditions and slope geometry are very well investigated and understood, or
- the use of conservative design F.S. may have a large economic impact on the slope design or land-use, or
- there is considerable historic evidence of slope stability.

TABLE 7.2 - Design Minimum Factors of Safety

	LAND-USES	DESIGN MINIMUM FACTOR OF SAFETY
A	**PASSIVE**; no buildings near slope; farm field, bush, forest, timberland, woods, wasteland, badlands, tundra	1.10
B	**LIGHT**; no habitable structures near slope; recreational parks, golf courses, buried small utilities, tile beds, barns, garages, swimming pools, sheds, satellite dishes, dog houses	1.20 to 1.30
C	**ACTIVE**; habitable or occupied structures near slope; residential, commercial, and industrial buildings, retaining walls, storage/warehousing of non-hazardous substances	1.30 to 1.50
D	**INFRASTRUCTURE and PUBLIC USE**; public use structures or buildings (i.e. hospitals, schools, stadiums), cemeteries, bridges, high voltage power transmission lines, towers, storage/warehousing of hazardous materials, waste management areas	1.40 to 1.50

8. SUGGESTED LEVEL OF GEOTECHNICAL INVESTIGATION REQUIRED TO ASSESS SLOPE STABILITY

The following section describes the reasoning and basis for a suggested method of site evaluation to assist regulating agencies within MNR in determining the level of geotechnical investigation required to assess slope stability. In all cases, the responsibility for providing the geotechnical investigation is that of the proponent who might be a land developer, a pit operator, or a government agency. Part of the proposed development may be located close to a slope crest and there may be concerns about risks of ground loss in the event of a slope slide.

The level of geotechnical investigation required to determine the stability of a slope involves an understanding of:

- the physical and hydrological site conditions
- the type of development or land-use proposed, which may be put at risk.

8.1 Physical and Hydrological Site Conditions

Slope stability analysis and the calculation of Factors of Safety, requires certain basic information that can be determined in several manners or can be estimated with reasonable accuracy;

a) the slope configuration; height and inclination or shape. These can be estimated visually, or determined from topographic mapping, or measured by on-site survey of slope cross-sections (profiles).

b) the subsurface conditions within the slope; soil stratigraphy (types and layering), soil strengths (density and shear strength), groundwater levels. These can be determined in a general manner by visual inspection of exposed soil on the slope, or on the basis of geologic mapping. More specific information can be obtained by drilling boreholes (unlimited depth), or digging test pits (max. depth 3 to 5 m), or hand auger holes (max. 1 to 2 m depth).

c) any external loadings to the slope; structures, traffic, earthquakes,

d) site drainage and erosion conditions; surface run-off, ditches, channels, seepage, creeks, rivers, lakes,

e) vegetation cover.

The decision to use simple investigation (based on site inspection only) versus a detailed investigation (including boreholes, surveys or mapping) depends mostly on

- the slope height
- the consequence of slope failure on the adjacent land-use.

8.2 Suggested Procedure To Determine The Level of Investigation Required

To assist in determining the suggested level of investigation required, a "Slope Stability Rating Chart" is provided. This Rating Chart can used by either those reviewing proposals or by a proponent, however a site visit is required to complete the Chart. The Rating Chart must be completed for all slope assessments and be retained by the reviewer. The Rating Chart has 7 components that together provide a reasonable assessment of the slope stability. Some calibration may be required of the values in the chart, on the basis of extensive experience with its use. The 7 components are;

1. Slope Inclination

. The angle from the horizontal of the slope face, measured from the toe to the crest. If the slope is comprised of several different inclinations, provide details on each. Estimate visually, or use hand inclinometer to measure approximate inclination, or survey (also refer to available mapping).

2. Soil Stratigraphy

. Soil layering and soil types composing the slope. Confirm if visible in bare exposed areas. Refer to previous nearby boreholes or well established local geology. If several soil layers are present, provide details on each.

3. Seepage from Slope Face

. The quantity and location of groundwater on the slope face. Visually inspect slope for surface seepage (springs, streams, creeks).

4. Slope Height

. Measurement of the vertical height between the toe (bottom) and the crest (top) of the slope. Estimate visually, or measure by surveying, or refer to available mapping.

5. Vegetation Cover on Slope Face

. Indication of the type and extent of vegetation cover (trees, grass).

6. Table Land Drainage and Gullies

. Indication of surface infiltration and run-off over the slope face, which may cause a potential for surface erosion. Describe whether table land drains towards slope and whether drainage/erosion features are present.

7. Previous Landslide History

. Indicates past instability. Visually inspect slope for evidence or indicators of past instability (scarps, tension cracks, slumped ground, bent or bowed or dead trees, leaning structures such as walls etc.).

Toe Erosion

. Recognizes the presence of and potential for continued slope instability. Toe erosion must be eliminated or solved prior to solving slope instability.

The Rating Chart provides a general indication of the stability of a slope. Based on this chart, the level of investigation required, can be assessed. The chart is a guideline or tool only. In all cases, the consequences of slope failure must be carefully considered and may be an over-riding factor. The chart is not intended as a replacement to the judgement of experienced and qualified geotechnical engineers.

The Rating Chart identifies 3 levels of stability and associated investigation requirements. The three levels are:

1. Stable / Site Inspection Only

A rating of 24 or less, suggests stable slope conditions,

- no toe erosion,
- good vegetation cover
- no evidence of past instability
- no structures within ½ (slope height) of the crest

and that no further investigation (beyond visual inspection) is needed. This should be simply confirmed through a visual site inspection and estimate of the slope configuration and slope stratigraphy and drainage (i.e. no measurements). Confirmation of the slope stability should be provided in the form of a letter (signed and sealed with A.P.E.O. stamp) from an experienced and qualified geotechnical engineer. The letter should include a summary of the site inspection observations which could be recorded on a Slope Inspection Form (see enclosed) and should clearly identify;

- slope height and inclination,
- vegetation cover on slope face,
- toe erosion, or surface erosion on slope,
- structures near slope crest or on slope,
- drainage features near slope crest, on slope face, or near slope toe.

2. Slight Potential / Site Inspection, Preliminary Study

A rating between 25-35 suggests the presence of several surface features that could create an unstable slope situation. The stability of the slope should be confirmed through a visual site inspection only, without boreholes. In addition to recording the visual observations outlined in the section above, some direct measurements of site features are required.

The slope height and inclination should be determined either with a hand inclinometer, or by 'breaking slope', or from mapping, or by surveying. As well, more information about the soil stratigraphy of the slope, should be obtained (without drilling boreholes) based on either previous or nearby subsurface investigations, or geologic mapping, or hand augering or test pits to determine shallow depth soil type(s). Measurements should be taken (by hand tape or surveying) of the locations of structures relative to the crest, and other features such as vegetation, past slide features (tension cracks, scarps, slumps, bulges, ridges), and erosion features. If available, historical air photographs should be examined for evidence of any past instability over the long-term. Confirmation of the slope stability should be provided in the form of a detailed report (signed and sealed with A.P.E.O. stamp) from an experienced and qualified geotechnical engineer.

This report will include:

- Slope Inspection Record (Appendix)
- a Site Plan and a Slope Profile indicating the positions of the various measurements taken on site (slope crest, slope toe, location of structures relative to crest, drainage features, erosion features, vegetation cover, indicators of past instability or movements)
- photographs of the site and slope conditions
- a discussion of the site inspection and measurements taken, review of previous information
- preliminary engineering analysis of slope stability (i.e. calculation of Factor of Safety) based on the above information and measurements, but utilizing conservative soil strength parameters and groundwater conditions since boreholes were not carried out.

3. Moderate Potential / Borehole Investigation

A rating of more than 35 suggests a moderate potential for instability. This may result if the slope is either steep, high and/or has several features that could create an unstable slope situation. The stability of the slope should be assessed more precisely through topographic survey of slope configuration and boreholes for slope stratigraphy and penetration resistance tests. Piezometers must be installed in the boreholes and measurements must be taken for groundwater levels. Laboratory testing on the borehole samples must be conducted to measure Basic Index Properties (water contents, unit weights, grain size distribution, Atterberg Limits) described in Appendix D, or other properties as required.

A detailed engineering stability analysis must be conducted to determine if the Factor of Safety for the original slope conditions equals or exceeds a design minimum Factor of Safety. The analysis should be based on the information obtained from the site survey and the borehole information. Historical data such as air photographs should also be reviewed. Confirmation of the slope stability or instability (and the stable slope inclination) should be provided in the form of a detailed report (signed and sealed with A.P.E.O. stamp) from an experienced and qualified geotechnical engineer. This report will include:

- Slope Inspection Record (Appendix)

- a Site Plan and a Slope Profile indicating the positions of the various measurements taken on site (slope crest, slope toe, location of structures relative to crest, drainage features, erosion features, vegetation cover, indicators of past instability or movements)

- photographs of the site and slope conditions

- a discussion of the site inspection and measurements taken, review of previous information

- Borehole logs and piezometer monitoring data

- Laboratory test results (water contents, unit weights, grain size distribution, Atterberg Limits)

- the results of the detailed engineering Stability Analysis (Factors of Safety, failure surfaces, assumed slope data), stabilization alternatives, long-term stable slope inclination.

Where the local geology is well known (exposed stratigraphy or nearby boreholes), the requirement for numbers or depths of boreholes should be reviewed and possibly reduced.

Following is the Slope Stability Rating Chart (see Table 8.1).

TABLE 8.1 - SLOPE STABILITY RATING CHART

Site Location: File No.
Property Owner: Inspection Date:
Inspected By: Weather:

				Rating Value
1.	**SLOPE INCLINATION**			
		degrees	**horiz. : vert.**	
	a)	18 or less	3 : 1 or flatter	0
	b)	18 - 26	2 : 1 to more than 3 : 1	6
	c)	more than 26	steeper than 2 : 1	16
2.	**SOIL STRATIGRAPHY**			
	a)	Shale, Limestone, Granite (Bedrock)		0
	b)	Sand, Gravel		6
	c)	Glacial Till		9
	d)	Clay, Silt		12
	e)	Fill		16
	f)	Leda Clay		24
3.	**SEEPAGE FROM SLOPE FACE**			
	a)	None or Near bottom only		0
	b)	Near mid-slope only		6
	c)	Near crest only or, From several levels		12
4.	**SLOPE HEIGHT**			
	a)	2 m or less		0
	b)	2.1 to 5 m		2
	c)	5.1 to 10 m		4
	d)	more than 10 m		8
5.	**VEGETATION COVER ON SLOPE FACE**			
	a)	Well vegetated; heavy shrubs or forested with mature trees		0
	b)	Light vegetation; Mostly grass, weeds, occasional trees, shrubs		4
	c)	No vegetation, bare		8
6.	**TABLE LAND DRAINAGE**			
	a)	Table land flat, no apparent drainage over slope		0
	b)	Minor drainage over slope, no active erosion		2
	c)	Drainage over slope, active erosion, gullies		4
7.	**PROXIMITY OF WATERCOURSE TO SLOPE TOE**			
	a)	15 metres or more from slope toe		0
	b)	Less than 15 metres from slope toe		6
8.	**PREVIOUS LANDSLIDE ACTIVITY**			
	a)	No		0
	b)	Yes		6

	SLOPE INSTABILITY RATING	RATING VALUES TOTAL	INVESTIGATION REQUIREMENTS
1.	Low potential	< 24	Site inspection only, confirmation, report letter.
2.	Slight potential	25-35	Site inspection and surveying, preliminary study, detailed report.
3.	Moderate potential	> 35	Boreholes, piezometers, lab tests, surveying, detailed report.

TOTAL

NOTES:
a) Choose only one from each category; compare total rating value with above requirements.
b) If there is a water body (stream, creek, river, pond, bay, lake) at the slope toe; the potential for toe erosion and undercutting should be evaluated in detail and, protection provided if required.

8.3 Levels of Investigation

If a site slope is higher than 2 m and steeper than 3 to 1 (horiz. to vert.), an assessment of slope stability is warranted. Three basic levels of investigation have been identified, to be used in evaluating slope stability of sites. The Slope Rating Chart above is an aid to determine the appropriate level of investigation for a site, based on the physical features of the site slopes which are important to slope stability (height, inclination, groundwater, etc.).

The results of carrying out a Level 1 or Level 2 investigation may be that a Level 3 investigation is required. In general terms, the levels of investigation have been chosen on a basic premise that low height or gentle slopes can be analyzed sufficiently by general or observational methods, and that as slopes become higher and steeper more rigorous and intensive methods are required. The amount of field investigation increases with each level as follows,

Level 1 - site visit and inspection by engineer

Level 2 - site visit and inspection, mapping and site survey/measurements of physical features

Level 3 - site visit/inspection, mapping/surveying, borehole drilling.

For purposes of comparison only, the approximate engineering fees (1998 $Cdn.) for evaluating a single house lot on a slope site is estimated as follows;

Level 1	$ 500 - 1000
Level 2	$ 1,000 - 2,000
Level 3	$ 2,500 - 10,000 and up.

Some complex site conditions may result in higher costs for investigation than indicated above.

The following flow chart (see Figure 79) shows the typical steps that are taken in selecting an appropriate level of investigation for slope stability.

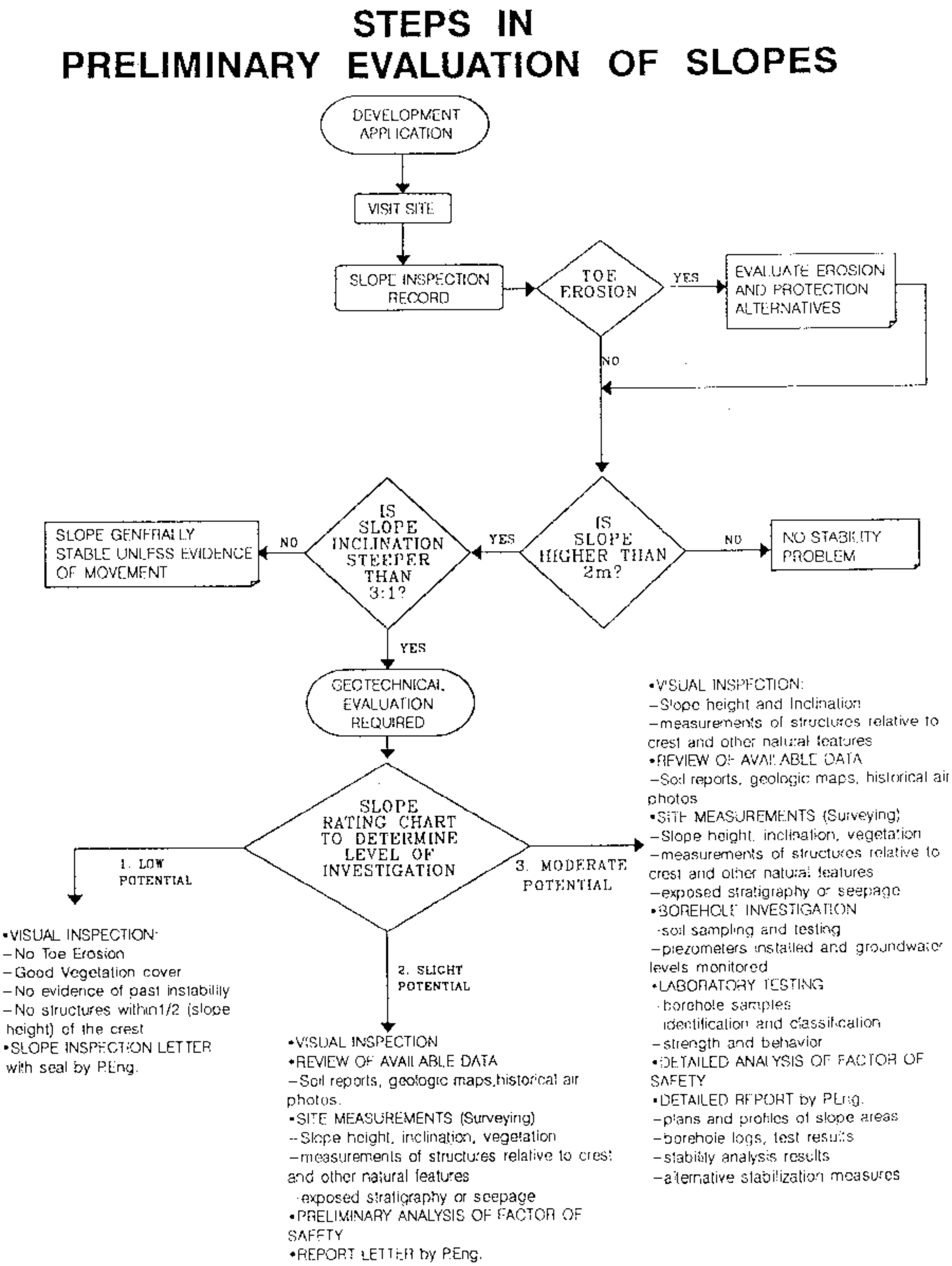


Figure 78 - Preliminary Evaluation Steps

9. GEOTECHNICAL REPORT GUIDELINES

It is important that detailed geotechnical reports on slope stability contain the necessary information to permit the reader to fully evaluate the slope stability and possible consequences of failure. The reports should be as complete as possible in collecting and summarizing all available factual information on a site. The following section describes the typical requirements for detailed geotechnical investigations of slope stability, the general approaches available, and the issues which should be discussed.

9.1 Review of Available Data

Regional geology should be considered at the outset of any slope stability investigation, along with any records of past slope instability situations. MNR geological mapping (bedrock geology and bedrock topography or drift thickness, Quaternary geology (see enclosed Map 2556), and MOE water well records) is available for many areas of the province, including most urbanized centres.

As well, many urbanized areas have had topographic mapping prepared from air photography interpretation and this is often available from the Engineering or Public Works Department in the municipal level government offices. The mapping should preferably be at a scale of 1:500 or 1:1000 in order to show sufficient detail of the slope profile. These government offices sometimes also possess records of historical air photographs which may document conditions of erosion, slope instability, land development, or land filling.

The Metropolitan Toronto Archives (and the University of Toronto, Robarts Library) has such air photographs for the Toronto-centred area which are available from 1947 on almost an annual basis. These air photographs are at a scale of about 1:4800. Conservation Authorities also have files which document past reports of slope failures or erosion.

9.2 Site Inspection and Mapping

As discussed in Section 8, a site inspection is always required when assessing slope stability, which produces an extensive basis of factual information for relatively little cost. A variety of other data including aerial photographs, topographic maps and so on can be used to support the field data.

The completion of the Site Inspection Record from a field investigation is very important because it establishes vital factual information on the slope height, slope inclination, exposed soil stratigraphy (if visible), vegetation cover, structures near the slope, and other important features which are relied on by the stability analysis in attempting to model or simulate the actual forces and strength resistance conditions at a site. A photographic record (still or video) should also be taken of the site slope conditions.

The Site Inspection Record (see next page) has the following components to be recorded about the site. Further description is found on Table 7.4.

- File No.

  record date and time of inspection, including weather conditions and visibility, site accessibility

- Site Location

  describe site location with respect to major roads or regional features; provide sketch

- Watershed

  record name of watershed site is located in

- Property Ownership

  obtain name and address of property owner, and legal description for property; describe current land-use of site and adjacent properties

- Slope Data

  record vertical height of slope from toe to crest; describe slope inclination (horiz. to vert. or angle from horizontal) and shape (also provide sketch at end of report and take photographs), whether slope angle is uniform or composite

- Slope Drainage

  describe locations and amounts of any seepage on the slope face or near the slope crest or toe; note location of any 'piping' if occurring, also provide sketch at end of report and take photographs

- Slope Soil Stratigraphy

  where visible or exposed, describe soil stratigraphy (location, thickness, colour of soil layers) and soil types (sand, clay, rock) if possible, also show on sketch and take photographs

- Water Course Features

  indicate location and proximity of any nearby drainage features or water bodies (marshy ground, swale, channel, gully, springs, stream, creek, river, pond, bay, lake), show on sketch

- Vegetation Cover

  describe location, amount, and types of vegetation cover on the slope (crest, face, toe) and on adjacent properties; show sketches, take photographs; grasses, weeds, shrubs, saplings, trees

- Structures

  describe location, types, and size of any man-made structures on the slope face or near the slope crest or slope toe; show on sketches, take photographs; buildings, retaining walls, fences, roads, stairs, decks, towers, bridges, buried utilities

- Erosion Features
  describe location, types, and size of any erosion features on the slope face or near the slope crest or slope toe; show on sketches, take photographs; bare exposed soil, rills, gully, toe erosion, scour, undercutting, piping

- Slope Slide Features
  describe location, types, and size of any past slope movements on the slope face or near the slope crest or slope toe; show on sketches, take photographs; tension cracks, scarps, slumps, bulges, ridges, bent tree trunks or stands of dead trees

- Comments
  record any other general observations

- Plan View Sketch
  show locations of slope crest, toe, structures, vegetation, stratigraphy, seepage, erosion, water course features

- Profile Sketch
  show slope height, inclination, and shape

TABLE 7.4 - Slope Inspection Record

1. FILE NAME / NO.
INSPECTION DATE (DDMMYY):
WEATHER (circle): ○ sunny ○ partly cloudy ○ cloudy
○ calm ○ breeze ○ windy
○ clear ○ fog ○ rain ○ snow
○ cold ○ cool ○ warm ○ hot
estimated air temperature:
INSPECTED BY (name):

2. SITE LOCATION (describe main roads, features)

SKETCH

3. WATERSHED

4. PROPERTY OWNERSHIP (name, address, phone):

LEGAL DESCRIPTION
Lot
Concession
Township
County

CURRENT LAND USE (circle and describe)
○ vacant - field, bush, woods, forest, wilderness, tundra,

○ passive - recreational parks, golf courses, non-habitable structures, buried utilities, swimming pools,

○ active - habitable structures, residential, commercial, industrial, warehousing and storage,

○ infra-structure or public use - stadiums, hospitals,
schools, bridges, high voltage power lines, waste management sites,

5. SLOPE DATA
HEIGHT ○ 3 - 6 m ○ 6 - 10 m ○ 10 - 15 m ○ 15 - 20 m
○ 20 - 25 m ○ 25 - 30 m ○ > 30 m
estimated height (m):
INCLINATION AND SHAPE
○ 4:1 or flatter ○ up to 3:1 ○ up to 2:1
25 % 14° 33 % 18½° 50 % 26½°

○ up to 1:1 ○ up to ½:1 ○ steeper than ½:1
100 % 45° 200 % 63½° > 63½°

6. SLOPE DRAINAGE (describe)
TOP

FACE

BOTTOM

7. SLOPE SOIL STRATIGRAPHY (describe, positions, thicknesses, types)
TOP

FACE

BOTTOM

8. WATER COURSE FEATURES (circle and describe)
SWALE, CHANNEL

GULLY

STREAM, CREEK, RIVER

POND, BAY, LAKE

SPRINGS

MARSHY GROUND

9. VEGETATION COVER (grasses, weeds, shrubs, saplings, trees)
TOP

FACE

BOTTOM

10. STRUCTURES (buildings, walls, fences, sewers, roads, stairs, decks, towers,)
TOP

FACE

BOTTOM

11. EROSION FEATURES (scour, undercutting, bare areas, piping, rills, gully)
TOP

FACE

BOTTOM

12. SLOPE SLIDE FEATURES (tension cracks, scarps, slumps, bulges, grabens, ridges, bent trees)
TOP

FACE

BOTTOM

13. PLAN SKETCH OF SLOPE

14. PROFILE SKETCH OF SLOPE

9.3 Site Plan and Profile

If there is insufficient existing topographic mapping on a site (at 1:500 scale or better), then detailed topographic surveying will be necessary to establish positions of surface features (slope crest, toe, structures and fences, vegetation and trees, drainage or seepage, scarps, ridges), as well as to measure slope profile (cross-section) or configuration (inclination). The plan should also show the locations of boreholes, auger holes, or test pits. The profile should show the soil stratigraphy (see enclosed examples and Figure 80).

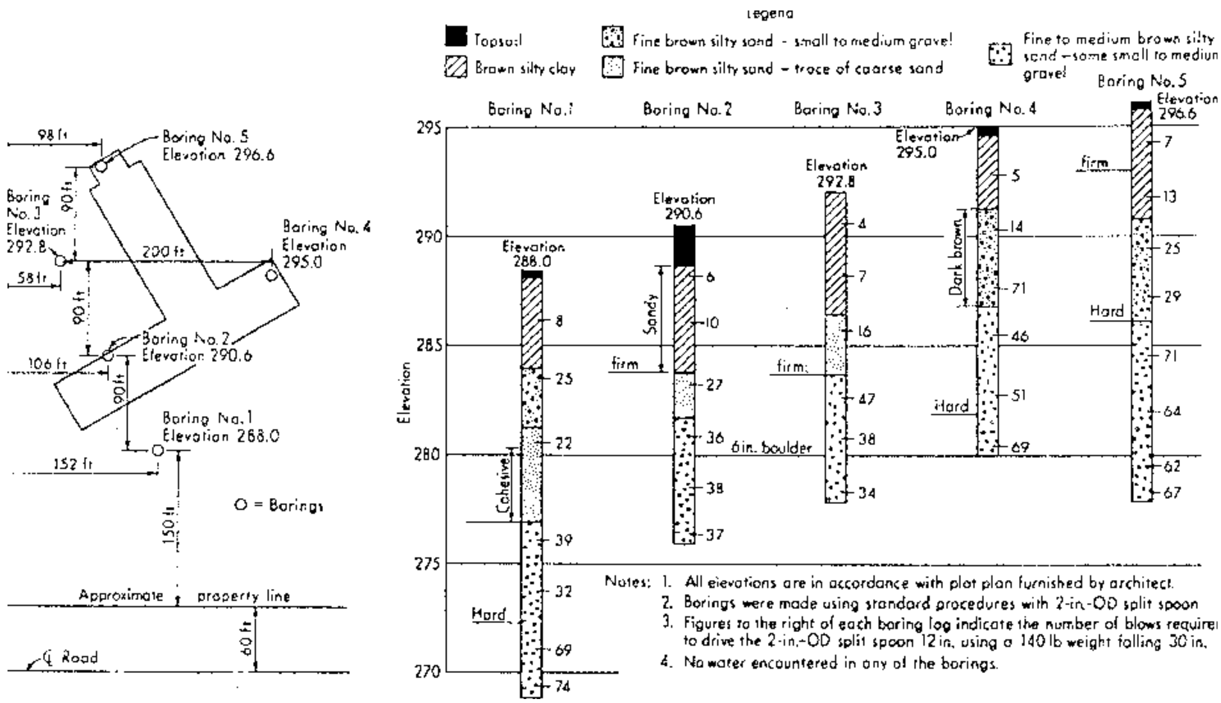


Figure 79 - Plan and Profile

9.4 Boreholes

The subsurface conditions of the slope should be investigated with boreholes and piezometers, to accurately establish the soil types, soil stratigraphy, soil relative density or consistency, groundwater levels, and obtain soil samples. Boreholes are more suitable for investigation than test pits, because excavated test pits are limited by the equipment to maximum depths of 3 to 5 m. Conventional boreholes can be drilled to depths of 30 m or more.

One or two boreholes may be sufficient for many small and simple sites, while many boreholes may be required for larger sites or complex site conditions. For example, boreholes for other engineering projects are often spaced as follows:

Road Pavements and Sewers	50 to 150 m
Buildings	10 to 30 m.

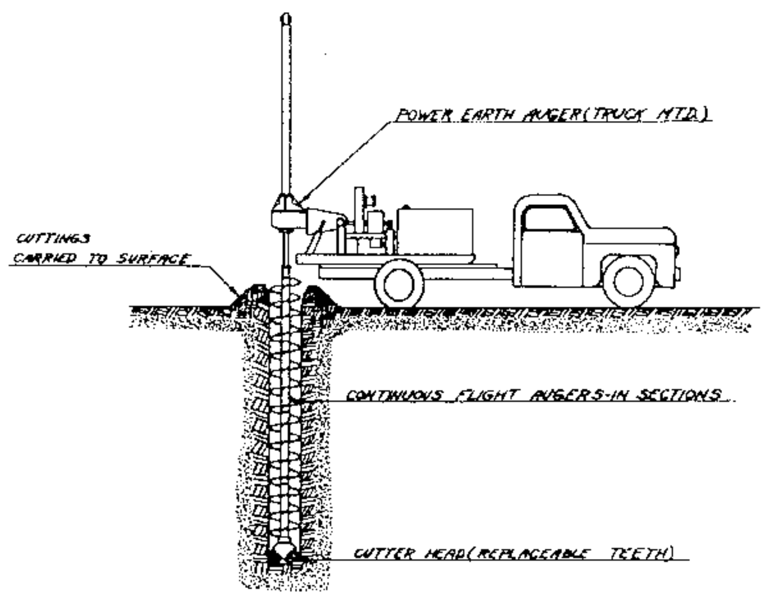
For uniform slope conditions, a reasonable maximum spacing of boreholes along the slope crest would be about 100 m (a closer spacing may be necessary for complex sites).

Generally the ground conditions should be established for the full height of the slope. Some judgement can be used where previous information is available, or where rock or other competent material is found at a shallower depth.

There are several methods of advancing boreholes,

- hand augers
- wash boring
- rotary auger (continuous flight)
  - solid stem
  - hollow stem.

The borehole is usually advanced by continuous flight solid-stem augers (see Figures 81 and 82) which are extracted at each depth interval to permit the insertion of a sampling device or test apparatus. These solid-stem augers typically result in borehole sizes of about 125 mm diameter. Hollow-stem augers (continuous flight) do not require extraction at depth intervals because a central hollow core serves as a casing to support the borehole. Sampling and testing equipment can be inserted through the augers to the bottom of the borehole. In very deep boreholes, the torque required to turn the augers may not be available and other means of borehole advancement are required.



Soil boring using auger method.

Figure 80 - Borehole Drilling

Figure 81 - Photo, Borehole Drill Rig

Another standard method of advancing boreholes (through very slow) is "wash boring" which involves the insertion of pipe casing to support the hole and the use of a chopping bit on drill rods to dislodge the soil at the bottom of the borehole. Water is pumped under pressure through the drill rods and chopping bit to wash out the dislodged soil. The casing is driven to the bottom successively as required.

Portable tripod equipment can be used in difficult access areas (i.e. on slope face), to advance boreholes to moderate depths such as 5 to 10 m. This is very slow work.

Boreholes can also be drilled off-shore in standing water (lake, river, pond, bay) with the aid of a barge or platform to carry the drilling equipment.

Shallow hand auger holes can also be carried out (1 m depths) on steep slopes or in difficult access areas, but these are of limited value due to the shallow depth.

The most common test in the borehole is the Standard Penetration Test (S.P.T.) which consists of driving a standard split-spoon sampler (50 mm diameter, 600 mm long) into the bottom of the borehole with a falling weight of 67 kg dropping over a height of 0.75 m (see Figure 83).

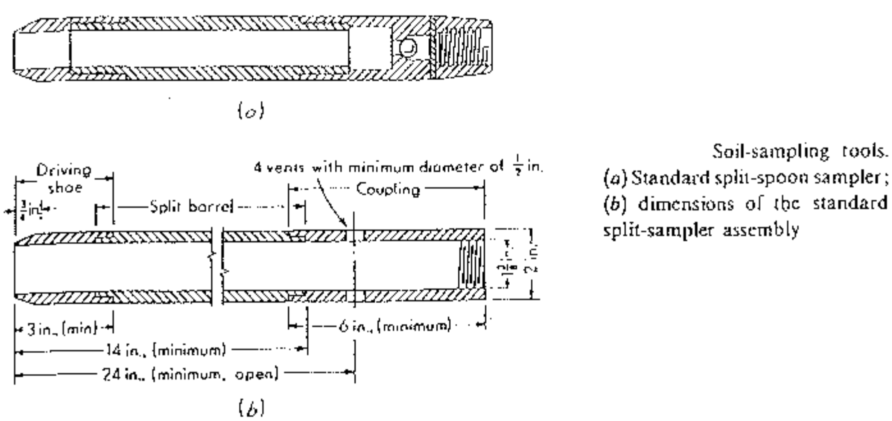


Figure 82 - Standard Penetration Test

This test is repeated at increasing depths as the borehole is augured out in increments of 0.75 to 1.5 m. The sampler penetration into the ground is measured and the number of blows required to obtain penetration increments of successive 150 mm are counted and recorded ("N" value). The "N" value or standard penetration resistance is expressed in blows per 0.3 m penetration. The initial 150 mm penetration is usually disregarded due to possible weakening or disturbance at the borehole bottom.

The measured "N" value provides a relative indication of penetration resistance which has been correlated to other soil properties such as density and shear strength. The Standard Penetration Test is commonly used in almost all ground conditions, except soft clay. The S.P.T. samples are considered disturbed due to the high area ratio of the sampler to the sample diameter.

In soft cohesive soils, thin-walled Shelby tubes are used for the extraction of relatively undisturbed samples(see Figure 84). The field vane apparatus is also used in soft, cohesive soils, in order to obtain shear strength values of the soil. The field vane consists of a vane-like device on the end of drill rods, which is inserted into the soil at the borehole bottom and then turned at the ground surface. The torque required to turn the vane is measured. The measured torque can be related to undrained shear strength based on the shape and size of the field vane.

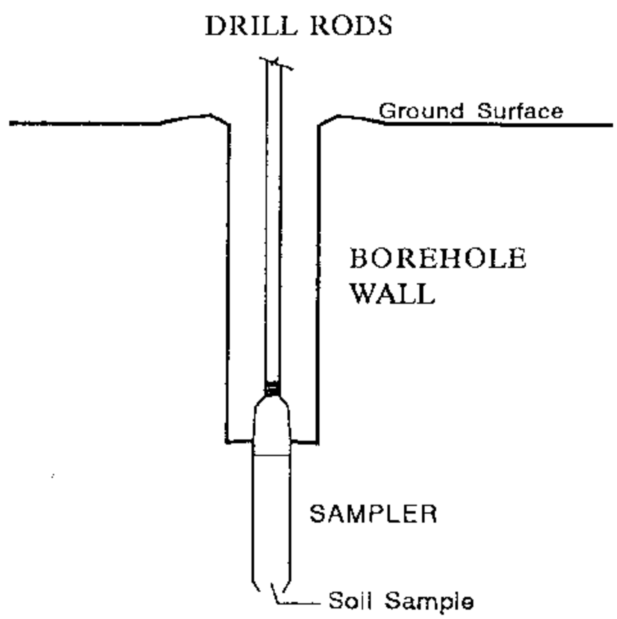


Figure 83 - Shelby Tubes

Other types of penetration resistance testing can be carried out (dynamic cone, static cone) and these are summarized on the following Table 9.1 (ref. Cdn.Fdn.Man.),

TABLE 9.1 - Borehole Test Methods

Type of Test	Type of Soil		Properties Obtainable	Remarks	References
	Best For	Not For			
1. Standard Penetration Test	Sand	Clay	Qualitative evaluation of compactness; comparison of subsoil stratification.	(See Section 4.5.1.1)	1) CSA A119.1 2) ASTM D1586 3) Fletcher (1965) 4) Peck et al (1963) 5) Tavenas (1971) 6) ISSMFE (1977)
2. Dynamic Cone Test	Sand and Gravel	Clay	Qualitative evaluation of compactness; comparison of subsoil stratification.		1) ISSMFE (1977)
3. Static Cone Test	Sand		Continuous evaluation of density and strength of sands and gravel; undrained shear strength in clays.	Test is best suited for design of piles in sand. Tests in clay only reliable with vane tests.	1) Sanglerat (1972) 2) Schmertmann 1970) 3) Ladanyi & Eden (1969) 4) ISSMFE (1977)
4. Plate Bearing Test	Sand		Modulus of subgrade reaction. Ultimate bearing capacity.	Strictly applicable in uniform deposits. Size effects must be considered in other cases.	
5. Vane Test	Clay	Silt Sand Gravel	Undrained shear strength c_u.	Test should be used with care particularly in fissured, varved and highly plastic clays.	1) ASTM D 2573 2) Bjerrum (1972) 3) Aas (1965) 4) Lo (1972) 5) Schmertmann 1975) 6) Lemasson (1976)
6. Pressure - meter Test	Soft rock Sand	-	Ultimate bearing capacity and compressibility	(See Section 4.5.1.3)	1) Menard (1965) 2) Eisenstein (1973) 3) Tavenas (1971) 4) Baguelin 1978)
7. Permeability Test	Sand and Gravel	Clay	Evaluation of co-efficient of permeability	Variable head tests in BH's have limited accuracy. Results reliable to one order of magnitude obtained only from long term large scale pumping tests.	1) Hvorslev (1949) 2) NAVFAC DM7 3) Sherard

The field technician keeps records of the drilling and sampling operations, along with sample descriptions and stratigraphy. The soil samples should be sealed and transported to a geotechnical laboratory for testing of index properties as noted below (see Figure 84).

Class	Sample Quality	Identification	Stratigraphy	Stratification	Organic Content	Grain Size Distribution	Atterberg Limits	Specific Gravity	Water Content	Unit Weight	Permeability	Compressibility	Shear Strength	Note
			Properties that can be measured											
1	Undisturbed	a - Block samples b - Stationary piston sampler 8 cm (3") minimum diameter	+	+	+	+	+	+	+	+	+	+	+	1 - 4 - 6
2	Slightly disturbed	Open thin-walled tube sampler 5 cm (2") minimum diameter	+	+	+	+	+	+	+	+	+	+	+	2 - 3 - 4 - 5 -6
3	Substantially disturbed	Open thick-walled tube sampler such as split-barrel sampler	+	+	+	+	+	+	+					3
4	Disturbed	Random samples collected by auger or in pits	+	+	+	+	+	+						

NOTES

1. Block samples are best when dealing with sensitive, varved or fissured clays. Whenever possible block samples should be taken in such soils.

2. 8 cm (3") diameter stationary piston samples may be impossible to obtain in some materials such as very stiff clays. If shear strength and compressibility of such materials are required they may be determined using class 2 samples but due consideration must be given to the lower quality of such samples.

3. Samples of classes 1b and 2 must be taken with tubes conforming to the following geometric requirements:
   The area ration $I = \frac{D_o^2 - D_i^2}{D_i^2} < 12\%$ where D_o = outside diameter of the tube
   The inside clearance $0.5\% < \frac{D_i - D_e}{D_i} < 1\%$ D_i = inside diameter of the tube
   The angle of the cutting edge must not be greater than 30° D_e = inside diameter of the cutting edge

Figure 84 - Appropriate Lab Tests for Field Samples

Enclosed are samples of field borehole logs and the office report logs (see Figure 85).

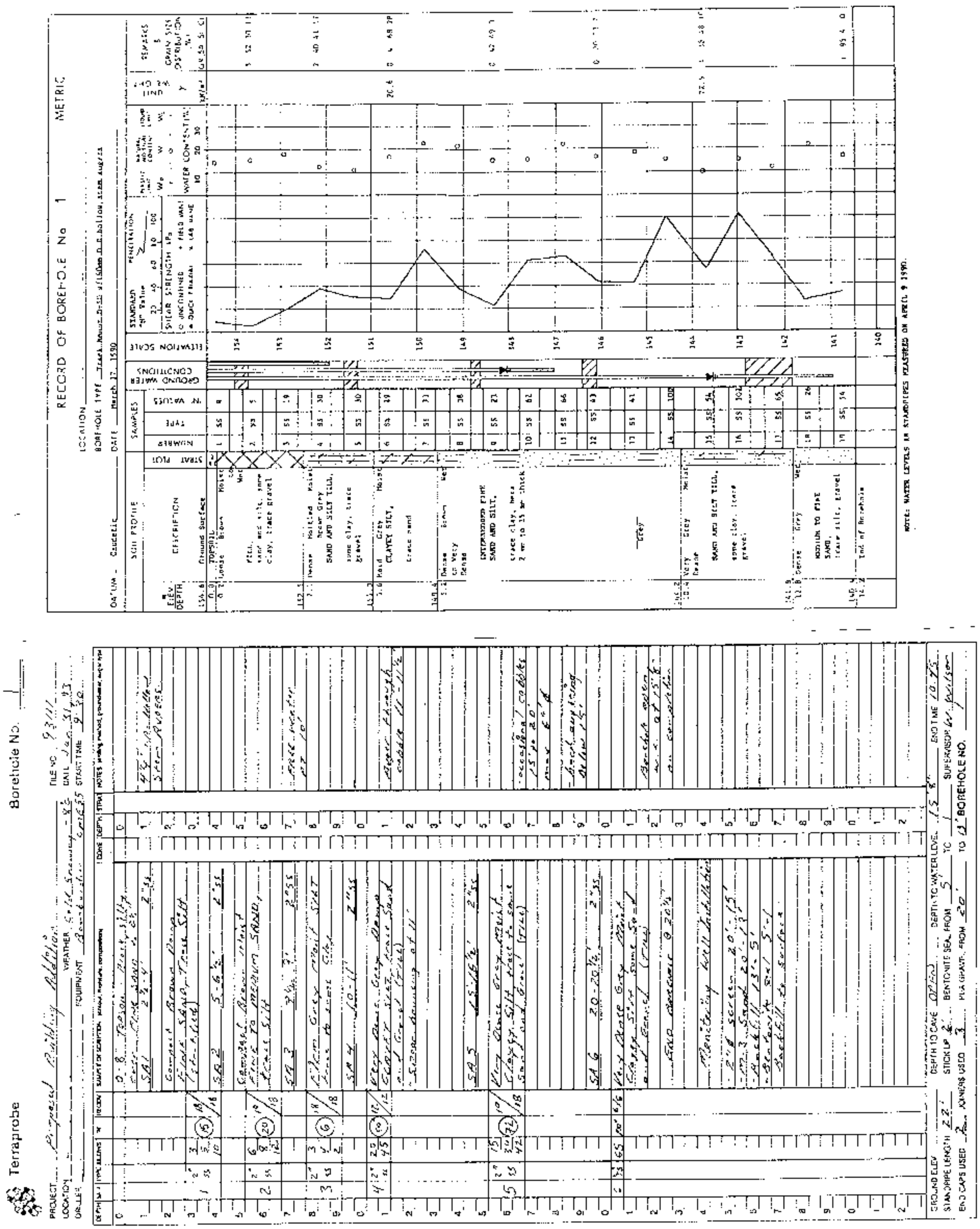


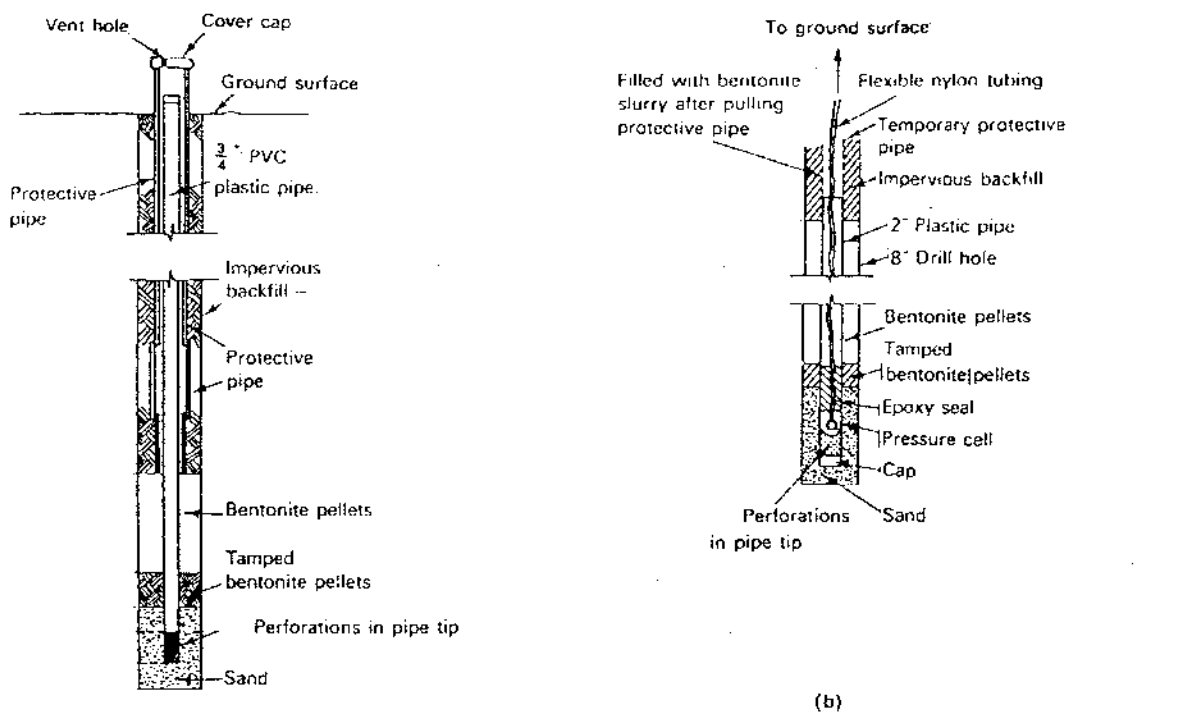
Figure 85 - Borehole Logs

9.5 Piezometers

Groundwater conditions are often measured by standpipe piezometers consisting of hollow plastic pipe or tubing (10 to 50 mm diameter), which are installed in the boreholes on completion of drilling (see Figure 87). Monitoring of groundwater levels is conducted after borehole drilling.

The most common standpipe installation consists of small diameter tubing which extends down to a filtered porous (or perforated) tip that is surrounded with granular material (sand or fine gravel). Groundwater is allowed to enter the standpipe through the filtered porous or perforated tip, and to rise to its static hydrostatic or piezometric level inside the piezometer tubing or piping. The groundwater level inside the tubing can be measured by lowering a calibrated coaxial cable with low electrical current (or other device) down to the water level.

A bentonite clay seal (swelling or expansive clay) is typically provided just above the filtered tip, to ensure that the measured hydrostatic pressure is representative for the level of the piezometer tip (protects against influence from different pressures at different depths. After backfilling the borehole to near the ground surface, an additional bentonite clay seal is usually provided, to protect against surface infiltration down the backfilled borehole.



(a) Illustrations of piezometer types: (a) open tube type for use in permeable soil; (b) pressure cell for use in impermeable soil.

Figure 86 - Piezometers

Other remote monitoring types of piezometers are available, such as pneumatic, or electric. The advantages of standpipe piezometers include inexpensive cost, no de-airing required, no calibration required.

Most standpipes are backfilled with the native soil, when the monitoring after drilling does not extend beyond a few months. For long-term monitoring (many months) the piezometer installations should be grouted to protect against backfill settlement and damage to the tubing or connections.

9.6 Laboratory Testing

In the geotechnical laboratory, the soil samples should all be subject to tactile examination by an experienced engineer who confirms the field descriptions on the borehole log, and who selects representative samples for detailed testing. There are several common laboratory tests to establish index properties of soils. The behaviour of soils types are often estimated on the basis of their measured index properties.

The most common laboratory tests and their recommended testing frequency for samples are:

A. Water contents, all samples

B. Atterberg Limits, cohesive strata

C. Grain size distribution, all strata

D. Soil unit weight, as required

E. Specific gravity, as required

F. Direct shear test, sand strata, as required

G. Triaxial compression test, cohesive strata, as required.

The following is a summary of each of the above laboratory tests, as well as example test results.

9.6.1 Water Content

The water content is a relatively simple measurement which should be undertaken on all borehole soil samples. The water content is defined as the ratio of the weight of water, to the weight of solids in a soil sample, and is often expressed in percent.

The test is usually conducted by weighing a sample in it's natural state, and then drying the sample in a oven until all of the water has evaporated. The sample is then re-weighed, and the weight of the water loss as well as the weight of the dried soil is determined.

The variability and accuracy of the measurement is relevant only to one digit, and should never be reported to more than one decimal point.

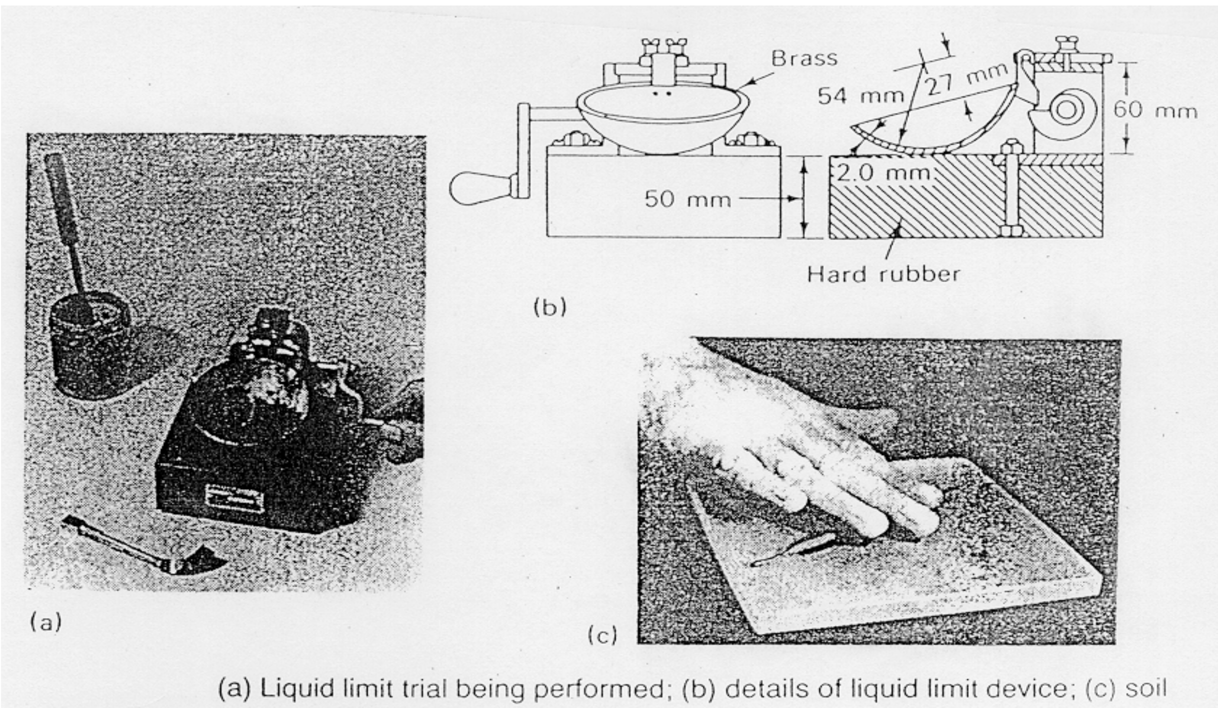
The limits on the water content are indirectly controlled by the void ratio or porosity within the soil mass, that being, the available void space between the individual soil particles. A soil is considered to be "saturated" if all of the void space has been filled with water. In natural environments, the water content of a soil mass may be saturated if natural water infiltration is present, or it may be partially saturated and consisting of air and water in the void space. For comparison, the saturated water content of various soil types are as follows (see Table 9.2).

TABLE 9.2 - Saturated Soil Water Contents

SOIL TYPES	SATURATED WATER CONTENT
sand and gravel	5 - 7 % by weight
sand	8 - 10 %
silt	16 - 20 %
clayey silt	20 - 30 %
clay	30 - 40 %

9.6.2 Atterberg Limits

The Atterberg Limit test is an empirical type test measuring the consistency of fine grained soils. Samples of a soil are mixed to a wide range of water contents and measurements of the water content are taken for ranges of the soil's consistency; 'liquid' or 'plastic'. Atterberg Limits consists of the liquid limit and the plastic limit, each representing a boundary or limit of water content that produces a different behaviour. Various tools and test equipment are used to conduct the Atterberg Limits test (see Figures 88 and 89).



(a) Liquid limit trial being performed; (b) details of liquid limit device; (c) soil thread being rolled to determine plastic limit.

Figure 87 - Atterberg Limits

Figure 88 - Photo, Atterberg Limits

The following Table 9.3 describes the different behaviour for the different limits.

TABLE 9.3 **-** Consistency / Behaviour of Cohesive Soil

Stage	**Description**	**Boundary or Limit**
liquid	slurry, vicious liquid, pea soup to soft butter	
		liquid
plastic	deforms but will not crack, soft butter to stiff putty	
		plastic
semi solid	deforms permanently but cracks, cheese	
		shrinkage
solid	fails completely upon deformation, hard candy	

The liquid limit is defined as the water content at which a pre-shaped groove cut in a moist soil contained in a specifically shaped cup, closes after 25 taps on a hard rubber plate. The plastic limit is the water content at which the soil begins to break apart and crumble when rolled by hand into threads 3 mm in diameter. The shrinkage limit is the water content at which the soil reaches its theoretical minimum volume as it drys out from a saturated condition. The difference between the liquid and plastic limits is termed the plasticity index, which represents the range of water contents through which the soil is in a plastic state.

The Atterberg Limits serve as indexes to other significant properties of the soil which are useful. For example, the liquid limit has been found to be proportional to the compressibility of the soil.

The Atterberg Limits are relevant only to fine grained soils possessing cohesion (i.e. having a significant percentage of clay-size particles), and are most relevant to weaker cohesive soils, as opposed to very stiff to hard cohesive soils. Soil classifications have also been developed based on plasticity (see Figure 90).

Primary letter		Secondary letter	
G:	Gravel	W:	Well graded
S:	Sand	P:	Poorly graded
M:	Silt	M:	With non-plastic fines
C:	Clay	C:	With plastic fines
O:	Organic soil	L:	Of low plasticity (LL < 50)
Pt:	Peat	H:	Of high plasticity (LL > 50)

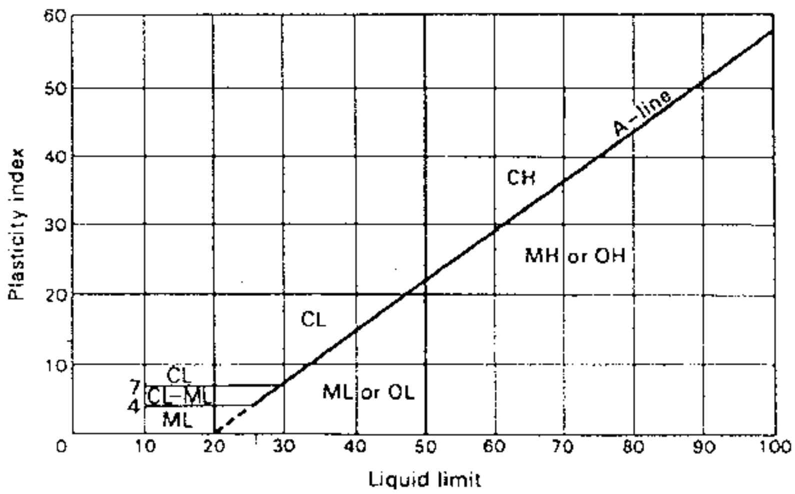


Figure 89 - Plasticity

9.6.3 Grain Size Analysis

There are 2 common methods of grain size analysis, depending on the range of the particle size being analyzed; these are mechanical sieve analysis and, hydrometer analysis.

Cohesionless soils, those being comprised of fine sand, sand, gravel, and coarser particle sizes, are commonly sieved mechanically, through a graduated series of sieves. Each sieve has a finer mesh than the one proceeding it (see Figures 91 and 92). The weight of the sample retained on each sieve is compared to the total weight as a percentage.

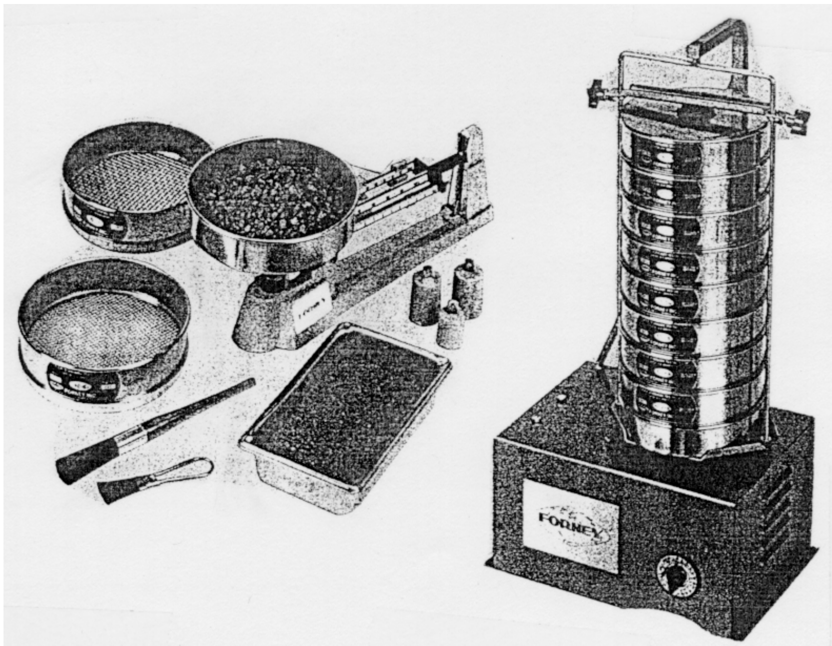


Figure 90 - Mechanical Sieves

Figure 91 - Photo, Mechanical Sieves

Grain size distributions for fine grained soils (finer than fine sand) are analyzed through a sedimentation experiment. The sedimentation procedure is referred to as a hydrometer analysis.

In a hydrometer analysis, fine grained soil is first dried, and broken down to it's natural particle size. The fine powder is then saturated and treated with a chemical deflocculating agent which will break up clumps, or flocs, of fine soil particles, and keep individual soil particles separate.

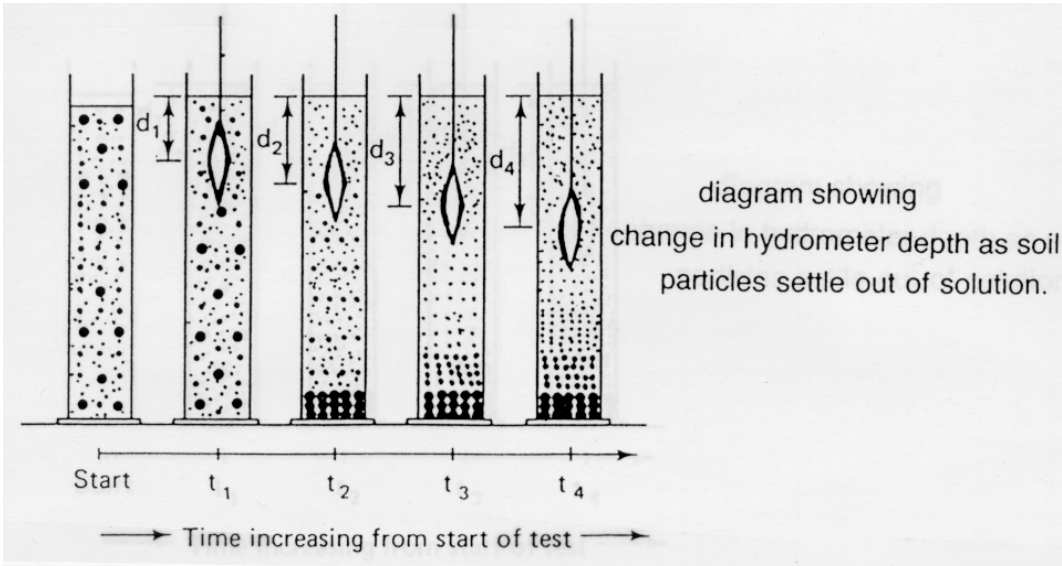


Figure 92 - Hydrometer Analysis

After treatment with the deflocculating agent, water is added to the sample to produce a fixed volume and the mixture is shaken into suspension thoroughly. The procedure is conducted using a glass cylinder, which is then set upright and a hydrometer is used to measure the density of the suspension (see Figures 93 and 94). Measurements of the suspension density, or specific gravity, are taken at time intervals which have been calibrated to various particle sizes falling through the water suspension. This permits similar calculation of weight of particle sizes as a percent of the total sample.

Figure 93 - Photo, Hydrometer Analysis

Both the sieve analysis and hydrometer analysis may be required for soil types that includes sand and gravel as well as silt and clay sizes. The grain size distribution is often plotted on a semi-log graph, represented by gradation curves of particle size versus percentage by weight of the total soil sample (see Figure 94).

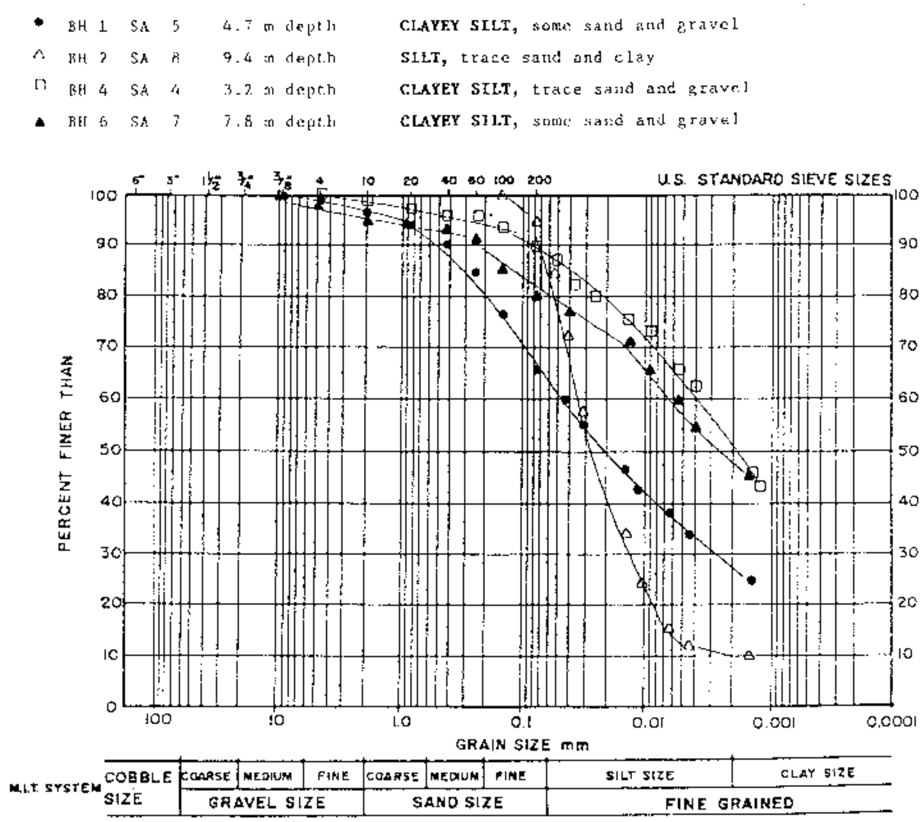


Figure 94 - Grain Size Gradation Graph

Grain size distributions may range from uniform (all of a similar particle size), to well graded (a range for various soil particle sizes), to gap graded, which is a combination of two or more uniform soil sizes.

The grain size distribution of a soil type helps to classify the soil and gives insight as to other properties of the soil such as permeability and density, as well as shear strength.

9.6.4 Soil Unit Weight

The soil unit weight is a direct measurement of the unit weight or density of the whole soil sample (soil particles, moisture, voids). This measurement is usually taken on uniformly shaped soil samples, either directly from split spoons or from shelby tubes samples obtained from boreholes. Samples are typically cylinder shaped, of a relatively constant diameter, and of different lengths. The unit weight can be measured directly by a physical measurement of the dimensions and weighing the sample and obtaining a unit weight, or by waterproofing the sample and weighing it in air and weighing it submerged in water to get a unit weight.

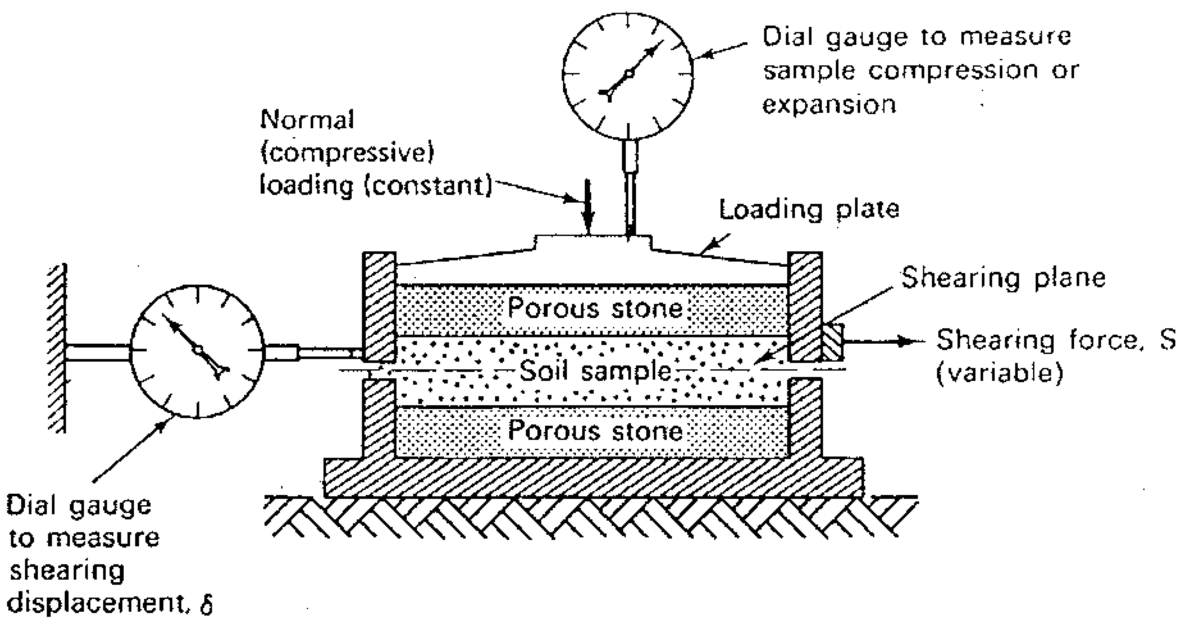
For most of the soil types, the range of unit weight is relatively narrow, between about 17 and 20 kN / m^3. The density will vary, partly dependant on the compactness or consistency of the soil mass, as well as the water content of the soil mass, and specific gravity of the individual soil particles.

9.6.5 Specific Gravity

The specific gravity test is a measurement of the unit weight or density of the individual soil particles rather than the aggregate soil mass. The individual particles are often made up of other minerals such as calcite and quartz. The specific has been well established for various soil types, and has been found to be generally about 2.65 to 2.67 for cohesionless sandy type soils, and can vary from about 2.68 to 2.72 for clay mineral type soils.

9.6.6 Direct Shear Test

Direct shear tests are usually carried out on cohesionless soils, such as sands or silts. The tests are normally undertaken in dry conditions from dry samples and consists of placing and compacting the soil mass in thin layers into a square mould, placing a predetermined confining weight onto the sample, and measuring subsequent displacement. The mould is split in the centre so that it can be sheared along a constant plane, and measurements of the shearing resistance and deformation rate are obtained (see Figure 95).



Schematic diagram of direct shear apparatus

Figure 95 - Direct Shear Test

The test permits the testing of the shear stress at different levels of confining stress to obtain a straight line or slight curve referred to as a Mohr-Coulomb plot. This plot of shear stress versus confining stress, or normal effective stress, permits the direct measurement of the angle of internal friction phi, ϕ'.

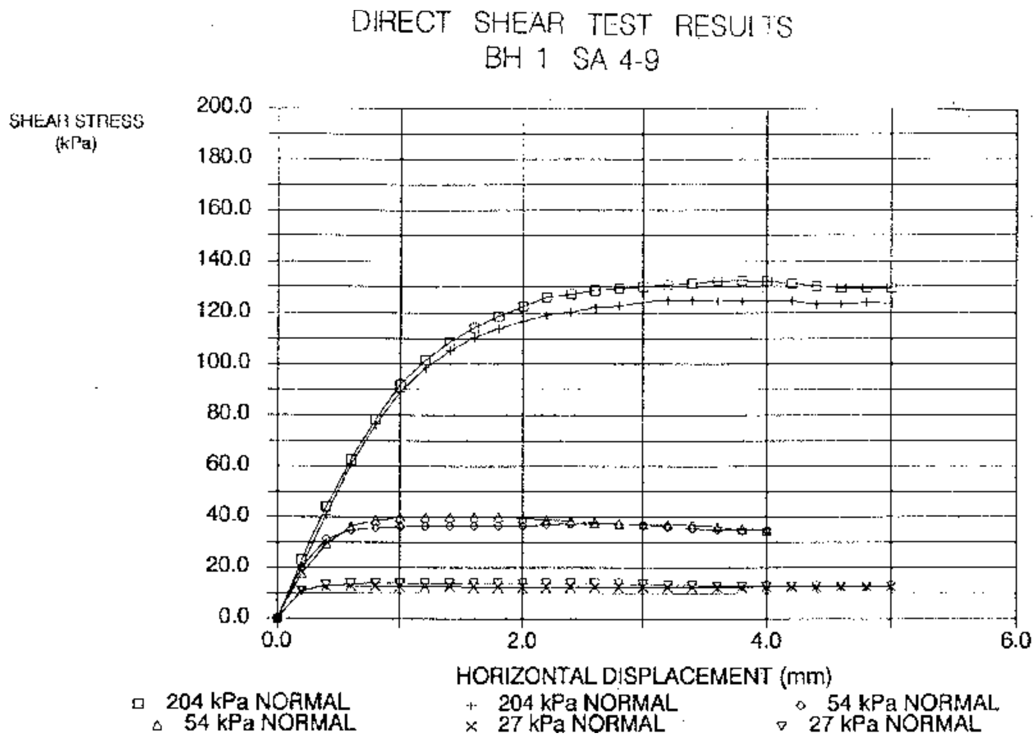


Figure 96 - Direct Shear Test Results

This test provides a general indication of the angle of internal friction for a soil type. In some instances, such as very dense soils, it is difficult to reproduce the level of density in the preparation of the sample during the test. For this reason, the measurement of the true friction angle may not be possible due to the inadequacies of the test equipment and methods to simulate the field conditions. A general trend of the angle of internal friction in the higher density or confining stress levels can be obtained by conducting a number of tests at increasing confining levels (see Figure 96). This test method usually undertaken only for complex problems, with the angle of internal friction more commonly estimated on the basis of borehole standard penetration test results.

9.6.7 Triaxial Compression Tests

The 'effective stress' shear strength parameters of ϕ' and c' for cohesive soils are commonly measured in the triaxial apparatus. The triaxial test is conducted on relatively undisturbed soil samples obtained from the field using thin walled Shelby tubes. This limits the triaxial testing to soils which can be sampled using Shelby tubes. Only firm to stiff or softer cohesive soils can be tested. Very stiff to hard consistency soils can not be sampled using shelby tubes due to their dense nature.

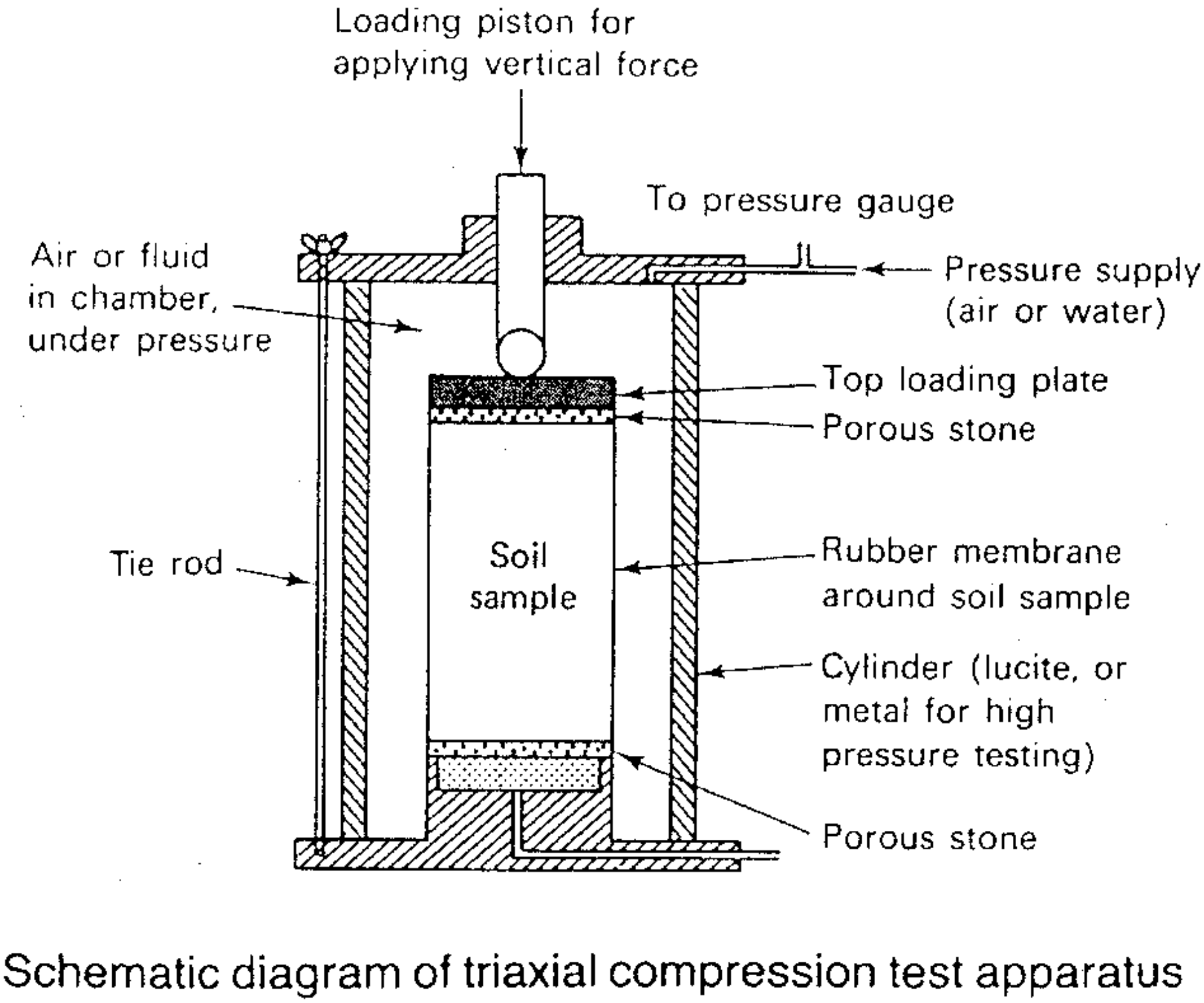


Figure 97 - Triaxial Test Apparatus

Properties for very stiff to hard soil types are often estimated on the basis of standard penetration test results or on previous triaxial testing on soils of a softer or less dense consistency.

The test consists of the extrusion of a cylindrical shaped sample from a Shelby Tube, and enveloping the sample with a rubber membrane. Permeable platens are placed at each end of the sample, to permit pore water drainage, and the application of loads to the sample. The entire sample with rubber membrane and platens (see Figure 97) is placed in a plastic cell which can be filled with fluid to apply a confining pressure to the entire sample during axial loading.

This configuration permits the saturation of samples which are not fully saturated. Saturation is required to simulate the weakest possible soil strengths. This test also permits the measurement of pore water pressure so that the drained values for shear strength and internal friction can be measured. The triaxial test can typically take several days to complete, while the direct shear test can usually be completed in one day (see Figure 98).

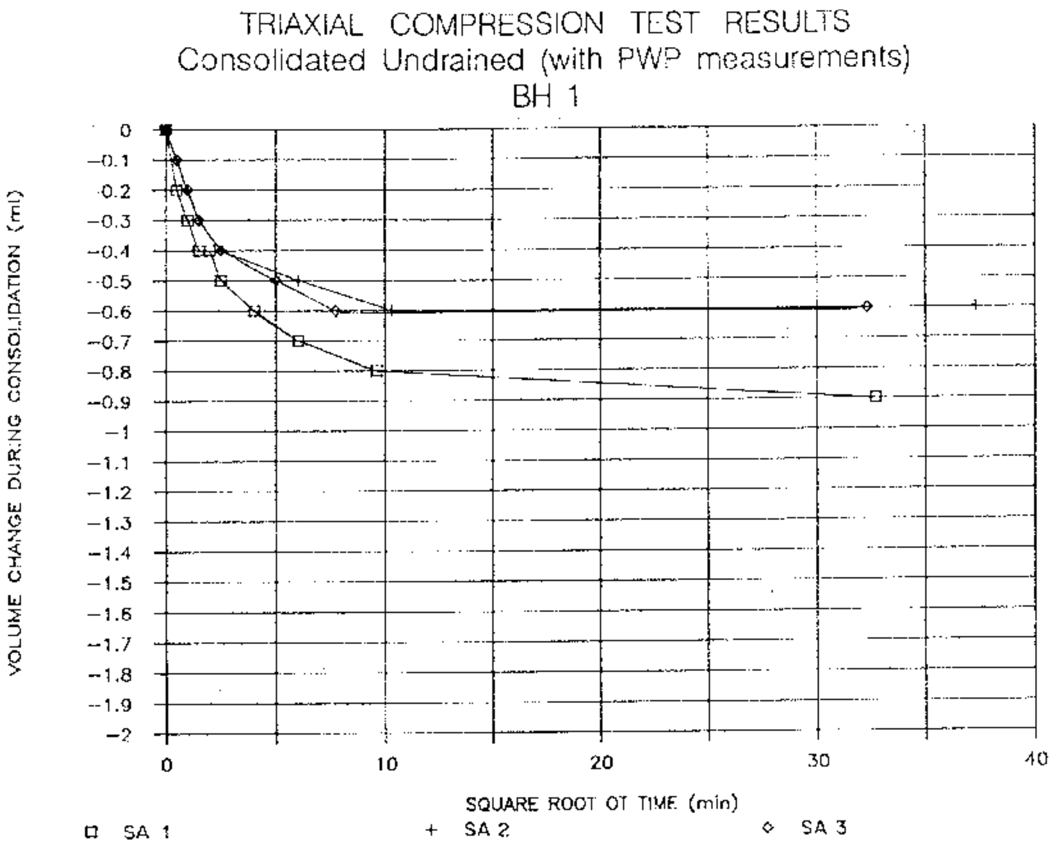


Figure 98 - Triaxial Test Results

9.7 Engineering Analysis

The engineering analysis should be conducted with a recognized method (Bishop's, Janbu, Morgenstern-Price, Spencer, Sarma) in accordance with the guidelines provided in the previous Section 6. The geotechnical report should provide details on the analysis method, the model basis and all assumptions made, and the extent of calculations with an overall summary. Both in the report text and on suitable figures or drawings the following basic information and discussion should be provided,

a) slope height, slope inclination,

b) location of structures near slope,

c) assessment of erosion risks,

d) soil stratigraphy and strength,

e) groundwater conditions and drainage,

f) vegetation cover and species,

g) Factor of Safety calculations,

h) potential causes of instability,

i) alternative slope stabilization methods, and comparison of benefits,

j) discussion of erosion on or near the site; locations, extent, severity, rates, suitable protection alternatives

k) discussion of potential impacts on surrounding properties

l) if required, discussion of cost-benefit analysis of stabilization measures including:

- 'do nothing'
- partial stabilization
- full stabilization

m) long-term stable slope crest position and inclination, based on engineering analysis.

Following is a suggested outline for Terms of Reference as seen in Figure 99 for a detailed geotechnical investigation and report on slope stability (Level 3).

SLOPE STABILITY STUDY
GEOTECHNICAL TERMS OF REFERENCE

TABLE OF CONTENTS

1. PURPOSE

The purpose of the slope stability study for (site location): ____________ is to review and determine the long term stable slope inclination and slope crest position for the section of slope along the (river /lake /pit). The study will also evaluate various alternative stabilization measures which should be considered by the proponent. The study will collect and review existing available information for the existing slope conditions as well as analyzes the proposed changes in land use and long term conditions in the slope area.

The study will review, update, and expand the factual information data base as to the causes, effects, extent and associated hazard in connection with erosion or slope instability along the study area. As well as reviewing various alternative solutions, the study will recommend a optimum solution and will develop a final design for stabilization and protection of the sloped areas.

2. STUDY AREA LOCATION

The study area is located:

The slope area to be considered has a length of about m and the condition and land use on neighbouring properties should also be considered (see Figure for location).

3. BACKGROUND

The study area is comprised primarily of (residential neighbourhood/industrial land/park) and is located along . Based on existing information, the site area has been subject to past erosion or slope instability.

4. SCOPE OF WORK

The study should be carried out as three general sections,

1. A geotechnical site investigation.
2. Engineering analysis and Preliminary design.
3. Recommended stabilization works and final design recommendations.

4.1 Geotechnical Site Investigation

This section of the study includes the following:

a. Review existing air photography for the site as well as any previous reports or mapping carried out in the site vicinity.

Figure 99 - Geotechnical Terms of Reference

b. Conduct field mapping of the existing conditions of the slope area and record for reporting the general slope conditions including height, inclination, seepage, vegetation, bare areas, structures, erosion features, filling, and any visible stratigraphy. Evaluate any indicators of slope movement including scarps, tilted structures or trees or irregular topography. Evaluate surface drainage conditions.

c. Boreholes should be drilled from near the slope crest to the full depth to the bottom of the slope. At least one borehole should be drilled for small sites and boreholes should be spaced at about 100 m along the slope or closer if site conditions require. The boreholes should include sampling at depth intervals of not more than 1.5 m. The sampling should consist of Standard Penetration Tests or Shelby Tubes if appropriate including field vanes. On completion of drilling and sampling of the boreholes, one or more piezometers should be installed in the borehole and backfilled to permit monitoring of ground water levels at various depths within the borehole. Bentonite clay seal should be provided near the ground surface to prevent excessive infiltration into the piezometer installation. Protective caps and concrete may be advisable at the ground surface to protect against vandalism. The horizontal location and the vertical elevation of the ground surface at the borehole locations shall be surveyed. Where possible and if accessible several shallow hand auger probes should be put down through the slope face to confirm shallow soil conditions on the slope face as well as along the slope toe.

d. The borehole samples shall be tested in a GEOTECHNICAL laboratory for basic index properties as well as any additional appropriate tests. These will include water content, Atterberg Limits, grain size distribution, unconfined compressive strength, direct shear tests, over triaxial compression test.

e. Several representative slope cross sections should be surveyed to accurately determine the slope profile; inclination and height. On a site plan of the study area, a report of the surveyed slope inclinations and height shall be summarized along with the results of the visual mapping carried out including vegetation cover, bare areas, erosion or slope instability features, structures, slope inclinations, seepage, drainage and borehole locations.

f. Prepare detailed borehole logs summarizing the soil stratigraphy, field test data, laboratory test data, piezometer installation, ground water levels, site plans and profiles showing slope crest position, slope toe position and ground water levels.

4.2 Geotechnical Laboratory Testing

Promptly after transportation of the borehole samples (jars and shelby tubes) to the geotechnical laboratory, a testing schedule shall be prepared. Water contents shall be measured for all samples. Representative samples shall be tested for basic index properties;

- grain size distribution
- Atterberg Limits, if cohesive
- unit weight
- specific gravity

Figure 100 - Geotechnical Terms of Reference

- Direct Shear Test, if cohesive
- Triaxial Compression, Consolidated Undrained with Porewater Pressure Measurements; for undisturbed cohesive soils, stiff or softer consistency.

4.3 Section 1 - Engineering Analysis and Preliminary Design

In this section an engineering analysis should be carried out of the existing Factor of Safety as well as the Factor of Safety for different stages of the proposed development or work including short term stability and long term stability. An accepted slope stability analysis method shall be used and details should be provided of the basis, and assumptions, and background for the method. Engineering analysis of slope stability shall include incorporation of the information from mapping, surveying, borehole drilling, laboratory testing, and previous available information, and ground water monitoring. The various special information will be used to model or simulate the site conditions on which to carry out calculate Factor of Safety and sensitivity analysis to the different factors.

The analysis shall be carried out to determine the minimum Factors of Safety for existing conditions, as well as conditions in the long term, and the minimum Factors of Safety for possible alternatives or solutions for stabilization. Other external factors should also be considered such as potential for erosion or other phenomena. The study should include consideration of various levels of solution including the following options,

a) Do nothing and allow for self stabilization to occur by the slope and environmental factors,

b) Two partial stabilization to reduce or minimize the amount of addition future slope movement,

c) Or be full stabilization to preserve and protect existing conditions and minimize loss of property.

Various types or methods can be used in consideration in either of the above three options or solutions. For each consideration prepare and present a rough concept sketch or drawing and short description of the method and benefits. The report to be prepared at this stage of all the findings and considerations of alternatives and solution shall include description of the site conditions, mapping of field work conducted, the results of the field work, description of the soil stratigraphy and soil types, ground water levels, laboratory tests, results and surveying information and summary of previous available information. The model for slope stability analysis should be presented on a diagram as well as discussed and the results of the Factor of Safety calculation shall be presented in table form and discussed as well. The discussion should include the Factor of Safety for all conditions including temporary short term conditions, existing conditions, construction conditions, and long term conditions. The site conditions and analysis results shall be shown on site plans and profiles.

The final report will also include comments and recommendations for construction procedure including possible set backs or erosion protection. The engineering analysis should also include consideration for construction accessibility, aesthetics, environmental impacts, future maintenance, MNR Policy, safety. The final report will also discuss the potential failure mechanism or failure modes as well as the extent of susceptible areas. Hazard consideration to existing buildings dwelling or structures should be clearly be identified as well as criteria for risk assessment.

Figure 100 Geotechnical Terms of Reference Continued

9.8 Long Term Monitoring

For site conditions where safety may be critical and ground movements sensitive, long-term monitoring may be appropriate to assist in warning of dangerous ground movements. Instrumentation can be installed to permit monitoring of slope (ground) movements. This monitoring can be undertaken by measuring the horizontal position of surface features on or near a slope, with respect to bench marks or datum points that are located well outside the possible areas of movement. This can be accomplished by tape measurements or optical survey instruments such as electronic distance measurements (EDM) and lasers.

As well, tilt meters can be used to detect tilt or rotation of fixed points. These tilt meters are usually utilized in rock slope environments.

Monitoring of ground movements can assist in accurately defining the actual mode of failure, thereby permitting the most appropriate modelling for analysis and design of stabilization. Further, it is common that very small ground movements precede larger movements (i.e., tension cracks, bulging, creep, etc.) and therefore monitoring may provide a means of warning of a larger ground movement. This warning may allow evacuation or stabilization prior to a catastrophe. Slope inclinometers are commonly used for the monitoring of slope movements.

Slope inclinometers consist of the installation of a grooved casing in a borehole, to permit the insertion of an electronic probe that can measure small changes in inclination of the casing. They are regularly used to monitor the performance of slopes, retaining walls, sheet piling and shoring (see Figure 101).

Should ground movements occur, they would cause tilting or deformation of the casing, that can be measured by the sensor probe. Successive measurements over time permits the detection of even small ground movements around the casing. The casing is available in aluminium or plastic and is grouted into the borehole at the completion of drilling.

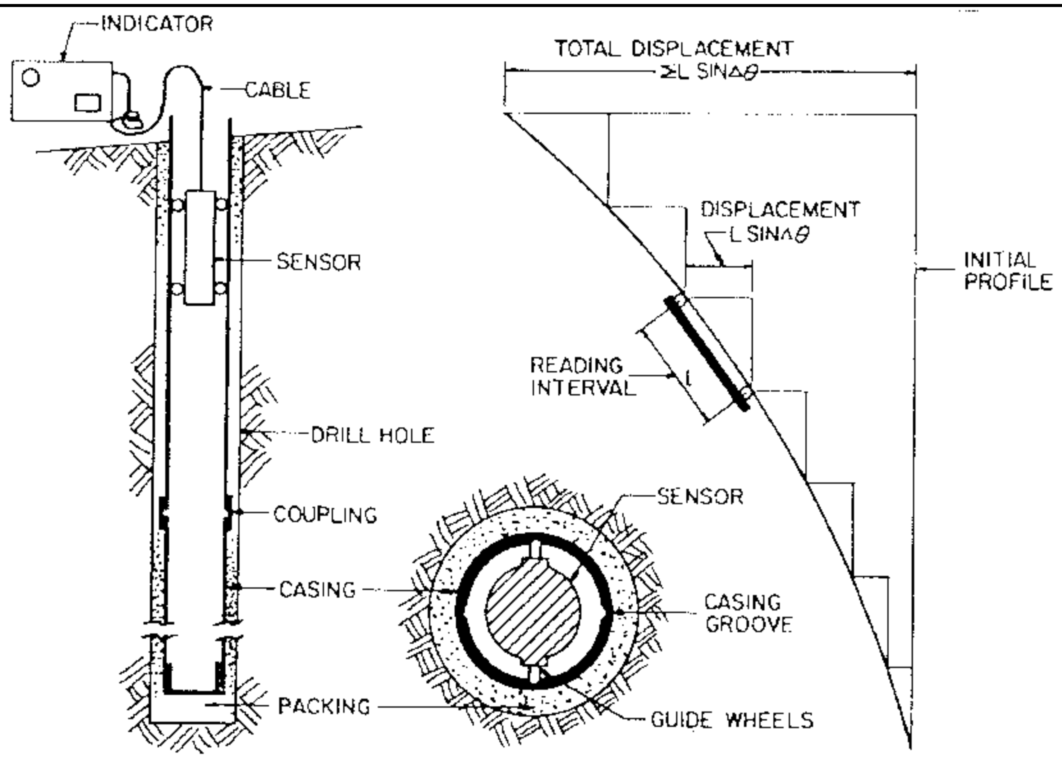


Figure 101 - Slope Inclinometer

10. POSSIBLE RESPONSES TO UNSTABLE SLOPE CONDITIONS

The following section describes various alternatives which can be undertaken to improve the stability of a slope. This section does not describe toe erosion protection measures which may also be necessary if water flow (streams, creeks, rivers) or wave action (lakes, bays, ponds) can affect the slope toe.

If a slope is not stable, there are 3 general approaches to stabilization;

a) do nothing - stay away from the slope and permit self-stabilization through slope failure or movement,

b) stabilize the slope by reducing the forces tending to cause instability,

c) stabilize the slope by increasing the forces resisting instability.

There are a wide variety of methods of slope stabilization. The most suitable may be governed by criteria such as site access, costs, or aesthetics. The common causes of insufficient stability which are usually dealt with, are summarized as follows;

- the slope is too high or steep for the soil stratigraphy, or
- the soils are too weak to adequately support the current profile, or
- the groundwater levels are too high.

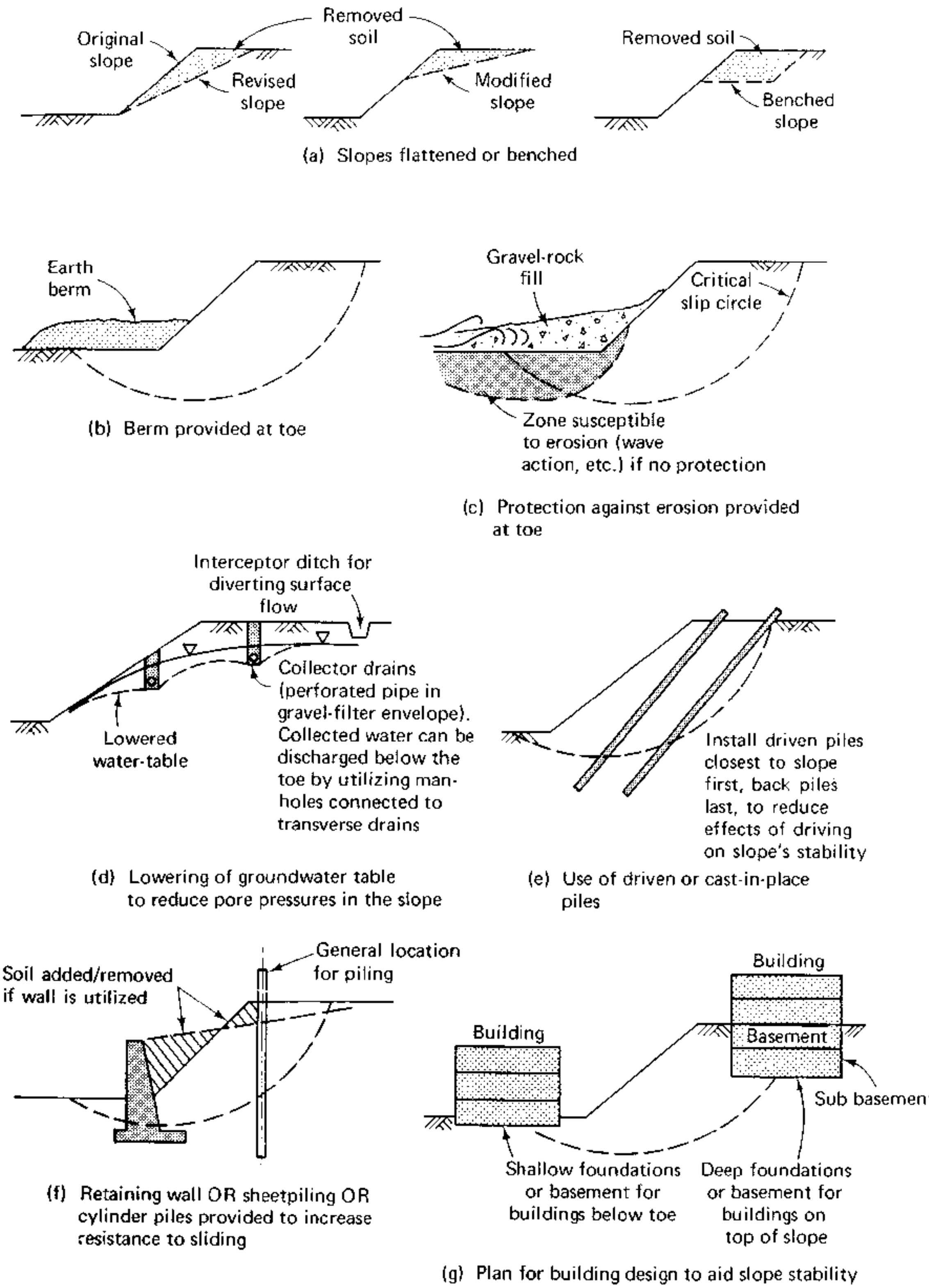
10.1 Altering Slope Geometry

A slope can be re-graded to alter its geometry and improve stability by (see Figure 102 a,b,c);

a) grading to a uniform flatter inclination; most effective for stabilizing shallow slides (less effective against deep-seated slides), or

b) constructing a toe berm or bench; most effective against deep-seated slides, or

c) by reducing the slope height; most effective against deep-seated slides but often not practical.



Methods to improve and protect slope stability.

Figure 102 - Regrading to Improve Stability

Benching of a slope face (regular short flat areas parallel to the slope) is effective only in controlling surface run-off or to provide access across a slope. It generally has little effect on overall stability.

Lightweight fill such as polystyrene board, sawdust, slag, or cinders can be used to replace heavier existing earth and thereby reduce the forces tending to cause movement.

10.1.1 Filling Against Slopes

Stabilizing a slope by filling against it, can be undertaken in several manners (see Figure 103);

- using sandy soil material (cohesionless) material, or
- using cohesive or fine-grained soil material which is different from the pit slope material

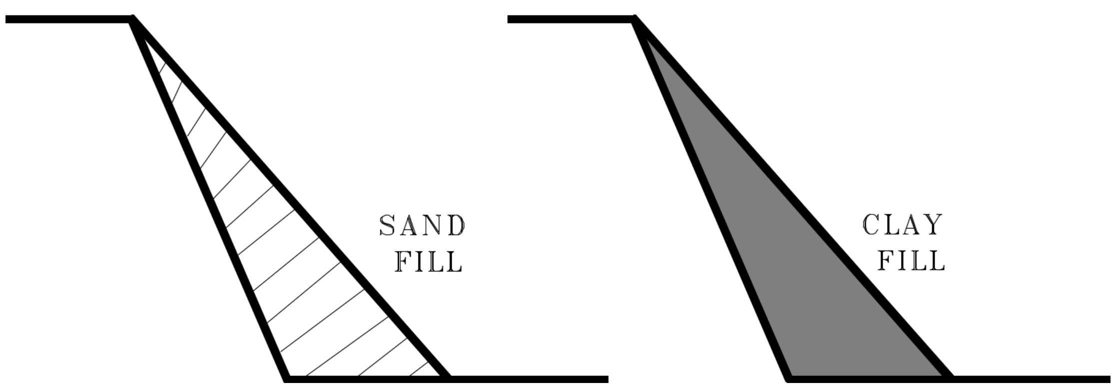


Figure 103 - Fill Types

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When filling against a slope, there is often concern that the fill material might block seepage from the underlying natural slope face. Blockage of seepage could lead to build up of water pressure behind the fill and potential instability of the filled slope.

Slopes comprised of massive sand deposits, are generally well drained and seepage from the slope face is not likely. The groundwater table in massive granular deposits is relatively flat and in a steady state condition with little gradient of flow on a local scale (unless the groundwater regime is disturbed by pumping). Infiltration will tend to be vertical due to gravity, with no horizontal flow. Stratified soils may have layers that are water-bearing or impermeable layers, and precautions may be necessary to permit drainage to occur.

Sandy fill generally has good drainage characteristics and there should be little concern with blocking drainage from the slope. Cohesive or fine-grained fill (silt, clay) may block slope drainage if it is present (not likely), and a drainage layer of fill may be required to allow the seepage to drain without adversely affecting the slope stability (see Figure 104).

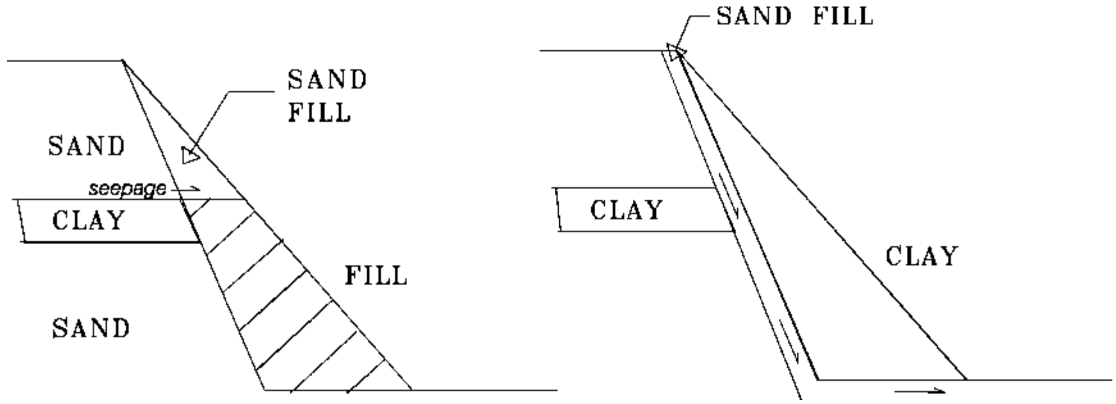


Figure 104 - Slope Filling and Drainage

While the filling must begin from the bottom upwards, the method of placement can be seen in Figure 105 either by,

- transporting the fill to the slope bottom with equipment, and placing it in layers; the equipment would build up the fill height by travelling on the fill material, stable slope inclinations of 2 to 1 or flatter can be achieved due to the compaction that occurs from the placement and spreading traffic,

- transporting the fill to the slope crest and top-dumping the fill over the slope crest down to the lower slope; the fill may not be as well compacted from this method and stable slope inclinations of 2 to 1 or flatter are recommended for cohesionless sand fill, and 3 to 1 or flatter for cohesive or fine-grained fill (clay, silt).

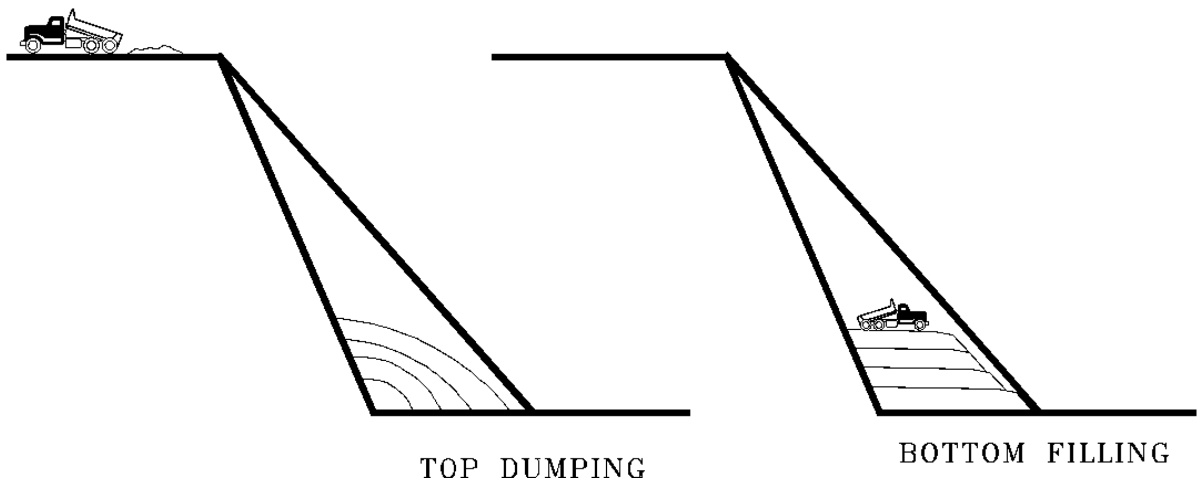


Figure 105 - Filling Methods

If top-dumping is undertaken from the slope crest, precautions should be taken to ensure that equipment and material weight near the slope crest does not result in a slope slide that could affect the equipment and any manpower. Fill should not be stockpiled within 10 m of the slope crest, unless it is to be immediately pushed over the slope crest during top dumping. Most of the equipment and fill materials should be kept several metres back (Ministry of Labour requires min. 3.66 m set-back for equipment) from the slope crest.

Assuming the fill material will be transported to the slope crest by dump trucks, the vehicles should be guided to the slope crest (in reverse), very carefully and slowly to permit discharge of the fill from the truck to within the 3.66 m set-back from the slope crest. Wheeled vehicles should not be allowed to travel regularly within 5 m of the slope crest. After dumping the fill near the slope crest, the final push of fill over the slope crest should be carried out by tracked bulldozers in order to prevent concentrated point loads (wheels) from near the slope crest.

Additional precautions should be taken by providing a 'spotter' during the top dumping while equipment is travelling close to the slope crest (ie. when dozers are pushing fill over crest). The 'spotter' should monitor the slope face and slope crest areas for any warning of potential slope slides or ground movements, so that equipment and men can be evacuated quickly from the slope crest area if required. Pushing fill over the slope crest will likely cause some minor crest loss due to the disturbance.

10.2 Slope Drainage

Drainage improvements can be effective in stabilization of granular soil slopes (permeable soils, sands) where seepage is a problem. When dealing with pit sites (except for erosion control) drainage improvements are seldom required since the granular soils are usually well draining with no horizontal seepage (unless groundwater is perched on impermeable layers).

Decreases in groundwater levels or pore water pressures, (see Figure 102 d) result in increased soil shear strength and increased Factor of Safety. Various methods are available to promote drainage (see Figure 102 and 107) ;

a)	trench drains	d)	vertical drains
b)	horizontal drains	e)	electro-osmosis
c)	drainage galleries or adits	f)	vegetation.

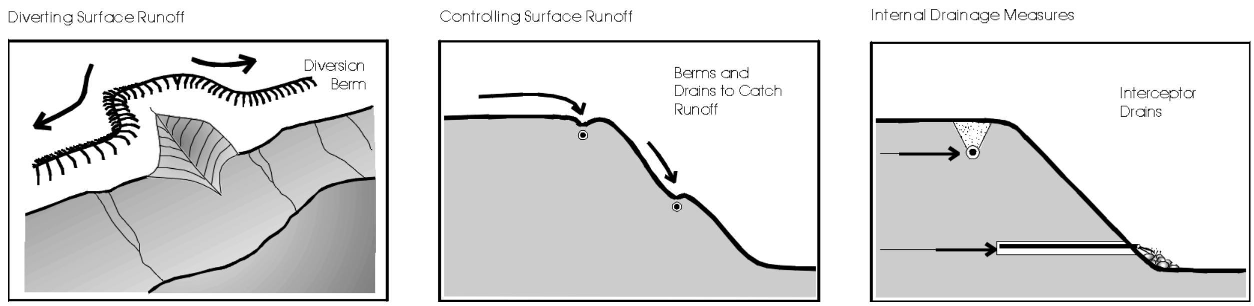


Figure 106 - Slope Drainage

Drainage improvements can be effective for granular soil slopes (permeable soils, sands) but often much less effective for slow-draining cohesive soil slopes (clays, silts). Drains can aid in several manners (see Figure 108);

- by controlling surface runoff and infiltration through shallow drains such as ditches, French drains, subdrains,
- by controlling seepage patterns and lowering levels through deep drains,
- by providing pore-pressure relief.

The following Figure 107 indicates various drainage measurements.

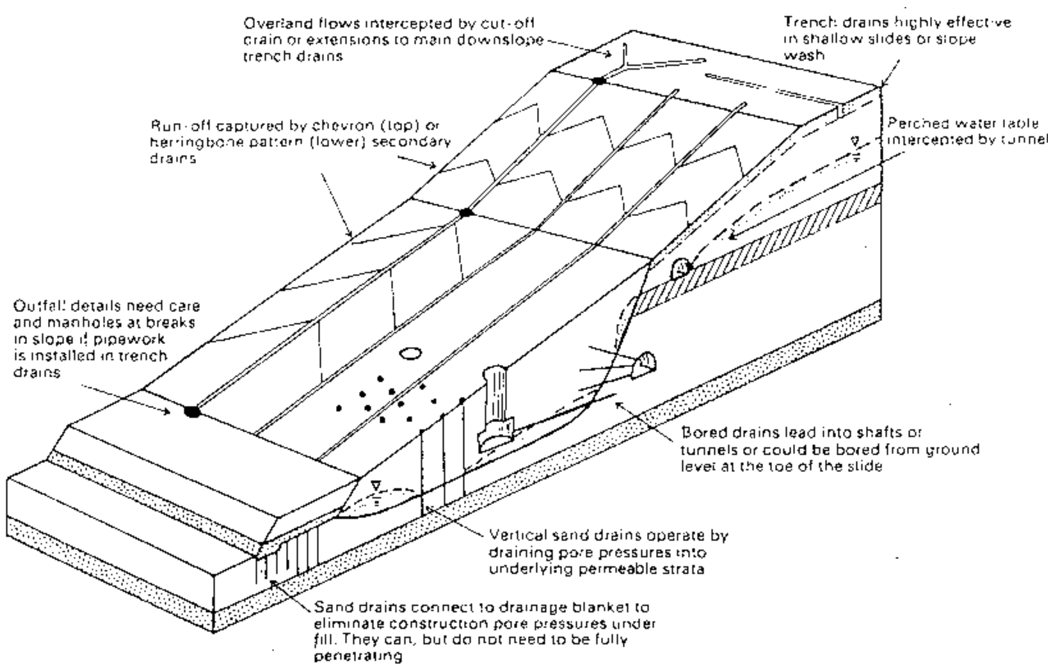


Figure 107 - Drainage Measures

Trench drains consist of narrow gravel-filled trenches which can be shallow or deep. The major use of trench drains in practice, is to stabilize shallow slides or translational slides in cohesionless wet soil slopes. The granular backfill can also provide a buttressing support to adjacent soil. The major use of trench drains in practice, is to stabilize shallow slides or translational slides (see Figure 108 a),b),c)).

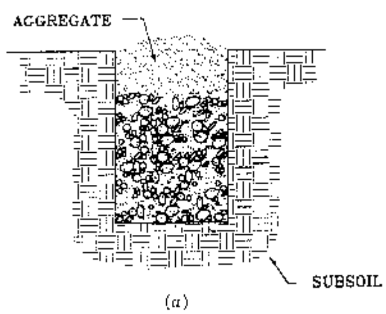


Figure 108 a) - Subdrains

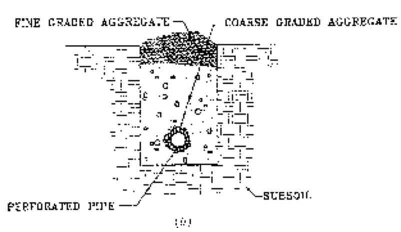


Figure b) - Subdrains

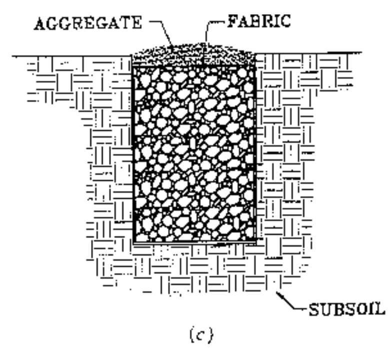


Figure c) - Subdrains

(from Gray and Sotir, 1996)

10.3 Soil Reinforcement

The soil in a slope may be reinforced with high strength material. The reinforcement may provide or mobilize additional strength in the soil and thereby increase stability. Geogrids (see Figure 109 and 110) are synthetic (polymers, plastics, strands) reinforcement materials which are buried as layers in earth fill to provide anchorage against ground movements or slips. The use of geogrids can enable higher or steeper stable slope configurations. This approach requires excavation and backfilling of the slope.

Geogrids are synthetic (polymers, plastics, strands) reinforcement materials which are buried as layers in earth fill to provide anchorage against ground movements or slips. The use of geogrids can provide sufficient reinforcement to enable higher or steeper stable slope configurations.

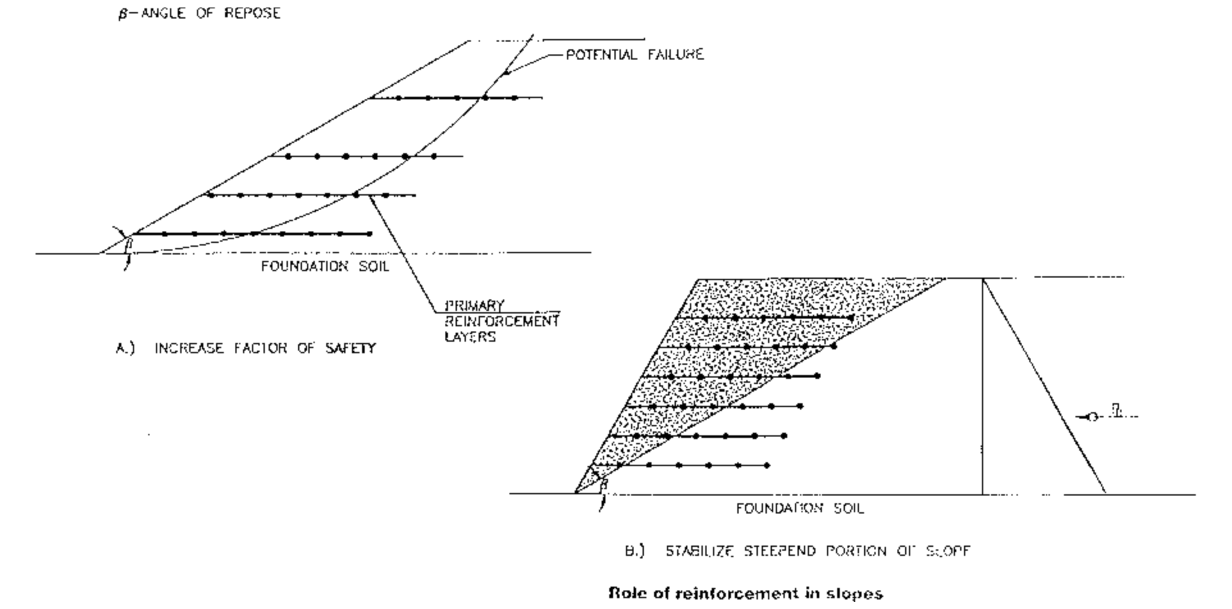


Figure 109 - Geogrid Reinforcement of Slopes

Typically, the geogrids are placed within the fill at vertical intervals of every 600 mm and the ends at the slope face are folded back and buried or anchored within the slope fill.

Figure 110 - Photo, Geogrid Reinforcement

Soil nailing (driving metal rods into ground) has been successfully used for temporary support of excavation slopes, and in more long-term applications in temperate climates. There has been little experience documented in cold winter climates. This stabilization approach does not require excavation and backfilling of the slope.

Closely spaced large diameter caissons have been utilized for slope stabilization works, but nominal diameter piles or caissons are generally ineffective for slope support or reinforcement.

10.4 Controlling Surface Run-off on Slopes

Surface run-off on slopes can result in high flow velocities that become erosive. It is important to reduce both the amount of run-off and the speed of its flow, by manipulating the slope surface to detain the water and increase the infiltration (see Figure 106) . Several means of reducing the flow volume and velocity are available,

- intercept run-off from the high table land, with ditches at and parallel to the slope crest, and divert it to locations where it can be drained away in a controlled manner (pipe or lined channel), or

- construct berms or terraces (stair step grading) on the slope face to reduce the length of uniform slope gradient, and hence the flow velocity, or

- cover the slope face with staked netting or mesh which would help confine the soil particles on the slope surface, and aid in reducing the flow velocity.

Ground cover for erosion protection can range from loose mulches (hay and straw), to hydraulic mulches, to geotextiles (nets, mats, jute, blankets, meshes), to cellular containment systems. Some of these can be anchored with pins or stakes (see Figure 111).

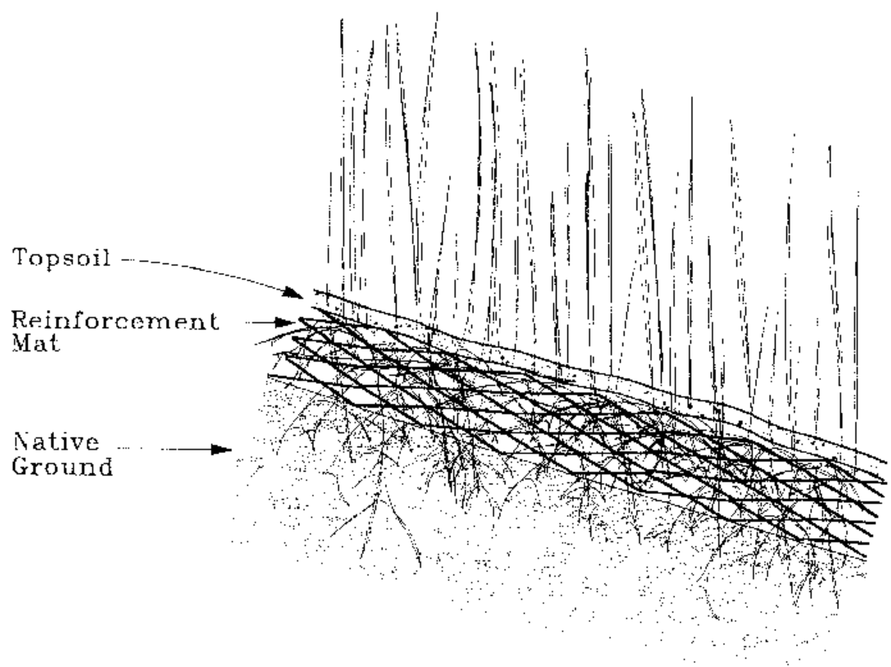


Figure 111 - Nets, Mats, Mesh (Gray and Sotir, 1996)

Other ground cover measures for erosion protection may consist of cellular containment systems such as Geoweb and Terraweb grids are seen on Figures 112.

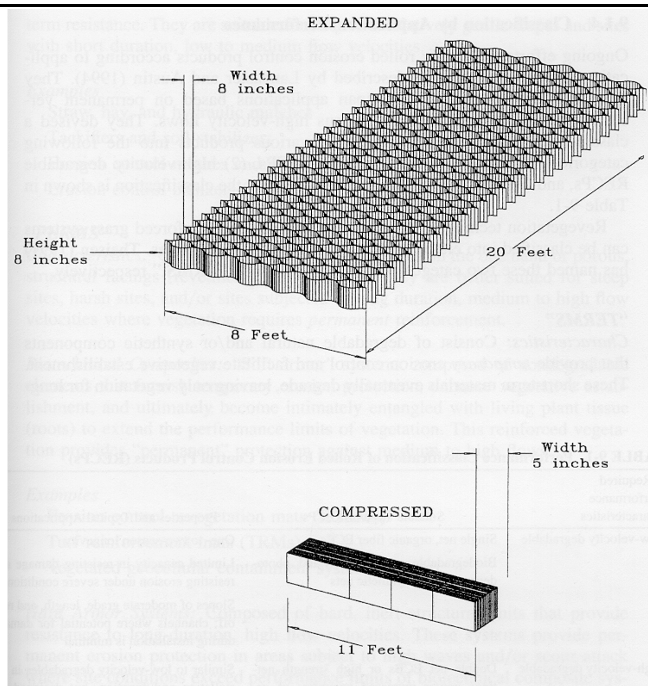


Figure 112 - Cellular Confinement (from Gray and Sotir, 1996)

The slope crest and any sharp changes in gradient are difficult to vegetate, and often where erosion begins (see Figure 113). These can be improved by rounding off the sharp edges.

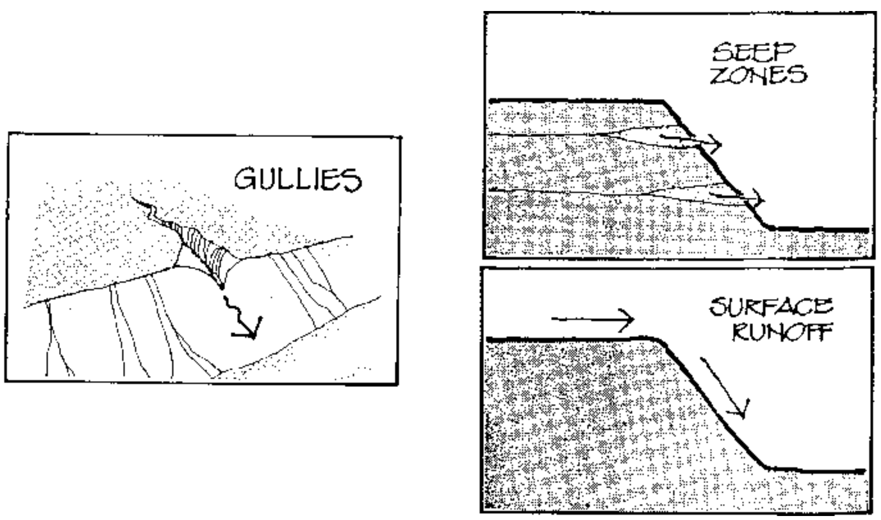


Figure 113 - Slope Drainage

Vegetation plays an important role in surface runoff erosion control, by reinforcement with roots and binding the soil particles near the ground surface and protecting against sheet erosion, rill erosion, and gully formation (see Figure 114).

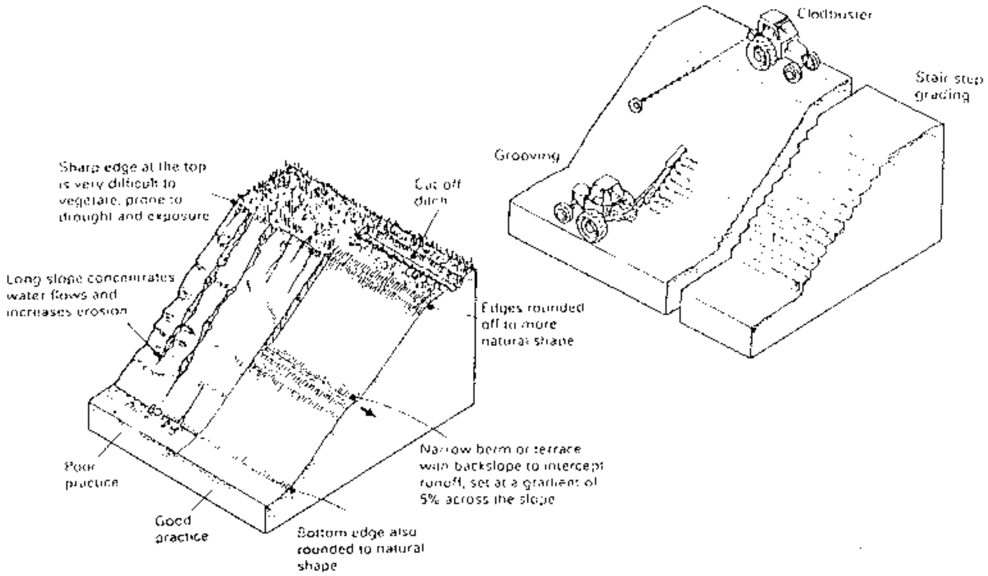


Figure 114 - Surface Run-off Control on Slopes

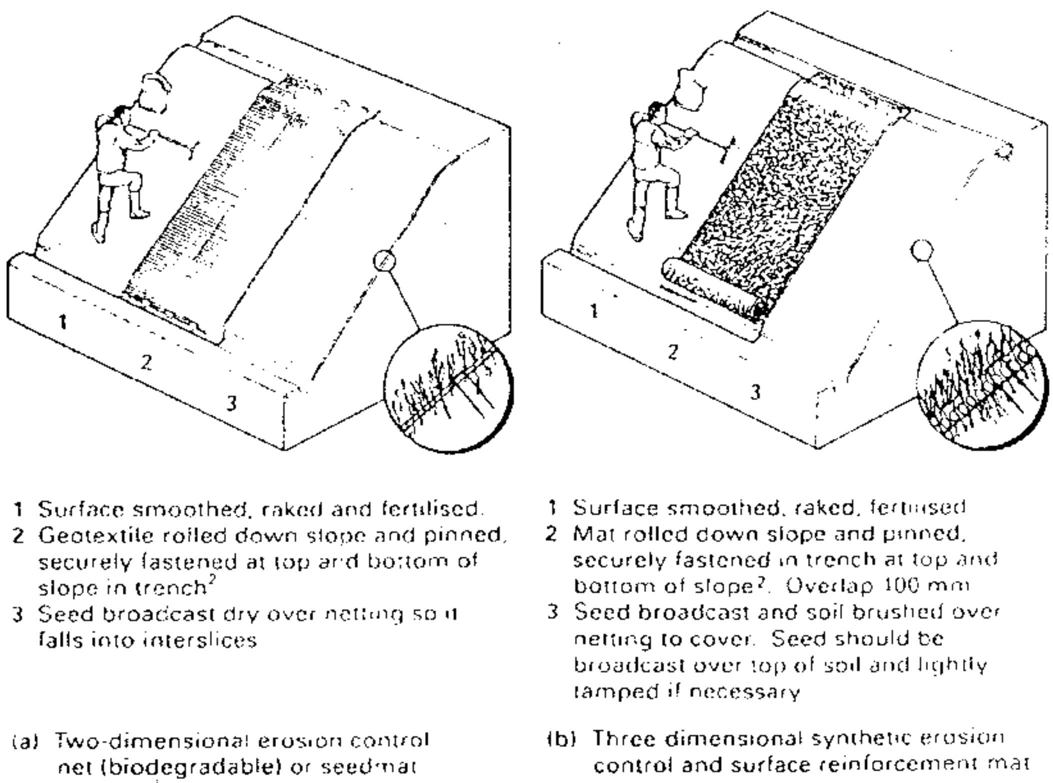


Figure 115 - Surface Cover on Slopes

10.4.1 Control of Shallow Slides with Vegetation
While vegetation provides limited protection against slope slides, vegetation is the primary defence against surface erosion. Vegetation plays an important role in erosion control by reinforcement with roots and binding the soil particles near the ground surface and protecting against rill erosion and gully formation. Root reinforcement can also aid in protecting against shallow slides within the depth of root penetration. Root penetration has been reported as ranging from about 0.5 to 0.75 m depth for grasses and weeds, to 1 to 3 m depth for shrubs and trees (see Figure 116).

The effect of root reinforcement on soil has been documented as having a typical shear resistance ranging from 1 to 17 kPa but more commonly from about 5 to 6 kPa, and only in the depth of root penetration.

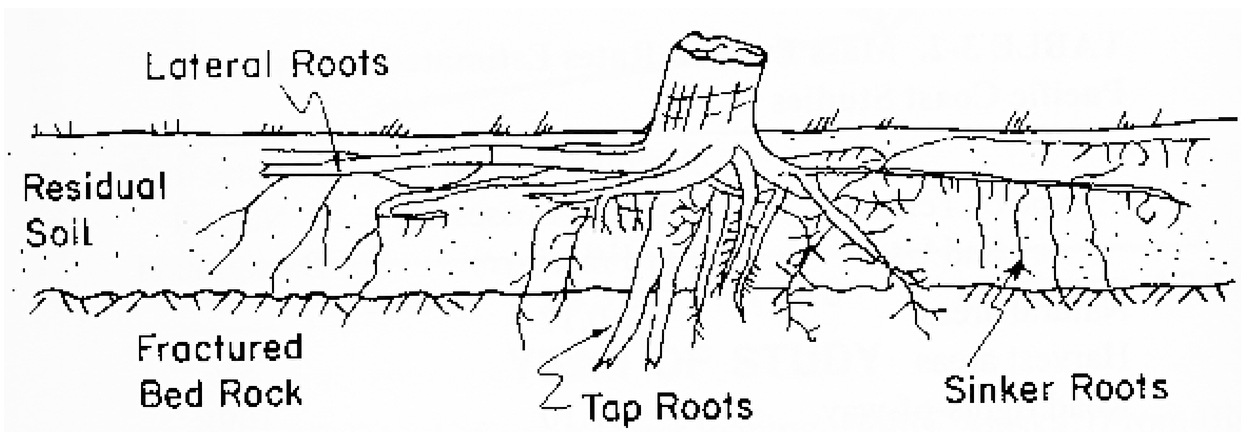


Figure 116 - Root Reinforcement (from Gray and Sotir, 1996)

A thin layer of topsoil is likely required to provide nutrients, and hydro-seeding is recommended to protect against washing away of the seed. Additional applications of fertilizer may also be required.

While grasses and legumes commonly provide good erosion protection at an early stage of rehabilitation, shrubs and small trees are also required. The shrubs and trees will provide future nutrients from leaf litter and are beneficial for screening (aesthetics), and for wildlife.

10.4.2 Slope Inclinations for Vegetation
Empirically it has been found that a 2 to 1 inclination (26°) is the steepest upon which vegetation can be established and maintained satisfactory. However a slightly flatter inclination of 3 to 1 (17°) is required to achieve maximum vegetative stability. Steeper inclinations cannot be suitably stabilized by vegetation alone and additional reinforcing or support is required.

On sloping surfaces, netting or mesh is required to be staked on the ground surface, for temporarily protection until the vegetation has established.

The above inclinations (2 : 1 and 3 : 1) relevant to vegetation, are also the same inclinations recommended in the Sand and Gravel Pit Rehabilitation in Northern Ontario (MNR, 1987) manual;

2 : 1 - maximum general slope inclination considered for long-term stability of a pit site

3 : 1 - generally considered to be the maximum gradient for safe side-hill vehicle travel, for effective surface erosion control, and for safe pedestrian access up and down the slope.

Therefore, while slope inclinations steeper than 2 to 1 (horiz. to vert.) can be stable in well-consolidated (compact to dense condition), it may be difficult to establish vegetation on the steep stable slopes. Other methods of surface cover can be used instead (i.e. gravel or crushed stone, cellular media) but costs would be relatively high.

10.5 Soil Bioengineering and Biotechnical Stabilization methods

Some slopes are also suitable for the application of soil bioengineering and biotechnical stabilization techniques. These terms have been defined as;

- soil bioengineering - the use of living plant materials to protect against erosion (brush layering or contour wattling).

- biotechnical stabilization protection - the combined or integrated use of vegetation and structural components to protect against erosion,

Detailed information on the various soil bioengineering and biotechnical stabilization methods can be found in the book titled Soil Bioengineering and Biotechnical Stabilization by Donald Gray and Robbin Sotir.

10.5.1 Soil Bioengineering

Non-structural protection works are essentially activities that do not involve the construction or placement of significant additional structures or materials within the area of provincial interest. Soil Bioengineering refers to the use of living plant materials it is used to protect against erosion (fascines, brush layering or contour wattling).

These approaches are most effective as surface erosion protection and are not very effective for improving overall stability against deep rotational slides or slides which extend deeper than the root reinforcement. As erosion protection, soil bioengineering approaches may be more pleasing to the eye, and may require less site access and disturbance to install.

Successful application requires harvesting and placement of live cuttings in the brush layers, during the dormant season (Nov. - Apr.). Stems and branches up to 75 mm diameter are typically used from the following species; willow, dogwood, alder, and poplar.

Vegetation by itself is vulnerable to frost action, trampling, and moisture or nutrient deficiencies. Further, it would be of little value in protecting against deep-seated slides, or against toe erosion from rivers or wave action.

These approaches are most effective as surface erosion protection by,

- intercepting raindrop impact,

- binding soil by rooting for reinforcement and anchoring,

- slowing and filtering surface runoff,

- maintaining infiltration.

However they are not very effective for improving overall mass (global) stability of potential deep-seated slides. Furthermore, these techniques are not effective against wave erosion or toe undercutting. As surface erosion protection, soil bioengineering approaches may be more pleasing to the eye, and may require less site access and disturbance to install. The benefits of vegetation for slopes also include visual aesthetics, low-cost stabilization, and natural habitat for wildlife.

While vegetation on its own generally requires slope inclinations flatter than 2 to 1 (26½°), soil bioengineering methods can be utilized to permit vegetation establishment on slightly steeper slopes.

Vegetation by itself is vulnerable to frost action, trampling, wildlife, and moisture or nutrient deficiencies. Further, it would be of little value in protecting against toe erosion from rivers or wave action where structural methods are necessary. Vegetation could be incorporated with structural protection, to improve aesthetics.

Various structural materials such as stone and timber can be used with vegetation to locally reinforce or steepen slopes,

- live staking
- wattle fencing, or fascine drains and plantings
- brush layering, or contour boarding
- slope grids and gratings
- post and railing walls
- live slope grating

10.5.2 Live Staking

Live staking consists of the inserting and tamping live plant cuttings into the ground surface. The live cuttings take root, leaf out, and propagate. The cuttings are typically 12 to 40 mm diameter and 0.6 to 0.9 m long. They are driven into the ground using a hammer, on a spacing of 0.6 to 0.9 m apart. A small part of the stake (1/5) should be left sticking out of the ground. Live staking is usually used in small localized areas (see Figure 117 and 118).

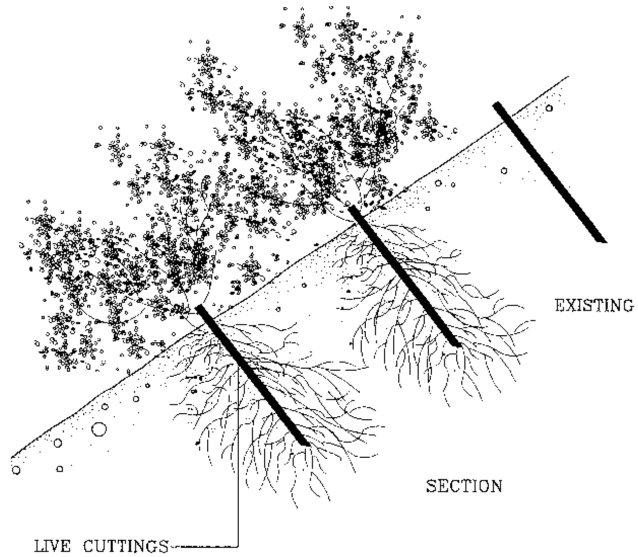


Figure 117 - Live Staking (from Gray and Sotir, 1996)

Figure 118 - Photo, Live Staking (from Gray and Sotir, 1996)

10.5.3 Contour Wattling

Wattles ('interwoven twigs') consist of string-tied bundles of plant stems up to 75 mm diameter (commonly willows). Each wattle is 2 to 3 m long, about 200 to 250 mm in diameter, and string tied every 300 to 450 mm. After initial cutting, the stems are stored in wet burlap or water (1 to 2 days) until the wattles are buried in shallow trenches parallel to and on the slope face, reinforced with wood staking. Wattling is considered to stabilize the slope face to depths of 0.25 to 0.5 m (see Figure 120 and 122).

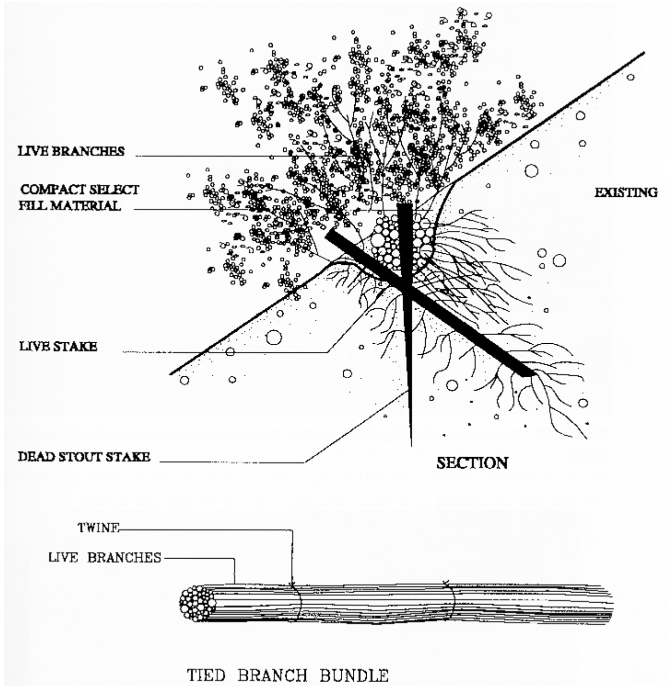


Figure 119 - Wattling (from Gray and Sotir, 1996)

The typical installation takes place from the bottom (slope toe) up to the top (slope crest) using manual labour. The slope inclination should not be steeper than 1½:1 (34°) and preferably 2:1 (26½°) or less. Wooden staking should be carried out along the contour at vertical intervals of 1 to 1.5 m and, using about 7 stakes for every 2 m length. The stakes should be at least 600 mm long. A shallow continuous trench should be dug on the upslope side of the stakes, about 200 mm deep. The excavated soil should be spread below the staked contour. The wattles or bundles of tied willow cuttings are then laid in the trenches and overlapped at the ends. The ends are also staked.

Figure 120 - Photo, Fascines, Wattles (Gray and Sotir, 1996)

Figure 121 - Photo, Contour Wattling (Gray and Sotir, 1996)

Figure 122 - Photo, Fascines, Wattles (Gray and Sotir, 1996)

The next higher trench is dug and the excavated soil is used as backfill for the lower wattle. Walking on the backfilled lower wattle contour is encouraged, to assist in compacting it. The installation continues to the slope crest. It has been suggested that areas with less than 300 mm of annual rainfall, should be irrigated. The nutrient levels in the soil should be verified and adjusted if necessary. A topsoil layer at least 100 mm thick should be applied to the final surface. The whole slope surface should then be hydro-seeded with a mixture of grasses and legumes.

Contour wattling has also been referred to as 'fascine drains'. The wattling serves as an energy dissipator for surface runoff, as surface drains and filters along the wattling, as anchoring and reinforcement. Regular inspection and maintenance should be carried out during the first year.

10.5.4 Brush Layering

This technique consists of burying plant cuttings on the slope face, on contour lines spaced 1.5 to 3 m up the slope face. The continuous 'brush layer' is buried at least 0.5 m, starting from the bottom or slope toe and proceeding up the slope. The work is carried out manually and begins with an initial trench or terrace being dug and the excavated material being spread below the trench. The plant brush is laid in the trench and the next higher trench is begun. The excavated soil is spread over the lower brush layer, leaving the outer ¼ length of brush exposed on the slope face (see Figure 123).

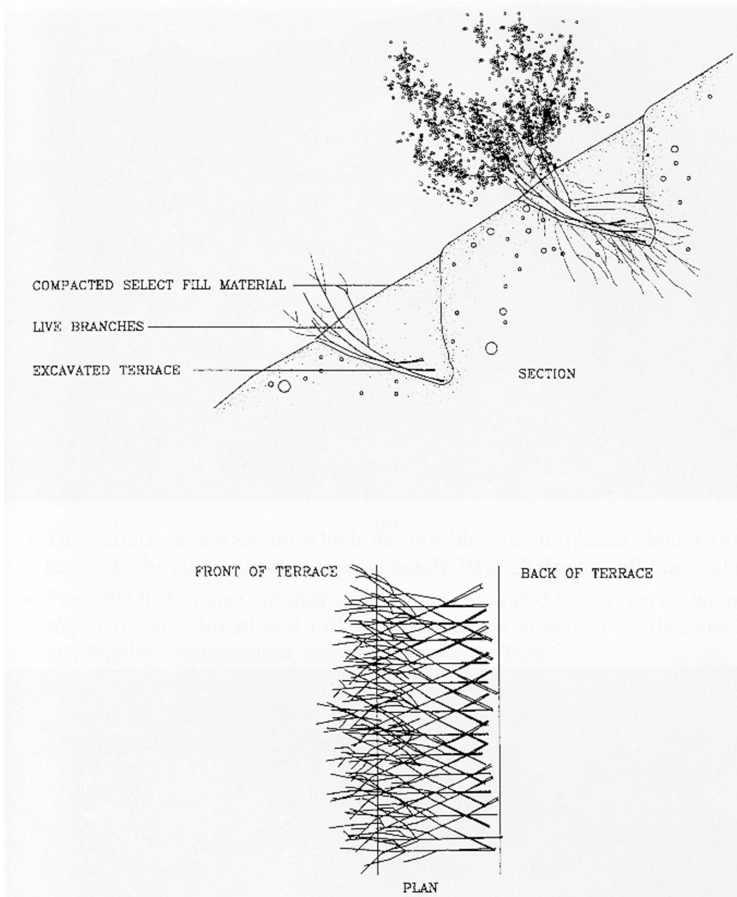
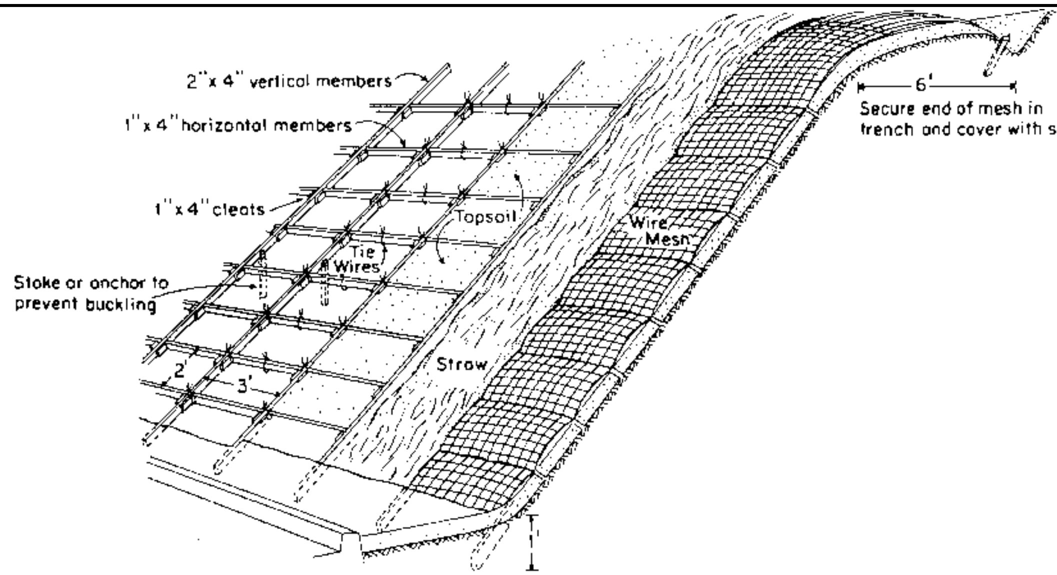


Figure 123 - Brush Layering (from Gray and Sotir, 1996)

Timbers or logs can also be used on this contouring principle, except without the natural rooting.

10.5.5 Live Slope Grating

A live slope grating is used to establish vegetation on very steep slopes, less than 1½ to 1 (H:V), and consists of a lattice horizontal and vertical timbers. The openings in the timbers are filled with common earth and live branch cuttings similar to “brush layering” (see Figure 124).



Cut slope stabilization using anchored timber grid to hold topsoil and slope plantings in place.

Figure 124 - Live Slope Grating (from Gray and Sotir, 1996)

10.6 Biotechnical Stabilization and Structural Components

Structural protection works involve the construction and/or placement of significant additional structures and/or materials within the area of provincial interest. Biotechnical stabilization protection (i.e., vegetated cellular grids, live crib walls and rock walls) refers to the combined or integrated use of vegetation and structural components to protect against erosion (see Figure 125).

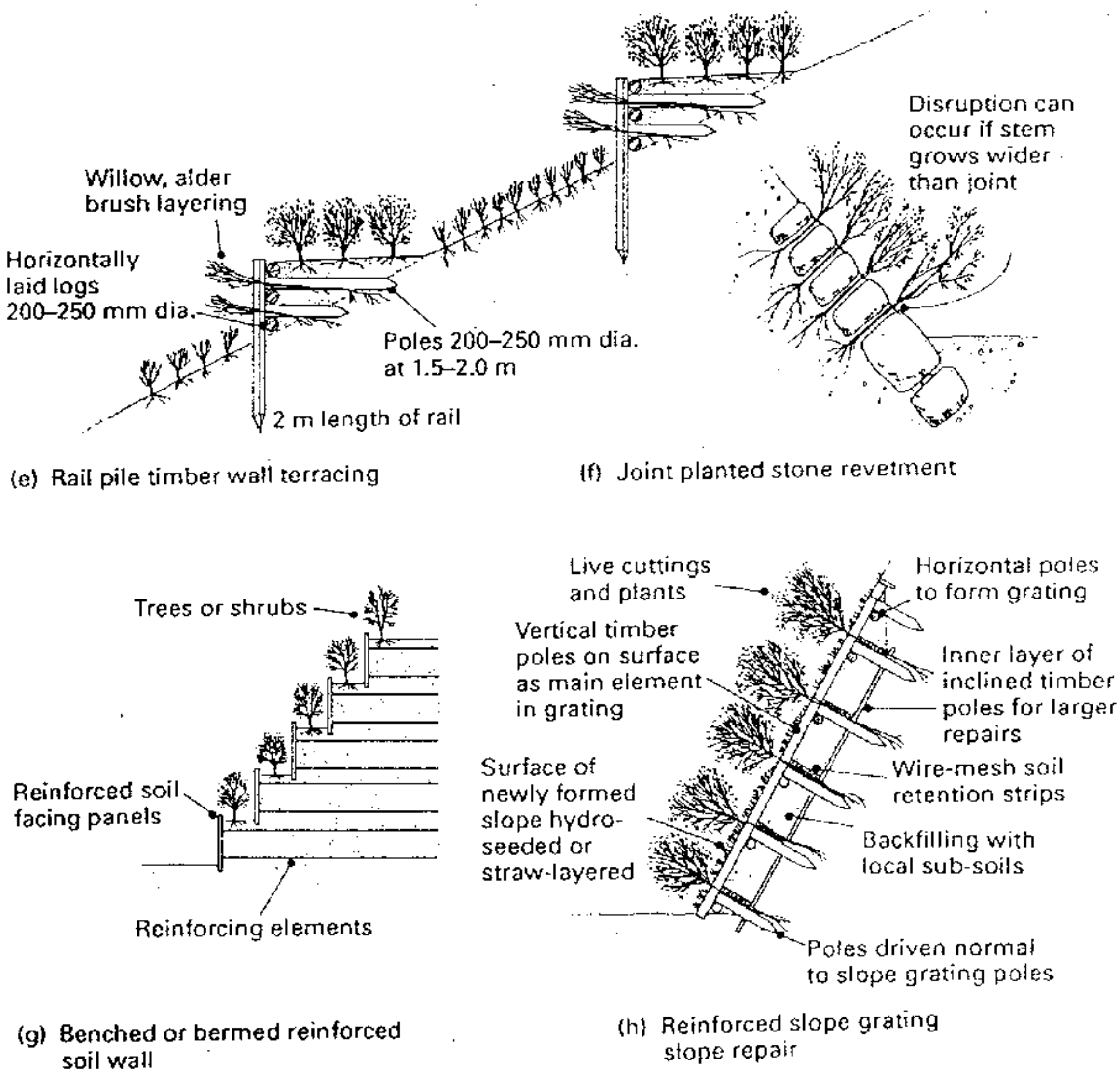


Figure 125 - Biotechnical Stabilization Methods

The common types of biotechnical stabilization include;

• vegetated rip rap;	live cuttings (25 to 40 mm diam.) inserted into openings between rip rap stones(2 to 4 cuttings/sq.m)
• vegetated rock breast walls;	live cuttings/branches inserted between stacked rock walls
• vegetated crib walls;	layers of live cuttings inserted and backfilled into the crib box structure
• vegetated cellular grids;	cellular grid structure anchored to the slope with planting of interior cells
• tiered retaining walls with bench plantings;	shrubs or small trees planted on the benches between walls.

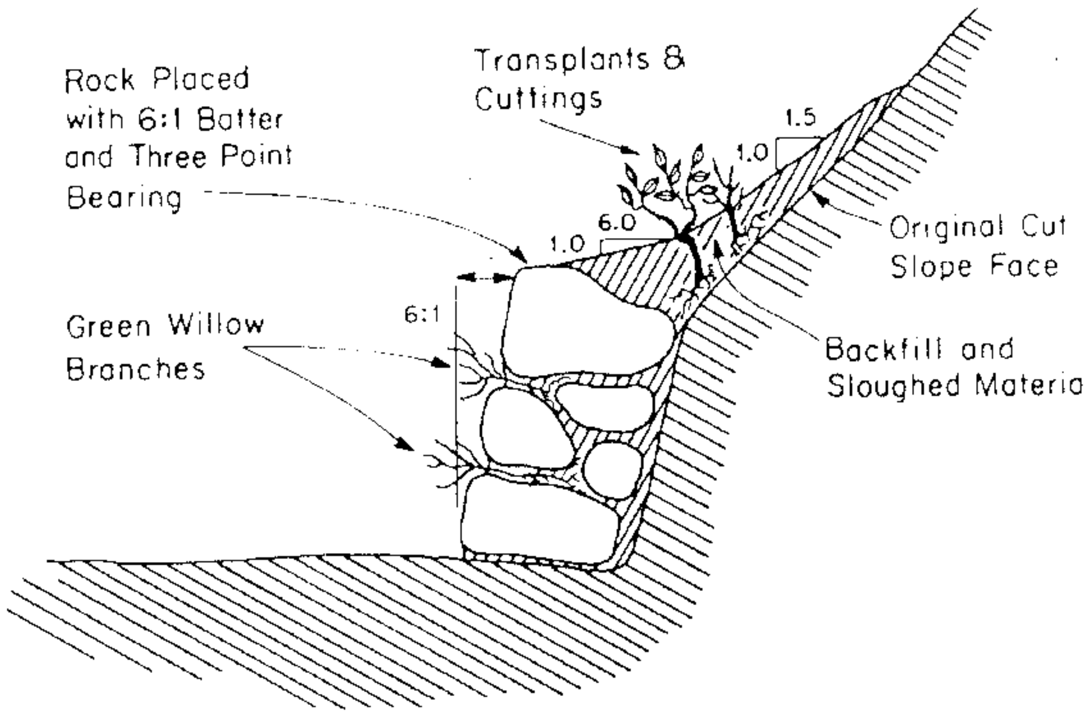
The incorporation of structural or armouring components such as rip rap, armourstone, gabions, field stone, masonry, timber, or concrete with re-vegetation, can be beneficial in several manners,

- by providing protection against toe erosion from water bodies, or
- by permitting much steeper inclinations than can be maintained by soil alone.

The following general types of retaining structures have been used in slope environments,

- gravity walls; stone or concrete, breast walls (1 to 1½ m high), toe walls
- crib or bin walls; timber, metal, precast concrete; cellular
- reinforced earth; strips, geogrids, geotextiles
- cantilever or counterfort walls; reinforced concrete (8 to 9 m high)
- rip rap or amour stone walls
- tie-back walls (anchored); steel sheeting, piles and lagging

The armour stone or rock breast wall is shown on Figure 126. Depending on the flow conditions at the site the amount of rock material may vary, allowing for further use of plant materials.

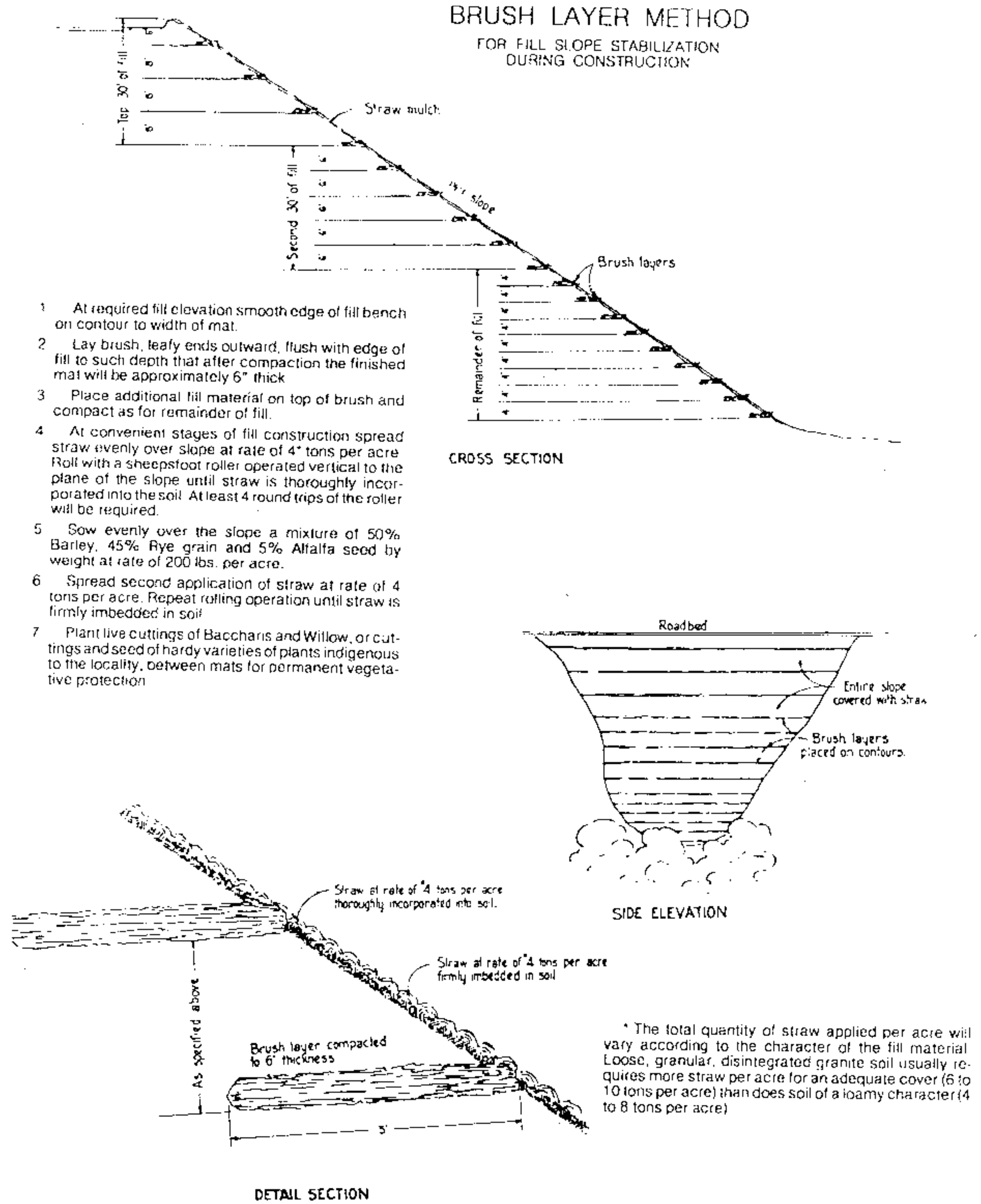


Typical rock breast wall design for stabilizing the toe of a steep, eroding slope.

Figure 126 - Rock Breast Wall

A biotechnical stabilization technique where rip rap was used along with anchoring trees and logs into the bank can be seen on Figure 127.

Figure 127 - Biotechnical Stabilization Protection



Specifications for fill slope stabilization using brush-layer method.

Figure 128 - Brush Layer Method

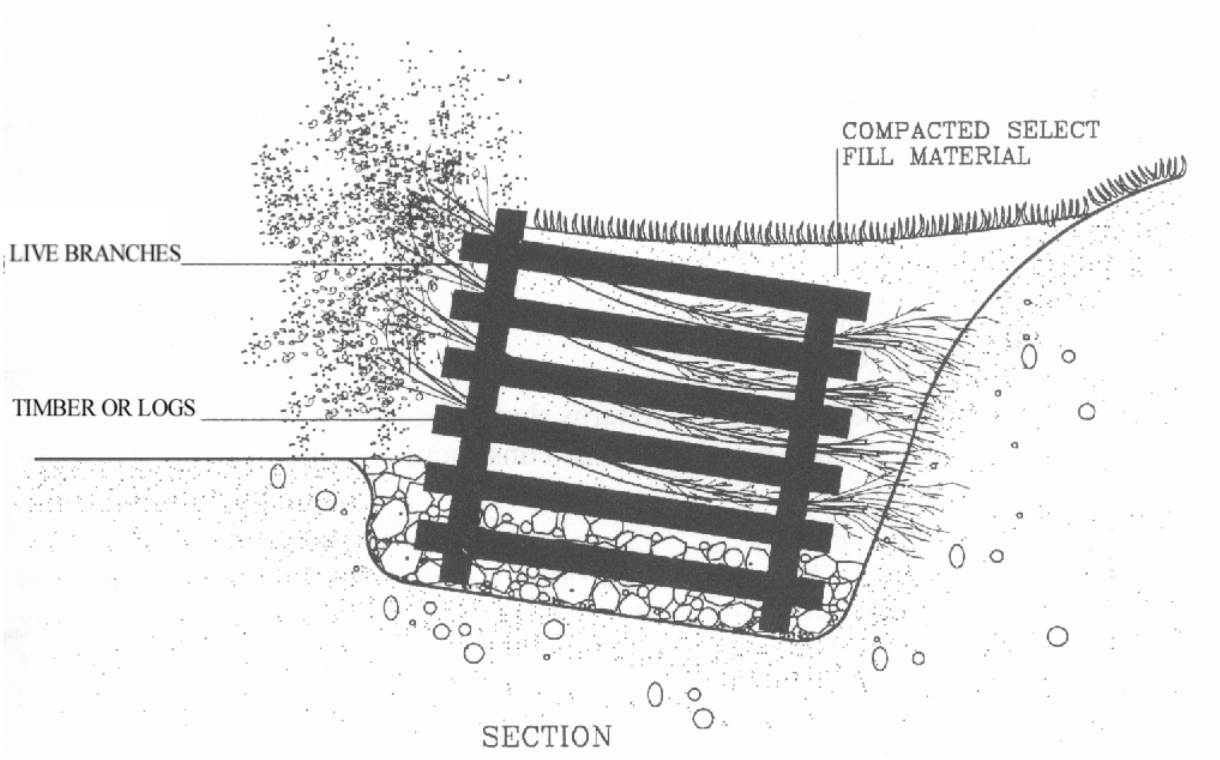
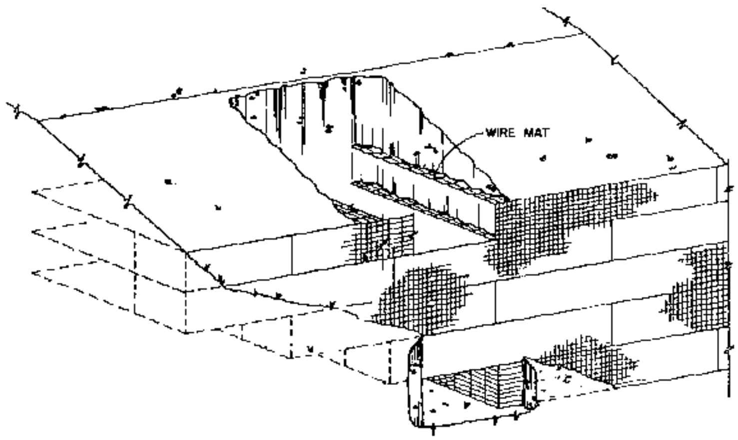


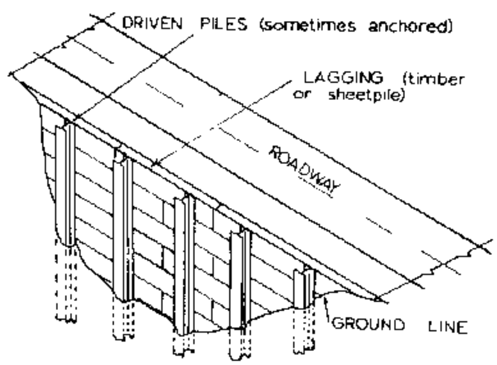
Figure 129 - Timber Grid (from Gray and Sotir, 1996).

Figures 130 to 133 demonstrate some of the traditional earth retaining structures.

Figure 131 - *not used*



(f) WELDED-WIRE WALL



(g)

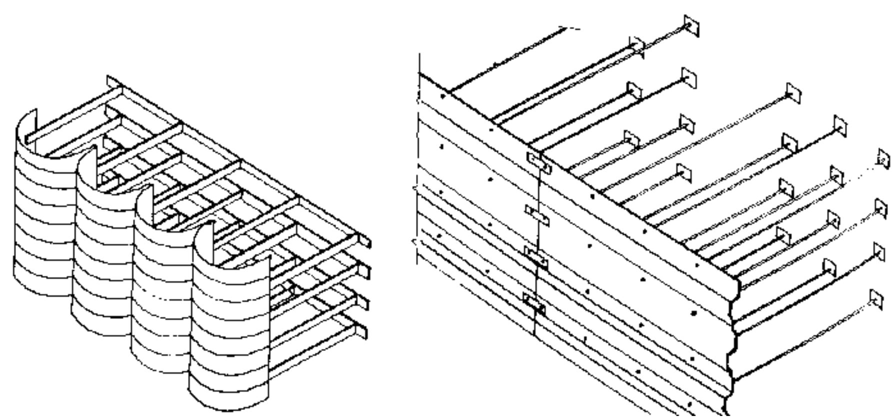


Figure 132 - Earth Retaining Sheetpile Wall Structures

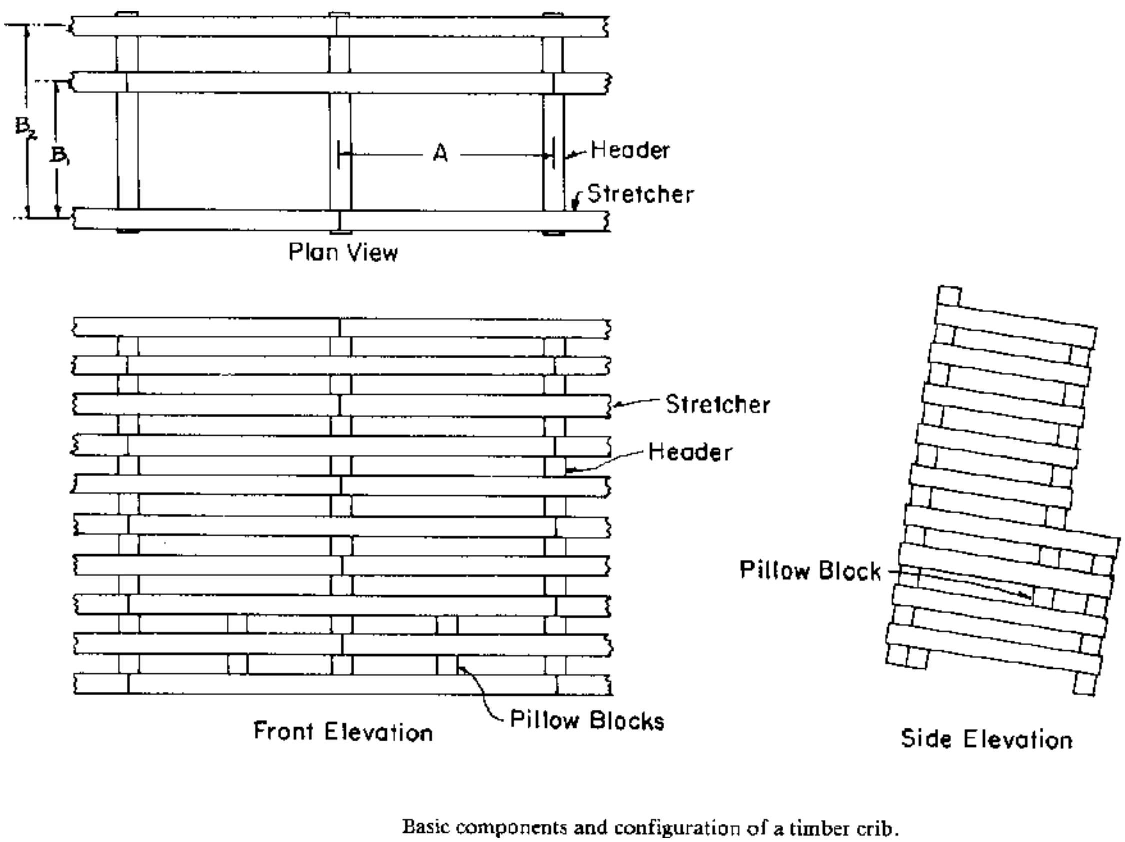


Figure 133 - Earth Retaining, Timber Crib Structure (from Gray and Sotir, 1996)

10.7 Gully Stabilization

The key to combatting gully erosion, is to first stop the downcutting toe erosion (reduce flow velocity or protect gully base). Stabilization of the toe erosion will then permit the banks or side-slopes to self-stabilize through shallow slides, to a flatter inclination. Once the slope toe and slope inclination are stable, vegetation cover can develop.

Branchpacking consists of the burial and staking of live branch cuttings, to repair small erosion or slumping areas. Live cuttings are used, 12 to 50 mm diameter and 1.5 to 2 m long. Wooden stakes should be used, 75 to 100 mm diameter poles or 2x4 lumber, 1.5 to 2 m long. The wooden stakes are driven into the ground on a spacing of about 0.3 to 0.5 m apart and live cuttings are buried with common earth fill (see Figure 134).

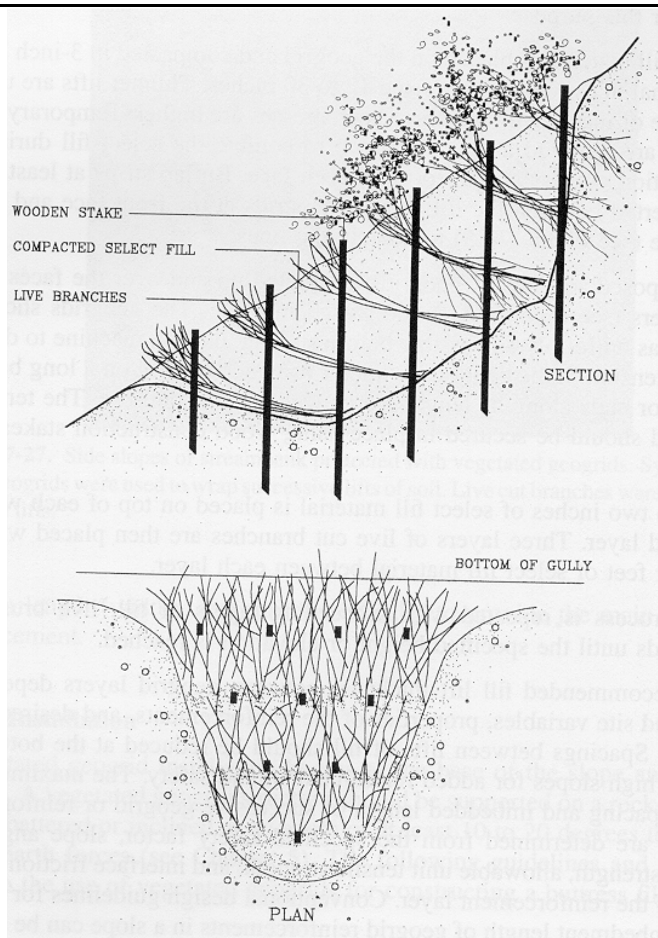


Figure 134 - Branchpacking (from Gray and Sotir, 1996)

Flow velocity can be reduced by constructing check dams (see Figure 135), or, the gully base can be lined with a durable lining such as 'rip rap' stone or pre-cast blocks. Stone size will be dependent on the design flow velocity. Alternatively, the water flow over the slope face can be eliminated entirely by transporting it in a pipe or diverting it. There are many methods of stabilization available, including soil bioengineering and biotechnical protection.

If the gully erosion is surface drainage related it may be controlled by collecting the flow in a pipe or subdrain, or by lining the gully base, or by constructing check dams. Check dams function by decreasing the flow velocity. Several check dams are usually used to achieve a 'stair stepped' gradient rather than a uniformly steep grade. Various materials can be used in the construction of check dams,

- brush

- timber; wood posts, boards

- stone; rip rap, field stone, armourstone

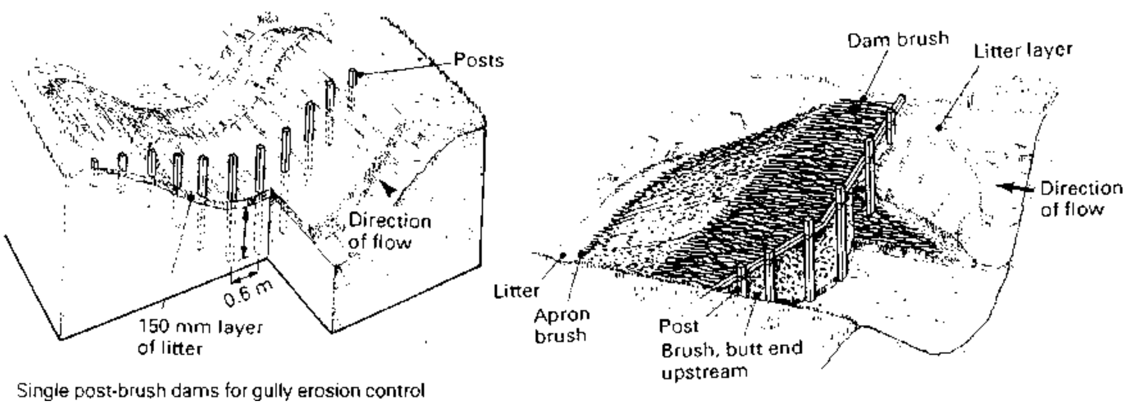


Figure 135 - Gully Stabilization

10.8 Retaining Walls, Anchors

Retaining walls on slopes (see Figure 136 and 137) support the loads imposed from upslope soil. Mobilization of the wall resistance may require some additional deformation (of soil mass) to occur. A cantilever or gravity wall can improve slope stability if founded in competent strata. Good drainage and frost protection are also important in the design.

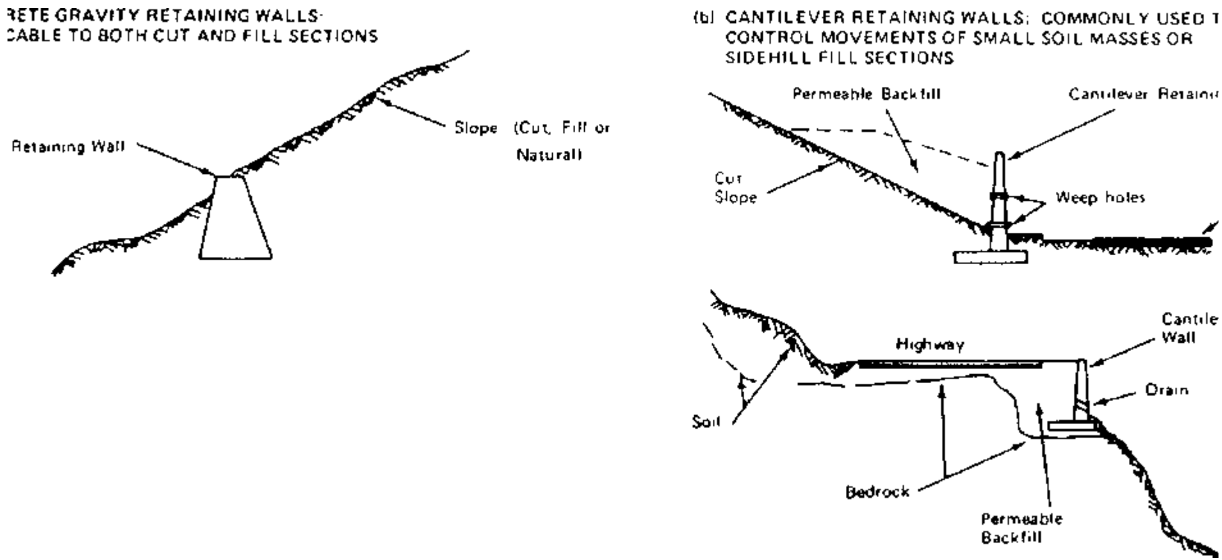


Figure 136 - Retaining walls

Figure 137 - Photo, Rock wall

Buttress design stabilization provides sufficient deadweight or reinforced restraint near the slope toe, so as to increase the Factor of Safety against sliding. The buttress design must also be stable against overturning, sliding along the base, and internal shear (see Figure 138).

Figure 138 - Photo, Timber wall

Anchors in soil slopes whether pre-stressed or not, may be difficult to design due to the complex nature of the horizontal forces. Performance anchor loads should be monitored to permit re-stressing if necessary, and corrosion protection should be considered (see Figure 139).

Figure 139 - Photo, Geogrid Reinforced Wall

11. SLOPE REHABILITATION OF SAND & GRAVEL PITS

This section presents an overview of existing rehabilitation of sand and gravel pit slopes, and, a summary of geotechnical principles of stable slopes and of surface erosion control methods.

The requirement of sand and gravel pit rehabilitation includes the grading of slopes, to make the slopes adjacent to surrounding land owners sufficiently stable. The stability/instability of the slopes around sand and gravel pits may affect surrounding properties or structures or utilities, depending on the height of the slopes and the proximity of the slope crest to the property boundaries (see Figure 140).

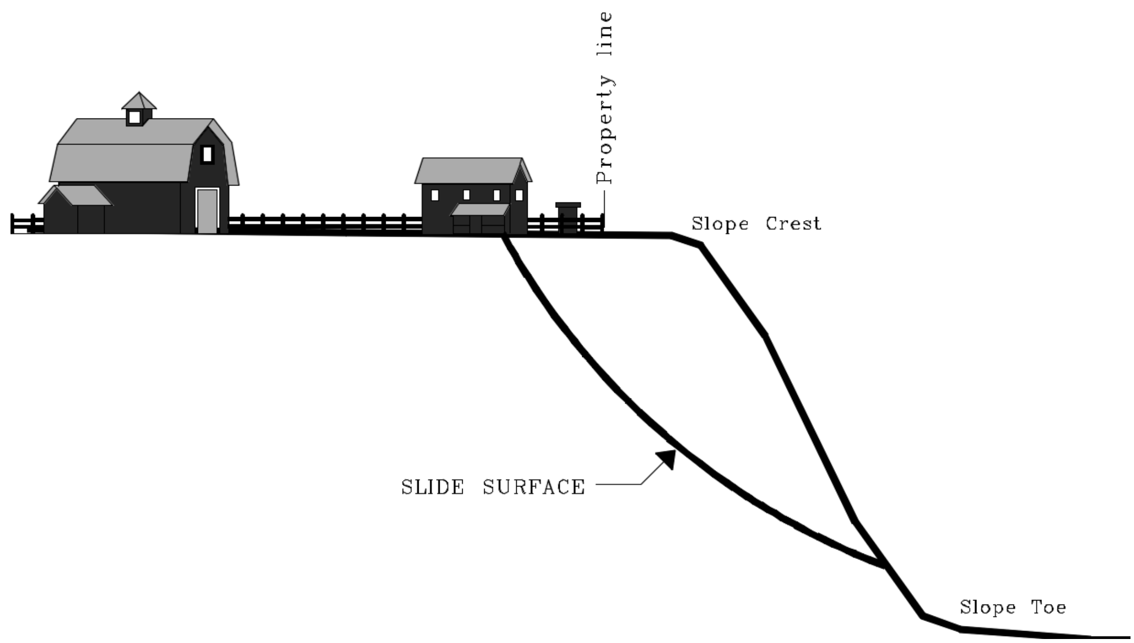


Figure 140 - Slope Proximity to Property Lines

The previous sections described how the determination of an appropriate stable slope inclination is dependent on the following three basic criteria (discussed in further detail below);

CRITERIA		PURPOSE
a)	the land-use proposed for the slope face itself	access capability to support pedestrian or vehicle traffic or wildlife or agriculture on the slope
b)	the characteristics of the soil comprising the slope	mass (global) stability of the inclined slope surface, to protect against slope slides or ground movements
c)	the land-use on top of the table lands adjacent to the pit or quarry	mass stability of the slope crest and, adjacent table land which may support structures or buildings

There are several aspects common to slopes in sand and gravel pits, which help in reducing the variables that must normally be considered in the analysis of slope stability;

ASPECTS / FEATURES		DESCRIPTION
a)	cohesionless sandy soil	by their nature, sand and gravel pit operations are commonly located within extensive natural deposits of granular soils; cohesive soils are not common
b)	high and uniformly steep slopes	the pit slopes are commonly very high (10 to 50 m) and therefore very long; the slope inclination is uniformly steep and often the slope may extend below local groundwater levels (lower slope submerged)
c)	bare and exposed slopes	due to the extraction operations, the pit slopes are no longer vegetated but are bare and exposed, subject to surface erosion

11.1 Form of Land-Use Proposed For the Graded Slope

The Sand and Gravel Pit Rehabilitation in Northern Ontario (MNR, 1987) manual recommends the following slope inclinations (horiz. to vert.);

26½°	2 : 1	50 %	maximum general slope inclination considered for long-term stability of a pit site
18½°	3 : 1	33 %	generally considered to be the maximum gradient for safe side-hill vehicle travel, for effective surface erosion control, and for safe pedestrian access up and down the slope
6°	10 : 1	10 %	slopes in the range of 3 : 1 to 10 : 1 are generally satisfactory for forestry, recreation, and some agricultural uses

Most pit operations are based on maximum extraction of aggregate and therefore the steepest slopes possible are typically excavated except for access roads and the like. If regular access on the slope face is not required, stable slope inclinations steeper than 3 : 1 may be feasible for the graded material, but the inclination angle will be dependent on the material composition (i.e. gravel, sand, silt, clay) and on the type of surface cover (i.e. vegetation only, reinforcement, bioengineering).

11.1.1 Submerged Slopes

Many pit operations are extracting granular resources from below the local groundwater level, using dragline equipment under water. An under-water or submerged slope is created by the extraction operations, which can often be higher than the slope portion which is not submerged (see Figure 141).

Generally the appropriate stable slope inclination is flatter for submerged portions of the slope than for non-submerged. The flatter submerged inclination is due to additional external forces on the slope such as fluctuating water levels, wave action, currents, frost and ice forces. The potential for erosion of the slope along the water shoreline (undercutting) must be considered. The magnitude and frequency of these forces are difficult to analyze and predict.

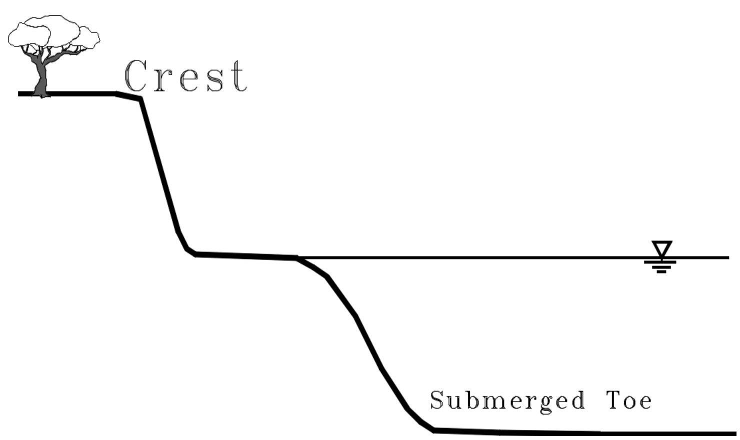


Figure 141 - Submerged Slopes

The following table presents a comparison of typical 'angles of repose' for various soil types, submerged and non-submerged. The angle of repose represents the slope inclination the soil naturally forms when dumped or placed in an uncompacted state (relatively loose). The angle of repose may be steeper than the long-term stable slope inclination, but it provides a good representation of the relative strength or stability of various soil types.

SOIL TYPE	ANGLES OF REPOSE, H:V			
	NON-SUBMERGED		SUBMERGED	
Sand, clean	1½ on 1	34°	3 on 1	18½°
Gravel, clean	11/3 on 1	37°	2 on 1	26½°
Hard rock, rip rap	1 on 1	45°	1 on 1	45°

Since the magnitude of these water body forces is partly proportional to the water depth near the shoreline, the effects of these various water forces can be protected against, by maintaining a shallow foreshore (i.e. shallow water depth) for a minimum distance out from the water's edge (see Figure 142). Alternatively, hard armouring of the submerged slope could provide protection but would be relatively expensive.

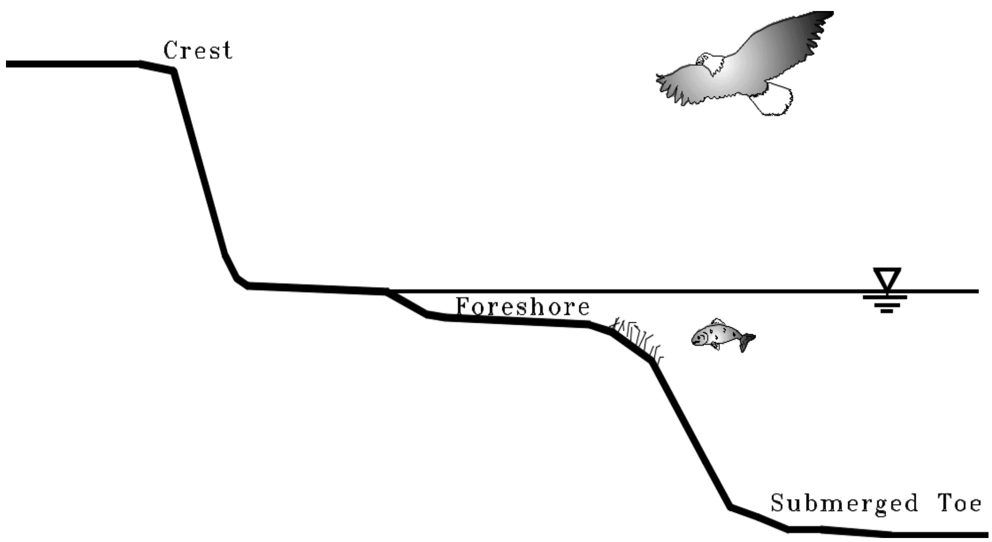


Figure 142 - Foreshores for Submerged Slopes

Some pit operations may not limit their extraction along the shoreline nor would they wish to provide costly shoreline erosion protection measures. Accordingly an alternative method of addressing the erosion risk would be to require an additional minimum set-back of the slope crest from the property line. This would not eliminate the erosion risk but would permit erosion to occur for an extended period of time, before it might pose a risk to the slope crest and adjacent table lands.

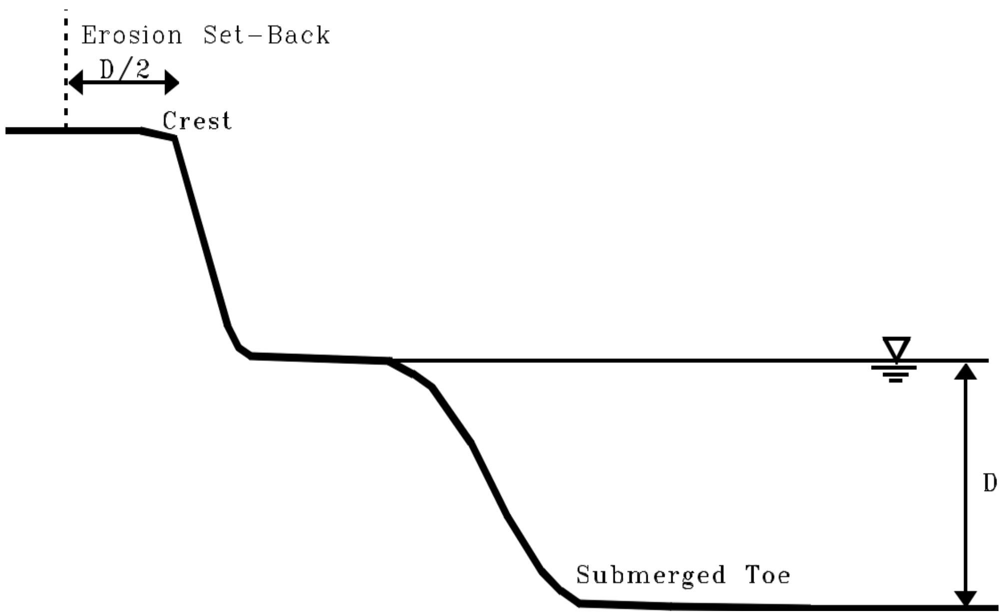


Figure 143 - Erosion Set-back

This erosion set-back should be proportional to the water depth in the foreshore area. For preliminary design purposes, a minimum erosion set-back of one half the water depth or the height of submerged slope, is recommended for the slope crest to the property line (see Figure 143). This minimum set-back could be reduced if suitable erosion protection measures were provided along the shoreline. The erosion set-back could also be reduced on the basis of detailed engineering analysis of the potential water body forces (currents, waves).

11.2 Characteristics of the Graded Slope Material

On pit sites, the final slope surface is often prepared by grading of the 'in situ' material (undisturbed native soil), or by grading of earth fill placed up against an exposed over-steepened pit face (see Figure 144).

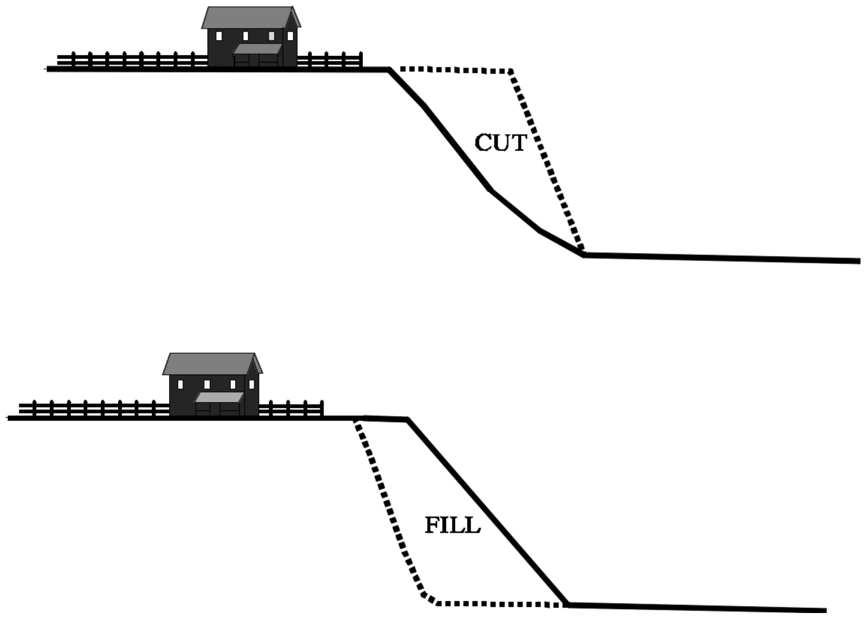


Figure 144 - Slope Grading

The soil type also influences the failure mode or shape of slope slides. Slope slides or failures in cohesionless soils (gravels, sands, silty sands) tend to be relatively shallow and translational involving smaller soil masses. In comparison, slides in cohesive soils (clays, clayey silts) tend to be much deeper and involve larger soil masses. Accordingly slides in cohesive soil slopes can cause more damage to property near the slope crest (see Figure 145).

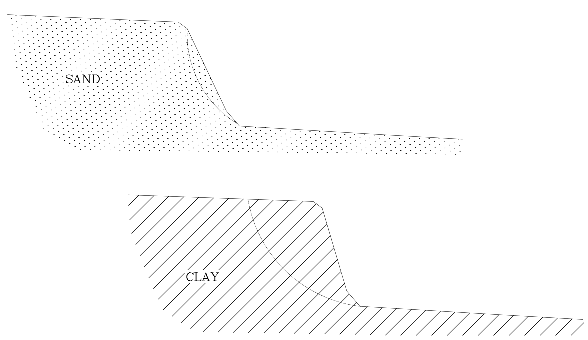


Figure 145 - Failure Surfaces

The stable slope inclination can be increased (i.e. made steeper) if engineering works are undertaken (soil reinforcement, retaining structures, drainage improvements) that are appropriate for that material and site location. For most pit operations, costly engineering works on the slopes are seldom undertaken and stabilization is usually accomplished by flattening of the inclination.

11.3 Land-Use of Adjacent Lands

The appropriate slope inclination of a pit slope along a property boundary, is dictated by the land-use of the adjacent properties which might be impacted by a slope failure. In geotechnical engineering, consideration of the slope failure consequences are built into the design minimum Factor of Safety for the stable slope inclination.

The choice of design minimum Factor of Safety (FS) shown on Figure 146 has the following influence on the stable slope inclination;

- a higher FS has a flatter inclination or a larger horizontal distance between the slope toe and slope crest,

- a lower FS has steeper inclination or a smaller set-back horizontal distance between the slope toe and slope crest,

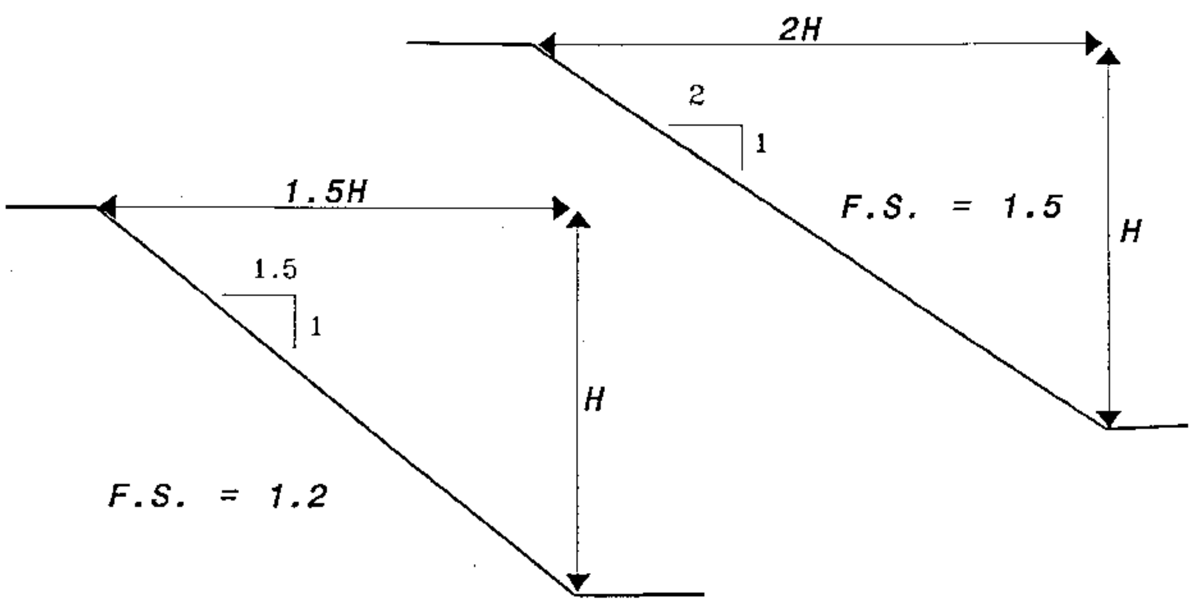


Figure 146 - Affect of Factor of Safety

For detailed engineering analysis, the following typical design minimum Factors of Safety are shown on Table 11.2 for different types of land-use have been suggested in previous sections of this document. These should be used for guidance only and they do not replace the judgement and experience of a qualified professional. Detailed investigation and analysis of site conditions may warrant the use of slightly lower Factors of Safety if supported by a qualified professional.

TABLE 11.2 - DESIGN MINIMUM FACTORS OF SAFETY

	LAND-USES	DESIGN MINIMUM FACTOR OF SAFETY
A	**PASSIVE**; no buildings near slope; farm field, bush, forest, timberland, woods, wasteland, badlands, tundra	1.10
B	**LIGHT**; no habitable structures near slope; recreational parks, golf courses, buried small utilities, tile beds, barns, garages, swimming pools, sheds, satellite dishes	1.20 to 1.30
C	**ACTIVE**; habitable or occupied structures near slope; residential, commercial, and industrial buildings, retaining walls, storage/warehousing of non-hazardous substances	1.30 to 1.50
D	**INFRASTRUCTURE and PUBLIC USE**; public use structures or buildings; hospitals, schools, stadiums, cemeteries, roads, railroads, bridges, high voltage power transmission lines, towers, storage/warehousing of hazardous materials, waste management areas, extraction activities (i.e. aggregates)	1.40 to 1.50

Using the above design minimum FS, the appropriate stable slope inclination or set-back can be determined through an engineering stability analysis. The slope stability analysis should consider only slides which are at least 2 m deep since shallower slides can be controlled with surficial stabilization techniques such as planting or reinforcing (tie-backs, cellular media, granular fill). These design minimum Factors of Safety should be used to determine the 'long-term stable slope inclination' or slope configuration, rather than for short term changes in slope or site conditions created for instance during construction projects or aggregate extraction.

11.4 Summary of Criteria for Stable Slope Inclinations

In summary, an appropriate stable slope inclination is dependent on three criteria;

a) the form of land-use proposed for the slope; maximum 2 : 1 for long-term stability; maximum 3 : 1 for pedestrian or vehicle traffic, and 3 : 1 to 10 : 1 for forestry, recreation, and some agricultural uses; Sand and Gravel Pit Rehabilitation in Northern Ontario (MNR, 1987) manual,

b) the characteristics and properties of the soil slope; these influence the type of potential slide or failure and the stable slope inclination through the Factor of Safety in engineering analysis of slope stability

c) the land-use on top of the table lands adjacent to the pit or quarry; this influences the risk and consequences of slope failure as well as the stable inclination, through the Factor of Safety.

11.5 Selection of Appropriate Stable Slope Inclination

The selection of the appropriate stable slope inclination and method of stabilization

- grading in situ material, or
- grading filled overburden, or
- installation of engineering works

should be determined at the Site Plan proposal stage and should be part of the rehabilitation plan. Procedures for re-vegetation of the slope face should also be defined during the Site Plan proposal stage.

Several pieces of information that are collected to develop the Site Plan proposal can be used in determining the appropriate slope inclination. This information includes; subsurface stratigraphy and test data from boreholes (and laboratory test data); groundwater information from wells, well records, piezometers; and, ground surface topography from maps and surveys.

The above information is utilized in modelling the site conditions as part of a detailed engineering analysis of slope stability. The popular methods of analysis are based on soil mechanics theory and the results produce a 'Factor of Safety' which is used to select an appropriate stable slope inclination.

Once the appropriate stable slope inclination is determined, the location of the toe of the stabilized slope should be mapped on the "Progressive Rehabilitation and Final Rehabilitation Plans" map, and the slope stabilization technique noted and described.

In the case of abandoned pits, the selection of the appropriate stable slope inclination and slope stabilizing technique may depend on a number of factors such as the distance between the pit/quarry slope toe and the adjacent property boundary, the amount of overburden present on site, the proposed use of the rehabilitation pit and/or the amount of money available for pit rehabilitation.

In most pits and quarries, the excavations have taken place gradually and the slope conditions and drainage have generally reached steady state conditions so that slope stability analysis should be based on an effective stress analysis using 'drained strengths' for long-term conditions.

In most cases, extraction has proceeded by excavating into the slope face and progressively advancing the slope position. The slope inclination is often very steep and usually at the limit at which the parent soil materials will stand temporarily without experiencing shallow translational slides. Consequently, the minimum Factor of Safety of these pit and quarry soil slopes for long-term conditions, is often very minimal for stability, typically between 1.0 and 1.2 (or less) and the slope inclination may experience future slides or ground movements to a more stable inclination (flatter). This slope flattening typically results in additional crest loss or recession, which can endanger the nearby structures, property lines, or utilities. The slope face is often bare and exposed, permitting visual identification of soil stratigraphy and seepage from the slope face.

11.5.1 Site Evaluation

As discussed in previous sections, in evaluating the stability of high and/or steep slopes, certain basic information about the site is required,

- the <u>subsurface conditions</u> should be accurately determined with the aid of boreholes and piezometers (soil stratigraphy or layering, soil index properties, and groundwater levels and seepage), and
- the <u>slope configuration</u> should be accurately established (slope height, slope inclination or shape), and
- any <u>external loadings</u> should be considered (structures, traffic, earthquakes).

Information on the subsurface conditions are best determined with boreholes for at least the full height of the slope. However, it is difficult to specify the number of boreholes required, beforehand. Some judgement must be exercised but in general, the number of boreholes and piezometers increases with increasing complexity in site conditions (uniform subsurface conditions - fewer boreholes, complex conditions - more boreholes).

Boreholes for route alignment and buried sewers are usually spaced at intervals of 50 to 150 m depending upon the complexity of the site conditions. On aggregate resource sites, boreholes have often been previously conducted as part of the site evaluation process to assess the resource. On sites with older extraction operations, the soil stratigraphy on the slope may be clearly exposed for visual inspection if not submerged.

Laboratory testing should be undertaken on the borehole samples to measure basic Index Properties (water contents, unit weights, grain size distributions, Atterberg Limits) described in the Appendix.

The slope configuration should be precisely determined using existing topographic mapping (scale of 1:500 or better) if available, and ground surveying of the slope profile. The above information are essential variables which are incorporated in all analytical methods of slope stability, to calculate Factors of Safety.

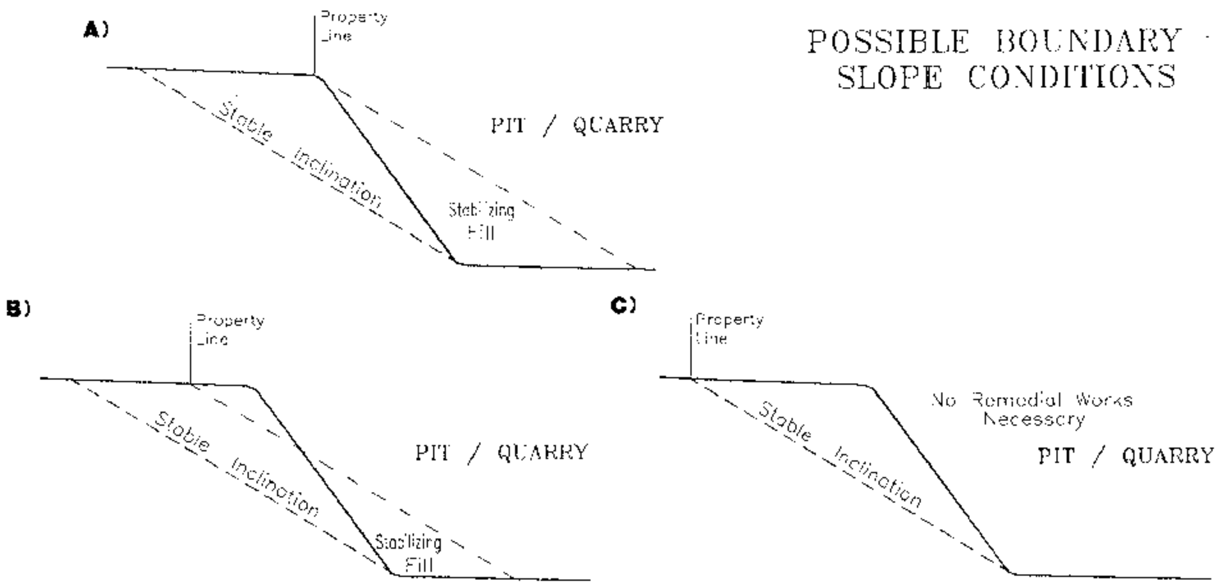


Figure 147 - Control of Slope Inclination

11.5.2 Basic Design Considerations for Typical Pit Slopes

In some old abandoned pit and quarry sites, past extraction activities may have extended very close to the property boundaries and there may be neighbouring houses or structures very close to the slope crest. The minimum Factor of Safety of the existing slopes may be less than that required in the above table, and stabilization measures may be required to improve the minimum Factor of Safety.

Conversely, in active pit sites it is important to determine the long-term stable slope inclination for the site conditions, in order to limit extraction activities and the position of the slope toe. This will ensure that the design minimum Factor of Safety will be maintained for slopes along property boundaries.

Where the slope crest can be allowed to recede without jeopardizing the minimum Factor of Safety, the slope can be allowed to naturally self-stabilize without undertaking extensive/expensive stabilization measures. In many cases the over-steepened slope may be very close to the property boundary and additional crest loss is unacceptable so that the slope must be stabilized.

The stabilization alternatives generally consist of control of the overall slope inclination by either;

- limiting excavations so as to maintain the slope inclination to the design stable angle or flatter, or
- if excavations are taken to a slope inclination steeper than the design stable angle, backfilling against the excavated slope with earth fill to achieve a design stable slope angle which is suitable for the fill material.

11.5.3 Typical Stable Slope Inclinations

Due to the high slopes associated with most pits, the slope soils are usually in a well-consolidated condition from the weight of the overlying strata. One exception to this is 'sand dune' deposits which can be in a very loose state for considerable depth.

The following stable slope inclinations have been observed for vegetated natural soils. Complex stratigraphy (layering) including thin weak layers or hard layers within the slope, requires detailed analysis to assess stability and not the following simplified tables. These inclinations do not pertain to earth fills which may be placed on slopes to flatten the overall inclination (discussed in next section).

Natural Slope Material	**Steepest Stable Inclination Observed** (horiz. to vert.)
Shale, Limestone	near vertical to 1 : 1 90° to 45°
Glacial Tills	1½ : 1 to 2 : 1 34½° to 26½°
Cohesionless Sands, Gravels	1½ : 1 to 2 : 1 34½° to 26½°
Glaciolacustrine Clays and Silts Marine Clays (Leda Clays)	2 : 1 to 4 : 1 26½° to 14°

For well consolidated soils, steeper inclinations may be appropriate if the slope surface is reinforced, to protect against surface erosion and still permit establishment of vegetation on the slope face. This reinforcement is often too costly for large slope areas.

The long-term stable slope crest position can be calculated, based on the slope height and can be measured from the slope toe or from the slope crest (see Figure 148).

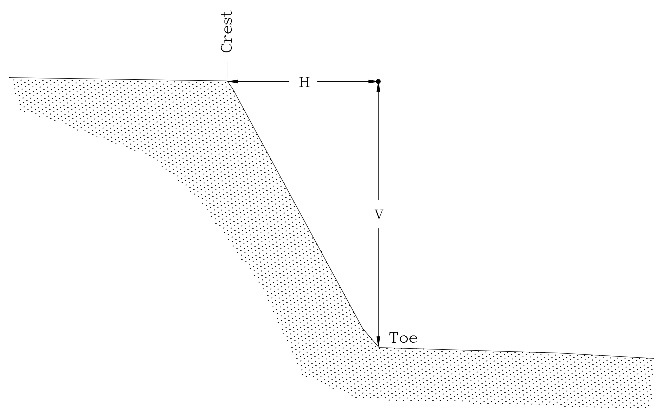


Figure 148 - Slope Crest Position

For 'wet slopes' where significant seepage is present on most of the slope face, slightly flatter inclinations may be required or, improved drainage measures could be undertaken. It should be noted that stable slope inclinations steeper than 2 to 1 may have difficulty in establishing and maintaining vegetation without some local reinforcement of the slope surface.

11.5.4 Summary of Slope Assessment Procedure

In order to assess the 'long-term stable inclination' of soil slopes for a specific pit or quarry site using the above simplified tables, the following steps are required;

1) determine the <u>physical dimensions</u> of the slope; horizontal position of the slope toe, slope crest, and property boundary, as well as the slope height,

   the dimensions can be visually estimated or scaled from mapping produced by air photo interpretation for preliminary assessment, but should be accurately surveyed for final design purposes,

2) determine the <u>subsurface conditions</u> of the slope mass; soil stratigraphy, soil types, soil density or consistency, groundwater conditions,

   the conditions can be visually estimated from the exposed slope conditions for preliminary design, but should be more accurately ascertained on the basis of boreholes and piezometers,

3) examine the <u>land-use</u> along the slope crest (within a horizontal distance from the crest of 1 x 'slope height') to select the appropriate design minimum Factor of Safety from Table 11.2 above.

With the above basic information, Table 11.2 can be used to select the minimum design Factor of Safety for engineering analysis of the 'long-term stable slope inclination'. This inclination may be steeper than the maximum inclination on which vegetation will establish and can be maintained. It is not usually economical to artificially reinforce the slope surface to a steeper angle.

The stable slope inclination should therefore be selected on the basis of the conditions required to permit establishment of vegetation on the slope face.

11.6 Slope Stabilization Methods

For the high slopes typically found on aggregate extraction sites, the most economic stabilization method is control of the slope inclination. Retaining structures, reinforcement of the slope soil, or improved drainage of the slope, are often too costly. In considering slope stabilization, the most effective method is to reduce the influence of gravity (main force causing instability) on the slope, by flattening the overall slope inclination. Please refer to Chaper 10 for further details on slope stabilization methods.

Stabilizing a slope by filling against it, can be undertaken in several manners which have been outlined in Chapter10. Cohesive or fine-grained soil material are different from the pit slope material. Sandy soil material (cohesionless material) is similar to the underlying pit slope material and the applications for these types of materials are appropriate for pit slopes.

11.7 Surface Erosion Control and Vegetation

Pit slopes are often steep, long, and bare of vegetation. They are therefore very prone to surface erosion. Due to the high and long slopes, surface erosion can have a greater adverse affect on the slope condition than slope movements related to mass stability.

Most exposed pit slopes are composed of cohesionless soils (sand, sand and gravel) which are very susceptible to surface erosion from rain splash, sheet run-off, rilling, and gullying. Please refer to Chapter 10 for further details.

11.8 Soil Bioengineering and Biotechnical Stabilization methods

Pit slopes are also good candidates for the application of biotechnical stabilization and soil bioengineering techniques. These terms have been defined as;

- biotechnical stabilization protection - the combined or integrated use of vegetation and structural components to protect against erosion,
- soil bioengineering - the use of living plant materials to protect against erosion (brush layering or contour wattling).

Further details on these approaches can be found in Chapter 10.

GLOSSARY

ANGLE OF INTERNAL FRICTION, ϕ'

The maximum angle of obliquity between the normal and the resultant stress acting on a surface within a soil.

ANGLE OF REPOSE

The angle with a horizontal plane at which loose material will stand on a horizontal base without sliding.

ARCHING

The transfer of stress from a yielding part of a soil mass to an adjacent, less yielding or restrained part of the mass.

ATTERBERG LIMITS

See plastic limit, liquid limit.

BACKFILL

Soil material placed back into an area that has been excavated, such as against structures and in pipe trenches.

BEARING CAPACITY

The pressure that can be imposed by a foundation onto the soil or rock supporting the foundation.

BENTONITE

A naturally occurring clay material, with the ability to swell significantly with the addition of water.

BORING

The method of investigating subsurface conditions by drilling into the earth. Frequently, soil or rock samples are also extracted from the boring for classification and testing.

BORROW

Soil or rock material obtained from an off-site source for use as fill on construction projects.

BOULDER CLAY

A dense unstratified deposit of glacial origin consisting of a hard matrix (usually rock flour) packed with subangular stones of varied sizes.

CAISSON

Large cylindrical concrete foundation unit, usually augered and cast-in-place; typically 1-2 m in diameter and possibly 5 m or more deep.

CALCAREOUS

An adjective applied to rocks containing calcium carbonate.

CAPILLARITY

The movement of water, due to the affinity between soil and water which acts to increase the boundary of contact between the two materials, and the surface tension property developed by water in contact with air. The height of capillary suction for various soils ranges from a few centimetres in gravelly soil, to about 1 m in sands, to several metres in silt or clay soils.

CLAYS (CLAY MINERALS)

Very small soil particles having a crystalline (layered) structure, created as the result of the chemical alteration of primary rock minerals. Most clay particles, because of their mineralogical composition, are flat or plate-like in shape, with a large surface area to mass ratio. Clay particle dimensions are often smaller the 2 microns.

COARSE-GRAINED SOIL
Those soil types having particles large enough to be seen without visual assistance. The coarse-grained materials include sand and gravel (or larger) soil particles.

COEFFICIENT OF PERMEABILITY, k
The rate of discharge of water under laminar flow condition through a unit cross-sectional area of a porous medium under a unit hydraulic gradient and standard temperature (20° c).

COHESION
The bonding or attraction between particles of fine-grained soil that creates shear strength.

COHESION, C
The portion of the shear strength (s) indicated by the term c in Coulomb's equation: $s = c + \Phi \tan \Phi$, where Φ is the angle of internal friction. It has the nature of an intergranular binding force. Also see shear strength.

COMPACTION
The process of increasing the density or unit weight of a soil (frequently fill soil) by rolling, tamping, vibrating, or other mechanical means.

COMPRESSIBILITY
The change, or tendency for change, that occurs in the thickness of a soil mass when it is subjected to compressive loading.

COMPRESSION
The reduction in volume of a soil mass caused by the application of external forces.

CONSOLIDATION
The reduction in volume of a soil mass that is achieved with the passage in time, caused by naturally occurring forces.

CREEP
An increase in plastic strain with time is usually called creep. Also associated with the long term, gradual movement of a slope or embankment.

CREST
The top of a slope or embankment.

CRITICAL SURFACE (CRITICAL FAILURE SURFACE)
The sliding surface for which the factor of safety is at a minimum in an analysis of a soil slope.

DEGREE OF SATURATION, S
The ratio of the volume of water in the soil voids to the total volume of voids.

DENSITY (MASS DENSITY)
The mass of a material in relation to a unit volume. See also Relative Density.

DEWATERING
The procedure utilized to remove water from an area, such as pumping from an excavation or location where water covers the planned working surface; the procedure used to lower the groundwater table in order to obtain a "dry" area in the vicinity of an excavation which would otherwise extend below water.

DISPERSIVE CLAYS

Clay soils deflocculate in still water and erode when exposed to a flow of water. A clay-poor water system that has a high concentration of sodium ions tends to have a high dispersivity.

DISPLACEMENT

The straight line distance between two points.

DRAWDOWN

The lowering of the level of the groundwater table that occurs in the vicinity of a water well (or dewatering equipment) when it is pumped.

DYNAMIC COMPACTION

Procedure whereby surface and near-surface zones of soil or fill are compacted (densified) by dropping a weight (commonly 5 to 15 tons) from a relatively great height (drops of 30 to 100 feet are typical). Multiple blows are provided at each drop location, and closely spaced drop locations are utilized to improve a construction site.

EARTH PRESSURE

Normally used in reference to the lateral pressure or force imposed by a soil mass against an earth-supporting structure such as a retaining wall or basement wall, or on a fictitious vertical plane located within a soil mass.

EFFECTIVE STRESS

The actual particle-to-particle contact stress (or pressure) existing between soil grains. This stress compensates for the possible buoyancy influence of water pressure. Effective stress relates directly to the shear strength possessed by a soil.

FACTOR OF SAFETY

The ratio of available shear strength to shear stress on the critical failure surface.

FILL

Earth placed in an excavation or other area to raise the surface elevation. Also referred to as earth fill or soil fill. Structural earth fill refers to material which is placed and compacted in layers in order to achieve a uniform and dense soil mass which is capable of supporting structural loading.

FINES OR FINE-GRAINED

Refers to silt and clay sized particles which exist in a soil mixture.

FINGER DRAINS

A drainage system which consists of strips of pervious drainage materials.

FILTER

Natural soil or artificial material with pore sizes such that it will permit the free passage of water, yet sufficiently small to prevent the passage of fine soil particles from the protected soil.

FLOW LINE

The path of travel of moving water as it flows through a soil mass.

FLOW NET

A grid which is formed by the intersection of two sets of orthogonal lines, representing the loci of particles of liquid as they pass through the porous medium and the loci of points having the same potential. The flow net is used to estimate the rate of seepage flow and to predict the piezometric pressure at any point within the grid.

FOOTING

Type of foundation typically installed at a shallow depth and constructed to provide a relatively large area of bearing onto the supporting soil.

FRICTION, INTERNAL

The particle (solid to solid) friction developed by cohesionless soils, and the property responsible for most of the shear strength which this type of soil can develop.

GABIONS

Stone-filled steel wire baskets which can be assembled or stacked like to act as retaining walls or provide slope and erosion protection.

GEOFABRICS

Synthetic fabrics used as filters, drains, or reinforcements in earthwork projects.

GROUND WATER TABLE

The surface of the underground supply of water. Also referred to as the phreatic surface. The level below which the pores of the subsoil, down to indefinite depth, are full of water.

HEAD

Shortened form of the phrase pressure head, referring to the pressure resulting from an elevated column or supply of water. Pressure would be computed from $y_w h$, where y_w is the unit weight of water, and h is the height or elevation of the water supply.

HEAVE

Upward movement, caused by expansion occurring as a result of such factors as freezing or swelling due to increased water content, or by the removal of confining stresses. Frost heave refers to the vertical soil movement that occurs in freezing temperatures as ice layers or lenses form within the freezing soil and cause the soil mass to expand.

HYDRAULIC GRADIENT

Mathematical term indicating the difference in pressure head existing between two locations divided by the distance between these same locations. Given the designation i.

IN-SITU

Refers to soil when it is at its natural location and condition in the earth.

ISOTROPIC

Pertaining to a soil whose properties are the same in all directions.

LANDSLIDE

The relatively rapid downhill movement of a generally well-defined earth mass or landform due to gravitational forces.

LIMIT EQUILIBRIUM

A method of analysis used to evaluate the stability of a soil mass that could be involved in movement associated with failure. The method involves determining the soil shear strength required to maintain equilibrium or stability on an assumed failure surface, and compares this value with the actual shear strength of the soil; this comparison indicates if the limit of equilibrium will be exceeded.

LIQUID LIMIT, W_L

The water content at which a soil exhibits low shearing strength is taken to be the boundary between the soil's liquid and plastic behaviour.

LIQUEFACTION

The loss of strength occurring in saturated cohesionless soil exposed to shock or vibrations, when the soil particles momentarily lose contact. The material than behaves as a fluid.

MECHANICAL WEATHERING

The process of weathering whereby physical forces, such as frost action and temperature changes, break down or reduce rock to smaller fragments without involving chemical changes.

METHOD OF SLICES

A general procedure used for slope stability analysis in soil. A trial surface is chosen and the potential sliding mass is divided into a number of vertical slices. Each slice is acted on by its own weight which produces shearing and normal forces on its vertical boundaries and along its base.

METHOD OF INFINITE SLICES

A procedure involving a circular sliding surface for which the stability of the potential sliding mass is considered as a whole.

MINERAL

A naturally formed chemical element or compound having a definite chemical composition and usually a characteristic crystal form.

MOHR ENVELOPE (FAILURE ENVELOPE)

The envelope of a series of graphically plotted Mohr circles representing stress conditions at failure for a given material. According to Mohr's strength theory, a failure envelope is the locus of points the co-ordinates of which represent the combinations of normal and shear stresses that will cause a given material to fail.

MUSKEG

A Canadian term of Cree Indian origin, meaning a moss covered muck or peat bog.

NON-CIRCULAR SLIDING

The failure surface in a soil slope which follows a non-circular path.

OUTCROPS (OF ROCK)

Those places where bedrock is exposed at the ground surface.

STANDARD PENETRATION TEST

Term generally applied to subsurface investigation methods for determining a strength related property of a soil by measuring the resistance to advancement of penetration or test equipment.

PERMAFROST

The permanently frozen ground located in the northern regions of the Earth.

PERMEABILITY

The ability of water (or other fluid) to flow through a soil by travelling through the void spaces. A high permeability indicates flow occurs rapidly, and vice versa.

PHREATIC

Pertaining to ground water. See Ground Water Table.

PIER

Category applied to column-like concrete foundations, similar to caissons. The pier is generally considered the type of deep foundation which is constructed by placing concrete in a deep manually-excavated excavation.

Pier is also used frequently to indicate heavy masonry column units which are used for basement-level and substructural support.

PIEZOMETER
A device for measuring the hydrostatic pressure at a point in the ground. Simple piezometers are open holes or standpipes for measuring the groundwater table.

PILE
A relatively long, slender, column-like type of foundation which obtains support capacity from the soil or rock some distance below the ground surface. May be comprised of timber, pre-cast concrete, or steel elements.

PIPE BEDDING
Material that is used to support a buried conduit. Generally, preparation of bedding reduces the stresses that will develop in the pipe (conduit) from an overlying fill.

PIPING
The internal erosion and carrying away of fine material from within a soil as the result of a flow of water. Refers to the pipe-shaped discharge channel left by erosion which starts at the point of exit of a flow line which exits on the ground surface; typically beneath embankments or on slopes where perched groundwater may seep out.

PLANE STRAIN
Term applied to fine-grained soils (particularly clays) to indicate the soil's ability to flow or be remoulded without ravelling or breaking apart.

PLASTIC LIMIT, W_p
A boundary region of water content representing a change in characteristics of the soil from those of a plastic to those of brittle material. This water content is called the plastic limit.

POISSONS RATIO
The ratio of lateral unit strain to the longitudinal unit strain in a body that has been stressed longitudinally within its elastic limit.

PORE WATER PRESSURE, u_w
Stress transmitted through the pore water.

PHREATIC SURFACE
The surface along which the pressure in the fluid equals atmospheric pressure.

PHOTOGRAMMETRIC MAPPING
The mapping of surface exposures or geological structures done by photogrammetric techniques.

PRESSUREMETER
An instrument used to determine the in-situ strength of a soil zone through measurement of the pressure-related lateral expansion of a flexible cylinder which is at a known depth in a borehole.

REINFORCED EARTH
Earth structures such as embankments, retaining walls, and dams which are constructed in layers reinforced with geofabrics to increase the strength of the soil mass.

RELATIVE DENSITY
Term applied to sand deposits to indicate a relative state of compaction compared to the loosest and most dense conditions possible.

RELIEF WELLS

Wells drilled to control pore water pressure beneath an embankment, soil slope, or dam structure.

RETAINING WALL

A vertical structure designed to resist the lateral pressure of soil and water behind it.

REVETMENT

A facing or layering of stone, concrete or other durable material built to protect an embankment or shore structure from wave erosion.

ROCK FLOUR

A general term for fine-grained, non-plastic material corresponding in size to silt, comprised of more or less equidimensional grains of unweathered mineral particles.

ROLLERS, COMPACTION

The category of construction equipment utilized to compact (or densify) soil by rolling it. The compaction force typically results from the heavy weight of the equipment and / or vibrations transmitted from the equipment into the soil.

ROTATIONAL SHEAR SLIDE

A slide resulting from the yielding and redistribution of shear stresses in a soil so that a more or less circular surface of failure envelope develops before the cohesion breaks down and permits a comprehensive, circular sector of the slope to fail by rotating.

SAND

The category of coarse-grained soil whose particle sizes range between about 0.07 mm and 5 mm in diameter.

SEEPAGE

Generally refers to the quantity of water flowing through a soil deposit or soil structure such as an earth dam. Also may refer to the quantity of subsurface water leakage into a building's underground (basement) area.

SEISMIC

Pertaining to an earthquake or earth vibration, including one that is artificially induced.

SETTLEMENT

The downward vertical movement experienced by a structure or a soil surface as the underlying supporting earth compresses or consolidates.

SHALE

A laminated sedimentary bedrock in which the constituent particles are predominantly of the clay type. The characteristic feature is that of bedding.

SHEAR FAILURE

Failure resulting from shear stresses.

SHEAR STRENGTH

The internal resistance offered to shear stress. It is measured by the maximum shear stress, based on original area of cross section, that can be sustained without failure.
Peak shear strength - at a certain level of shear stress the shear strength of the surface is exceeded and further displacement will take place without any further increase in shear stress. This limiting value defines the peak shear strength at that particular normal stress.
Residual shear strength - after a surface of sliding forms and extensive slip occurs, the particles along the slide surface assume an orientation favourable to a low resistance to shear, and shear displacement takes place at a constant shear stress level. This shear stress is called residual shear strength.

SHRINKAGE LIMIT, W_s

The water content of a soil below which further loss of water by evaporation does not result in a reduction of volume of the soil.

SIEVE

Pan or tray-like equipment having a screen or mesh bottom; used in laboratory or field work to separate particles of a soil sample into their various sizes.

SILT

The category of fine-grained non-cohesive soil particles whose individual soil particle size is smaller than 0.07 mm.

SOIL SAMPLER

The equipment used to extract soil samples from borings or test pits made in a subsurface investigation.

SOIL STABILIZATION

Treatment of soil improve its properties; includes the mixing of additives and other means of alterations such as compaction or drainage.

SPECIFIC GRAVITY

The ratio of the weight in air of a given volume of soil particles to the weight in air of an equal volume of distilled water at a temperature of 4° c.

STRATUM

A layer that is discernable along bedding planes from layers above and below it. The separation arises from a break in deposition or a change in the character of the material deposited.

STRESS

An intensity of force. The ratio of forces per unit area, acting within a body.

Effective stress - The average normal force per unit area transmitted from grain to grain in a granular mass. It is the stress that is effective in mobilizing internal friction.

Principal stresses - Stresses acting normal to three mutually perpendicular planes, intersecting at a point in a body, which is at equilibrium.

Shear stress (Shearing stress) - The stress component tangential to a given plane.

Total stress - The total force per unit area acting within a mass. It is the sum of neutral and effective stresses.

SUMP

Small excavation or pit provided in the floor of a structure or in the earth to serve as a collection basin for surface water and near-surface underground water.

TALUS

A term used for the accumulation of mixed fragments and particles fallen at or near the base of cliffs or slopes.

TILL

Description given to glacially transported soil formations which consists of a heterogeneous mixture of fine-grained and coarse-grained material.

TILLITE

A term applied to consolidated boulders clays formed during glacial epochs anterior to that of the Pleistocene.

TRAP ROCK

An old Swedish name originally applied to igneous rocks that were neither coarsely crystalline, like granite, nor cellular and obviously volcanic, like pumice. The word is often popularly used to describe local rock types and so is one to be generally avoided.

UNIT WEIGHT

Weight per unit volume. See Density.

Dry unit weight **-** The unit weight of a material containing no water.

Saturated unit weight **-** The weight of a saturated mass.

Submerged unit weight **-** The weight of a mass in air minus the weight of water displaced by the mass per unit volume of the mass.

VOID RATIO, E

The ratio of the void volume to solid volume in a soil mass.

WATER CONTENT

The ratio of the weight of water in a soil to the weight of the soil solids, typically expressed as a percentage.

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