ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

OPTIMUM DESIGN OF HIGHWAY CUT SLOPE IN SOME SELECTED SOILS FOUND IN ETHIOPIA

BY

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DECEMBER 2010, ADDIS ABABA

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Optimum Design of Highway Cut Slope in Some Selected Soils

Found in Ethiopia

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LIST OF TABLES

Page

LIST OF FIGURES

Page

LIST OF SYMBOLS

c'	cohesion interms of effective stress;
c	cohesion interms of total stress;
ϕ'	angle of internal friction of soil interms of effective stress;
ϕ	angle of internal friction of soil interms of total stress;
ϕ_r	residual internal friction angle;
σ	normal stress
τ	shear stress
R_i	the total normal force on the base of the slice;
W_i	the weight of a slice of width b and height h;
θ	the angle between the vertical line to the center of the base of each slice and the normal force;
S_u	the shear force mobilized on the base of each slice;
F	Factor of Safety;
FOS	Factor of safety;
H	the height of each interslice surface;
h	the height of each slice from the center of the base;
L	the length of the base of each slice;
r_u	pore pressure ratio;
u	the excess pore pressure acting at the base of each slice;
γ	Unit weight of soil

LIST OF ABBREVATIONS

AASHTO	American Association of State Highway and Transportation
ASTM	American Society for Testing and Materials
BS	British Standard
CD	Consolidated Drained test
CDSCo	Construction Design Share Company
CU	Consolidated Undrained test
DED	District Engineering Division
ERA	Ethiopian Roads Authority
FEM	Finite Element Method
NC	Normally consolidated clay
OC	Over consolidated clay
SATCC	South African Transport and Communications Commission
Slope/W	One component of a complete suite of geotechnical products called Geostudio that has been designed and developed to be a general software tool for the stability analysis.
TCDSCo	Transport Construction Design Share Company
TRL	Transport Research Laboratory
UU	Unconsolidated Undrained test
USCS	Unified Soil Classification System

ABSTRACT

This thesis presents the safe and economical highway cut slope design in some selected soils found in Ethiopia. The preparation of the standard has been carried out by determining the soil shear strength parameters in the laboratory and determining the safe and economical slope using the factor of safety contour charts.

In this work, first, factor of safety contour charts are produced. Using these charts existing slopes are assessed. In addition to collecting soil parameters from previously analyzed soil samples, three soil samples were collected for laboratory analysis. Using the compiled soil parametes and prepared factor of safety contour charts, cut slope standard tables are prepared for the different soils. Lastly, the output of this standard is compared with ERA 2002 cut slope standard for cost – benefit analysis.

Generally, it has been observed that the newly developed highway cut slope standard is safe and economical. For the cases investigated in this work, the reduction in cost ranges from 19 to 32% f that of the current ERA 2002 standard.

Key Words: Slope Stability, highway cut slope, Factor of safety, Design Chart

TABLE OF CONTENTS

Page

1.0 INTRODUCTION

1.1 Background and Objective

Slope stability analysis is carried out to minimize the occurrences of slope failure or landslide. Slopes along the highways undergoing deep excavations are very susceptible to failure. Engineers must therefore give serious considerations before any construction or development is executed to ensure the stability of slopes.

Shallow and deep cuts are important features in road projects that are undertaken in rolling and mountainous terrain found in a country like Ethiopia. Steep cuts often are necessary because of right-of-way and property line constraints. Flat cut slopes, which may be stable for an indefinite period, are often uneconomical and impractical. Slopes that are too steep may remain stable only for a short period of time. The design must consider measures that will prevent immediate and sudden failure as well as protect the slope over the long term.

Different highway consulting firms use different cut slope standard in designing roads. The current ERA road standard uses cut slopes based on height of cut only without considering the type of soils. This state-of-affairs had been instrumental for slope failures and has motivated the researcher to look into the problem.

In practice, it is often necessary to form a rapid assessment of stability conditions for a large number of slopes, making direct application of the model impractical. This can be achieved by preparing design charts.

The objective of this thesis is to develop a simple mechanism for the design of safe highway cut slopes that may be deployed by institutions which are involved in design and construction of roads.

1.2 Scope of the Study

Within the framework of this study, charts of factor of safety contours shall be developed taking into consideration cut height, soil type and slope. From these charts safe and economical slope is determined using predefined factor of safety.

The appropriate soil parameter will be collected from previous works and laboratory analysis of samples collected from randomly selected areas. The selected areas are Lemmi, Amanuel, and Armenia which are 123 km, 331 km and 202 km respectively from Addis Ababa.

1.3 Organization of the Thesis

The thesis is organized in the following four main chapters.

The introduction chapter highlights the background of the problem, objectives and scope of the thesis. Chapter two is devoted to the literature review of the general concept of slope stability analysis. Chapter three deals with data collection, presentation of results from laboratory tests, factor of safety contours, and cut slope standard tables. Overall conclusions and recommendations of the thesis are presented in Chapter four.

1.4 Limitations

Due to the financial constraints, it would not be possible to conduct exhaustive geotechnical tests. As a consequence of these very few laboratory tests are conducted. Additional laboratory test results are taken from the works of (Taddesse 1989), (Tarekegn 2009) and other laboratories in Addis (TCDSCo, CDSCo).

2.0 GENERAL SLOPE STABILITY CONCEPT

2.1 Introduction

Slopes either occur naturally or are engineered by humans. Man made slopes include road way cut and fill slopes, embankments, dams and other similar constructions.

Slope stability problems and associated catastrophes have occurred throughout history when the delicate balance of natural slopes has been disrupted by human beings or by nature. Failure can occur due to faulty designs of engineered slopes or unforeseen natural hazards, which cause the disruption of even engineered slopes because they were not anticipated during the design process.

Increased demands, particularly for engineered cut and fill slopes on various types of construction projects over the years have increased the need to understand failure mechanism. Analytical tools and stabilization methods have been developed to solve and mitigate slope stability problems. In general, a basic understanding of geology, hydrology and soil properties is required for the proper identification of underlying principles, conditions and applications of slope stability principles to particular or general problems.

Slope stability analyses are carried out with the aim of conducting safe and economical design of excavations, embankments, earth dams, landfills etc. and understanding of nature, magnitude and frequency of potential slope problems.

Topography, geology and material properties often relating to whether the slope was naturally formed or engineered are taken into account in defining and formulating a given slope stability problem.

Through slope stability analysis:

- The development and form of natural slopes and the process responsible for the different natural features are understood;
- Stability of slopes under short term, or long term conditions is assessed;
- The possibility of landslides involving natural or existing engineered slopes are assessed;
- Failure mechanisms and causes of environmental factors in existing landslides are assessed;
- Effects of seismic loads on slopes and embankments are analyzed;
- Redesign of failed slopes and devising remedial measures, where necessary, are carried out.

Many projects intersect ridges and valleys, and these landscape features can be prone to slope stability problems. Natural slopes that have been stable for many years may suddenly fail because of changes in topography, seismicity, ground water flows, and loss of strength, stress changes, and weathering [2].

Any slope stability analysis, regardless of the method used, demands reasonably accurate modeling of the site subsurface condition, ground behavior and applied loads. Analysis results must be judged based on acceptable risk or safety margins and validity of solution with respect to accepted trends of soil behavior.

2.2 Method of Slope Stability Analysis

2.2.1 General

The problem of slope stability involves consideration of wide variety of parameters such as body forces, porewater pressures, soil strength parameters, topographic and geologic conditions, etc.

Slope stability problems are statically indeterminate, as the available conditions of static equilibrium are insufficient to determine the stress state within the soil mass. Generally, the solution to such problems requires application of methods founded on basic continuum mechanics and should employ representative constitutive models of the material involved. To obtain solutions for loading conditions varying from small to sufficiently large to cause collapse of a portion of a soil mass, complete elasto-plastic analysis considering the mechanical behavior of the soil mass until failure should be conducted. However, this leads to very complex computations and is not routinely used.

In continuum mechanics, three types of equations are needed to determine the stresses and strains in a certain body, under the influence of given stresses and displacements on the surface of that body. These include equilibrium equations, constitutive relations and compatibility conditions. This is typically a complex and formidable task even for the simplest types of materials (linear elastic materials).

For soils, which are nonlinear and inelastic, various solution methods have been developed and put to use over the years. Such methods include limit equilibrium methods, limit analysis methods, variational calculus methods, finite element method and more recently discrete element methods of analysis and probabilistic analysis approaches.

2.2.2 Limit Equilibrium Methods

At present several methods of stability analysis in existence which apply the limit equilibrium principle. Most of these methods apply the technique of slices. These methods of slope stability analysis are based on plasticity theories and consider the conditions of equilibrium at impending failure of the soil mass in a slope. The materials involved are assumed to conform to Mohr – Coulomb failure criterion and to have rigid – plastic deformation properties. These methods mainly differ in the shape of the assumed slip surfaces and in the handling of the indeterminacy of the problem.

In the method of slices, a failure surface is assumed and the soil mass above the failure surface divided into slices. A set of stresses is estimated along the potential failure surface so that conditions of global and local static equilibrium are satisfied for the soil mass overlying the slip surface. This state of mobilized stress, which is not necessarily the true state along this surface, provides an approximate solution that can be used to evaluate the factor of safety. A critical slip surface is then searched, for which the factor of safety is minimized. For the computation of the factor of safety, which is defined in terms of strength, the available strength along the slip surface is computed based on Mohr – Coulomb failure criterion.

2.2.3 Finite Element Method (FEM)

The Finite Element method of analysis is a powerful general-purpose method, which can be used for analyzing stresses and deformations, pore pressures, groundwater flows etc. in a soil mass.

With the FEM, it is possible to model complex conditions with a high degree of realism, including non-linear stress-strain behavior, non-homogeneous conditions, and sequences of events such as changes in geometry during construction of embankments or excavations, changes in pore-water pressures, etc.
Unlike limit equilibrium methods, which employ assumptions, Finite Element Method solves the statically indeterminate problem of slope stability analysis by introducing additional equations that consider the stress strain characteristics of the soil and the requirements of compatibility of deformations.

Although the FEM provides a powerful technique, it is associated with various complexities that have limited its application to solve practical problems [2,21,57]. The major point in this respect is that, it requires accurate information and input data regarding initial stress states involved, correct constitutive models for the slope materials, and correct parameters for the constitutive models. If uncertainties exist about the input data, the obtained results would be as doubtful and conducting such expensive, rigorous and sophisticated mathematical analysis becomes a futile exercise.

2.2.4 Limit Analysis Methods

Limit analysis methods are based on the lower and upper bound theorems of plasticity to provide relatively simple bounds to the true solution. The methods assume perfect plasticity behavior of soil and consider the Mohr-Coulomb failure criterion as the yield condition to be satisfied at incipient failure. Solution to the problem of stability is attempted by giving possible upper and lower limits to the stresses and deformations within a soil mass (not the complete stresses and deformations).The two basic theorems of plasticity employed in limit analysis methods are:

- Lower bound theorem – which states that the collapse load corresponding to an equilibrium system or a statically admissible field of stresses, is a lower bound to the actual collapse load. A statically admissible stress field is one that satisfies all stress boundary conditions, is in equilibrium and never violates the Mohr-Coulomb yield criterion.
- Upper bound theorem – which states that the collapse load calculated from a kinematically admissible field of displacements (a mechanism), is an upper bound to the actual collapse load. A kinematically admissible field of displacements is one that gives compatible field of displacements, satisfies the boundary conditions for the displacements and where deformations occur the corresponding stresses satisfy the yield condition.

For details of the methods, reference can be made to books on Plasticity theories and papers on limit analysis of slope stability problems [14, 32, and 34].

2.2.5 Methods Based on Variational Calculus

This method attempts to solve the slope stability problem by making use of the calculus of variations. The factor of safety equation is formulated as a functional, F, of two unknown functions: the potential slip surface function, y(x), and the stress distribution function, s(x). The factor of safety is obtained by minimizing on the safety functional, F.

Instead of making arbitrary assumptions on the stress functions or the kinematical functions as done by the limit equilibrium methods, it purports to derive the properties of these functions based on the limit equilibrium principles and variational calculus technique.

However, the application of this method has been found questionable both on theoretical basis and from evaluation of practical results. In addition, the method is a mathematically complex method, and is not as commonly used for routine applications [33].

2.3 Causes of slope failure and failure Modes

2.3.1 Causes of Slope Failure

Slope failures are often caused by processes that increase shear stresses or decrease shear strengths of the soil mass. The causes of failure of slopes may be external or internal. External causes are those which produce an increase in the stress at unaltered shearing resistance of the material. They include steepening of the slope, deposition of material along the edge of slopes, and earthquake forces [38].

Internal causes are those that lead to a slide without any change in surface conditions which involve unaltered shear stresses in the slope material. Some of these conditions are:

- The decrease in shearing resistance brought about by excess pore water pressure
- Leaching of salts
- Softening
- Breakage of cementation bonds, and
- Ion exchange

Processes that cause an increase and decrease in shear stresses acting on slopes are listed in Table 2.1 and 2.2 respectively.

Table 2-1: Factors that cause increase in Shear Stresses in Slopes [2]

No	Factor	Cause	Remark
1	Removal of support	Erosion	By streams and rivers, glaciers, action of waves, successive wetting and drying
		Natural Slope movement	Falls, slides, settlements
		Human activity	Cuts and excavation, removal of retaining walls, drawdown of bodies of water
2	Overloading	Natural causes	Weight of precipitation, accumulation of materials because of past slides
		Human activity	Construction of fill, building and other overloads at the crest, water leakage in culverts, water pipes , and sewers
3	Transitory effects	Earthquakes	

No	Factor	Cause	Remark
4	Removal of underlying materials that provide support		
5	Increase in lateral pressure	Water in cracks and fissures	
		Freezing of the water in the cracks	
		Expansion of clays	

Table 2-2: Factors that Cause Reduced Shear Strength in Slopes [2]

No	Factor	Cause	Remark
1	Factors inherent in the nature of the material	Composition, Structure, stratification, and secondary or inherited structures	
2	Changes caused by weathering and physicochemical activity	Wetting and drying processes, hydration, removal of cementing agents	
3	Effect of pore pressures	Earthquakes	
4	Changes in structure	Stress release, structural degradation	

Materials on slopes that are of concern to highway agencies are soil and rock. Many slopes are composed entirely of soil or entirely of rock while other slopes may contain various mixtures of soil and rock.

Soil often fails along distinctive surfaces that may be observed and/or estimated by field inspections. Many soil slope failures have been observed to occur along a surface approximated by a circular arc. Other slopes have been observed to fail along a series of circular arcs. Also, many soil slope failures have been observed to occur along a surface consisting of a series of planes.

2.3.2 Types of Slope Movements

Mud Flow

This type of slope failure is usually a shallow failure involving mostly surface soils. It usually consists of highly saturated soils that simply flow (like water) downward. The flowing mass is saturated with water and most often occurs without warning during or after very heavy rainstorms or when snowmelt occurs. Mud flows frequently occur in soils that are sandy and silty with or without small amounts of clays. They oftentimes occur in steep slopes (Fig. 2-1).

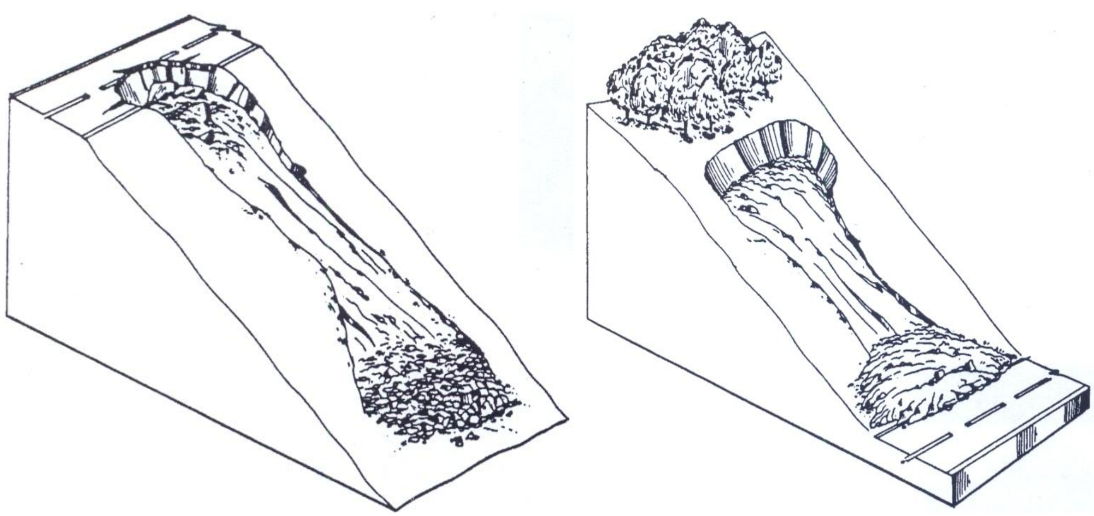

Figure 2-1: Mud flow slide below and above roadway [25]

Wedge - type Slide

This type is usually larger than a mud flow and the failure plane is wedge shape. Wedge-shaped slides usually occur along a distinctive failure plane. Usually the failure plane in the middle and lower portion of the slide is a soft clay layer of low strength or silt layer sandwiched in between two clay layers. The failure mass of soil often times slides along bed rock (Fig. 2-2).

Rotational Failure

Rotational failure is similar to a wedge failure except the failure surface is more circular (Fig. 2-3). Rotational slides usually occur in fairly homogeneous materials. The failure plane is usually circular and deep, and the mass of soil fails as a unit, although several scraps may be observed in the upper potions of the slide. Rotational slides may occur in the fill or foundation soils (Sub-Surface failure).

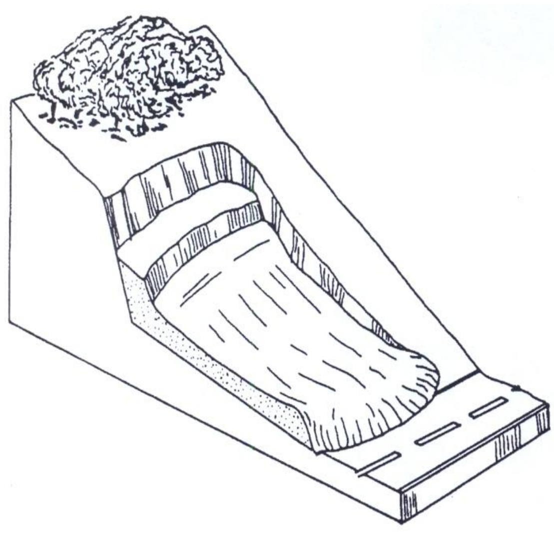

Figure 2-2: Wedge-type slide above roadway [25]

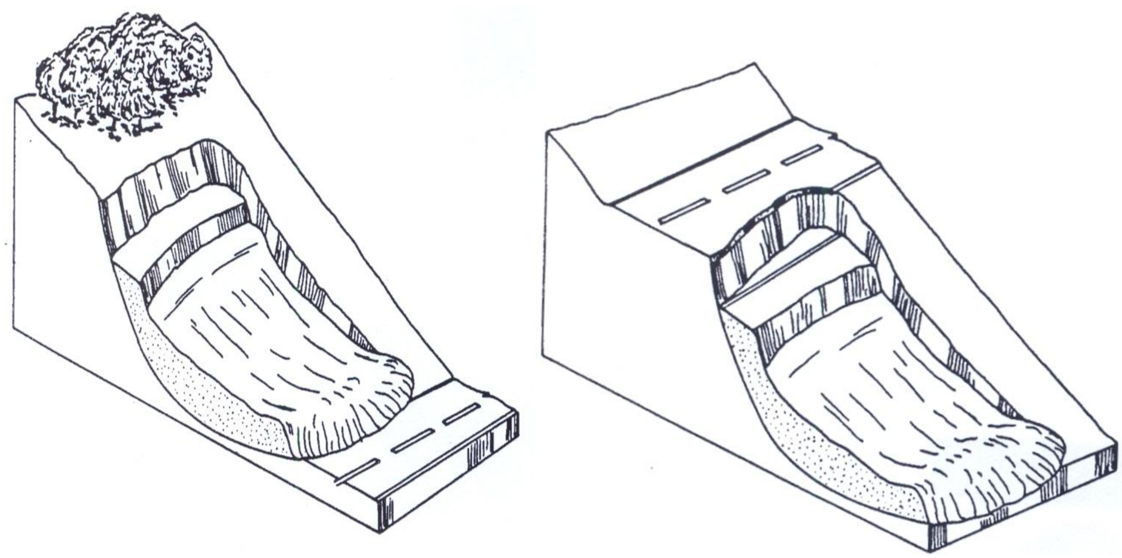

Figure 2-3: Rotational slide above and below roadway [25]

Block slide above Road

This type usually consists of a massive block of soil or rock moving as a unit. Block slides usually occur along a distinctive failure plane, or natural joints in rocks of soils (Fig. 2-4). The failure plane usually consists of weak materials or joints. A block slide may fail as a single unit of material or numerous units may fail at different times. These types of slides are extremely dangerous because they fail instantaneously without any warning.

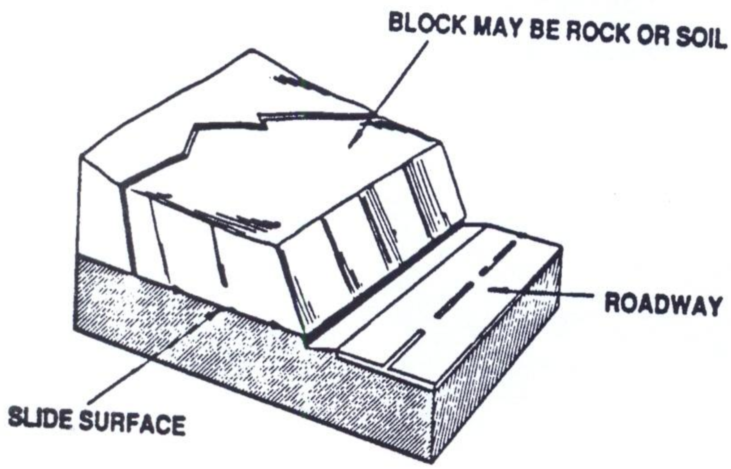

Figure 2-4: Block slide above roadway [25]

Rock fall from Differential Weathering

This type usually consists of large boulders falling onto the roadway (Fig 2-5). These boulders are mostly competent (hard) rock underlain by rocks that weather more quickly. This causes the competent rock to lose support and fall onto the road way.

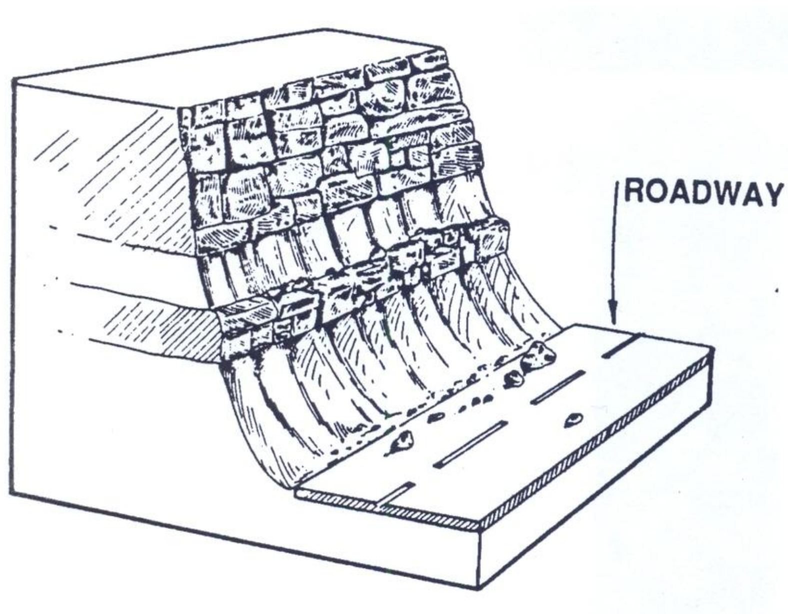

Figure 2-5: Rock fall from differential weathering [25]

Rock fall From Massive Rock Slopes

This consists of rock debris that falls from rock slopes that are close to the road way (Fig 2-6). Rock fall from massive slopes occurs is a result of rock weathering, rainfall, snowmelt, and freezing and thawing.

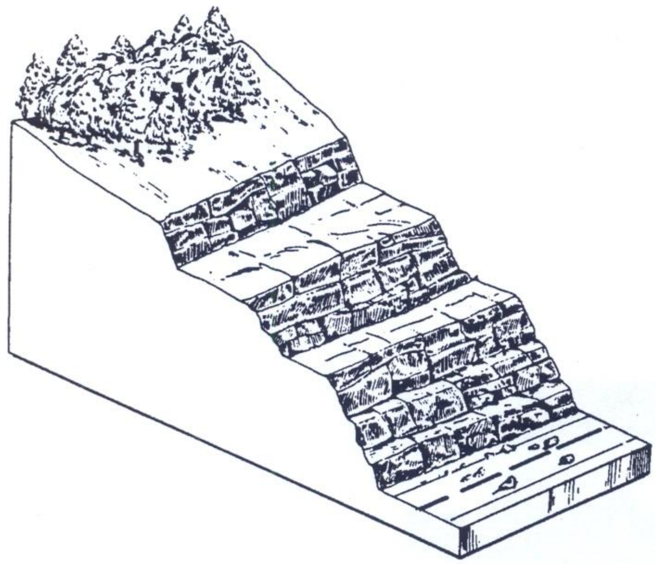

Figure 2-6: Rock fall from a massive rock slope [25]

Rock fall from Talus Slopes

This type of failure consists of a mass of highly weathered boulders and some fine materials that move onto the roadway (Fig 2-7). This type of material is prone to failure because of ground water seepage and seepage into the talus pile from rainfall and snowmelt.

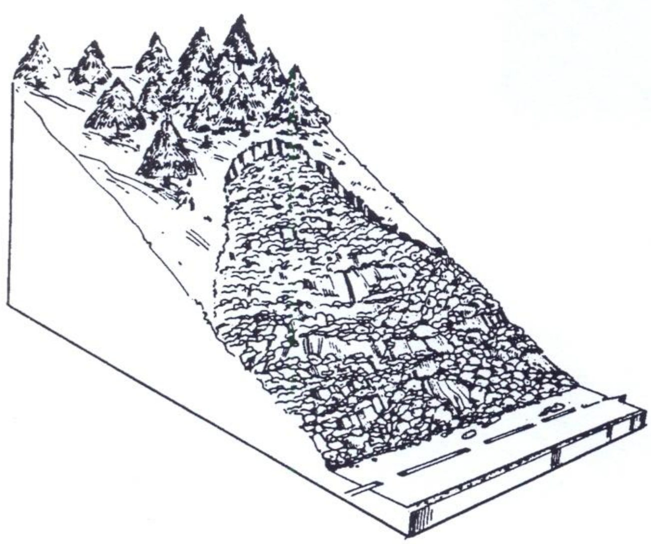

Figure 2-7: Rock fall from a talus slope [25]

2.4 Factors to Consider in Stability Analysis

The selection of the proper method for soil slope stability analysis is based on the geometric and time factors. Geometric factors refer to the project type (cut slopes, earth dam embankments, etc) and stratigraphy (isotropic, homogeneous; isotropic, non-homogeneous; or anisotropic). The time factor of short or long term conditions controls whether undrained or drained conditions prevail and thus, whether total stress or effective stress analysis is more suitable. The inputs to the analytical process are soil strength parameters, pore-water pressures, and in some cases earthquake forces.

2.4.1 Strength Parameter Selection

Measured soil strength parameters are not unique. Soil type, strain history, and loading time are the major factors to consider in the selection of strength parameters. Strain history controls whether the peak, ultimate, or residual strength will prevail. The time element refers to short term or undrained conditions or long term or drained conditions.
Without the knowledge of appropriate shear strength values the stability of slopes cannot be analyzed. The two types of shear strength used in stability analysis are the undrained and the drained shear strength.

A general summary of applicable strength parameters for various geologic conditions are given in the following Table 2-3.

Strength measurements are obtained from tests performed either in the laboratory or in the field. Laboratory tests are used to evaluate index and engineering properties of soil which are related to the ultimate engineering behavior of a cut slope and the recommended design. Test method selection is based on soil type and drainage conditions to be evaluated. For details of the laboratory and insitu determination of shear strength parameters of soil, reference can be made to books on soil mechanics and slope stability analysis, [2, 11, 19].

Table 2-3: Field Condition and Strength Parameters acting at failure [27]

Material	Field Condition	Strength Parameter*
Cohesionless sands	Dry	ϕ (i= ϕ)
	Submerged slope	ϕ' (i= ϕ')
	Slope seepage with top flow line coincident with and parallel to slope surface	ϕ' (i= $\phi'/2$)
Clays (except stiff fissured clays and clay shales)	Undrained conditions	S_u ,(ϕ = 0)
	Drained conditions	c', ϕ'
Stiff fissured clays and clay shales and existing failure surface	Without slope seepage	ϕ'_r (i≈ ϕ'_r)
	With slope seepage	ϕ'_r(i≈ $\phi'_r/2$)
Cohesive mixtures	Undrained conditions	c_u, ϕ_u
	Drained conditions	c', ϕ'

* i= stable slope angle

After evaluating the quality of laboratory data, it is advisable to check the shear strength value with available correlations for comparison. In some situations shear strength correlations are used for preliminary design studies. For granular soils, tabulated values or values from insitu testing are commonly used [2, 10].

The drained angle of internal friction of cohesive soils can be estimated from correlations between plasticity index and peak drained angle of internal friction. Figures 2.8 and 2.9 may be used to correlate the PI with residual angle of internal friction [2, 11].

The major parameters in relation to design of cut slopes are the slope angle and height of the cut. For dry cohesionless soil, stability of a cut slope is independent of height and therefore slope angle becomes the only parameter of concern. For purely cohesive ($\phi = 0$) soils, the height of the cut becomes the critical design parameter. For c'-ϕ' and saturated soils, slope stability is dependent on both slope angle and height of cut [2].

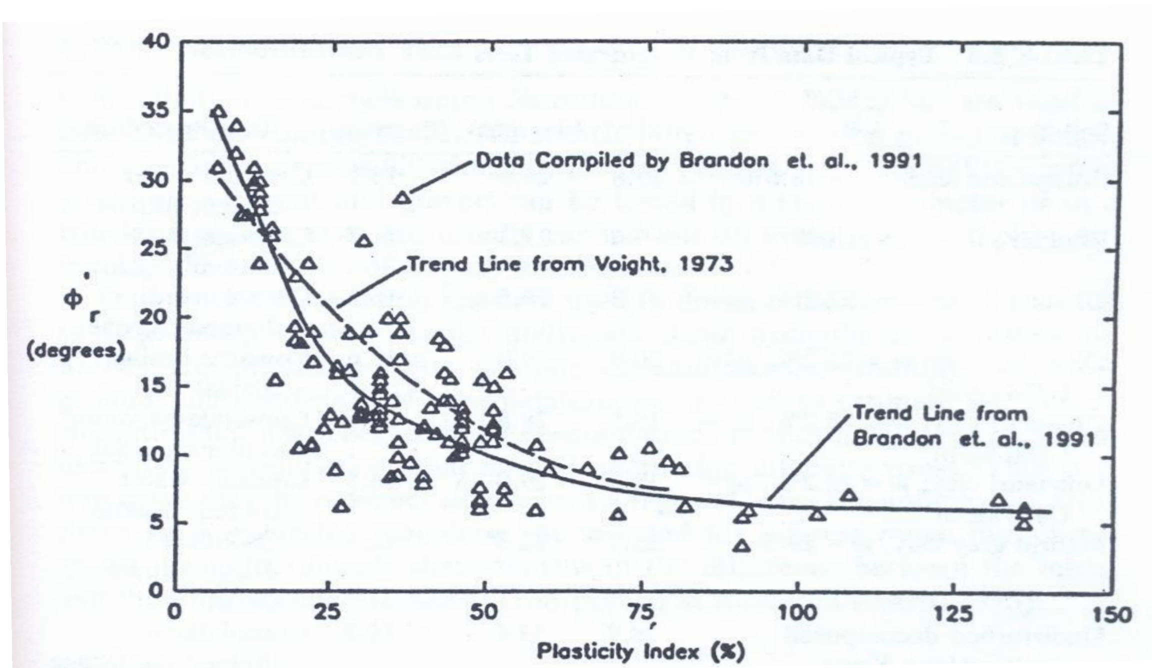

Figure 2-8: Correlation between plasticity index (PI) and the residual angle of friction, ϕ_r' [2]

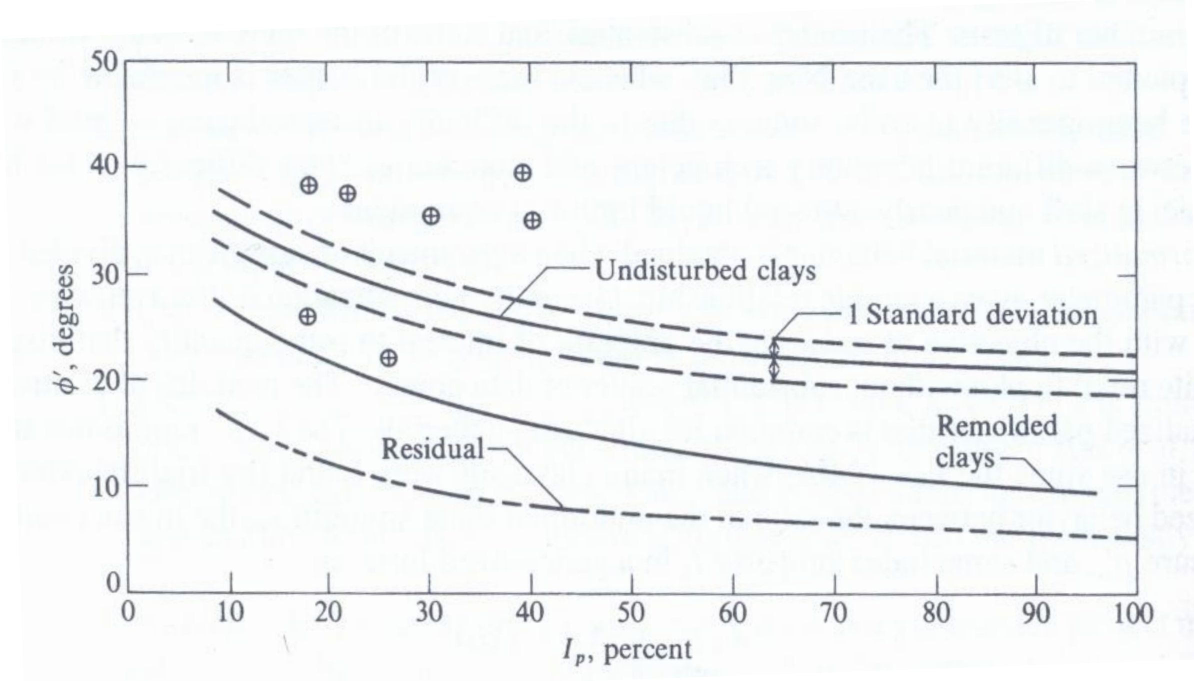

Figure 2-9: Correlation between plasticity index (PI) and ϕ' for normally consolidated clays. Approximately 80 percent of the data falls within one standard deviation. Only few extreme scatter values are shown [11]

2.4.2 Pore water Pressures

Changes in loading condition (excavations or embankment loads) and associated soil volume changes will result in a pore pressure change,u, which could be negative or positive. The change in pore pressure may increase or decrease with time depending on the type of soil and the type of stresses involved. Under fully drained conditions, porewater pressure is equal to zero. For partially drained or undrained conditions, evaluation of porewater pressure depends on the relative rate of loadings as compared to the rate of drainage within the soil. Evaluation of internal pore pressures is required for effective stress analysis [20].

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

(a) Phreatic Surface

The pore-water is given by

$u=\gamma_w\left(h_w\cos^2\theta\right)$, Where γ_w is the unit weight of water

This is a reasonable assumption for a sloping straight-line phreatic surface, but will provide higher or lower estimates of pore-water pressure for a curved (convex) phreatic surface.

(b) Piezometric data

Specification of pore pressures at discrete points, within the slope, and use of an interpolation scheme to estimate the required pore-water pressures at any location. The piezometric pressures may be determined from:

- Field piezometres
- A manually prepared flow net, or
- A numerical solution using finite differences or finite elements. This approach is the best method for describing the pore-water pressure distribution [2].

(c) Pore water pressure ratio

The pore-water pressure ratio is a coefficient that relates the pore-water pressure to the overburden stress. The coefficient is defined as:

$$r_u = \frac{u}{\gamma_t H_s}$$

$$u = r_u \gamma_t H_s$$

Where:

u = the pore-water pressure

γ_t = the total unit weight

H_s= the height of the soil column

The difficulty with r_u concept is that the coefficients vary throughout a slope if the phreatic surface is not parallel to the ground surface. Often, the slope will require an extensive subdivision into many regions with different r_u values [2].

(d) Piezometric Surface

The most common way of defining pore-water pressure is with a piezometric line. The pore-water pressure at the base of the slice is calculated by multiplying the vertical distance from the slice base mid-point up to the piezometric line with the unit weight of water.

(e) Constant Pore water pressure

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

2.4.3 Effective and Total Stress Analysis

Slope stability analyses may be performed using either total stresses or effective stresses. The use of total stress as opposed to effective stress analyses and the various ways in which design shear strengths can be selected can produce a wide range of safety factors. Bishop and Bjerrum [8] have set forth basic guidelines on the specification of shear strength for use in limit equilibrium slope stability analyses.

1. "Effective stress analysis is a generally valid method for analyzing any stability problem and is particularly valuable in revealing trends in stability which would not be apparent from total stress methods. Its application in practice is limited to cases where the pore pressures are measured or can be estimated with reasonable accuracy, such as long-term stability where the pore pressure is controlled either by the static water table or by a steady-state flow pattern."
2. "Where a saturated clay is loaded or unloaded at such a rate that there is no significant dissipation of the excess pore pressures set up, the stability can be determined by the $\phi=0$ analysis, using the undrained strength obtained in the laboratory or from in-situ tests. This is essentially an end of construction method, and in the majority of foundation problems, where the factor of safety increases with time; it provides a sufficient check on stability. For cuts, on the other hand, where the factor of safety generally decreases with time, the long term stability must be calculated by the effective stress method."
3. "For saturated soils the values of c' and ϕ' are obtained from drained Triaxial tests or consolidated undrained tests with pore pressure measurements, carried out on undisturbed samples. The range in stresses at failure should be chosen to correspond to those in the field. Values measured in the laboratory appear to be in satisfactory agreement with field records with two exceptions. In stiff fissured clays the field value of c' is lower than the value given by standard laboratory tests; in some very sensitive clay the field value of ϕ' is lower than the laboratory value."

Problems in slope stability can be broadly grouped in two classes: short-term problems and long-term problems. When a saturated or partially saturated soil with a low permeability undergoes a change in stress there will generally be a corresponding change in pore pressure. The stage at which the excess pore pressures (positive or negative) resulting from the change in stress is fully developed is referred to as the short-term condition. With the passage of time these out-of-balance pore pressures are redistributed until eventually they are everywhere in equilibrium with the steady state pore pressures appropriate for the new stress conditions. This final stage is referred to as the long-term condition and the continuing stability of the slope under gravity or applied loads is a problem with drained loading conditions.

Long-term or drained stability problems are usually simpler than short-term or undrained stability problems since they always involve drained or effective stress strength parameters and for a given soil these do not vary very much with the type of test that is used to determine them. However, it should be noted that even the effective stress Mohr-Coulomb envelope is curved, rather than straight, for most soils and that the values of ϕ' are thus lower at higher confining pressures.

In general, short-term stability problems involve undrained loading and they can be addressed using total stresses and undrained strengths or effective stresses, drained strengths and porewater pressures. It is commonly believed that both approaches should give the same answer but this is not necessarily so [8].

Since the limit equilibrium method is most applicable at failure, in effective stress analyses one should in fact use the pore pressures "at failure," rather than the "actual" pore pressures for the short-term loading condition, and then both approaches will give similar answers. In this connection one should note that the pore pressures specified in effective stress analyses affect only the resisting forces that are computed and not the driving forces. This occurs because the total normal force at the base of each slice is an unknown and an increase in the specified pore pressure decreases the effective normal force but has no effect on the total normal force. Similarly, changes in the pore pressures created by shearing under undrained loading conditions are not included as driving forces in total stress analyses.

The traditional argument for using effective stress analyses for short-term, undrained problems is that it is the effective stresses which really count in determining deformations and therefore effective stress analyses provide greater insight into the problem at hand. However, effective stress analyses require determination of either the "actual" pore pressures or the pore pressures "at failure" and this is not an easy task. Indeed, it is about as easy as it is to determine the undrained strength that should be used in a total stress analysis since the reason that undrained strengths vary with sample orientation, the type of test and the details of the loading conditions is largely that the excess pore pressures are sensitive to these factors. In other words, it is about equally as difficult to predict excess pore pressures for use in effective stress analyses as it is to determine the appropriate undrained strengths for use in total stress analyses.

The preferred method for determining undrained strengths has changed over the years. Bishop and Bjerrum recommended the use of UU triaxial tests and cautioned against the use of CU triaxial tests [8].

2.4.4 Factor of Safety

In any stability analysis, some measure of the degree of safety has to be provided. The function of the factor of safety is to account for uncertainty, and thus to guard against ignorance about the reliability of the items that enter into the analysis, such as strength parameters, pore pressure distribution, and stratigraphy. The choice of factor of safety is greatly influenced by the accumulated experience with a particular soil mass. Since the degree of risk that can be taken is also greatly influenced by experience, the actual magnitude of the factor of safety used in the design will vary with material type and performance requirement [2, 19].

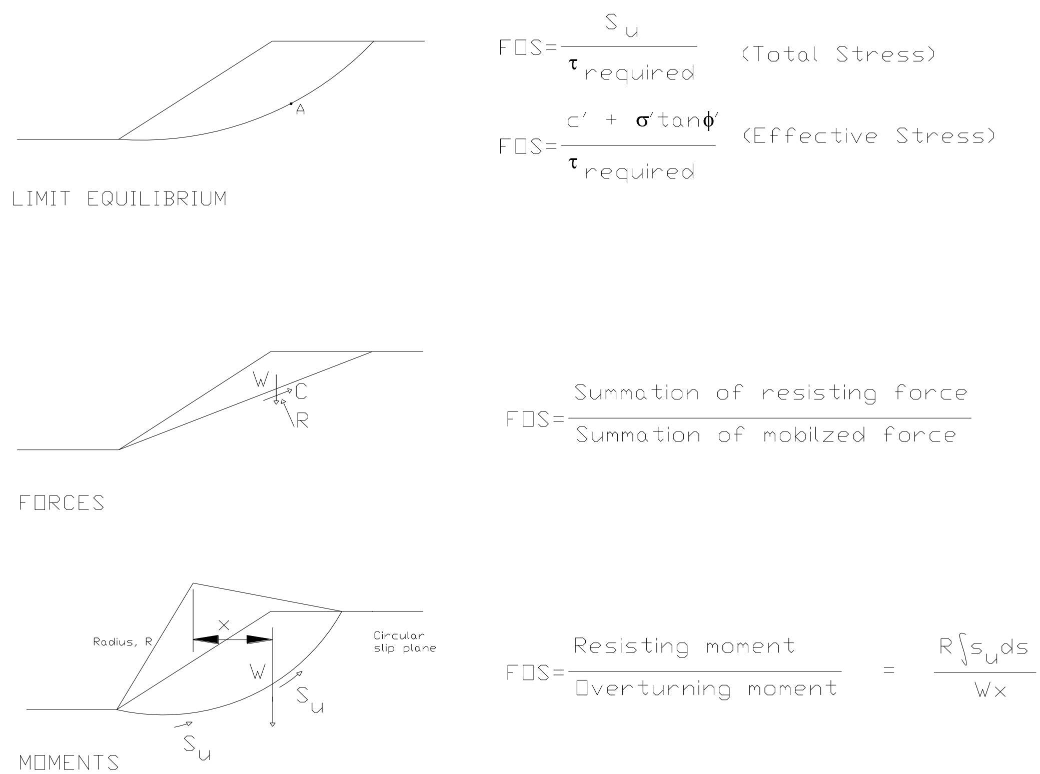

Figure 2-10: Various definitions of factor of safety [2]

In most limit equilibrium analysis, the factor of safety is assumed to be constant for the entire failure surface. Different values of factor of safety may be obtained using one of the three methods (Fig 2-10): mobilized strength, ratio of forces, or ratio of moments. All these methods will not give identical values for c- ϕ soils [2]. The last definition is the most widely used.

A factor of safety 1.1 is reasonable for lightly-trafficked roads, where the consequences of slope failure are relatively low. Where the consequences of failure are high, such as in the situation where road failure could cause significant traffic disruption, then a factor of safety of

1.2 to 1.3 might be appropriate [36]. Abramson [2] suggested the required factor of safety (non-seismic) for highway cut slope designs, in the range of 1.25 to 1.5. Cornforth [20] and American State Highway Agency also recommend factor of safety of 1.5 for long term stability to avoid creep movements that eventually may lead to failure after several years. During construction of highway slopes, a temporary factor of safety as low as 1.4 to 1.2 may be acceptable if long-term factor of safety is equal to or greater than 1.5. A factor of safety 1.25 is recommended by design guide for loess soils [30]. Some agencies for mining operations suggested a factor of safety of 1.5 and 1.3 when the design is based on the peak and residual shear strength respectively, where there is a risk of danger to persons or property [3].

Cut slopes are generally not designed for seismic conditions unless slope failure could impact adjacent structures. These factors of safety should be considered as minimum values. The geotechnical designer should decide on a case by case basis whether or not higher factors of safety should be used based on the consequences of failure, past experience with similar soils, and uncertainties in analysis related to site and laboratory investigation.

2.5 Comparison of Limit Equilibrium Methods

The common methods of slope stability analysis are presented in Table 2-4. The conditions of static equilibrium that are satisfied in determining the factor of safety are also presented.

Table 2-4: Static Equilibrium Conditions Satisfied by Limit Equilibrium Methods [2]

	Force-Equilibrium		Moment Equilibrium
	X	Y	
Ordinary method of slices	No	No	Yes
Bishop's simplified	Yes	No	Yes

	Force-Equilibrium X	Y	Moment Equilibrium
Janbu's simplified	Yes	Yes	No
Corp's of Engineers	Yes	Yes	No
Janbu's generalized	Yes	Yes	No
Bishop's rigorous	Yes	Yes	Yes
Spencer's	Yes	Yes	Yes
Sarma's	Yes	Yes	Yes
Morgenster-Price	Yes	Yes	Yes

The differences in the various methods are the assumptions on inter slice forces. For example, the ordinary Method ignores inter slice forces (V=H=0), Simplified Bishop Method assumes inter slice forces are horizontal (V=0, H>0), Spencer's Method assumes all inter slice forces are parallel (V>0, H>0) with an unknown inclination which is computed through iterations, Morgenstern and Price method relates the shear force, V to the normal force, H where $V=\lambda f(x)$ H.

The moment equilibrium approach gives factor of safety that are less sensitive to angle of thrust to horizontal of the interslice force than force equilibrium solutions [18]. Since an estimate of thrust of the interslice force has to be assumed in all analyses, it seems that a method satisfying moment equilibrium should be used for preference. In this thesis the Morgenstern-Price method of analysis is used in preparing the charts.

There have been different methods for slope stability analysis. Some of them fail to satisfy horizontal force equilibrium and some of them fail to satisfy moment equilibrium like Bishop's and Janbu's method respectively. However, the factors of safety calculated from these methods differ only by ±15% as compared to the methods that satisfy complete force and moment equilibrium methods [2].

2.6 Design Charts

Slope stability charts provide a rapid method of determining the factor of safety for simple slopes. They are useful for preliminary analysis, to compare alternates that can later be examined by more detailed analyses. Chart solutions also provide a rapid means of checking the results of detailed analysis.

Slope stability charts are also used to back calculate strength values for failed slopes to aid in planning remedial measures. This can be done by assuming a factor of safety of unity for the conditions at failure and solving for the unknown shear strength.

Charts for investigating the stability of simple homogeneous earth slopes for soils with cohesion and friction have been available. Of these Taylor's [53], Bishop and Mongenstern's [9], Spencer's [47], Janbu's [16], Cousins [19] can be mentioned. All these charts have limitations and drawbacks that restrict their use.

Taylor's charts are based on the total stress and do not take into account pore pressure. Bishop and Mongenstern's charts are based on effective stresses. However, the charts are for limited slope angle range ($11\text{-}27^0$). All the charts require a considerable amount of interpolation and extrapolation to determine the factor of safety.

Previously developed charts have the following assumptions: Two-dimensional analysis, simple homogeneous slopes, and slip surfaces of circular shapes only. Regardless of these assumptions, there is a practice of using these charts for non-homogeneous and non-uniform slopes with different geometric configuration. This can be done by using an average inclination, weighted average of *c* and ϕ, and the value of shear strength calculated on the basis of the proportional length of slip surface passing through different relatively homogeneous layers [2].

In most limit equilibrium analysis, the factor of safety is assumed to be constant for the entire failure surface.

2.7 Computer Analysis

Computers are used routinely to perform stability analysis of slope failures. Computer programs are now available to determine the slope factor of safety. The use of computer allows the precise ground slope profile to be incorporated, together with any variation in ground water profile. Thus, the individual slope characteristics, if known, can be accurately represented. Programs also allow a large number of potential slip surfaces to be considered. Some allow random generation of non-circular surfaces, the most prevalent form of failure on steep slopes formed in granular and heterogeneous soils [36].

There are many computer codes currently available for analyzing the stability of slopes such as GALENA, GSLOPE, CLARA-W, TSLOPE3, STABL, UTEXAS, and SLOPE/W. Program SLOPE/W is formulated in terms of moment and force equilibrium factor of safety equations. In this thesis the SLOPE/W computer program is used for the stability analysis.

Limit equilibrium methods include Morgenstern-Price, General limit equilibrium, Spencer, Bishop, Ordinary, Janbu etc. The program locates the critical slip surface by using search optimization technique. This program also allows integration with other applications. For example finite element computed stresses from SIGMA/W or QUAKE/W can be used to calculate a stability factor by computing total shear resistance and mobilized shear stress along the entire slip surface. Then a local stability factor for each slice is obtained. Using a Monte

Carlo approach, program computes the probability of failure in addition to the conventional factor of safety [45].

2.8 Comparison of Highway Cut Slope Standards

It is advantageous at this stage to examine some of the design standards employed for cut slope in Ethiopia and outside Ethiopia. Table 2.5 gives a comparison of different cut slope standards.

Before the development of new ERA Geometric Design Manuals (2002) the TCDE Geometric Design Manual was in use. Most of the roads in Ethiopia are designed using this manual. This practice has never shown sever slope stability problem up to now for roads constructed strictly in accordance with this manual.

In order to compare the Ethiopian standards with other African countries, Kenyan, Tanzanian and South African Design Manuals are examined. In addition AASHTO Geometric Design Manual and Highway Design Reference Guide are included.

Table 2-5: Comparisons of different cut slopes Standards

Material type	Height of Slope (m)	ERA Geometric Design Manual (2002)		TCDE Geometric Design Manual		Kenyan Road Design Manual		Tanzanian Road Design Manual	SATCC Geometric Design Manual	Highway Design Reference Guide
		Side Slope (V:H)	Back Slope (V:H)	Side Slope (V:H)	Back Slope (V:H)	Side Slope (V:H)	Back Slope (V:H)	Back Slope (V:H)	Side/Back Slope (V:H)	Side/Back Slope (V:H)
Earth or Soil	0.0m-1.0m	1:4	1:3	1:2	1:3	1:2	1:3	1:2 to 1:1.5	1:2 for Non Cohesive soils	1:3 for poor clay
	1.0m-2.0m*	1:3	1:2		1:2		1:2		1:1.5 for Soft cohesive soils	2:1 for good clay
	Over 2.0 m*	1:2	1:1.5		1:1.5		1:1.5		1:1 for Hard cohesive soils	2:1 for granular material

* The 2.0m limit is 3.0m for Kenyan and Tanzanian Road Design Manuals

H – Horizontal

V - Vertical

As shown in Table 2.5, the side slope provision by the new ERA Geometric Design manual is flatter in comparison to the other design manuals. AASHTO [1] recommends the side slopes to be designed for a slope not steeper than 1V:4H to ensure roadway stability and to provide a reasonable opportunity for recovery for an out-of-control vehicle.

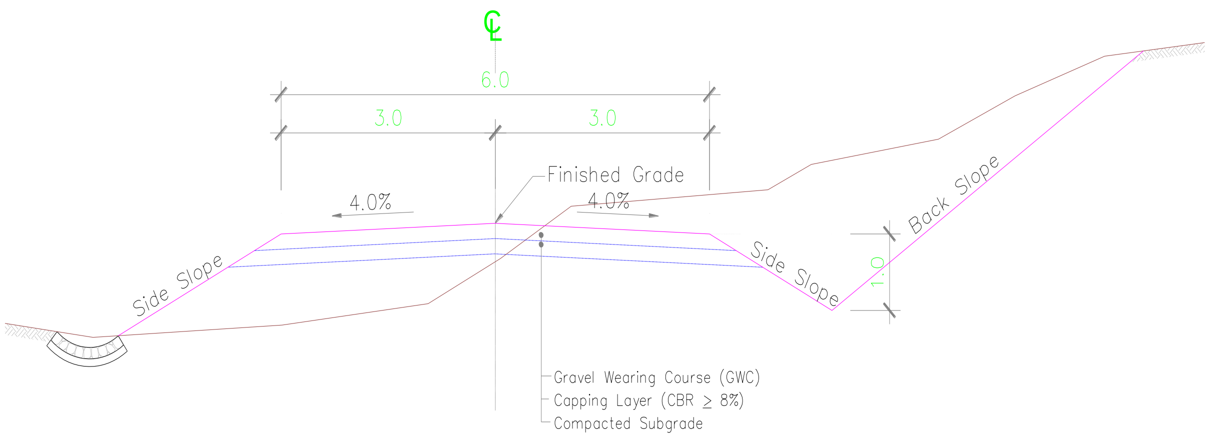

Figure 2-11: Typical Normal Cut/Fill Rural Road Section with road side designation [ERA DS-6]

The back slope ratio provision by ERA Geometric Design Manual is the same with TCDE and Kenyan geometric Design Manuals. But it is flatter as compared to Tanzanian and South African Geometric Design Manuals.

All the Geometric Design Manuals except the South African Manual do not identify soil type. The standards only consider the height of cut. This problem is crucial in slope stability analysis, since cut slopes are going to be designed in view of the slope height, material type and pore water condition.

By considering the soil type in cut slope design an economical design may be obtained which is the main work of this thesis.

3.0 CUT SLOPE DESIGN

3.1 Design Approach and Methodology

Safe design of cut slopes is based either on past experience or on more in-depth analysis. Both approaches require accurate information regarding geologic conditions obtained from standard field and laboratory classification procedures. For slope stability analysis the information that will be needed for analysis include: an accurate cross section showing topography, proposed grade, soil unit profiles, unit weight and strength parameters (c', ϕ') or (c, ϕ) for the soil , and location of the ground water table. A typical geometry of slope is given in Fig 3.1.

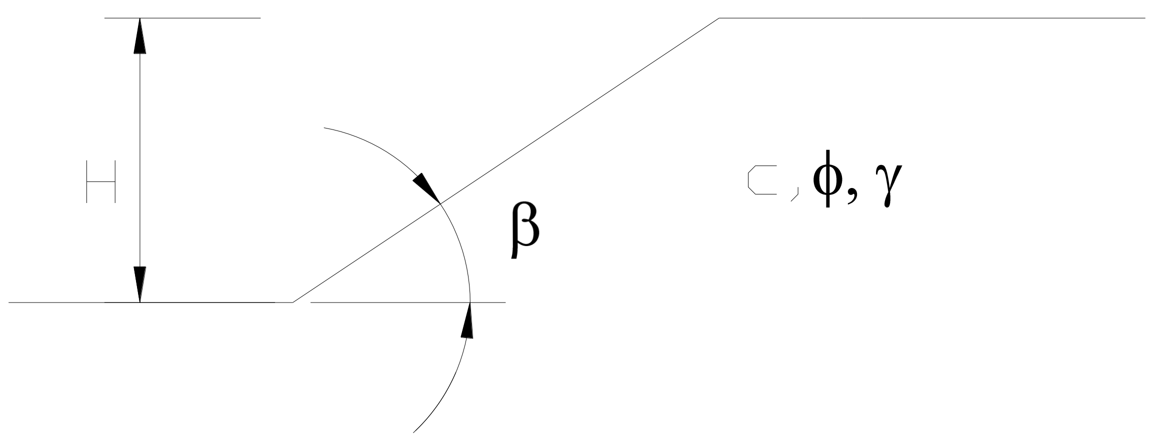

Figure 3-1: Typical Geometry of a slope

Initial slope stability analysis can be performed using stability charts. These charts can be used to determine if a proposed cut slope might be subject to failure. In this thesis, slope stability charts are produced using range of parameters as shown in Table 3.1. SLOPE/W computer software is used to determine the minimal factor of safety. These safety factor outputs with their corresponding ϕ and ($c/\gamma H$) values for each slope are used to draw factor of safety contours. This will be done with the help of Civil Designer software. The parameters ($c/\gamma H$), ϕ and FS are imported as *x*,*y* and *z* respectively and the FS contour is drawn.

In order to prepare cut slope design tables for soils, shear strength parameter and unit weight of each soil type is required. These parameters are collected and compiled from previous works and laboratory tests made on collected samples. Using this shear strength values and factor of safety contour charts prepared in this work as shown in Appendix-A, safe cut slope standard tables that relate height and inclination is produced. Predefined factor of safety 1.5 is used for static slope stability.

3.2 Preparation of Stability Charts

In developing the stability charts, it is assumed that the slope is homogeneous and made of a single material with shear strength parameter *c* and ϕ. The range of parameters adopted is tabulated in Table 3-1.

Table 3-1: Range of parameters adopted

Slope angle, β	5, 10, 15, 20, 25, 30,35,40,45,50,55,60
Slope height, H in m	1, 2,3,4,5,6,7,8,9,10
Cohesion, *c*	5,10,15,20,25,30,35,40,45,50
Angle of friction, ϕ	5,10,15,20,25,30,35,40,45,50
Porewater pressure ratio	$r_u = 0$, $r_u = 0.25$, $r_u = 0.50$

Conceptually, r_u is around 0.5 if the phreatic surface is at the ground surface. This is because the unit weight of water is about half the total unit weight of the soil. $r_u = 0$ indicates the soil is dry.

The stability analysis is carried out using the available software Slope/W (Geoslope Int Ltd) programme incorporating the Morgenstern price method. For detailed procedure of Slope /W programme one can refer GeoSlope manual [45].

The effect of soil parameters is brought out by graphs showing contours of equal factors for various values of c and ϕ. The geometry of the slope considered in these graphs is confined to twelve slopes which are usually encountered in highway cuts.

The graphs are easy to operate. In most cases, they eliminate the need for detailed mathematical calculation required for the determination of critical stability condition. The factor of safety contour graphs are plotted with the angle of internal friction ϕ as abscissa and the dimensionless parameter $c/(\gamma H)$ as ordinate. Such a graph will readily provide the effect of the various combinations of c and ϕ on the stability of the slope.

A typical factor of safety contour is given in Figures 3-2, 3-3, and 3-4 for r_u =0, r_u =0.25 and r_u =0.50 respectively. The graphs for other slopes are shown in the Appendix-A.

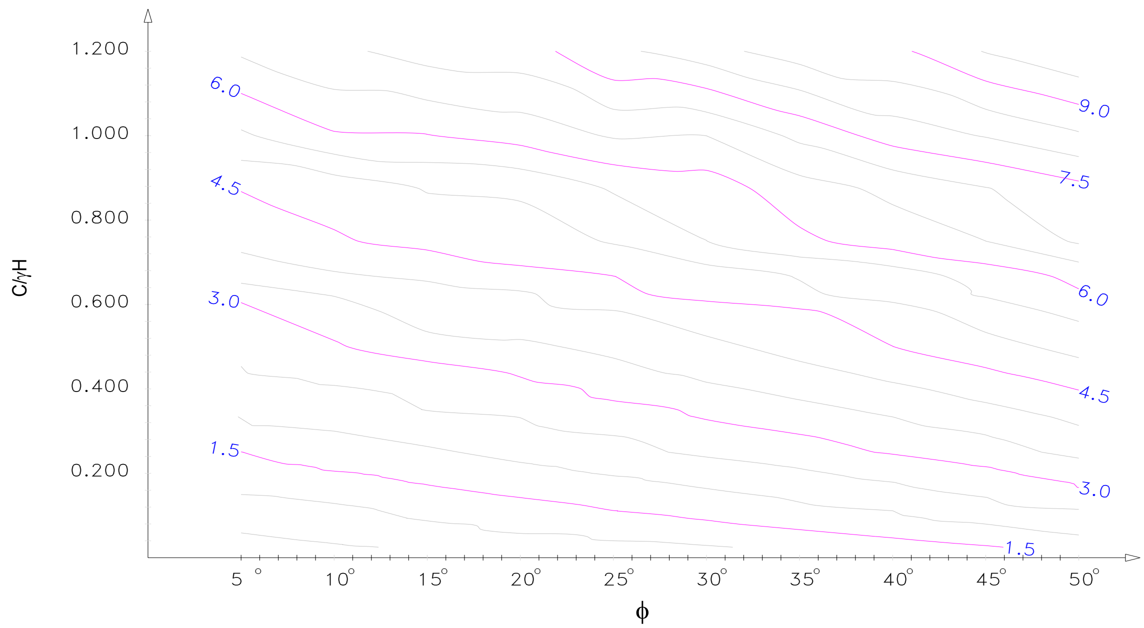

Figure 3-2: Factor of safety Contour for slope 45 ^{o} and r_u =0

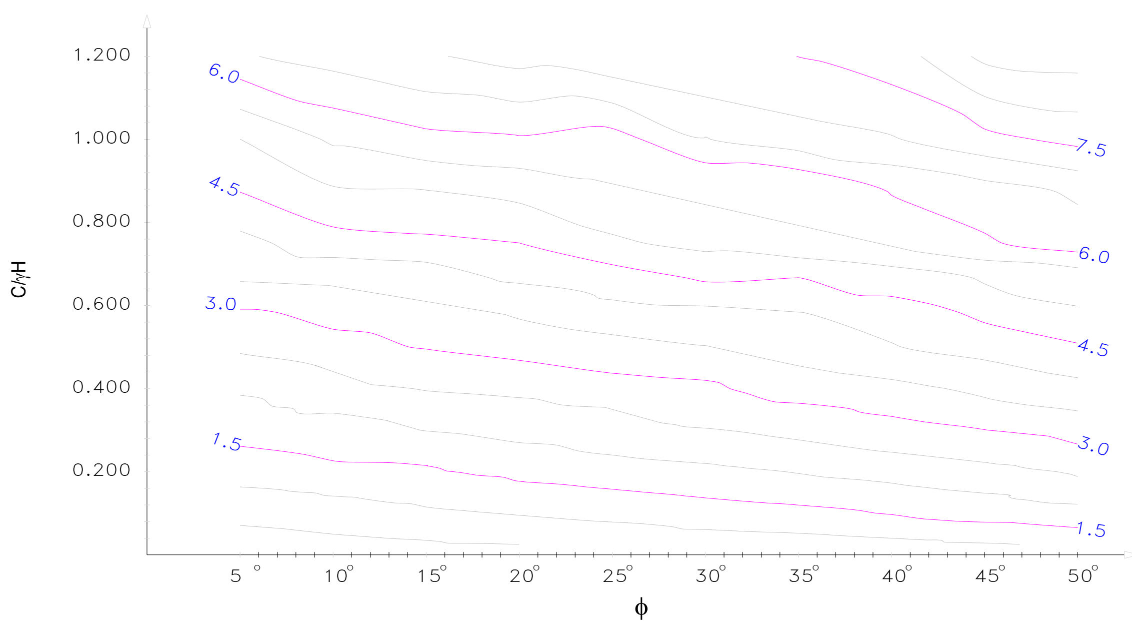

Figure 3-3: Factor of safety Contour for slope 45 ^{o} and r_u =0.25

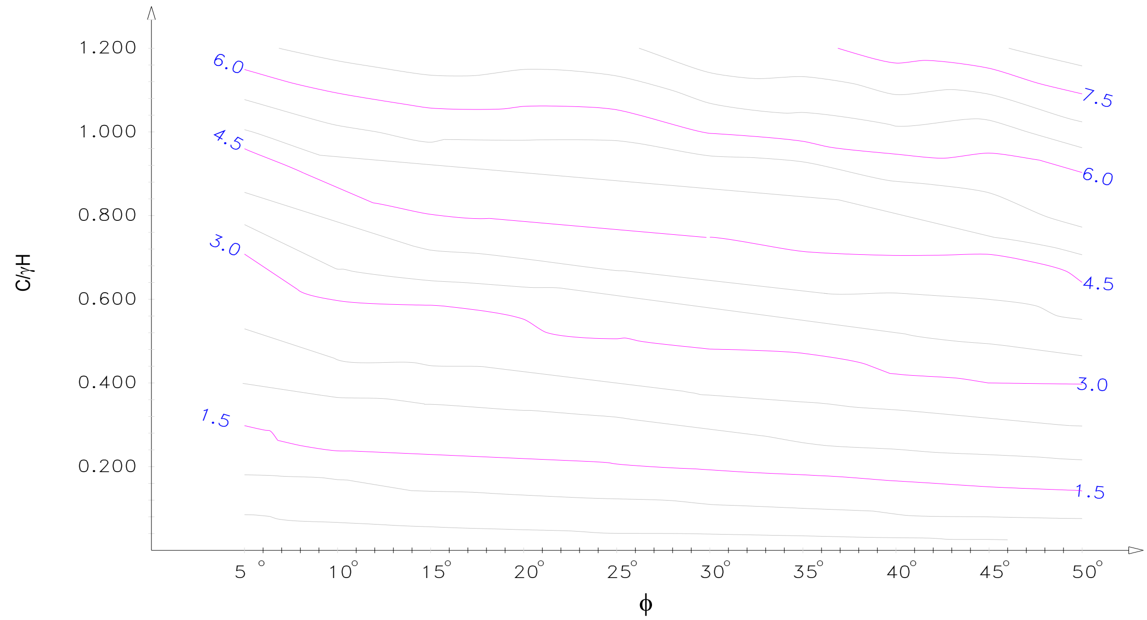

Figure 3-4: Factor of safety Contour for slope 45 ° and r_u =0.50

3.3 Validation of the Presented Chart

In order to assess the correctness of the charts, it is demonstrated by solving three problems. The first problem is given in Spencers [47] paper. The second comes from Baker [5]. The third is taken from Cousins [19].

Example 1**:** To find the slopes corresponding to safety factors of 1.5 and for an embankment 100 ft (30.5 m) high in a soil whose properties are *c*=870 psf (41.66 kPa); γ=120 pcf (18.85kN/m^3); ϕ=26^o and r_u =0.5. Now (*c*/γH) =0.07246. From the chart for slope=15^O, the corresponding (*c*/γH) =0.0547; for slope=20^O, (*c*/γH) =0.0826.

By interpolation, corresponding to a stability coefficient (*c*/γH) =0.07246, the slope angle is 18.2^O. This result is in close agreement with Spencer's [46] which is 18^O.

***Example 2*:** Consider a simple slope stability problem defined by the input data c=12 kPa, ϕ =25^O, γ=18 kN/m^3, and slope=14^O, H=12 m. Using the developed chart for (c/γH)=0.05556, ϕ =25^O and slope=10^O, the corresponding FS is 3.6. For slope 15^O, FS=2.5.

By interpolation the factor of safety is 2.72 for slope=14^O, which is the same as Baker [5].

***Example 3*:** Details of the geometry are given in Fig 3-5.

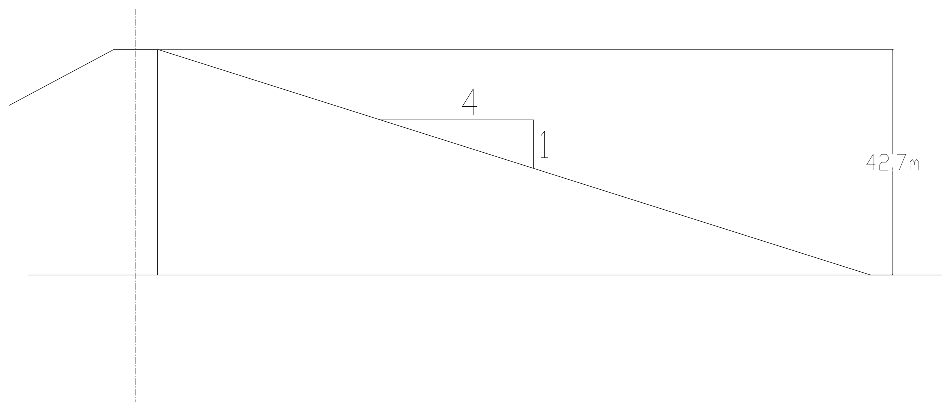

Figure 3-5: Earth Dam section for example 3

The soil parameters are: ϕ'=30^O, c'=28.3 kPa, γ=18.9 kN/m3 and r_u=0.50. Thus (c'/γH)=0.0351. Using the developed chart for (c/γH)=0.0351, ϕ'=30^O ,r_u=0.50 and slope=10^O, the corresponding FS is 2.1. For slope 15^O, FS=1.5.

By interpolation the factor of safety is 1.6 for slope=14.04^O. This result is approximately 7% lower than Cousins value of 1.72. Bishop and Morgenstern also obtained a value of 1.65 [19].

3.4 Practical use of Chart

In section 3.2 it is pointed out that factor of safety contour charts are produced in this work and helpful in assessing stability of slopes. Here an attempt is made to assess the stability of existing cut slopes in three locations: Lemmi, Amanuel and Armenia. The condition of each site will be discussed below.

i. Lemmi

Lemmi is located 52.2 km from Muketuri at the junction of Addis – Gohatsion trunk road. It is 123 km far from Addis Ababa. The geographical location of the slope is Easting= 0486642 and Northing= 1085478.

The road cross section at station 123+040 is shown in Figure 3-6.

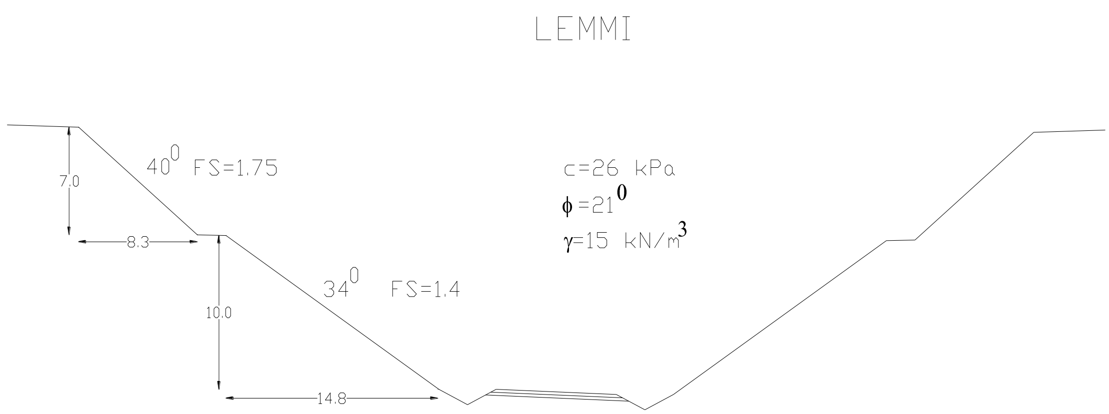

Figure 3-6 Cross-section at km 123+040

Calculating ($c/\gamma H$) =0.1733 and taking $\phi=21^o$, the factor of safety read from Figure A-21 for slope 35^o is 1.4.

ii. Amanuel

Amanuel is located 331 km on the major Addis – Bahidar trunk road. The geographical location of the cut slope is Easting= 0341658 and Northing= 1159043.

The road cross section at station 331+020 is shown in Figure 3-7.

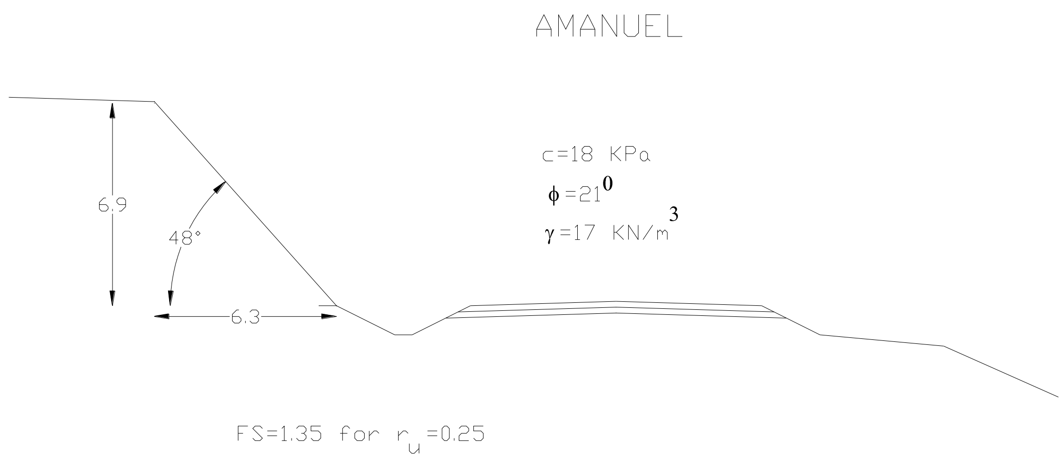

Figure 3-7: Cross-section at km 331+020

Calculating ($c/\gamma H$) =0.1733 and taking ϕ=21^o, the factor of safeties read from Figure A-27 and A-30 are 1.25 and 1.2 respectively. Interpolating for slope 48^o, the factor of safety is 1.2.

iii. Armenia

Armenia is located 201 km on the major Addis – Dessie trunk road. The geographical location of the cut slope is Easting= 0588897 and Northing= 1091732.

The road cross section at station 202+060 is shown in Figure 3-8.

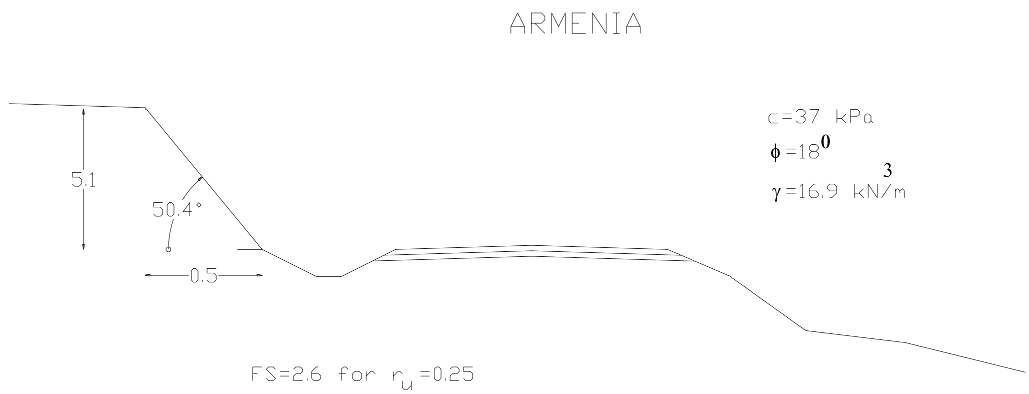

Figure 3-8: Cross-section at km 202+060

Calculating ($c/\gamma H$) =0.429 and taking ϕ=18^o, the factor of safety read from Figure A-29 is 2.6.

The above given examples are stable slopes, even though they are not cut strictly according to ERA cut slope standard. This shows a revised cut slope standard is required that satisfy safety and economy.

3.5 Recommended Cut Slope Standards

In order to propose a table that is similar to Table 2-5 but consider the soil type, it is found necessary to collect enough data from previous works and also conduct tests from randomly selected sites. From the data thus collected, classification test with the corresponding shear strength values (*c* and ϕ) shall be presented. The c and ϕ values shall then be used to estimate the cut slope for a given factor of safety.

3.5.1 Laboratory Test

The laboratory tests are made on samples collected from areas called Lemmi, Amanuel, and Armenia which are 123 km, 331 km and 202 km respectively from Addis Ababa.

First standard classification test is performed on the collected samples. These tests include gradation analysis, moisture content, and Atterberg limits. These tests will provide information to aid in determining appropriate slope inclinations.

For slope stability analysis, laboratory strength tests are performed using Triaxial and Direct Shear apparatus.

3.5.1.1 Grain size Analysis

The soil samples under investigation are almost fine since particle size retained in 2mm opening sieve was insignificant. Hence hydrometer analysis was used with sodium hexametaphosphate dispersing agent. The test is made according to ASTM D422-63. The result is shown in Appendix-B. The grain size distribution for the soils tested is presented in Figure 3-9.

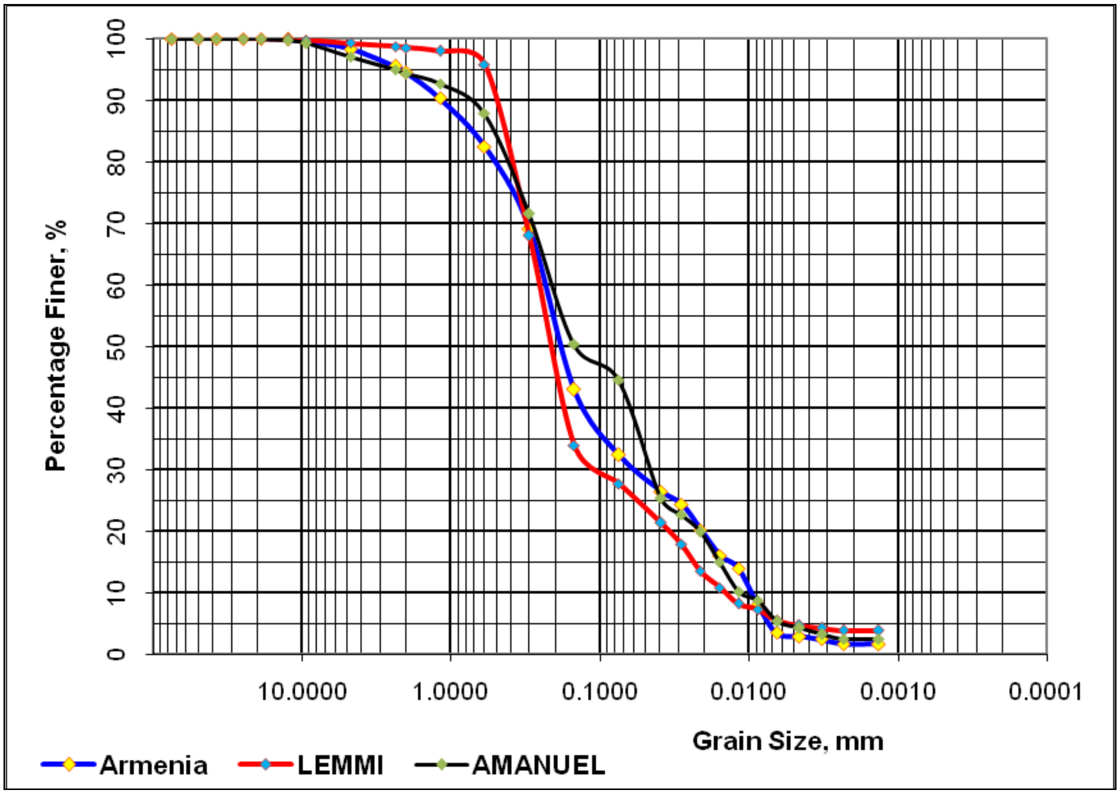

Figure 3-9: Grain Size distribution of the tested Sample

3.5.1.2 Atterberg Limits

Atterberg Limits are arbitrary boundaries between each of the two states such as liquid limit, plastic limit and shrinkage limit. The atterberg limit is determined according to the procedure outlined in ASTM D 4318 - 98 and the results are shown in Appendix-B. The summary of the test results is given in Table 3-2.

Table 3-2: Results of Atterberg Limit Test

Soil Sample	LL	PL	PI
Lemmi	86	53	33
Amanuel	68	40	28
Armenia	73	46	26

3.5.1.3 Soil Classification

A soil classification system is arrangement of different soils in to groups having similar properties. The purpose of soil classification is to make possible the estimation of soil properties by association with soils of the same class whose properties are known and to provide the engineer with accurate method of soils description. In this work the soils are classified according to unified soil classification system (USCS). The classification is made according to the procedure outlined in ASTM D 2487-98. The result of the classification is presented in Table 3-3.

Table 3-3: Soil Classification

Soil Sample	Classification according to USCS
Lemmi	Clayey Sand
Amanuel	Clayey Sand
Armenia	Clayey Sand

3.5.1.4 Shear strength

Shear strength parameters of a soil is generally obtained in the laboratory by shearing specimens either in a triaxial cell or in a direct shear box.

Direct shear tests were carried out on 60 mm diameter and 25 mm thick specimens. The specimen was set up in the shear box with porous stones at top and bottom and then filled with water and allowed for saturation. Normal load was applied on the specimen using a dead weight. When the vertical displacement stopped, the specimen was sheared at a constant displacement rate of 0.2 mm/min. Three specimens with different normal loads were sheared for each series. The shear strength is determined according to the procedure outlined in BS 1377-7. Direct shear apparatus used for the test is shown Appendix-C.

In triaxial test, three specimens of a sample were consolidated isotropically to different consolidation pressures; 100, 200 and 300 kPa, and then sheared under undrained conditions. The soil samples were sheared by increasing the axial load at a constant strain rate and maintaining the cell pressure constant. The pore water pressures in the specimens were measured during shear. The shear strength is determined according to the procedure outlined in BS 1377-8. The apparatus is shown in Appendix-C.

Specimens used in all the triaxial test series were 38 mm diameter and 76 mm high. Specimens were saturated with a back pressure technique until the pore water pressure response in the specimen was at least 95% of changes in the cell pressure.

The laboratory analysis results for the tested samples are shown in Annex-B. The summary of the strength parameters obtained from the tests are presented in Table 3-4.

Table 3-4: Strength Parameter

	Location	***c*, (kN/m^2)**	**ϕ, Degree**	**γ, (kN/m^3)**
Direct Shear	Lemmi	26	21	15
	Armenia	37	18	17
Triaxial	Amanuel	18	21	16.9

3.6 Data

In addition to the test results obtained from the three sites, data was collected from earlier works and compiled. Table 3-5 shows the compiled shear strength parameters.

Table 3-5: Compiled Shear Strength Parameters of Soils

Type of Soil	*c*, (kN/m^2)	ϕ, Degree	γ, (kN/m^3)	Test method	Sampling Area	Place of Laboratory Analysis done
Sandy silt	16	30	15.1	Direct shear	Bilate	CDSCo(2000)
Sandy silt	24	30	14.7	"	"	"
Sandy silt	24	27	12.7	"	"	"
Sandy silt	15	32	16.1	"	Gambella	"
Sandy silt	20	30	15.2	"	Gambella	"
Silty Sand	18	30	14	"	Bilate	"
Silty Sand	10	31	15.3	"	Gambella	"
Silty Sand	13	28	15.5	"	"	"
Silty Sand	13	31	16	"	"	"
Silty Clay	27	19	16.2	"	Addis Ababa	CDSCo(2011)
Silty Clay	16	29	16.7	"	Chena	"
Silty Clay	26	28	18.9	"	Ambaye	"
Silty Clay	32	18	15	"	Addis Ababa	"
Silty Clay	39	13	16.9	"	"	"
Silty Clay	28	18	16	"	"	"
Silty Clay	28	20	17.2	"	"	"
Silty Clay	26	16	16.1	"	"	"
Clayey silt	15	25	14.6	"	"	"
Clayey silt	15	28	14.6	"	Gonder	CDSCo(2010)
Clayey silt	15	26	15.3	"	Addis Ababa	"

Type of Soil	*c*, (kN/m²)	ϕ, Degree	γ, (kN/m³)	Test method	Sampling Area	Place of Laboratory Analysis done
Clayey silt	19	26	16.3	Direct shear	Addis Ababa	"
Clayey Sand	18	21	16.9	Triaxial, CU	Amanuel	AAU (2010)
Clayey Sand	26	21	15	Direct shear	Lemmi	"
Clayey Sand	37	18	17	"	Armenia	"
Red Clay	15	24	18.5	Triaxial, CU	Kolfe	AAU(Tadesse 1989)
Red Clay	25	23	18.2	"	Semen Gebeya	"
Red Clay	10	22	17.9	"	Rufael	"
Red Clay	17	23	-	"	Kolfe	AAU (Tarekegn 2009)
Red Clay	26	25.8	-	"	Semen Gebeya	"
Red Clay	11	21	-	"	Rufael	"
Red Clay	12	24.5	-	"	Asco	"

In order to carry out design of cut slopes, representative shear strength parameters c and ϕ for each soil type is required. In this thesis, those values which are listed in Table 3-5 are averaged for the type of soil considered and summarized in Table 3-6.

Table 3-6 : Summarized Shear Strength Parameter

Soil Type	c (kN/m²)	ϕ (Degree)	γ (kN/m³)
Sandy Silt	19	29	14.8
Silty Sand	13	30	15.2
Silty Clay	27	20	16.6
Clayey Silt	16	26	15.2
Clayey Sand	27	20	16.3
Red Clay	16	23	18.2

Using the shear strength parameter c and ϕ values in Table 3-6 and factor of safety contour graphs shown in Appendix-A, safe and economical cut slope standard tables are produced as shown in Table 3-7. Here a predefined factor of safety 1.5 and pore pressure ratio 0.25 are used for the design. For detailed usage of the chart one can refer Appendix-A.

Table 3-7: Recommended Soil Cut slope Standard

Soil Type	Height of slope (m)	Cut Slope (V:H)	ERA CUT SLOPE (V:H)
Sandy Silt and Silty Sand	0 - 1 1 - 2 2 - 5 5 - 6 6 - 7 7 - 8	1:0.8 1:0.8 1:0.8 1:1 1:1.2 1:1.4	1:3 1:2 1:1.5 1:1.5 1:1.5 1:1.5
Clayey Silt and Red clay	0 – 1 1 - 2 2 - 4 4 - 5 5 - 6 6 - 7 7 - 8	1:0.6 1:0.6 1:0.8 1:1.0 1:1.2 1:1.4 1:1.7	1:3 1:2 1:1.5 1:1.5 1:1.5 1:1.5 1:1.5
Clayey Sand and Silty Clay	0 – 1 1 – 2 2 - 4 4 - 6 6 - 7 7 - 8	1:0.6 1:0.6 1:0.6 1:0.8 1:1 1:1.2	1:3 1:2 1:1.5 1:1.5 1:1.5 1:1.5

3.7 Discussion

It is observed from Table 3-7 that the ERA Cut slope standard is flatter in comparison to the newly proposed cut slope standard. This new standard considers the type of soil involved and shows different cut slope condition for the different soil type.

For the soils consider in this thesis the cut slope for silty clay and clayey sand soils are steeper than the other soils. This is due to the higher drained shear strength values. The other soils have relatively similar cut slope.

These cut slope standards are obtained from the analysis based on laboratory test results. Before applying this standard it is recommended to make field trials and make engineering judgments.

The newly cut slope standard has the advantage of getting less earth work volume which makes it economical.

3.8 Cost – Benefit Analysis

Cost comparison is done using the recommended cut slopes produced in this thesis and ERA Geometric Design Manual (2002) by taking four recently designed road projects. The selected projects have the same road standard DS-6 according to ERA road classification.

The comparison is made by providing the newly cut slope standard in the road template and calculating the earth work volume for each project. The summarized computed volume of earthwork extracted from the earthwork computation for the project length is shown in Table 3-8.

Table 3-8: Earthwork Computation Summary

Project name	Length, Km	Terrain classification, %				Earth work quantity using ERA standard, m3	Earth work quantity using newly proposed cut slope standard, m3	% Decrease in earth work quantity
		Flat	Rolling	Mountainous	Escarpment			
Shenen-Silkamba	22.1	74.5	18.7	6.8	0	232,352	188,902	18.7
Ephrem-Chewaka	21.7	65.9	34.1	0	0	370,704	292,856	21
Gimbicho-Jacho	24.0	16.4	77.4	6.2	0	786,283	534,672	32
Agarpha-Chole	8.8	0	65.9	28.4	5.7	636415	483,675	24

The cost comparison showed that the newly recommended cut slope standard has cost reduction of 19-32% with respect to earthwork in comparison to the slope provided by the ERA Geometric Design Manual (2002).

4.0 CONCLUSION AND RECOMMENDATIONS

4.1 Conclusion

(a) A set of charts was produced for assessment of the stability of slopes for frictional soils. The data was obtained from the calculations based on the limit equilibrium method using the Morgenstern Price analysis. The slope assessment made on the three areas with the prepared factor of safety charts show that the charts can be used for rapid slope stability analysis confidently.

(b) The side slope provision by the new ERA Geometric Design manual is too flat in comparison to the proposed slope thus making uneconomical.

(c) The newly recommended cut slope standard has cost reduction of 19 to 32% with respect to earthwork in comparison to the slope provided by the ERA Geometric Design Manual (2002).

4.2 Recommendation

Though this research is limited in its scope, it can be a good starting point for further research in the area. For a further improvement in the design of highway cut slopes the followings are recommended:

(a) The cut slope standard table is prepared for saturated soils using the strength parameter c and ϕ obtained from triaxial and direct shear apparatus for saturated condition. It would have been more appropriate to conduct strength tests for unsaturated condition and develop charts for such soils.

(b) New refined cut slope standard should be developed by performing slope assessment in the country, using the practice of other countries cut slope design and making a pilot project in the field.

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APPENDIX - A

GRAPH OF FACTOR OF SAFETY COUNTOUR

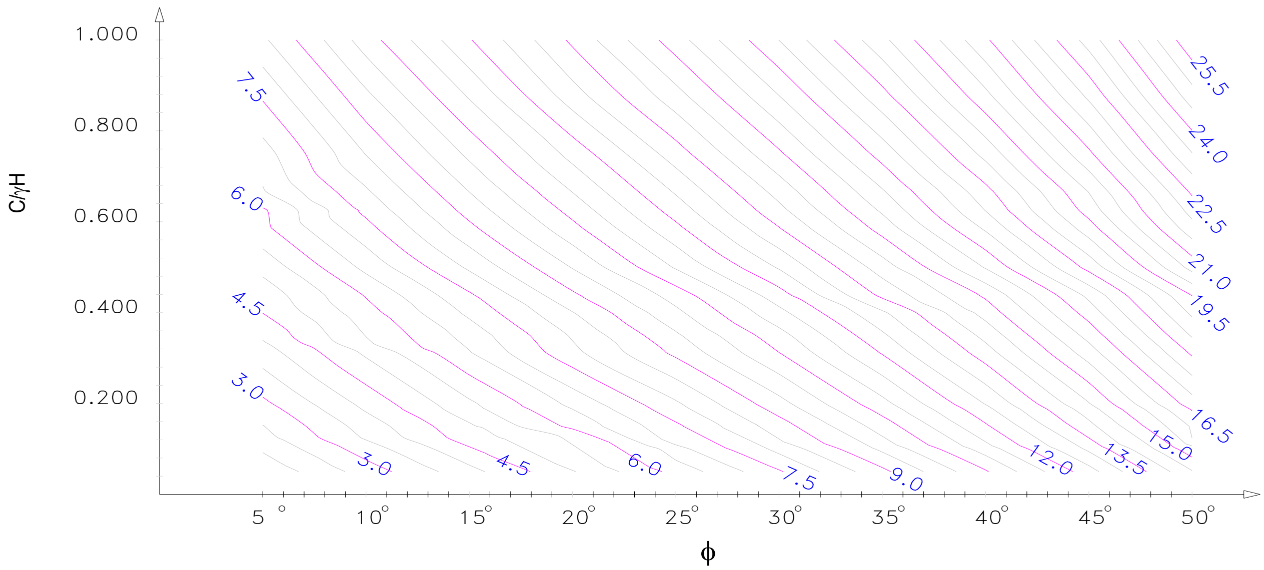


Figure A-1: Factor of safety Contour for slope 5 ^{o} and r_u=0

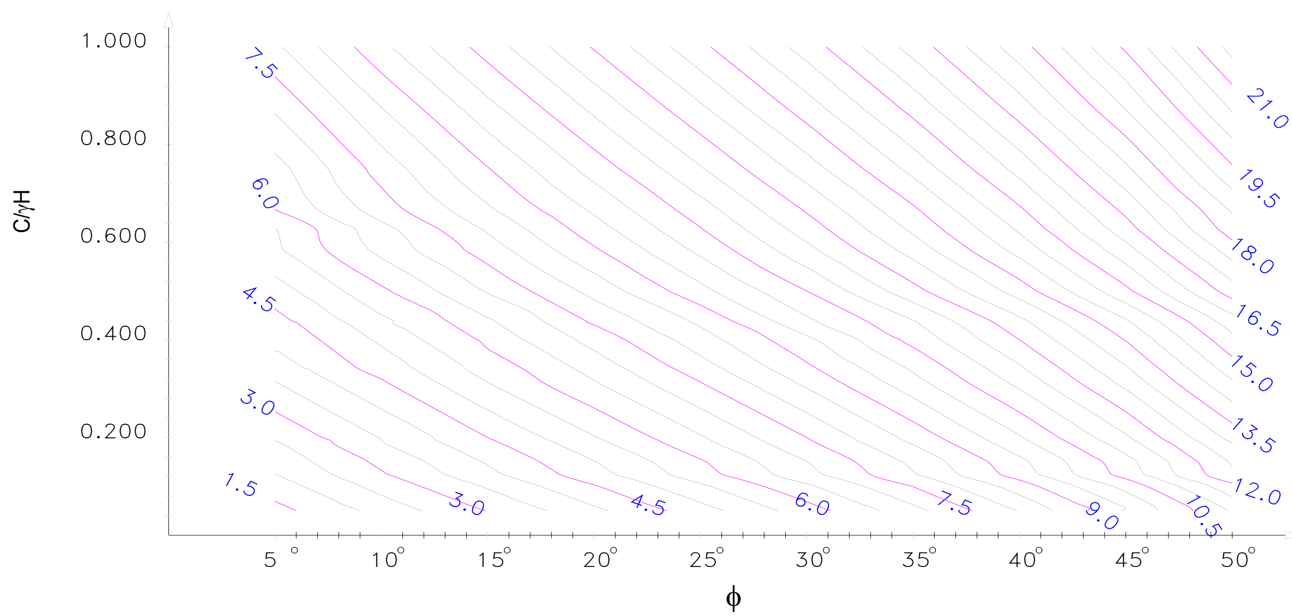


Figure A-2: Factor of safety Contour for slope 5 ^{o} and r_u=0.25

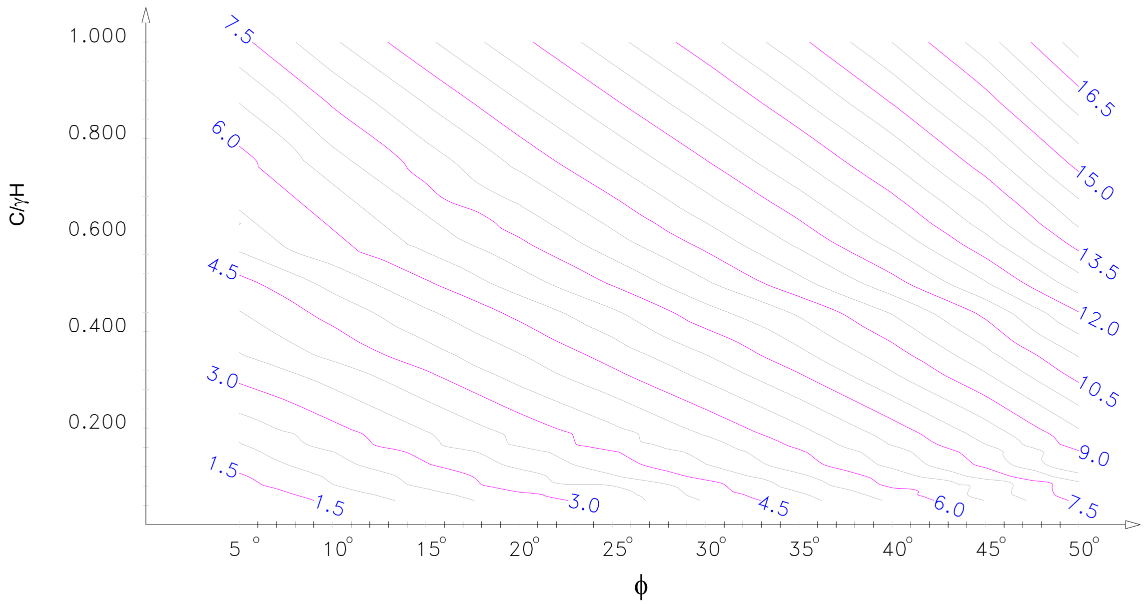


Figure A-3: Factor of safety Contour for slope 5 ° and r_u=0.50

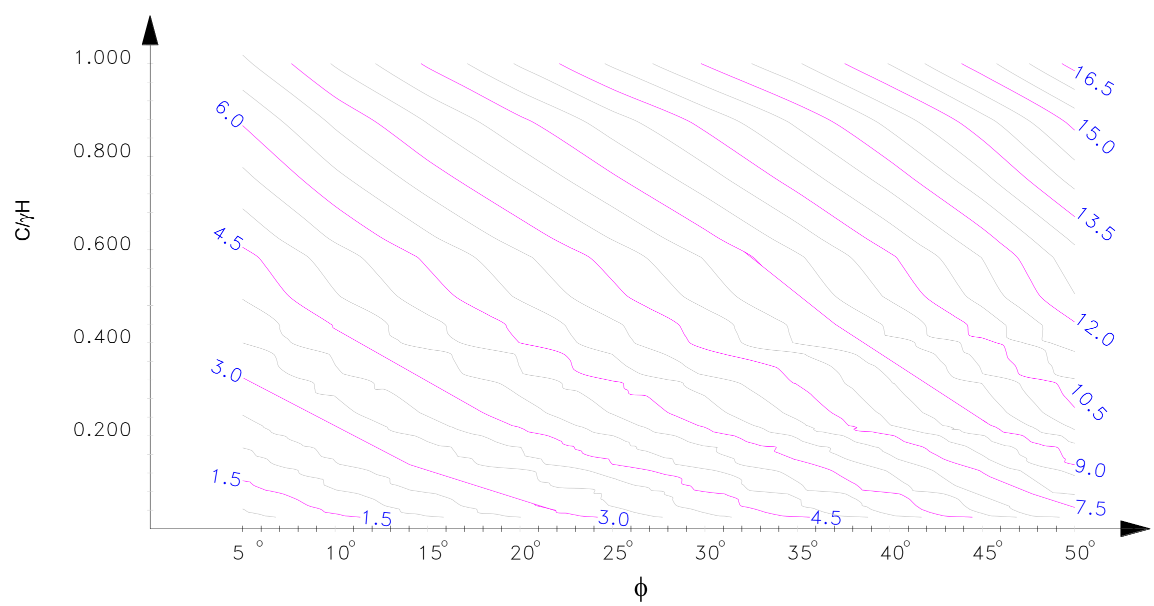


Figure A-4: Factor of safety Contour for slope 10 ° and r_u=0

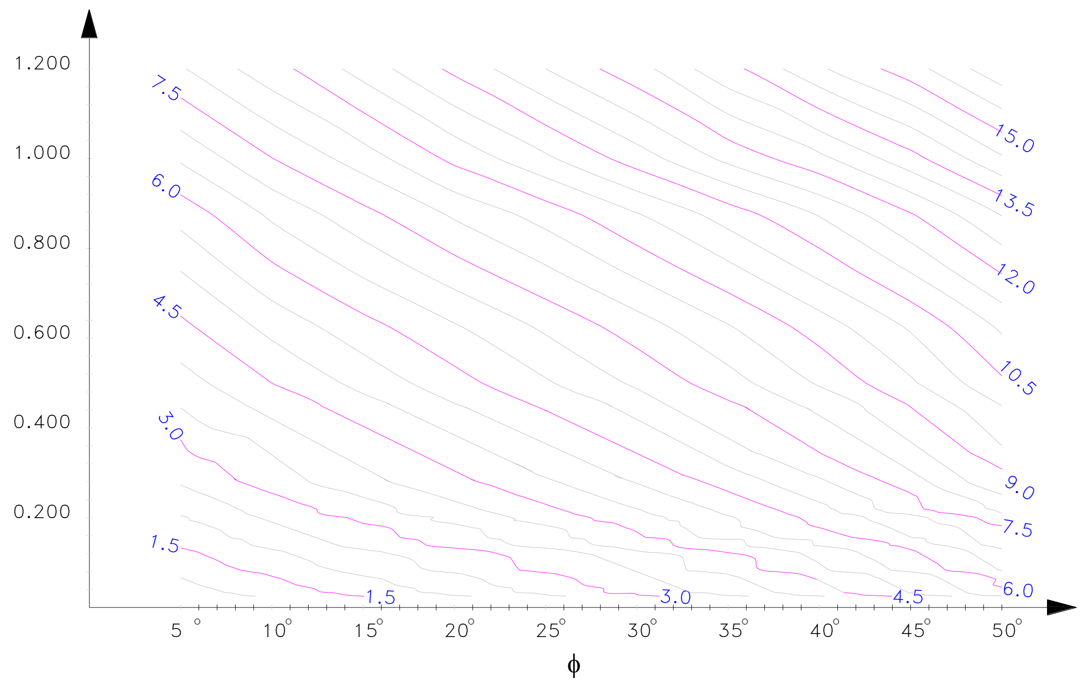


Figure A-5: Factor of safety Contour for slope 10 ° and r_u=0.25

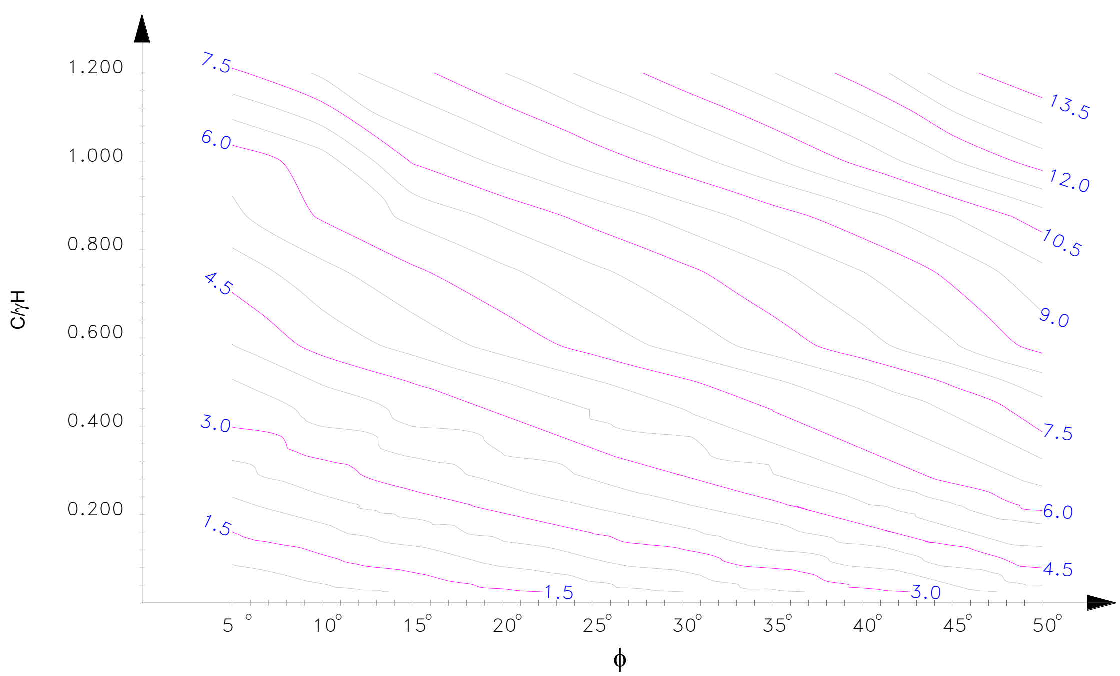


Figure A-6: Factor of safety Contour for slope 10 ° and r_u=0.50

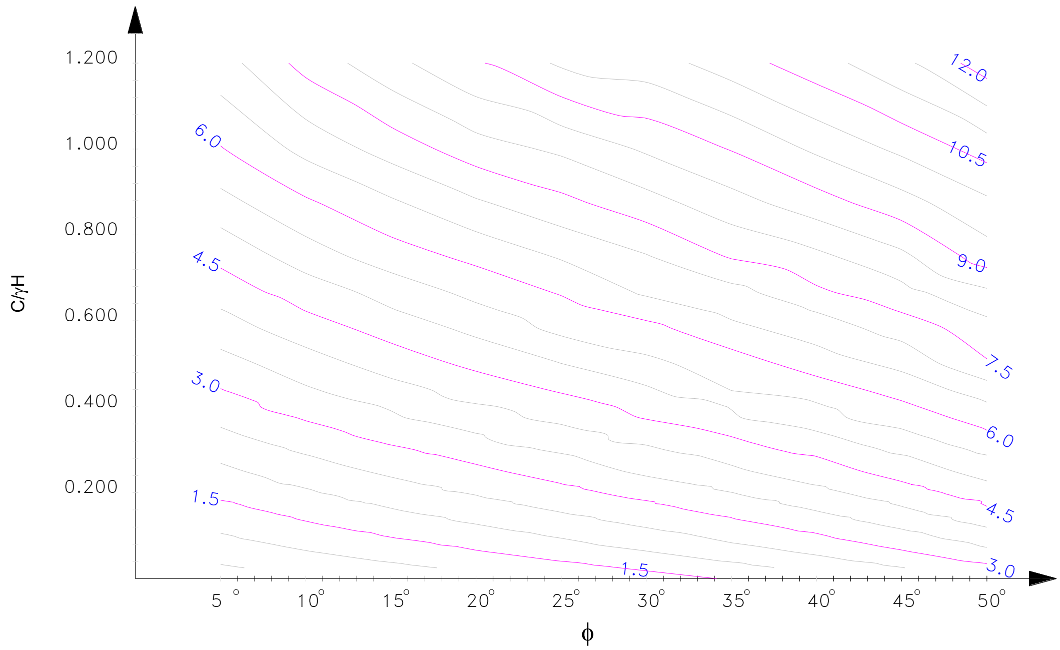


Figure A-7: Factor of safety Contour for slope 15 ^{o} and r_u=0

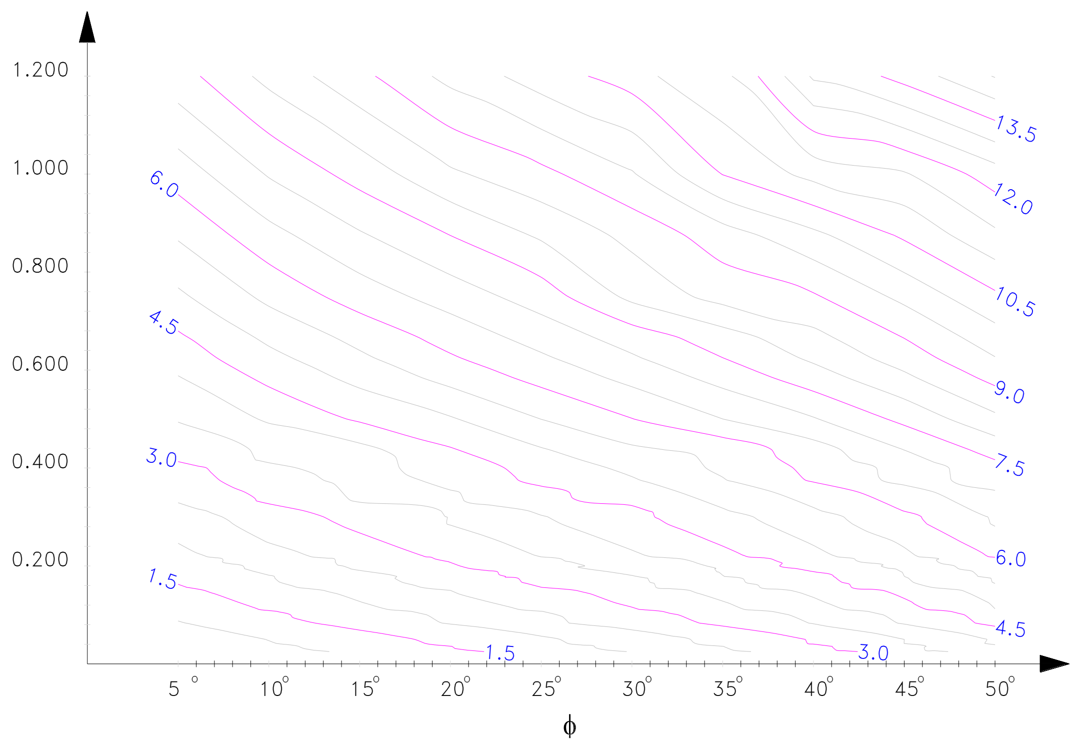


Figure A-8: Factor of safety Contour for slope 15 ^{o} and r_u=0.25

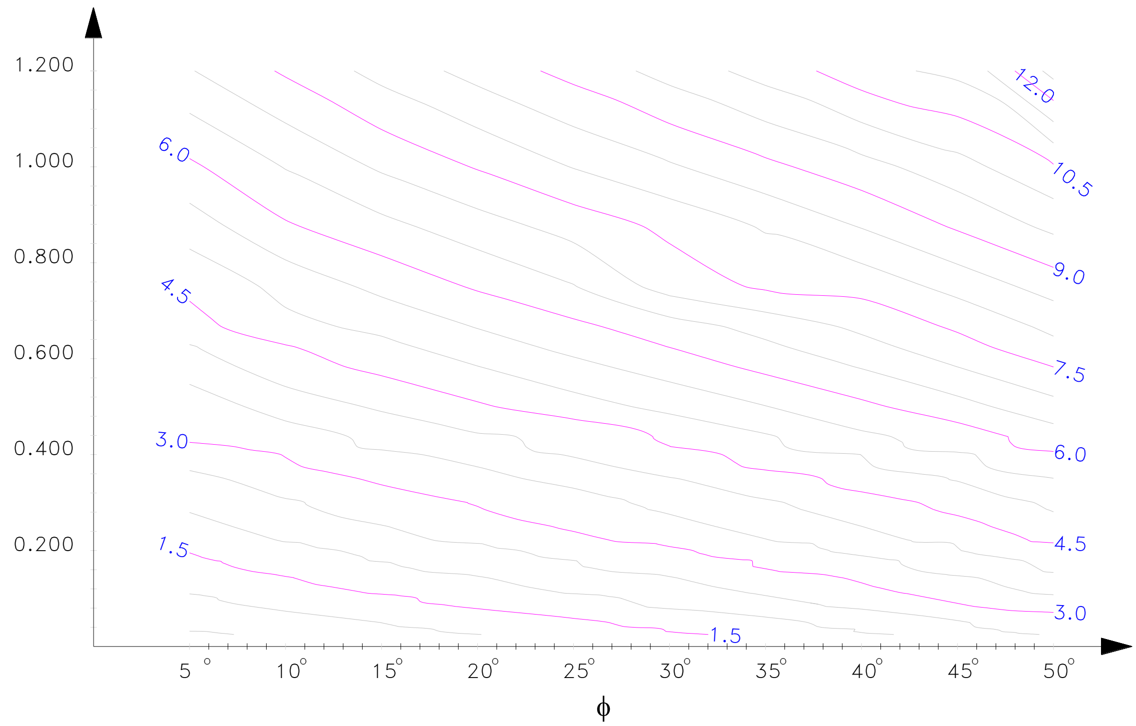


Figure A-9: Factor of safety Contour for slope 15 ° and r_u=0.50

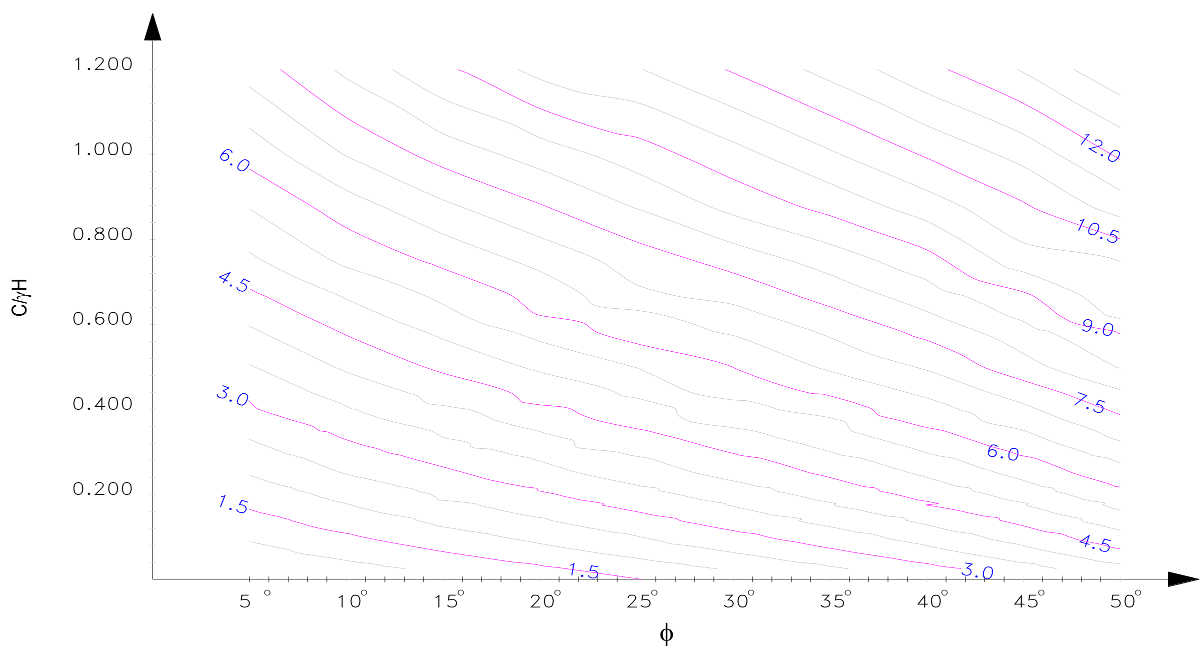


Figure A-10: Factor of safety Contour for slope 20 ° and r_u=0.0

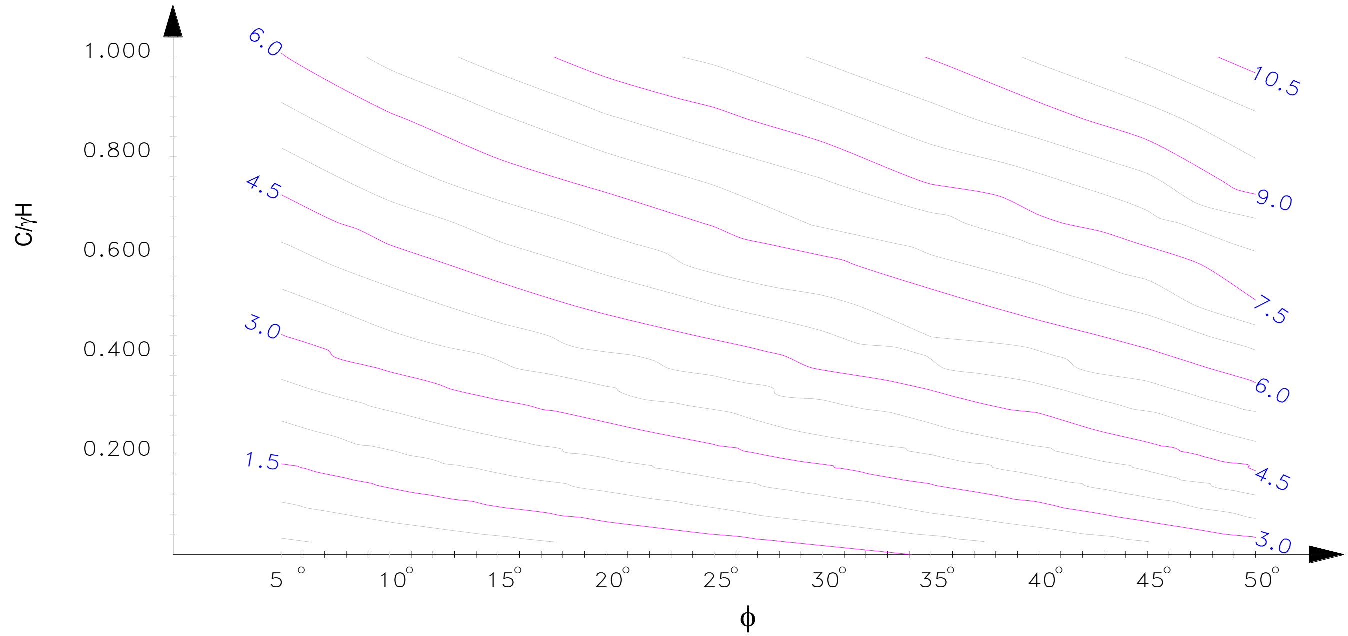


Figure A-11: Factor of safety Contour for slope 20 ^{o} and r_u=0.25

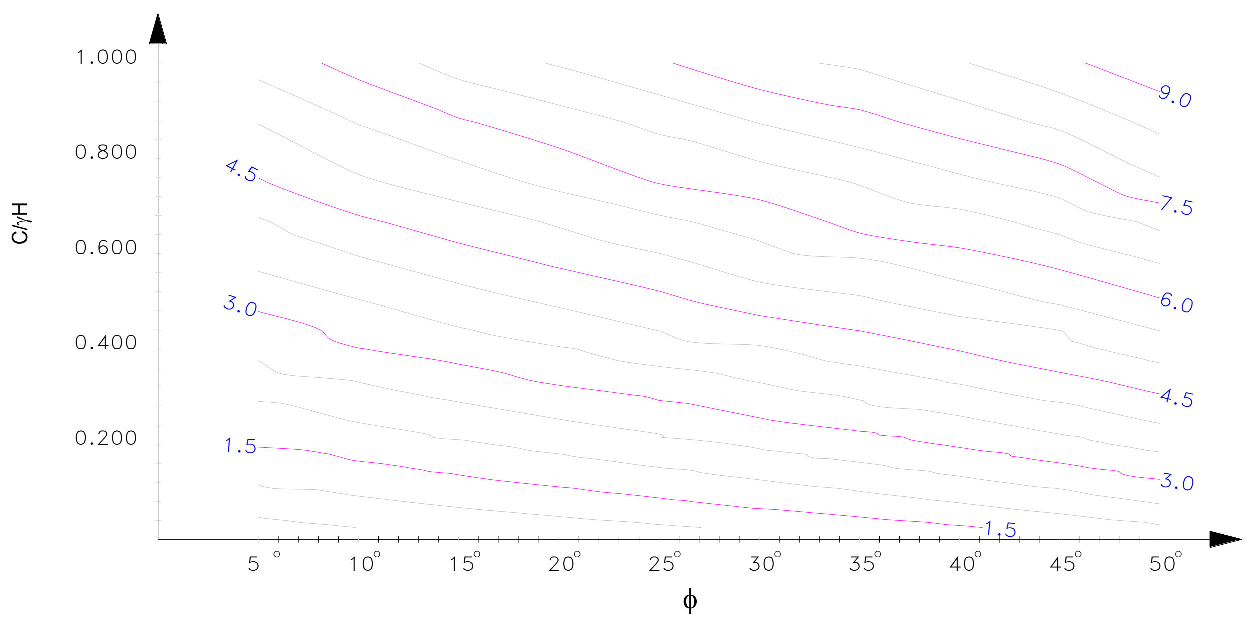


Figure A-12: Factor of safety Contour for slope 20 ^{o} and r_u=0.50

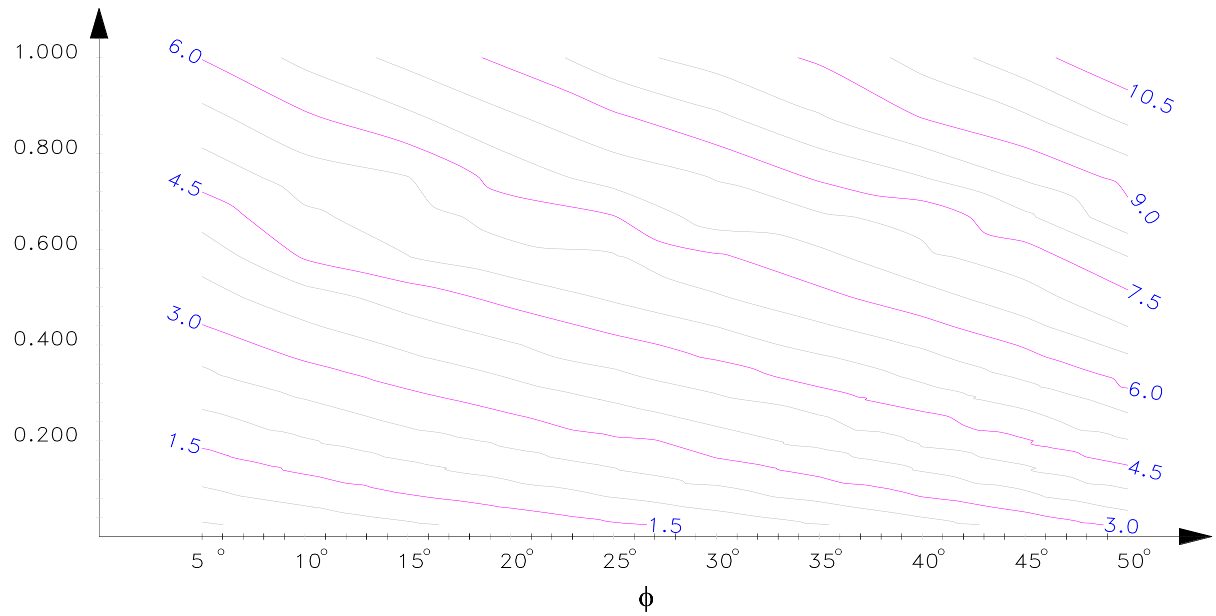


Figure A-13: Factor of safety Contour for slope 25 ^{o} and r_u=0.0

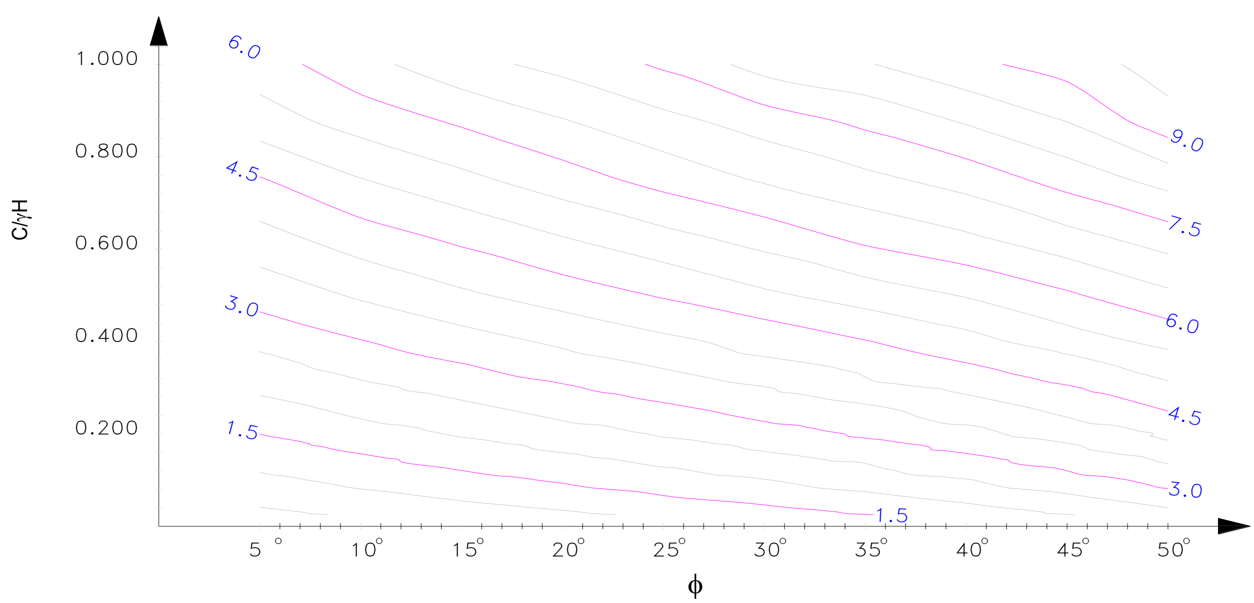


Figure A-14: Factor of safety Contour for slope 25 ^{o} and r_u=0.25

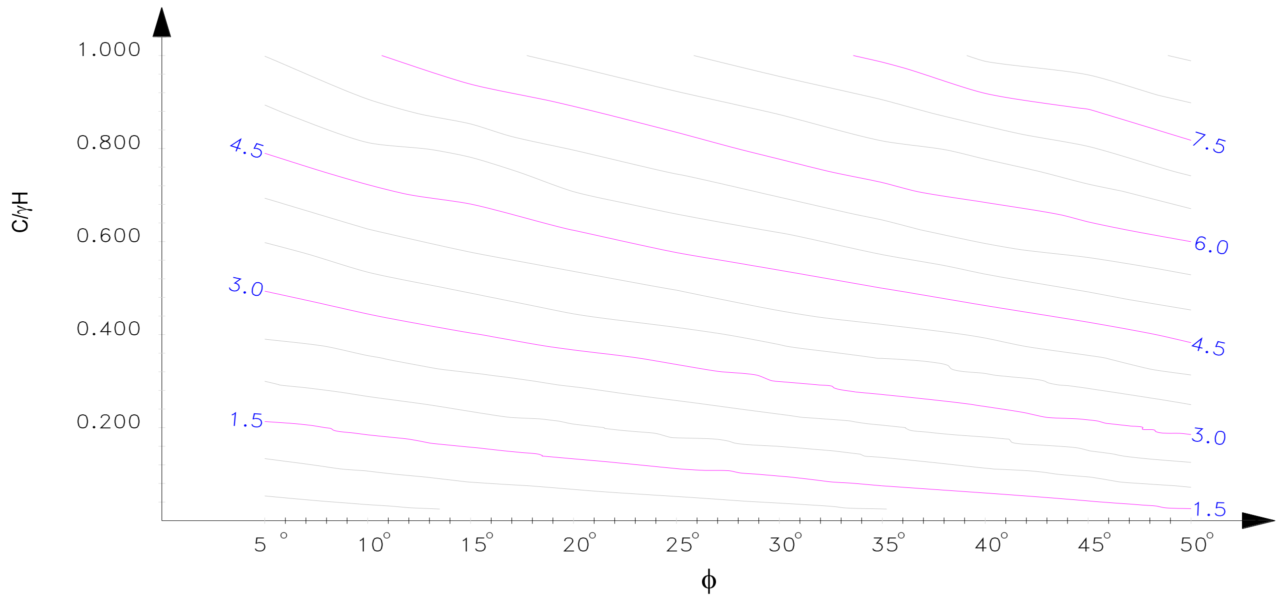


Figure A-15: Factor of safety Contour for slope 25 ^{o} and r_u=0.50

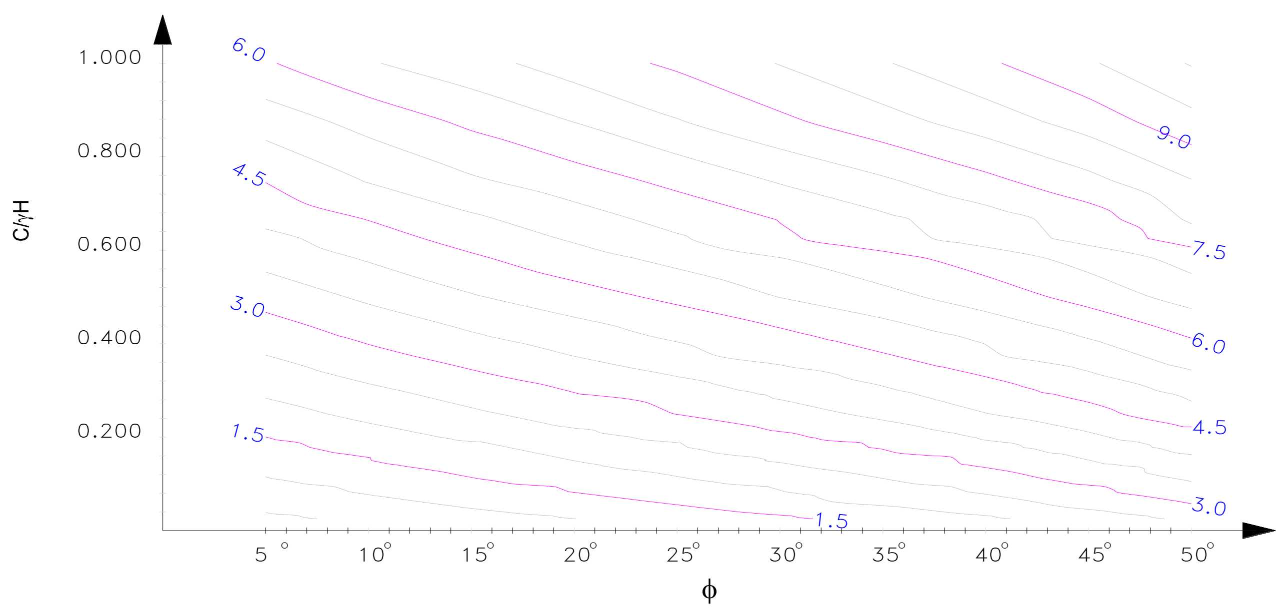


Figure A-16: Factor of safety Contour for slope 30 ^{o} and r_u=0.0

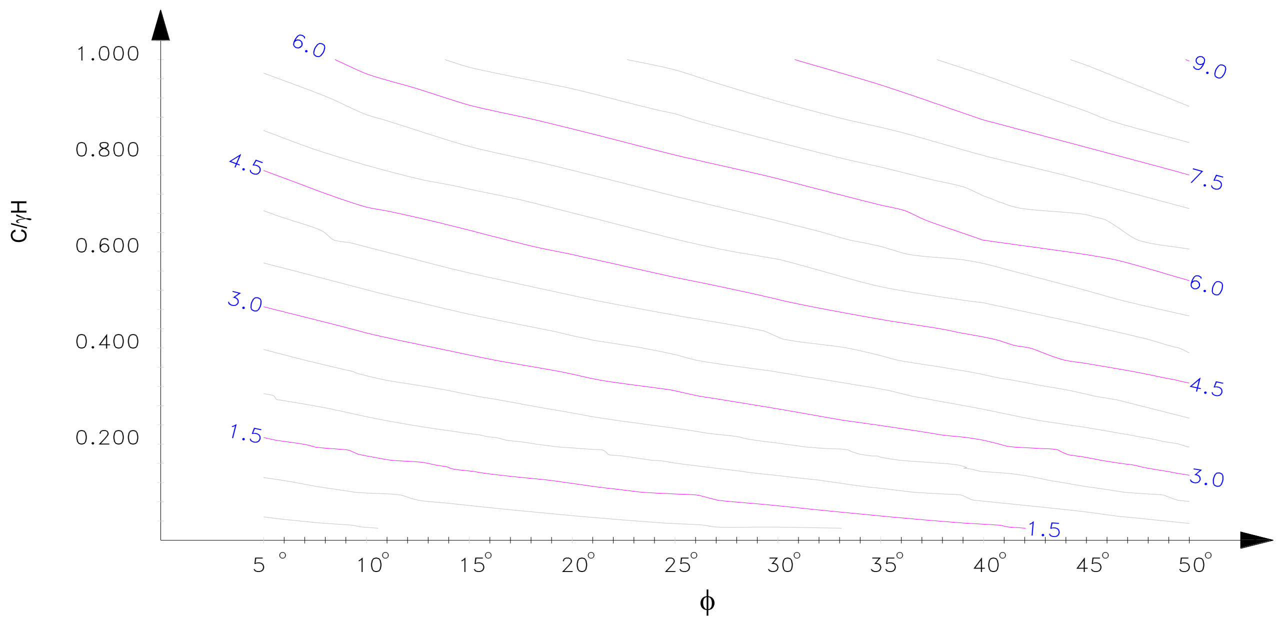


Figure A-17: Factor of safety Contour for slope 30 ^{o} and r_u=0.25

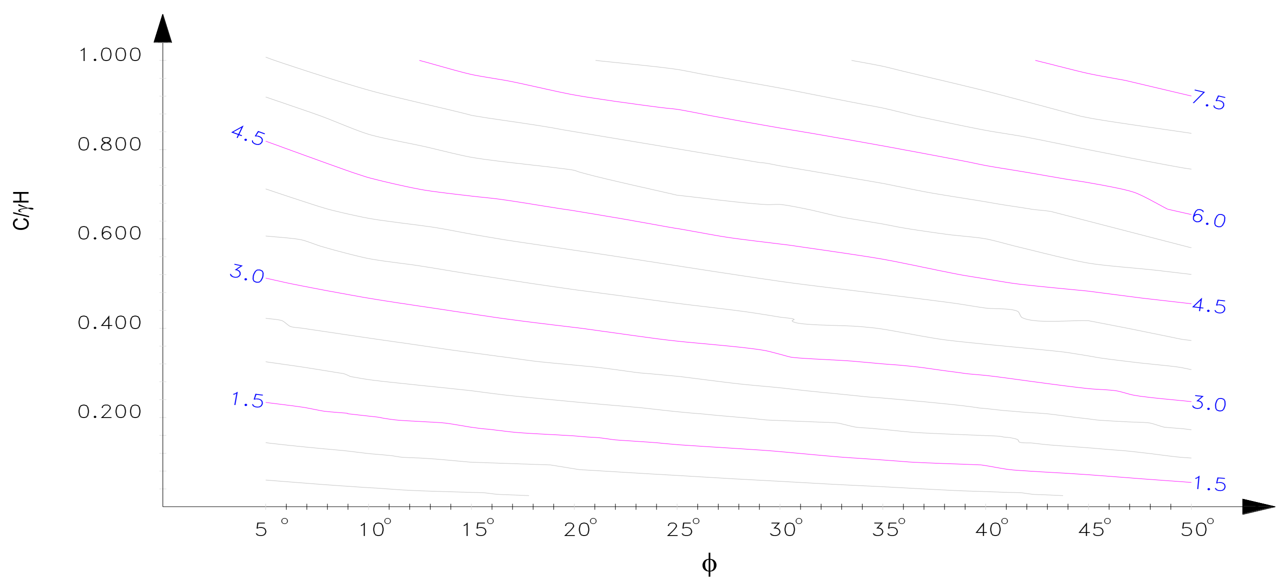


Figure A-18: Factor of safety Contour for slope 30 ^{o} and r_u=0.50

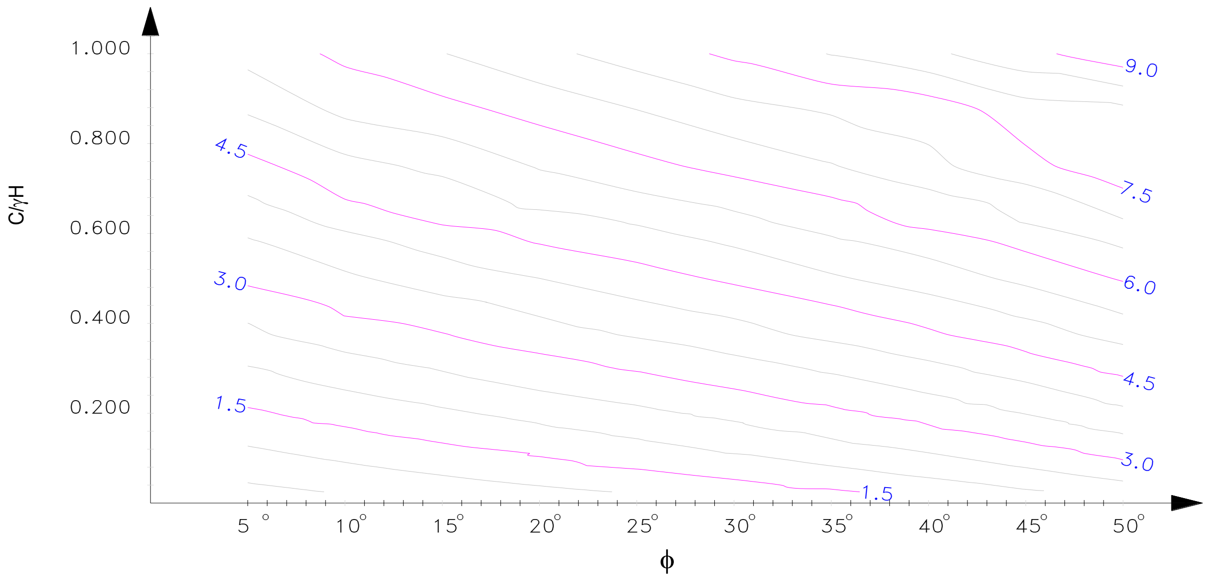


Figure A-19: Factor of safety Contour for slope 35 ° and r_u=0.0

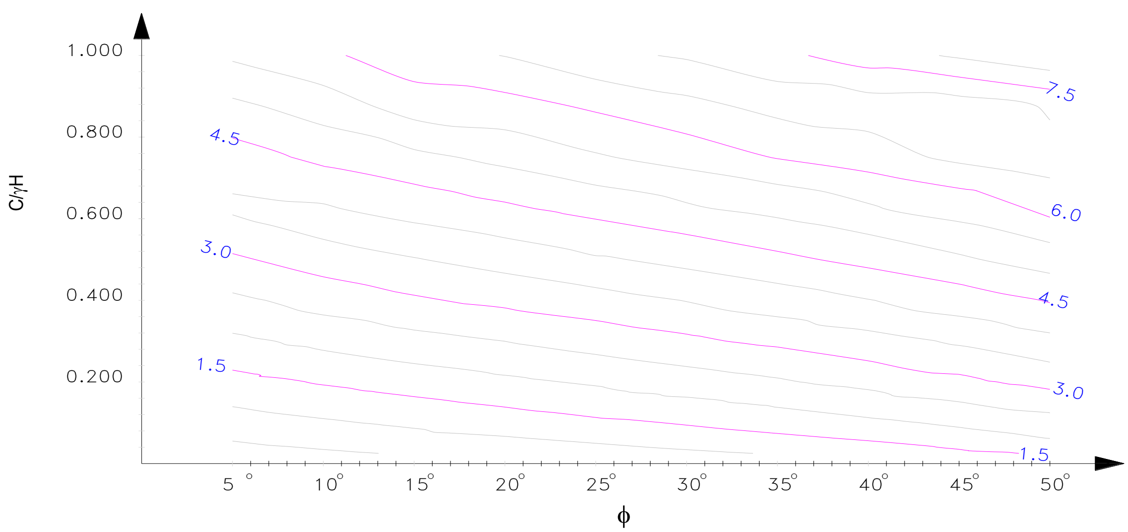


Figure A-20: Factor of safety Contour for slope 35 ° and r_u=0.25

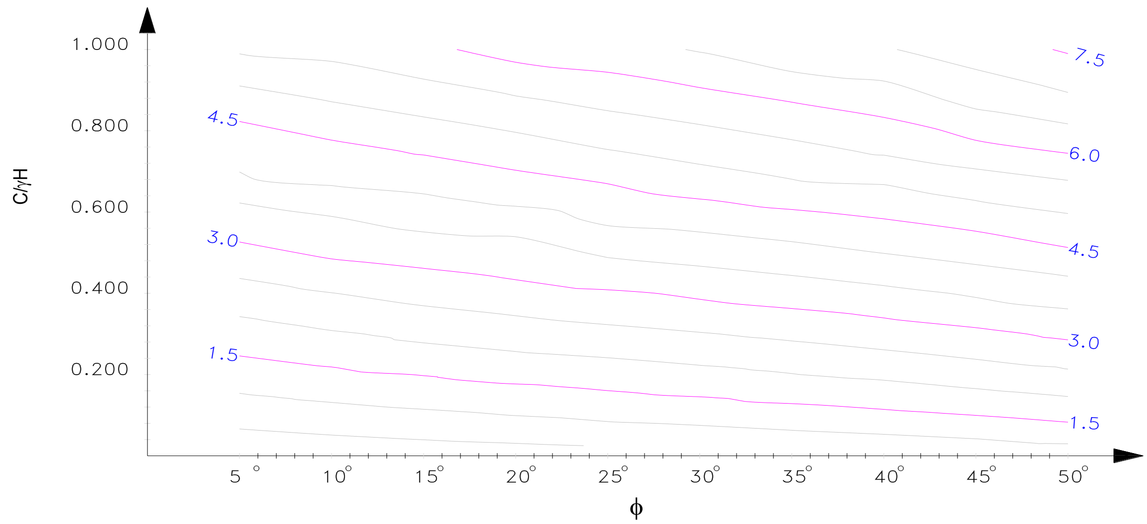


Figure A-21: Factor of safety Contour for slope 35 ^{o} and r_u=0.50

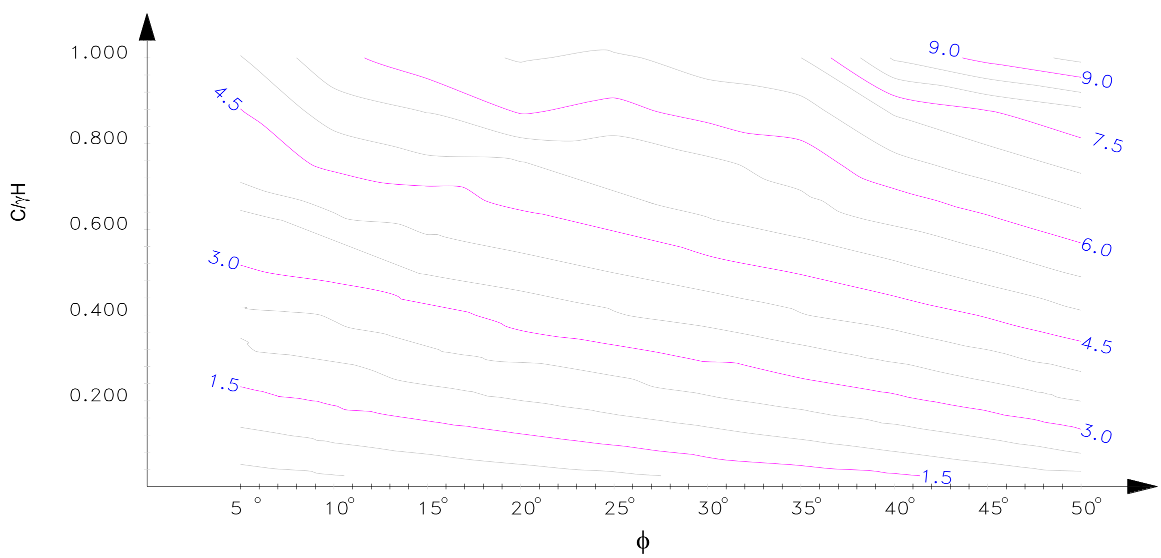


Figure A-22: Factor of safety Contour for slope 40 ^{o} and r_u=0.0

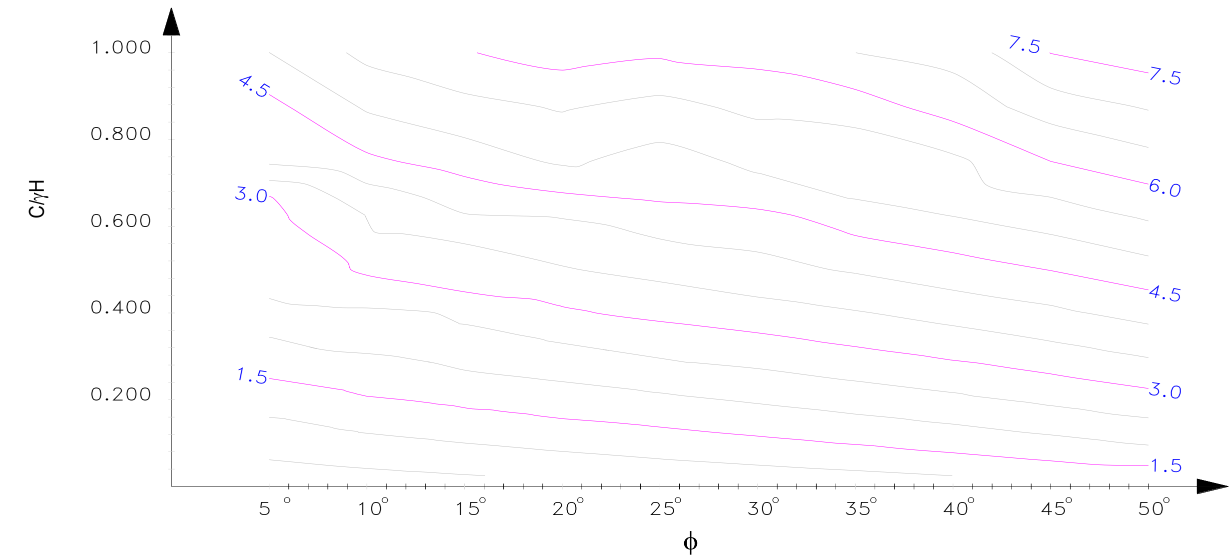


Figure A-23: Factor of safety Contour for slope 40 ° and r_u=0.25

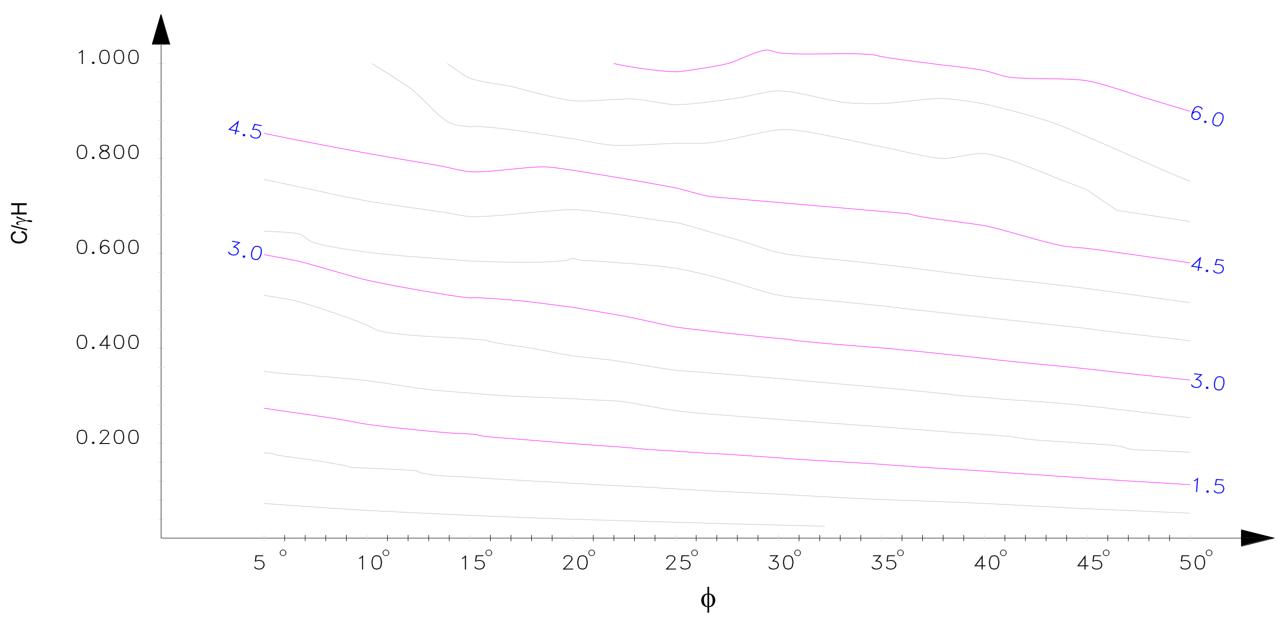


Figure A-24: Factor of safety Contour for slope 40 ° and r_u=0.50

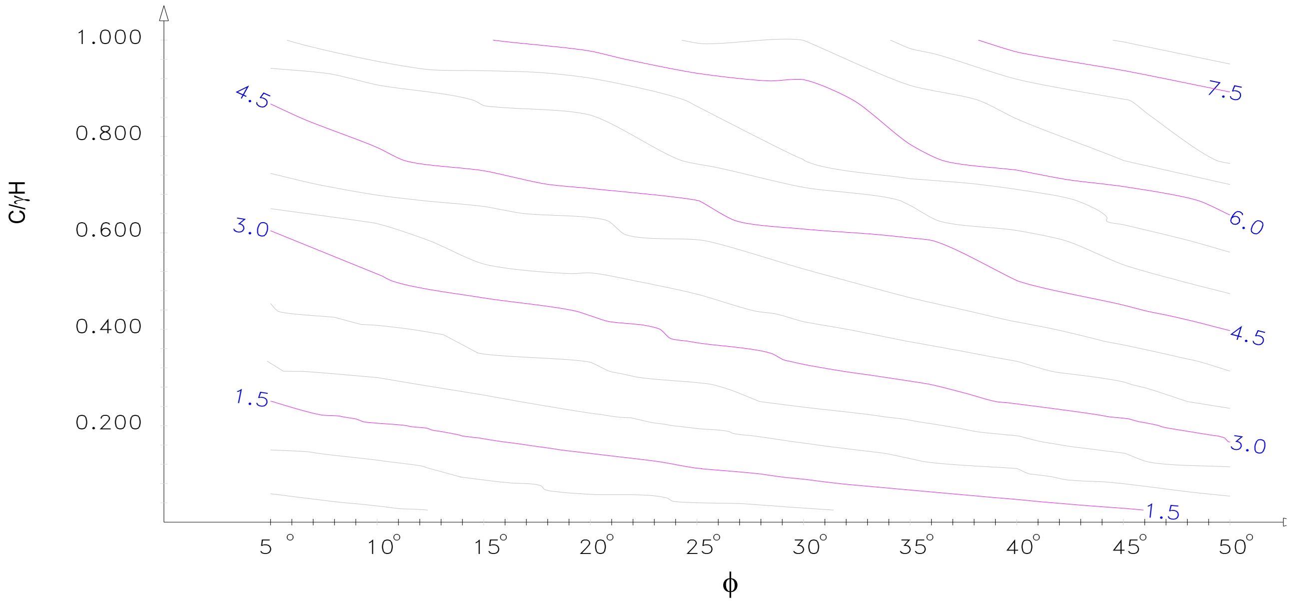


Figure A-25: Factor of safety Contour for slope 45 ° and r_u=0.0

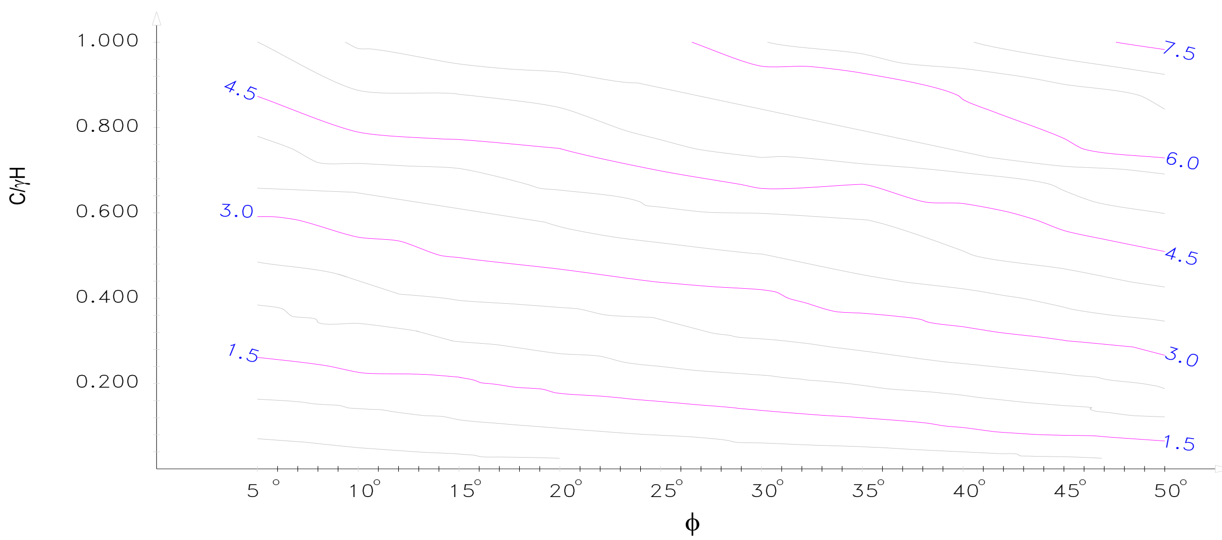


Figure A-26: Factor of safety Contour for slope 45 ° and r_u=0.25

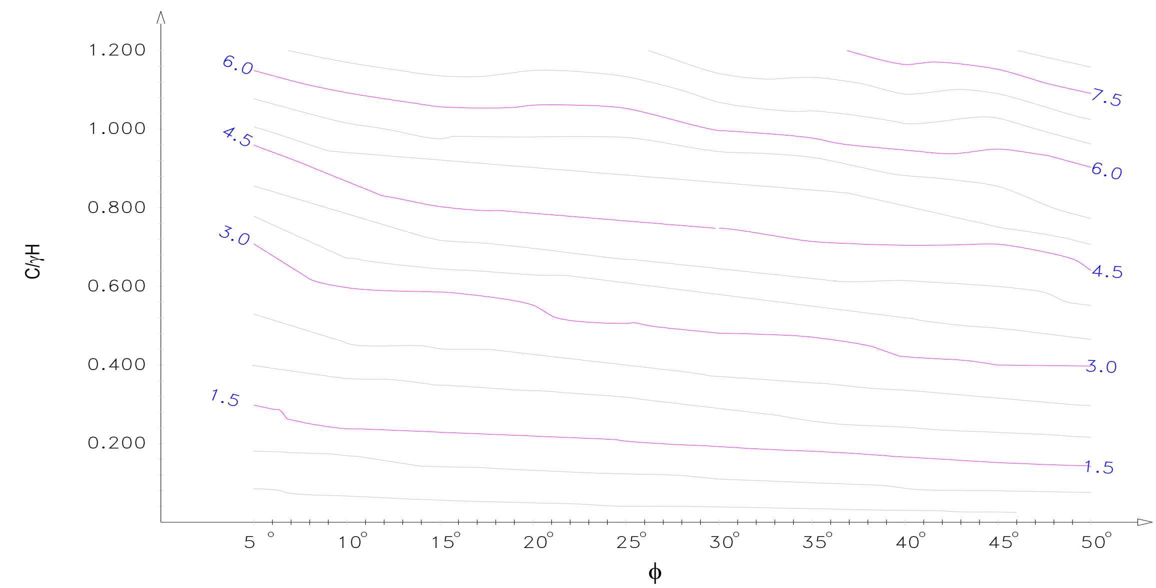


Figure A-27: Factor of safety Contour for slope 45 ° and r_u=0.50

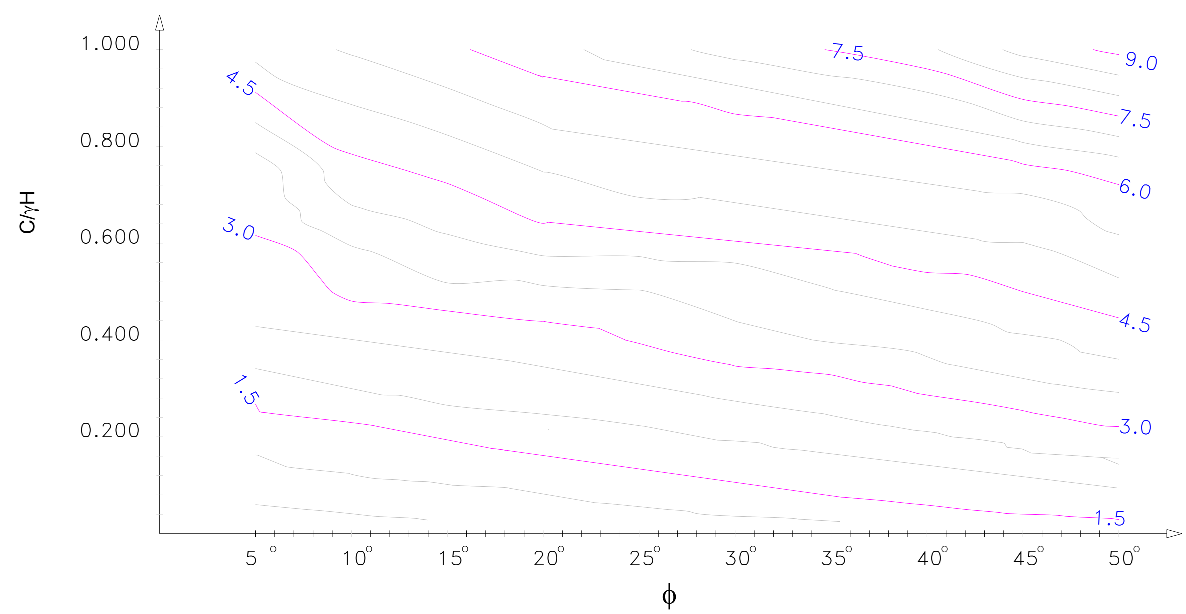


Figure A-28: Factor of safety Contour for slope 50 ° and r_u=0.0

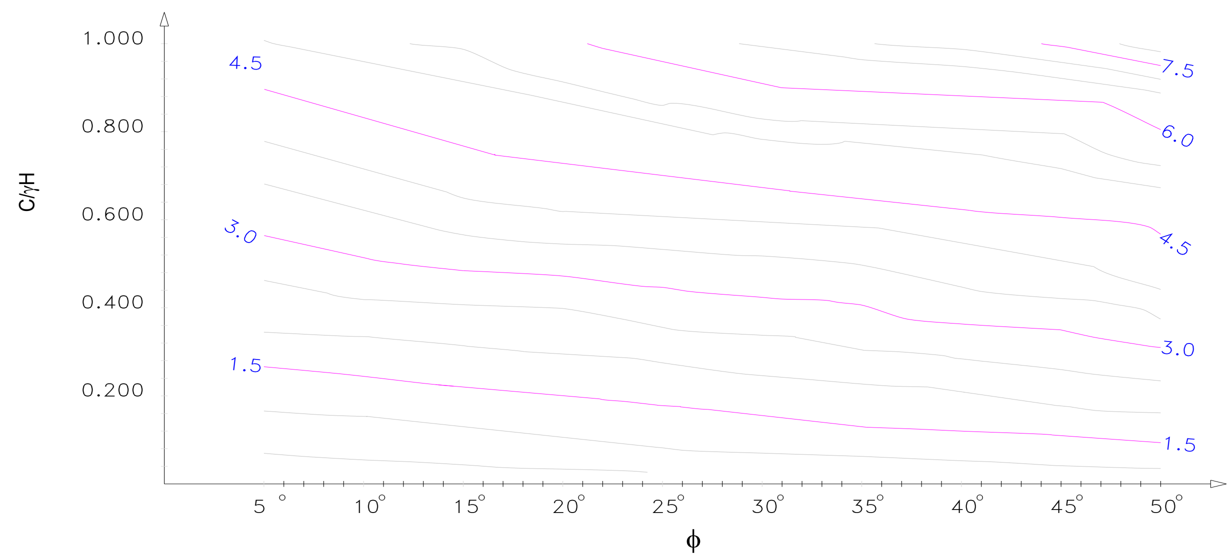


Figure A-29: Factor of safety Contour for slope 50 ° and r_u=0.25

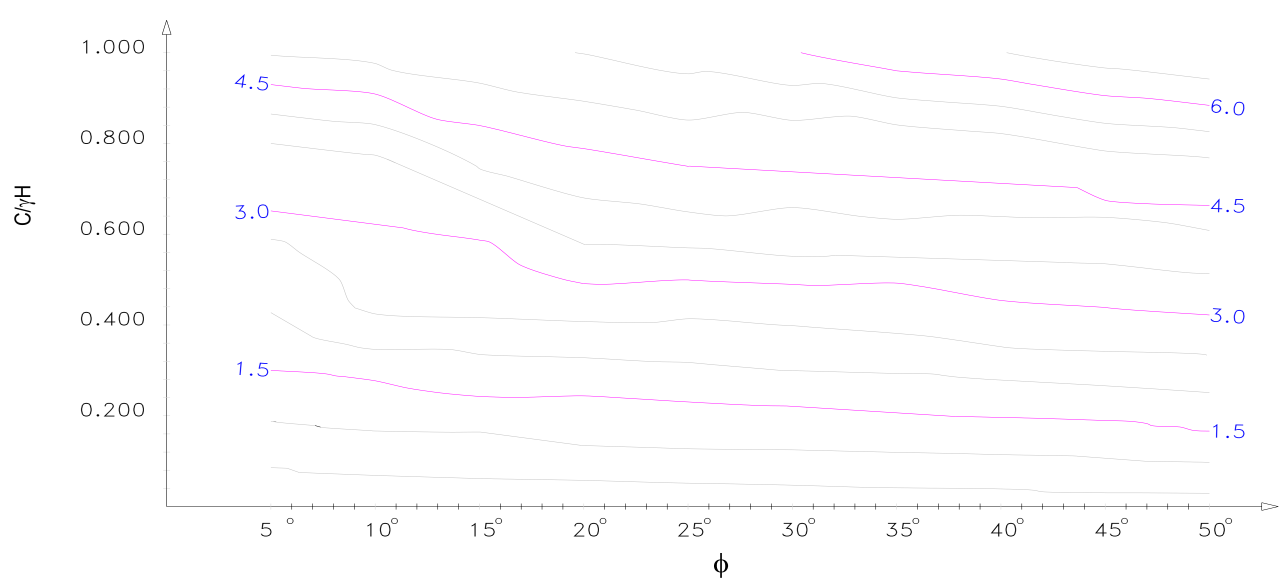


Figure A-30: Factor of safety Contour for slope 50 ° and r_u=0.50

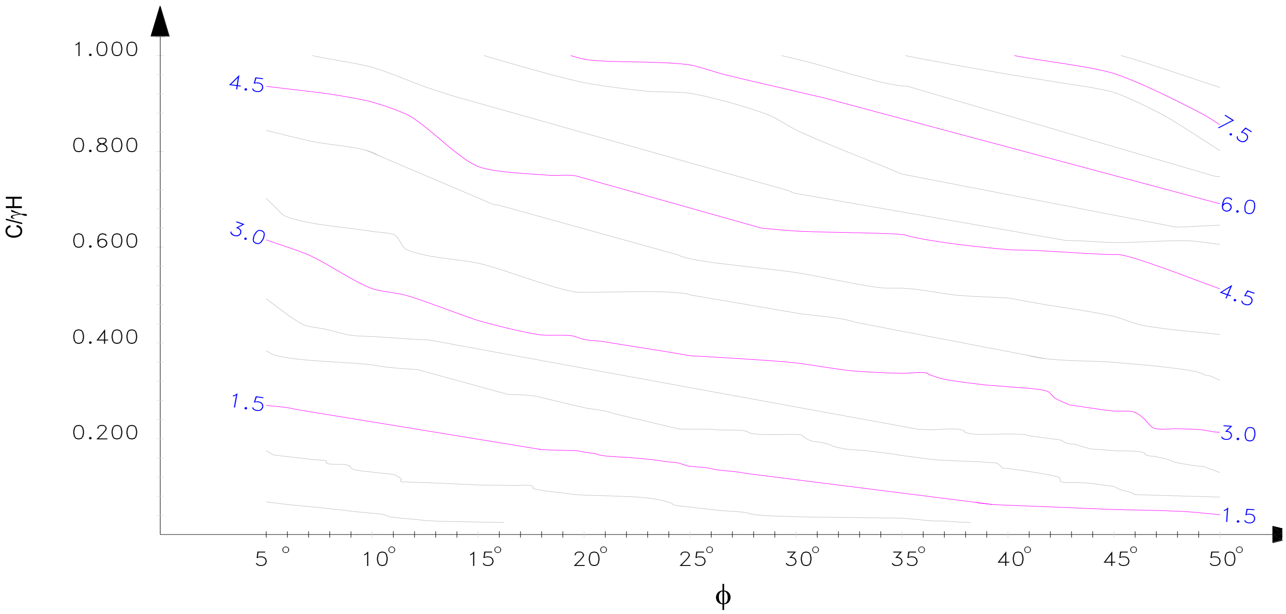


Figure A-31: Factor of safety Contour for slope 55 ^{o} and r_u=0.0

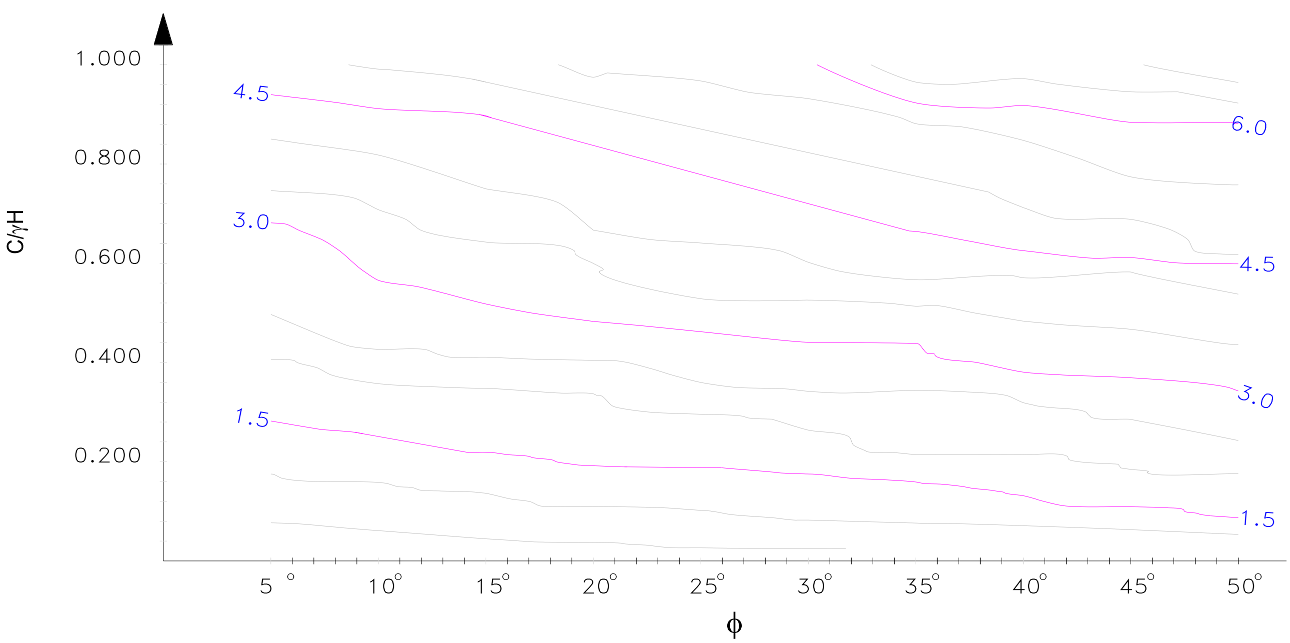


Figure A-32: Factor of safety Contour for slope 55 ^{o} and r_u=0.25

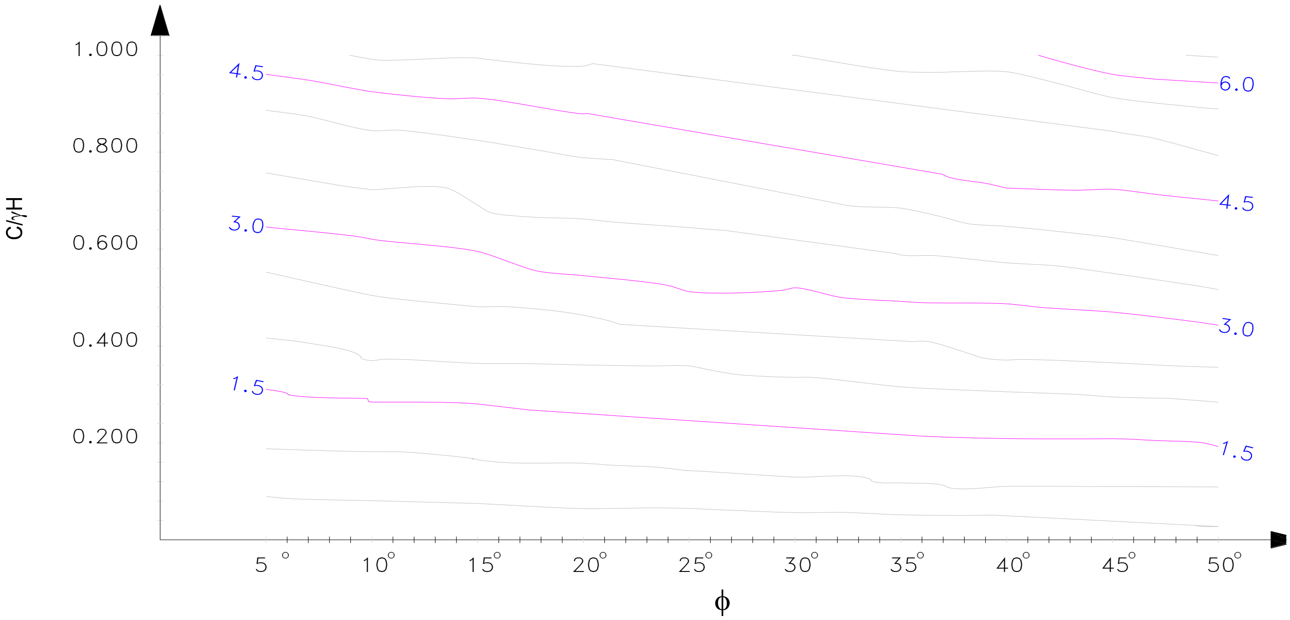


Figure A-33: Factor of safety Contour for slope 55 ^{o} and r_u=0.50

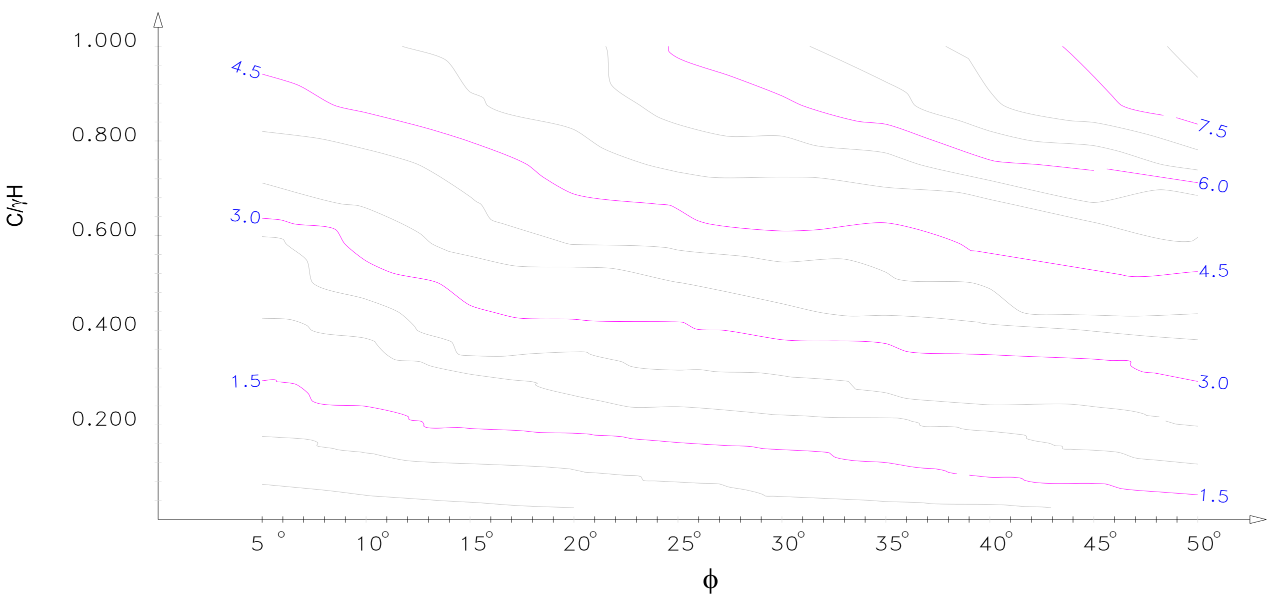


Figure A-34: Factor of safety Contour for slope 60 ^{o} and r_u=0.0

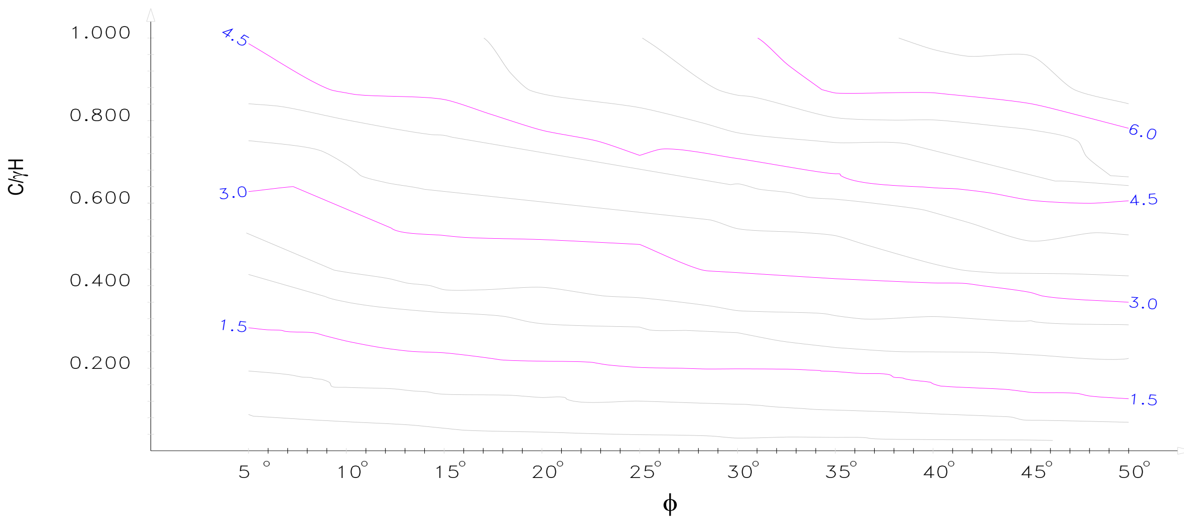


Figure A-35: Factor of safety Contour for slope 60 ^{o} and r_u=0.25

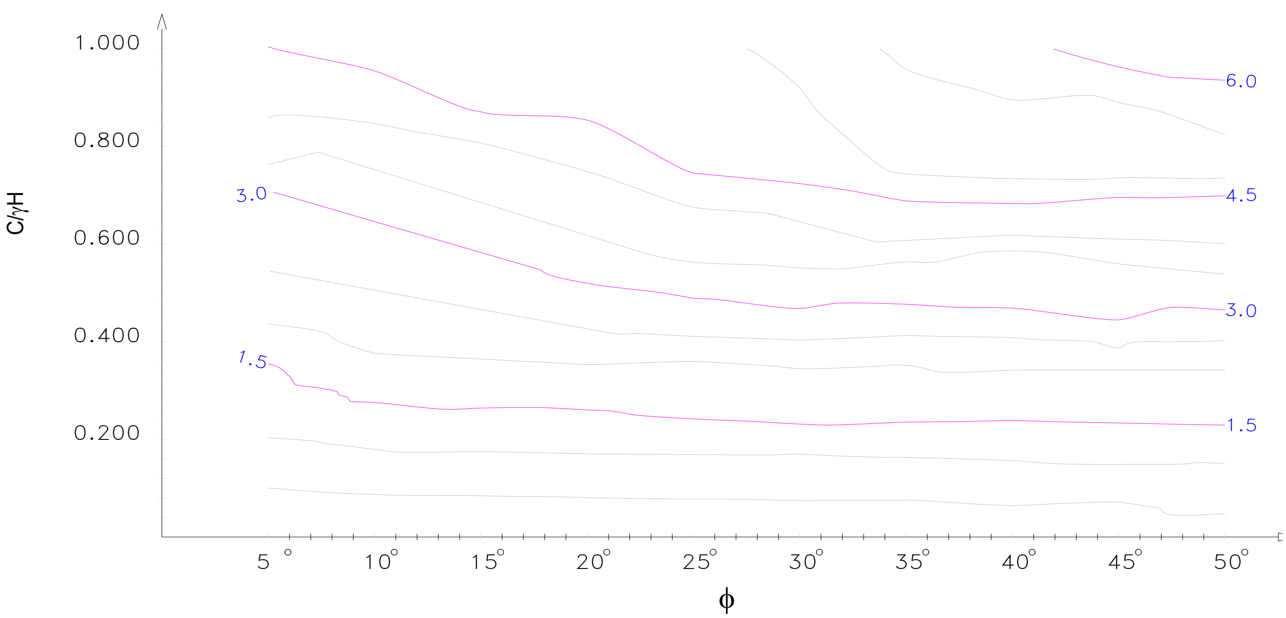


Figure A-36: Factor of safety Contour for slope 60 ^{o} and r_u=0.50

Method of Computation

Given soil strength parameter c and ϕ, height H, and unit weight γ, calculate ($c/\gamma H$). Plot a vertical line from the point ϕ on the x-axis and a horizontal line from the calculated $c/(\gamma H)$. Read the factor of safety on the accompanying graphs where the two lines meet and compare with predefined factor of safety.

Assume an engineer is called upon to design a high way cut slope to a specified factor of safety, say 1.5. Consider the following example: h= 12 m, γ= 17.6 KN/m^3, c= 26 KN/m^2 and ϕ= 20^0. With the help of such a graph one can design the most economical slope with the given specifications. For ϕ= 20^0, c/(γH) = 0.123, and r_u=0 a point plotted in Appendix-A Figure A-22 showing that of slope 40^0 (H:V =1.2:1) is good enough for this height for a predefined factor of safety of 1.5.

APPENDIX - B

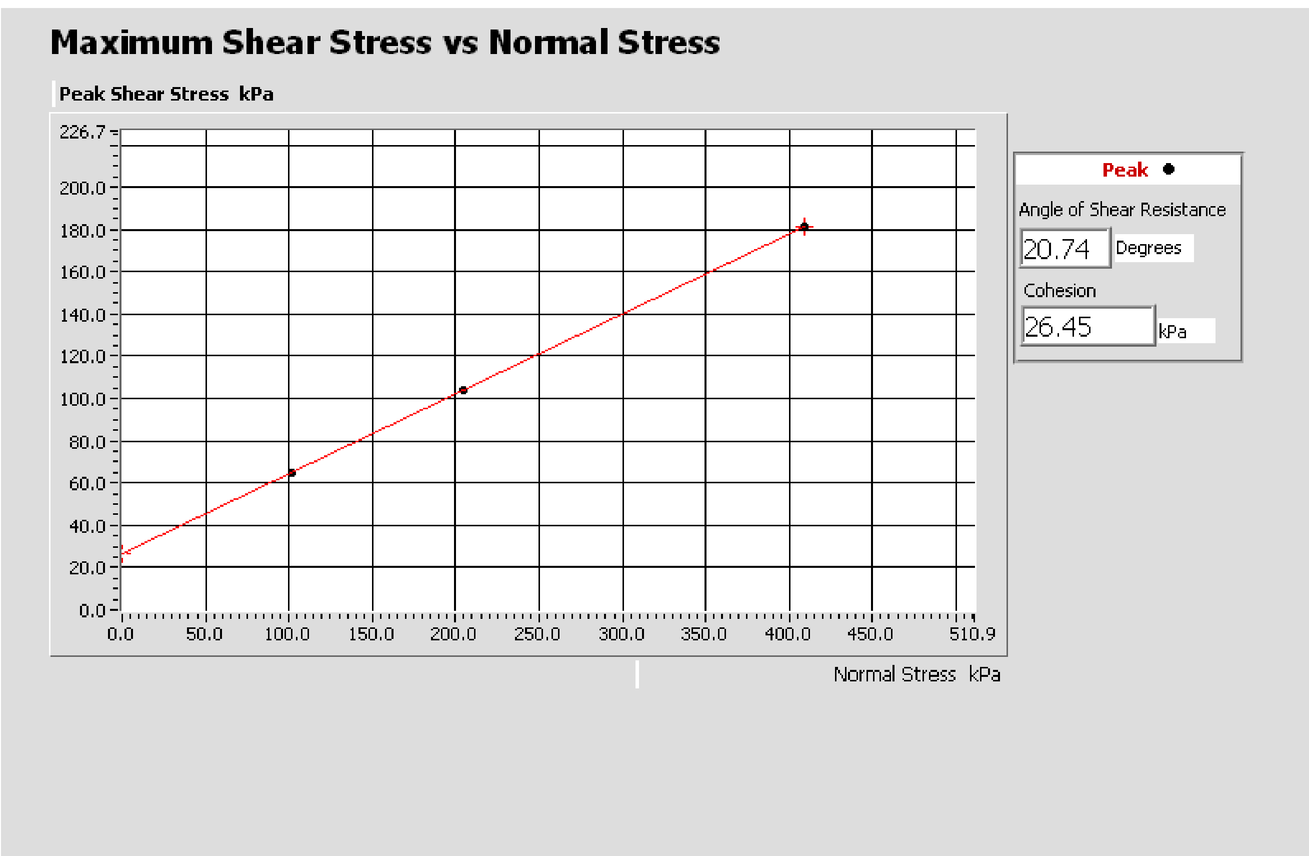
LABORATORY TEST RESULTS

Shear Strength by Direct Shear
(Small Shear Box)

ELE International

Test Summary

Sampling Area	Lemmi			
Reference	A	B	C	
Applied Normal Stress	102.2 kPa	204.4 kPa	408.8 kPa	
Peak Strength	64.9 kPa	104.2 kPa	181.2 kPa	
Corresponding Horizontal Displacement	8.086 mm	6.658 mm	3.625 mm	
Residual Shear Stress				
Rate(s) of Shear Displacement	Stage 1: 0.5000mm/min	Stage 1: 0.5000mm/min	Stage 1: 0.5039mm/min	
Final Height	21.99 mm	21.83 mm	19.73 mm	
Cumulative Displacement	8.086 mm	7.260 mm	6.048 mm	
Number of Traverses	1	1	1	



Shear Strength by Direct Shear
(Small Shear Box)

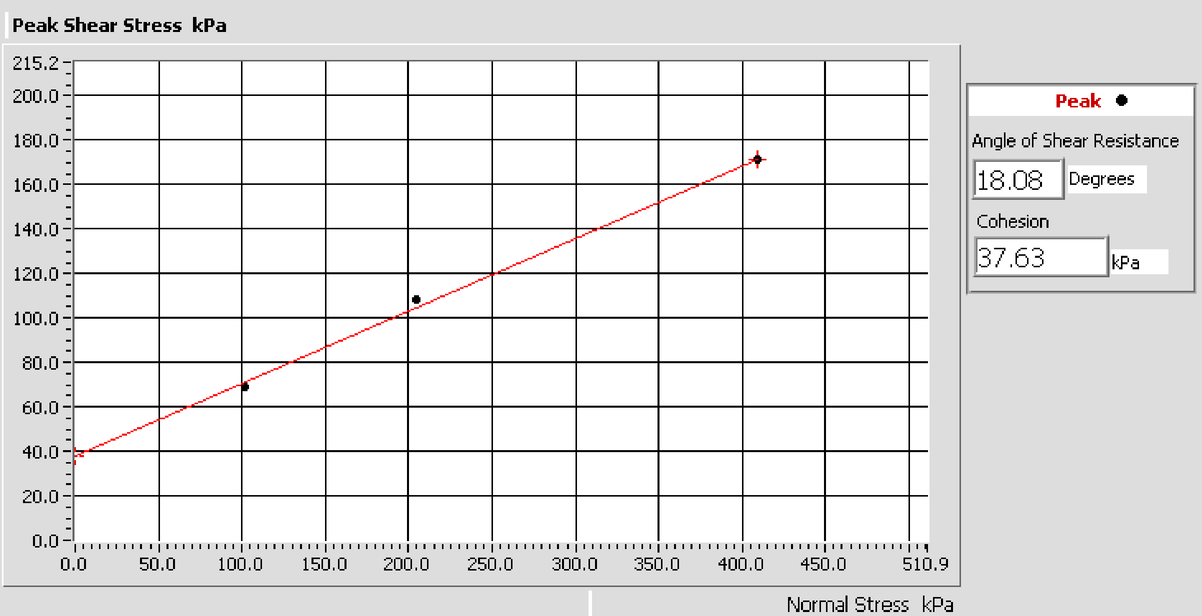
ELE International

Test Summary

Sampling Area: Armenia

Reference	A	B	C	
Applied Normal Stress	102.2 kPa	204.4 kPa	408.8 kPa	
Peak Strength	69.1 kPa	108.1 kPa	171.1 kPa	
Corresponding Horizontal Displacement	6.657 mm	6.668 mm	7.864 mm	
Residual Shear Stress				
Rate(s) of Shear Displacement	Stage 1: 0.5000mm/min	Stage 1: 0.5000mm/min	Stage 1: 0.5000mm/min	
Final Height	23.80 mm	23.56 mm	22.69 mm	
Cumulative Displacement	9.080 mm	9.079 mm	9.082 mm	
Number of Traverses	1	1	1	

Maximum Shear Stress vs Normal Stress



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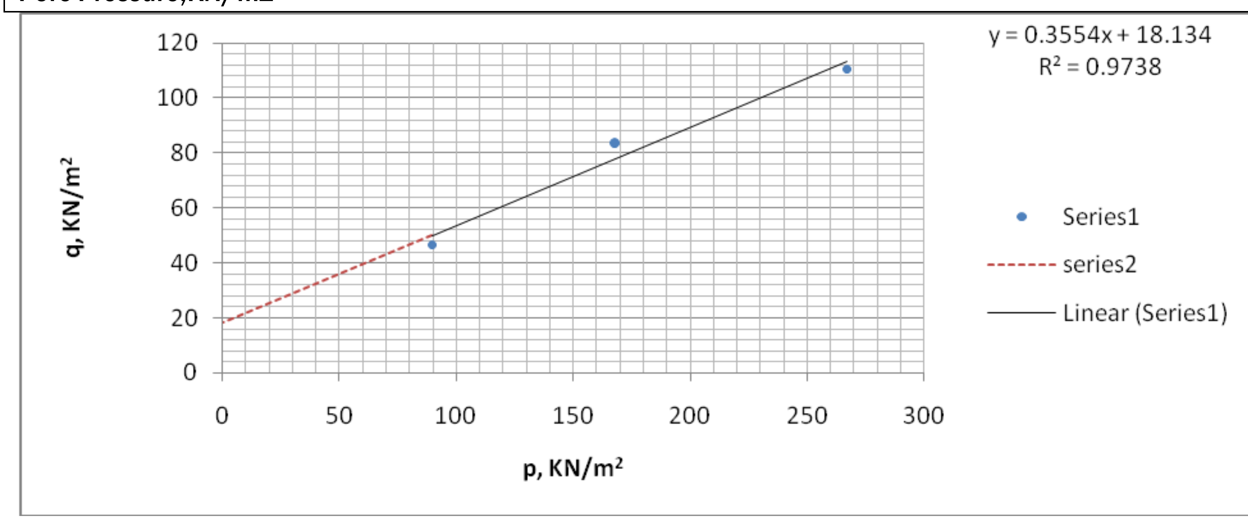
Attachment 3 = Shear Strength parameter determination

Project:	MSc Thesis	Test Type:	Triaxial CU
Location:	Amanuel	Specimen:	Soil Sample

BH No		Sample No	3
Depth (m)		Sample Condition	Undisturbed
Initial Height (mm)		Initial Diameter (mm)	38
Initial Area (mm2)		Initial Height (mm)	76
Initial Weight (gm)		Initial volume (cm2)	
Final dry weight (gm)		Bulk density (gm/cm3)	
Moisture Content (%)		Dry density(gm/cm3)	

				c'	ϕ'
Chamber Pressure, KN/m2, (σ_3)	100	200	300	18	21
Deviator Stress,KN/m2, (σ_1-σ_3)	93.3	167.4	220.9		
Pore Pressure,KN/m2	57	116	143.3		



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Attachment 1 = Grain Size Analysis

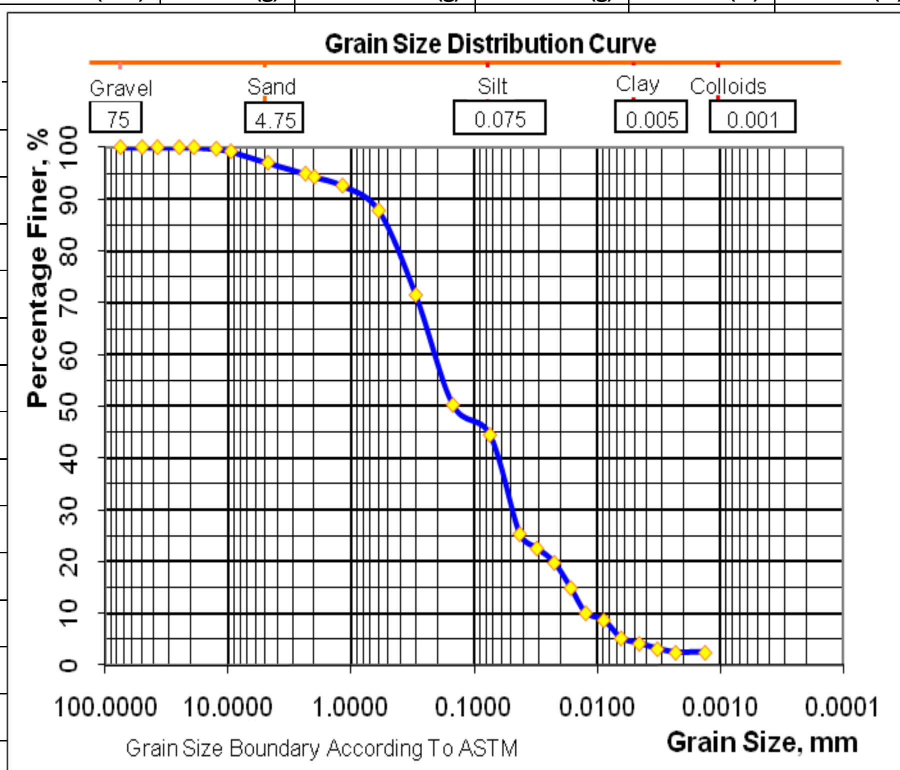
Project:	MSc Thesis
Owner:	-
Client:	-
Location:	Amanuel

Sample No : TP1   Sample Depth, m : 1.50

Total mass of sample, g 2000

Sieve Analysis

Sieve No	Sieve Opening (mm)	Mass of Sieve (g)	Mass of sieve + Retained soil (g)	Mass of Retained soil (g)	Percentage Retained (%)	Cum. Percentage Retained (%)	Perc. Passing (%)
3''							100.0
2''							100.0
1.5"							100.0
1"							100.0
3/4''							100.0
1/2"							99.7
3.8"							99.2
No 4							97.0
No 8							94.9
No 10							94.3
No 16							92.6
No 30							87.8
No 50							71.5
No 100							50.3
No 200							44.5
pan							-------



Hydrometer Analysis

Specific Gravity of soil 2.79 Test Temperature, deg.c 20

Elapsed Time (min)	Actual Hydrometer Reading	Composite Correction	Corrected Hydrometer Reading	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
3/4	1.0210	-0.0027	1.0183	10.75	0.01312	0.0430	57.05	25.39
1	1.0190	-0.0027	1.0163	11.27	0.01312	0.0312	50.81	22.61
2	1.0170	-0.0027	1.0143	11.80	0.01312	0.0225	44.58	19.84
4	1.0135	-0.0027	1.0108	12.73	0.01312	0.0165	33.67	14.98
8	1.0100	-0.0027	1.0073	13.65	0.01312	0.0125	22.76	10.13
15	1.0090	-0.0027	1.0063	13.92	0.01312	0.0089	19.64	8.74
30	1.0075	-0.0027	1.0048	14.32	0.01312	0.0064	14.96	6.66
60	1.0065	-0.0027	1.0038	14.58	0.01312	0.0065	11.85	5.27
120	1.0058	-0.0027	1.0031	14.78	0.01312	0.0046	9.51	4.23
240	1.0050	-0.0027	1.0023	14.98	0.01312	0.0033	7.17	3.19
480	1.0045	-0.0027	1.0018	15.11	0.01312	0.0023	5.61	2.50
1440	1.0045	-0.0027	1.0018	15.11	0.01312	0.0013	5.61	2.50

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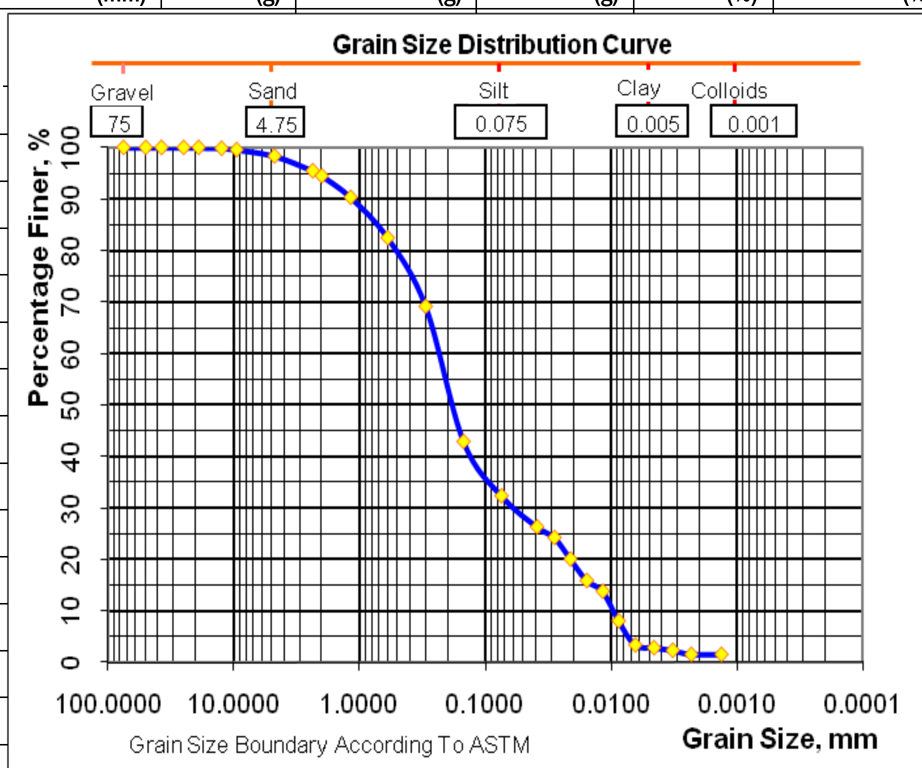
Attachment 1 = Grain Size Analysis

Project:	MSc Thesis
Owner:	-
Client:	-
Location:	Armenia

Sample No : TP1   Sample Depth, m : 1.20

Sieve Analysis   Total mass of sample, g 5040

Sieve No	Sieve Opening (mm)	Mass of Sieve (g)	Mass of sieve + Retained soil (g)	Mass of Retained soil (g)	Percentage Retained (%)	Cum. Percentage Retained (%)	Perc. Passing (%)
3''							100.0
2''							100.0
1.5"							100.0
1"							100.0
3/4''							100.0
1/2"							99.8
3.8"							99.6
No 4							98.3
No 8							95.5
No 10							94.5
No 16							90.3
No 30							82.5
No 50							69.2
No 100							42.9
No 200							32.4
pan	--------	240.0	290.0	50.0	1.0	68.6	-------



Hydrometer Analysis

Specific Gravity of soil 2.64 Test Temperature, deg.c 20

Elapsed Time (min)	Actual Hydrometer Reading	Composite Correction	Corrected Hydrometer Reading	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
3/4	1.0280	-0.0027	1.0253	8.89	0.01312	0.0391	81.45	26.38
1	1.0260	-0.0027	1.0233	9.42	0.01312	0.0285	75.01	24.29
2	1.0220	-0.0027	1.0193	10.48	0.01312	0.0212	62.14	20.12
4	1.0180	-0.0027	1.0153	11.54	0.01312	0.0158	49.26	15.95
8	1.0160	-0.0027	1.0133	12.07	0.01312	0.0118	42.82	13.87
15	1.0105	-0.0027	1.0078	13.52	0.01312	0.0088	25.11	8.13
30	1.0090	-0.0027	1.0063	13.92	0.01312	0.0063	20.28	6.57
60	1.0060	-0.0027	1.0033	14.71	0.01312	0.0065	10.62	3.44
120	1.0055	-0.0027	1.0028	14.85	0.01312	0.0046	9.01	2.92
240	1.0050	-0.0027	1.0023	14.98	0.01312	0.0033	7.40	2.40
480	1.0043	-0.0027	1.0016	15.18	0.01312	0.0023	4.99	1.62
1440	1.0043	-0.0027	1.0016	15.18	0.01312	0.0013	4.99	1.62

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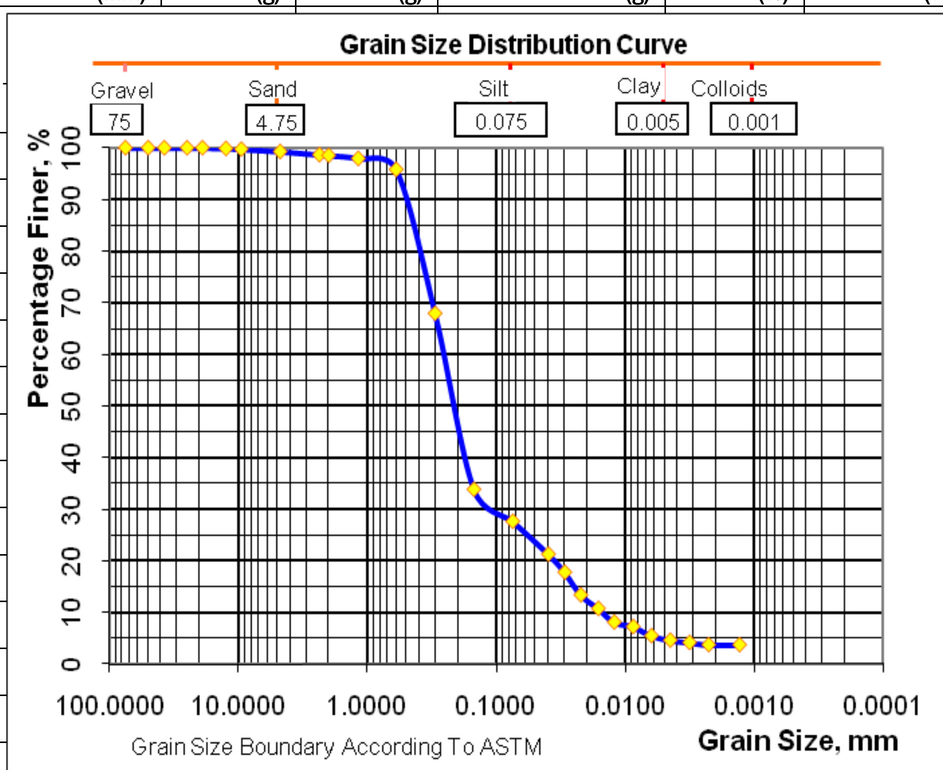
Attachment 1 = Grain Size Analysis

Project:	MSc Thesis
Owner:	-
Client:	-
Location:	Lemmi

Sample No : TP1 Sample Depth, m : 1.10

Sieve Analysis Total mass of sample, g 3000

Sieve No	Sieve Opening (mm)	Mass of Sieve (g)	Mass of sieve + Retained soil (g)	Mass of Retained soil (g)	Percentage Retained (%)	Cum. Percentage Retained (%)	Perc. Passing (%)
3''							100.0
2''							100.0
1.5"							100.0
1"							100.0
3/4''							100.0
1/2"							99.9
3.8"							99.7
No 4							99.3
No 8							98.7
No 10							98.5
No 16							97.9
No 30							95.8
No 50							67.9
No 100							33.9
No 200							27.7
pan							-------



Hydrometer Analysis

Specific Gravity of soil 2.71 Test Temperature, deg.c 20

Elapsed Time (min)	Actual Hydrometer Reading	Composite Correction	Corrected Hydrometer Reading	Effective Depth (cm)	Coefficient K	Grain Size (mm)	Perc. Finer (%)	Perc. Finer Combined (%)
3/4	1.0270	-0.0027	1.0243	9.16	0.01312	0.0397	77.02	21.31
1	1.0230	-0.0027	1.0203	10.22	0.01312	0.0297	64.34	17.80
2	1.0180	-0.0027	1.0153	11.54	0.01312	0.0223	48.49	13.42
4	1.0150	-0.0027	1.0123	12.33	0.01312	0.0163	38.99	10.79
8	1.0120	-0.0027	1.0093	13.13	0.01312	0.0123	29.48	8.16
15	1.0110	-0.0027	1.0083	13.39	0.01312	0.0088	26.31	7.28
30	1.0090	-0.0027	1.0063	13.92	0.01312	0.0063	19.97	5.52
60	1.0090	-0.0027	1.0063	13.92	0.01312	0.0063	19.97	5.52
120	1.0080	-0.0027	1.0053	14.18	0.01312	0.0045	16.80	4.65
240	1.0075	-0.0027	1.0048	14.32	0.01312	0.0032	15.21	4.21
480	1.0070	-0.0027	1.0043	14.45	0.01312	0.0023	13.63	3.77
1440	1.0070	-0.0027	1.0043	14.45	0.01312	0.0013	13.63	3.77

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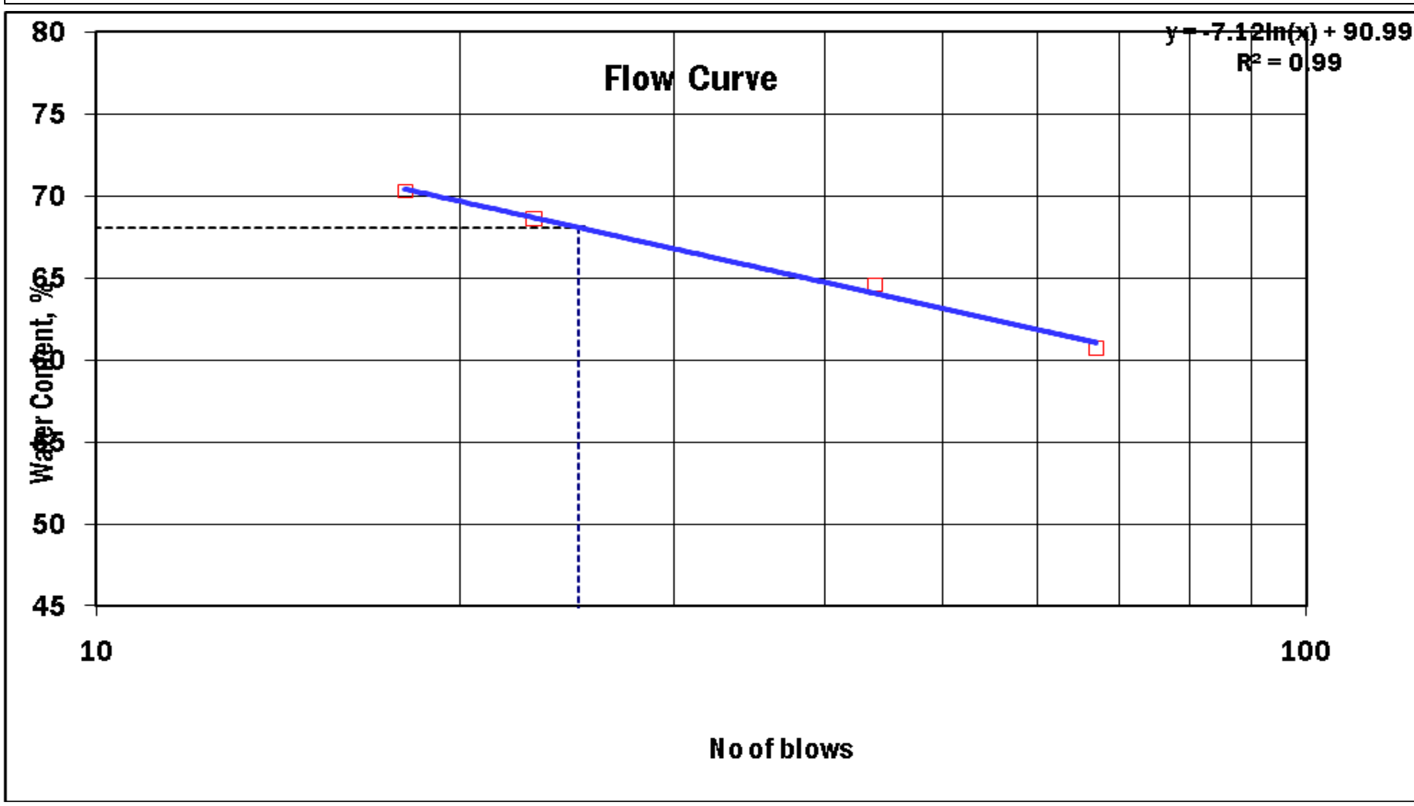
Attachment 2 = Liquid Limit and Plastic Limit Test

Project: MSc Thesis
Location: Amanuel

Sample No : TP1 Sample Depth, m : 1.5

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	D22	47	53	C-35	62	26
Mass of container, g	15.74	15.66	15.55	13.85	15.64	14.17
Mass of container + Wet soil, g	26.74	34.05	31.14	25.95	19.96	19.96
Mass of container + Dry soil, g	22.58	26.83	24.80	20.96	18.70	18.32
Mass of water, g	4.15	7.22	6.34	4.99	1.25	1.63
Mass of dry soil, g	6.84	11.17	9.25	7.10	3.06	4.16
Water content, %	60.71	64.57	68.61	70.30	40.83	39.25
No of blows	67	44	23	18	------	------

Liquid Limit, % = 68 Plastic Limit, % = 40 PI, %= 28



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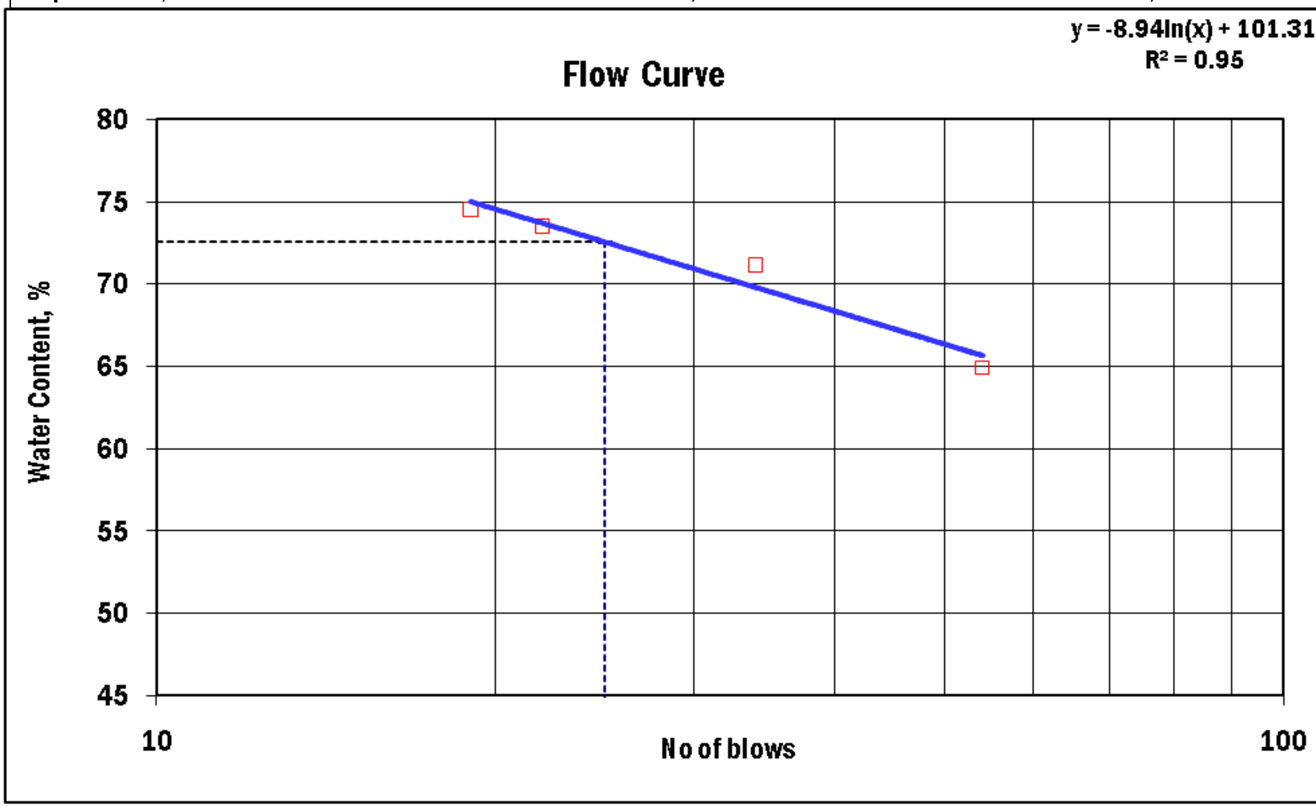
Attachment 2 = Liquid Limit and Plastic Limit Test

Project: MSc Thesis
Location: Armenia

Sample No : TP1 Sample Depth, m : 1.2

	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	D-34	C-2	24	66	95	20
Mass of container, g	15.37	13.71	15.64	15.63	15.60	15.62
Mass of container + Wet soil, g	25.50	25.04	29.01	27.21	17.20	17.59
Mass of container + Dry soil, g	21.51	20.33	23.30	22.30	16.69	16.98
Mass of water, g	3.99	4.71	5.71	4.91	0.51	0.61
Mass of dry soil, g	6.15	6.62	7.66	6.67	1.09	1.36
Water content, %	64.92	71.17	74.54	73.52	47.06	45.05
No of blows	54	34	19	22	------	------

Liquid Limit, % = 73 Plastic Limit, % = 46 PI, %= 26



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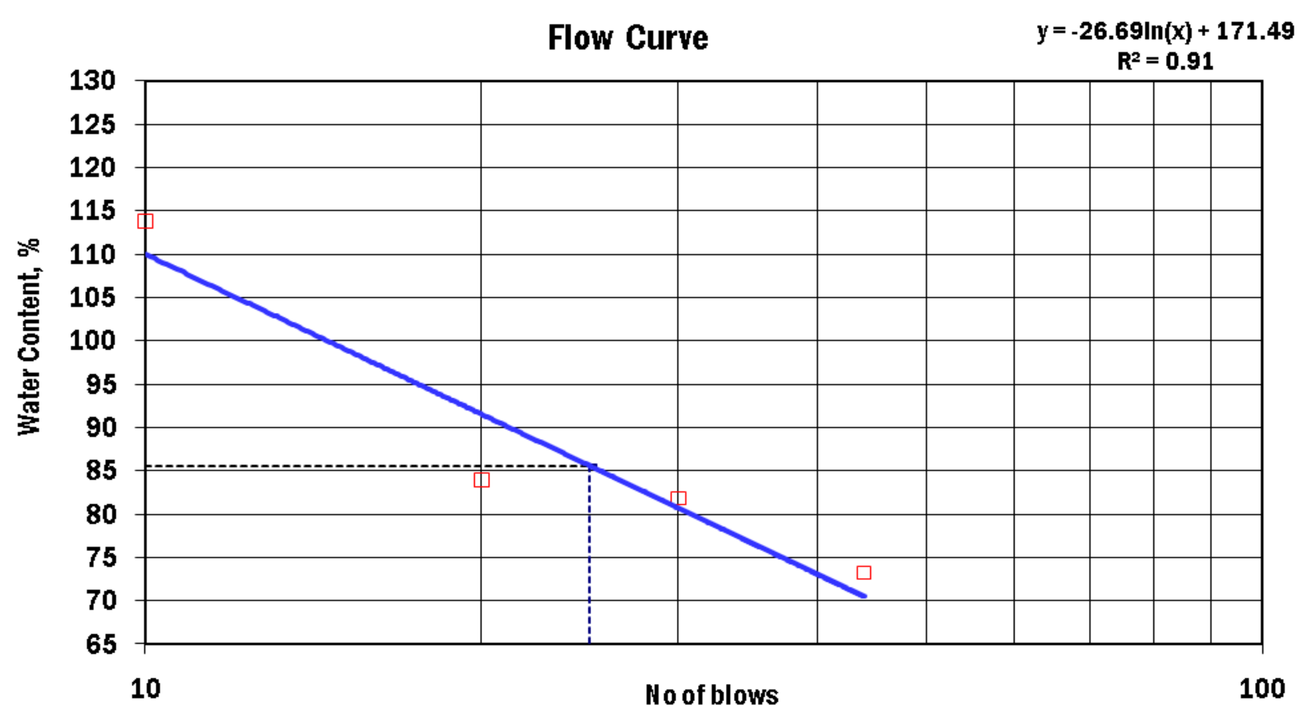
Attachment 2 = Liquid Limit and Plastic Limit Test

Project: MSc Thesis
Location: Lemmi

Sample No : TP1	Sample Depth, m : 1.1					
	Liquid Limit				Plastic Limit	
Trial No	1	2	3	4	1	2
Container No	98	69	D361	C18	A19	B1
Mass of container, g	15.78	15.66	15.56	15.55	15.61	14.17
Mass of container + Wet soil, g	23.86	24.63	25.46	24.20	18.41	18.36
Mass of container + Dry soil, g	20.45	20.59	20.94	19.59	17.44	16.92
Mass of water, g	3.41	4.03	4.52	4.60	0.97	1.43
Mass of dry soil, g	4.66	4.93	5.38	4.04	1.83	2.75
Water content, %	73.22	81.80	83.91	113.79	53.17	51.98
No of blows	44	30	20	10	------	------

Liquid Limit, % = 86 Plastic Limit, % = 53 PI, %= 33



APPENDIX - C

PHOTOS

Figure C-1: A Picture of Triaxial Test Apparatus in AAU soil Laboratory

Figure C-2: A Picture of Direct shear apparatus in AAU soil Laboratory