AN ABSTRACT OF THE THESIS OF

Randy C. Pratt for the degree of Master of Science in Civil Engineering presented on January 5, 1984.

Title: Simplified Stability Assessment for Low Volume Road Cut and Fill Slopes

Abstract approved: Redacted for Privacy

Dr. Marvin Pyles

Many miles of roads are built in the Pacific Northwest Forests to provide access for logging and fire protection. Logging roads often cross steep mountainous terrain. Logging roads and the associated cut and fill slopes represent a discontinuity in the natural stress state in the soil mantle. These conditions result in a greater incidence of slope failures associated with logging roads than with natural slopes.

Simple numeric methods were adapted to assess the stability of logging roads to aid in the evaluation of alternate road alignments during preliminary road design. The method of stability assessment consists of a numeric description of a chart solution for slope stability analysis. The chart solution addresses ground water conditions in a slope using the pore pressure ratio (r_u). An empirical determination of the relationship between r_u and ground water level within a slope was made for specific cases.

Literature relating to forest hillslope ground water conditions was reviewed and a simple ground water model was developed for

forested hillslopes with a soil mantle overlying impervious bedrock. The simple ground water model identifies general relationships between topography, precipitation events, and subsurface conditions.

Simplified Stability Assessment for Low Volume Road Cut and Fill Slopes

by

Randy C. Pratt

A THESIS

submitted to

Oregon State University

in partial fulfillment of
the requirements for the
degree of

Master of Science

Completed January 5, 1984
Commencement June 1984

APPROVED:

Redacted for Privacy

Assistant Professor of Civil Engineering in Charge of Major

Redacted for Privacy

Head of Department of Civil Engineering

Redacted for Privacy

Dean of Graduate School

Date thesis presented January 5, 1984

Typed by Cynthia Davis for Randy C. Pratt

ACKNOWLEDGEMENT

I would like to express my appreciation to the U.S. Forest Service for providing financial support for this project. I would like to thank Dr. Marvin Pyles and Dr. W.L. Schroeder for their guidance throughout the project; Cynthia Davis for typing this paper; and my family for their support.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

SIMPLIFIED STABILITY ASSESSMENT FOR LOW VOLUME ROAD CUT AND FILL SLOPES

INTRODUCTION

The U.S. Forest Service annually builds many miles of roads in the Pacific Northwest forests to provide access for logging and fire protection. These roads often cross steep mountainous terrain and are typically designed to much lower standards than state and federal highways. These roads generally experience a low volume of traffic.

Logging roads and the associated cut and fill slopes on steep terrain represent a major discontinuity in the natural state of stress in the soil mantle. Therefore, a greater incidence of slope failures is likely to be associated with roads than with natural field conditions. This observation has been verified by field studies (Swanson et al., 1981).

A landslide evaluation system has been proposed for U.S. Forest Service use (Prellwitz et al., 1983) to address the instability of hillslopes associated with timber harvest activities. This evaluation system has been separated into three levels of analysis based on the degree of complexity and required input data. These three levels are:

1. Resource Allocation - Level I
2. Project Planning - Level II
3. Critical Site Study - Level III

The purpose of the Level I analysis is to delineate areas susceptible to landslides on a broad scale. The Level II analysis is performed to identify unstable sites along proposed road alignments. The Level III analysis provides site specific road stabilization measures for unstable sites.

An important tool suitable for use in a Level II analysis is the PLANS (Preliminary Logging Analysis System) group of computer programs which presently operates on the Hewlett-Packard HP 9845B micro-computer (Twito and Mifflin, 1982). The PLANS group provides a convenient, thorough and rapid method of analyzing timber harvest alternatives based on digitized terrain data. The PLANS group includes the ROAD computer program. The ROAD program provides a rapid method of designing and evaluating alternative logging road alignments. This evaluation is based on road length, grades, and alignment. Presently, the slope stability of road alignments is not addressed in the evaluation of alternate routes.

The approach to slope stability analysis for a Level II analysis differs significantly from a detailed, site specific analyses in several ways. The available soil property data generally consist of average values for soil types on a regional basis for a Level II analysis. Topographic data used are typically at a scale where small features, which can be important to the stability of road cut and fill slopes, are not discernable. Ground water data may be non-existent, therefore requiring reliance on engineering judgement in an analysis. Due to this lack of precision of the data base, rigorous stability analyses are unwarranted and simplified approaches are generally used for Level II analyses. This level of stability analyses can be performed with the assistance of a computer program developed for this project which is operated in conjunction with the ROAD computer program.

A key factor to the value of a simplified stability analysis is the accurate prediction of design ground water conditions. Site specific ground water models have been developed by researchers but, presently, no generalized predictive ground water model exists. A simple ground water model was developed for this project.

Objective of Study

The primary objective of this study was to develop a computer code for assessing the stability of logging road cut and fill slopes. To accomplish this objective, completion of a number of tasks was required. These were:

1. Familiarization with the PLANS group of computer programs.
2. Review of the literature relating to simplified methods of slope stability analysis and subsequent adoption of the most appropriate method.
3. Review of the literature relating to predictive ground water models and the subsequent selection of an appropriate method.
4. Evaluation of the optimum manner for inclusion of the slope stability and ground water models into the PLANS group.
5. Development and testing of computer coding for the slope stability and ground water models.

CHAPTER 1. SLOPE STABILITY ANALYSIS

Introduction

Numerous methods for analyzing the stability of earth slopes have been developed in the last century. Methods of analysis can be divided into three general categories: (1) limit equilibrium, (2) limit analysis, and (3) finite element analysis. Limit equilibrium methods are the most commonly used because of the wide range of soil and pore pressure conditions that can be considered. Limit analysis methods have limited usefulness because of their inability to address varying soil and pore pressure conditions in a slope. Limit analysis methods generally require making simplifying assumptions to obtain a solution. Finite element methods require sophisticated computer programs and are warranted only on the more important and complex slope stability analyses. Simple limit analysis solutions compare favorably with more sophisticated finite element solutions not employing simplifying assumptions (Wright et al., 1973).

Factor of Safety

The end result of most stability analyses is a value for factor of safety (FS) which indicates the susceptability of a slope to failure for the analysis conditions. Several methods of expressing FS have been used in stability anlayses. Factor of safety can be expressed with respect to soil strength as:

$$FS = \frac{\text{available soil shear strength}}{\text{soil shear strength required for equilibrium}} . \quad (1)$$

Factor of safety is sometimes expressed in terms of overall moment equilibrium when circular arc failure planes are used in the stability analysis. This is expressed as:

$$FS = \frac{\text{sum of moments resisting failure}}{\text{sum of moments causing failure}} \quad . \tag{2}$$

Factor of safety can be expressed in terms of the height of the slope which is the same as the factor of safety with respect to soil cohesion (Taylor, 1948). This expression is:

$$FS = \frac{\text{actual slope height}}{\text{critical slope height (where FS=1)}} \quad . \tag{3}$$

or

$$FS = \frac{\text{available soil cohesion}}{\text{cohesion required for equilibrium with soil friction fully mobilized}} \tag{4}$$

Another way to express factor of safety is in terms of the frictional strength of the soil:

$$FS = \frac{\tan\phi}{\tan\phi_r} \tag{5}$$

where, ϕ is the angle of internal friction based on peak strength and ϕ_r is the angle of internal friction required for equilibrium with the soil cohesion fully mobilized. For the case where the FS for coehsion and the FS for frictional strength are equal, they are equal to the FS with respect to soil strength, as shown in equation (1) (Taylor, 1948).

Limit Equilibrium Analysis

Limit equilibrium analyses have the longest history of use. This method assumes that the soil strength is fully mobilized along the failure plane. This assumption implies that failure occurs simultaneously at all points on the failure plane and that soil strength is independent of shearing strain (Terzaghi and Peck, 1967). Despite this limitation, limit equilibrium analyses can provide relatively accurate and useful results for design and decision making purposes.

Numerous methods of limit equilibrium analysis have been developed. Each of these methods can be characterized by the assumptions that are made regarding the shape of the failure plane and the assumptions required to provide a determinant analysis. Table 1 summarizes the more common methods, some distinguishing features, and common references. Other limit equilibrium methods exist but have no significant advantages over the summarized methods.

Limit Analysis

The limit analysis is based on plastic limit theorems. The two main theorems for a elastic-perfectly plastic material describe the conditions when failure will not occur (lower bound) and conditions when failure will occur (upper bound) (Chen, 1969). Chen (1969) presented mathematical equations that related these theorems to slope stability and bearing capacity analyses. An extension of this work resulted in charts and graphs for the stability analyses of a slope with zero pore water pressure, varying slope and top slope angle, varying slope height, and a logrithmic spiral failure surface (Chen and Giger, 1971; Winterkorn and Fang, 1975). Use of these results are discussed in later sections.

Table 1. Summary of Common Methods for Slope Stability Analyses.

Name of Method	Type of Assumed Failure Surface	Basic Assumptions	Reference Texts
Culmann Method	Straight Line	Failure through toe of slope Failure mass analyzed as single rigid body	Taylor (1948)
Infinite Slope Method	Straight Line	The slope is constant and infinitely long Failure mass analyzed as single rigid body Seepage can be considered	Taylor (1948) Dunne et al. (1980)
Wedge Method	Straight Line Segments	Sliding blocks failure mechanism Lateral earth pressure between blocks Seepage can be considered	Terzaghi and Peck (1967) Sowers and Sowers (1970)
Ordinary Method of Slices	Circular Arc	Failure mass analyzed as several rigid bodies (slices) Forces between slices neglected Seepage can be considered	Sowers and Sowers (1970) Dunne et al. (1980)
Bishop's Simplified Method	Circular Arc	Failure mass analyzed as several rigid bodies (slices) Sum of the slice side forces = 0 Seepage can be considered	Terzaghi and Peck (1967) Winterkorn and Faug (1975)
Friction Circle Method	Circular Arc	Failure mass analyzed as single rigid body Frictional forces along failure plane act along circle concentric to radius point of failure plane Seepage can be considered	Taylor (1948) Terzaghi and Peck (1967)
Janbu Method	Irregular Surface	Failure mass analyzed as several rigid bodies (slices) Sum of the vertical sideforces = 0 Seepage can be considered	Terzaghi and Peck (1967)
Morgenstern and Price	Irregular Surface	Failure mass analyzed as several rigid bodies (slices) Indeterminant analysis requiring iterative solution Seepage can be considered	SSTAB1 User Manual (1974) Morgenstern and Price (1965)

Finite Element Analysis

This method of stability analysis is based on stress-strain relationships of an elastic soil model. The slope is generally analyzed as a continuous elastic-perfectly plastic medium using the finite element technique. Application of this methodology is complex and requires sophisticated computer programs.

CHAPTER 2. SIMPLIFIED SLOPE STABILITY ASSESSMENT

Introduction

One of the purposes of this research project was to determine a simple approach to stability assessment that could be used for road planning purposes and incorporated into a computer program. This required a review of available simplified solutions for slope stability analysis.

Simplified approaches to stability analysis are based on the observation that the factor of safety is a continuous function dependent on slope geometry, soil properties, and ground water conditions. Traditionally charts have been developed which summarize the relationships between factor of safety and simple slope geometry, soil, and ground water conditions. These charts provide a factor of safety without complex iterative processes which are typical for most methods of stability analysis.

The simplifications to the independent variables, fundamental to chart solutions, are summarized below.

1. Simple slope geometry. Slope must have a planan surface. Slopes above and below the slope being analyzed, generally, must be horizontal.
2. Soil must be homogeneous and isotropic throughout the slope (for most methods of analysis).
3. The bedrock surface must be horizontal, if bedrock is present.
4. Soil pore water pressure conditions are limited to a few particular cases.

Each of these assumptions are discussed more fully in the following sections which summarize the various simplified solutions.

Taylor Solution (Taylor, 1948)

This is one of the earliest simplified solutions to be developed. It is based on results obtained from the friction circle method of analysis. The key elements of this method are stability number (N_s) charts for soils with ϕ - c strength characteristics and for soil with the $\phi = 0$ condition, and tabulated values for the location of the critical failure circle. An important feature in this solution was the grouping of variables significant to the stability of a simple slope into a parameter called the stability number (N_s). This expression was subsequently used by others to extend the simplified solution method. The expression for N_s is:

$$N_s = \frac{c}{H\gamma} \tag{6}$$

where, N_s = dimensionless stability number

c = soil cohesion (F/L^2)

γ = soil bulk unit weight (F/L^3)

H = slope height (L)

Several limitations to the Taylor method restrict its usefulness. These limitations are that solutions are available only for cases with ϕ values between 0^o and 25^o, and for cases with either a submerged slope or zero pore water pressure. Only rough estimates of slope stability could be obtained using this method for cases with seepage forces in the slope.

Bishop and Morgenstern Solution (Bishop and Morgenstern, 1960)

This solution was based on slope stability analyses employing Bishop's Simplified Method. An important modification made in the analyses was expressing the pore water pressure (u) in terms of the pore pressure ratio (r_u). The pore pressure ratio is expressed as:

$$r_u = \frac{u}{h\gamma} \tag{7}$$

r = dimensionless pore pressure ratio

u = pore water pressure at a point (F/L^2)

γ = the unit weight of the soil above the point (F/L^3)

h = the vertical distance from the point to the ground surface (L).

The use of r_u simplified the analysis of slope stability with pore water pressure and allowed inclusion of varying pore pressure into a simplified method of analysis. However, it should be stressed that a constant r_u, a required input for this method, rarely describes real conditions. Figure 1 illustrates the unusual conditions required to obtain a constant r_u in a cut slope with seepage.

The expression for factor of safety with this method is:

$$FS = m - nr_u \tag{8}$$

where values for the dimensionless parameters m and n are obtained from charts. This expression implies that the factor of safety is linearly inversely proportional to r_u. Subsequent studies showed that the linear proportionality is valid only for 3:1 or shallower slopes (Cousins, 1978).

This method provides chart solutions for 5:1 to 2:1 slopes. Many low volume forest roads have cut and fill slopes much steeper than the

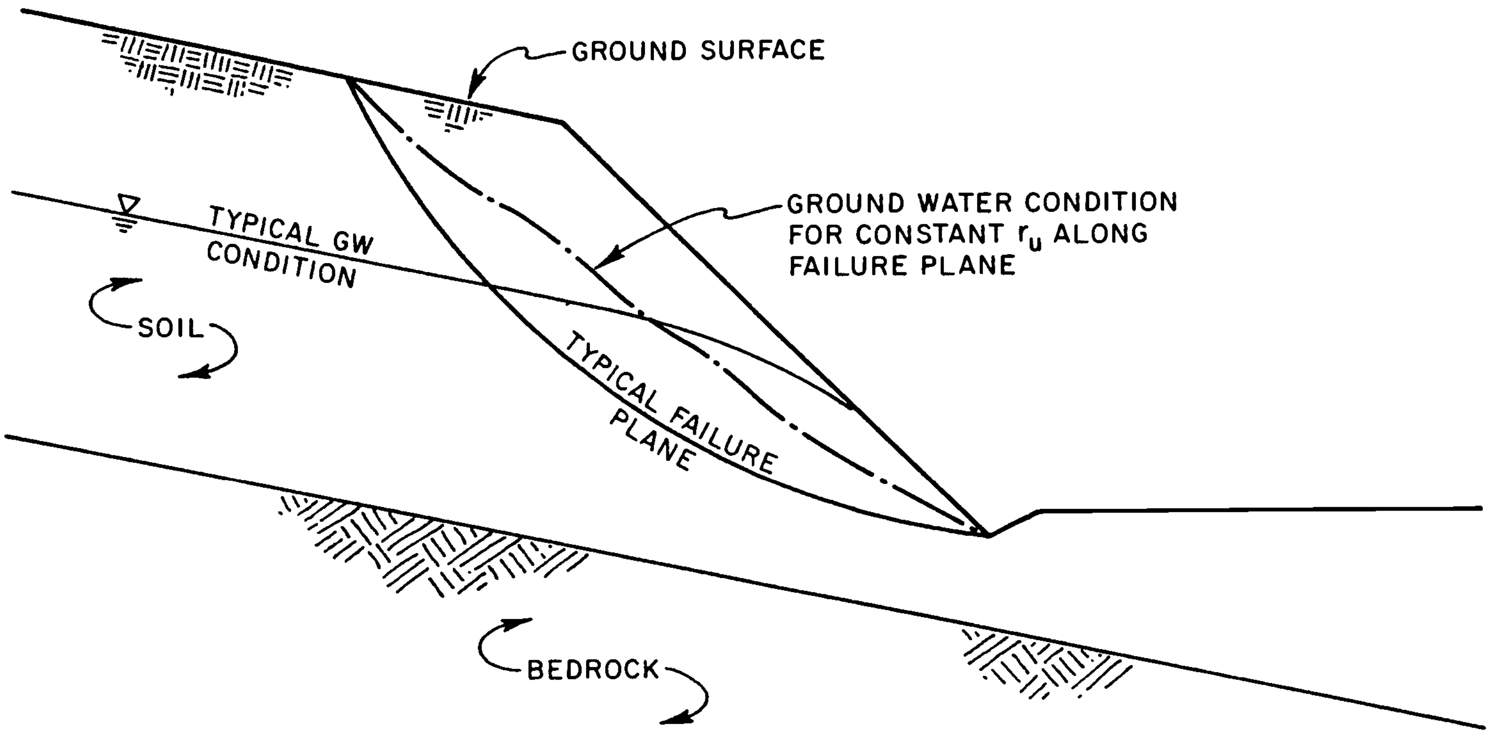


Figure 1. Comparison of Ground Water Conditions.

maximum of 2:1 for this method. Therefore, this method was considered to be unacceptable for our purposes.

Lo Solution (Lo, 1965)

This solution provides charts only for the $\phi = 0$ case. An interesting feature of this method is the ability to include an increase in undrained shearing strength with depth and anisotropic undrained shearing strength. However, this method was considered unacceptable for our purposes due to the limited failure conditions (rapid, undrained failure) addressed in this method.

Spencer Solution (Spencer, 1967)

This solution is based on slope stability analysis employing the Spencer Method of analysis. The chart solution for this method is complimentary to the chart solution for the Bishop and Morgenstern Method. Using the Bishop and Morgenstern charts, one can rapidly determine the slope height for a given factor of safety, soil strength, r_u, and slope angle. Using the Spencer charts, one can rapidly determine the slope angle for a given factor of safety, soil strength, r_u, and slope height. This method was considered unacceptable for our purposes because it does not provide solutions for slopes steeper than $1\frac{1}{2}$:1.

Bell Solution (Bell, 1966)

This solution is based on the Bishop and Morgenstern simplified method previously discussed. Bell reduced the number of dimensionless parameters required to obtain a solution and presented the results in a solution chart. This method incorporates pore pressures by using a constant pore pressure ratio (r_u). The usefulness of this method is limited by the narrow range of slopes (5:1 to 2:1) presented in the charts.

<u>Hunter and Schuster Solution (Hunter and Schuster, 1968)</u>

This solution provides charts for the undrained failure of slopes cut in saturated normally consolidated clays. This method is very similar to the Lo method except that it addresses the change in factor of safety with respect to ground water level.

<u>Chen and Giger Solution (Chen and Giger, 1971)</u>

This solution provides charts based on the upper bound theorem of limit analysis for a logrithmic spiral failure surface (Winterkorn and Fang, 1975). The logrithmic spiral failure surface is unique to this simplified solution, whereas all other simplified solutions discussed employed a circular arc failure surface. This method provides solutions for homogeneous and anisotropic soils, and for anisotropic soils with cohesion increasing linearly with depth. A significant feature of this method is the consideration of varying top slope angle. A review of the solution chart indicates, however, that changes in the top slope angle have only a small effect on the stability of a slope. Although this method has several useful features, solutions are provided only for the zero pore water pressure case. Therefore, varying ground water conditions cannot be addressed with this method.

Hoeke and Bray Solution (Hoeke and Bray, 1977)

This simplified solution is based on results obtained using the friction circle method. Chart solutions are presented for five specific ground water conditions which vary from zero pore water pressure to full saturation of the slope. Curved phreatic surfaces were derived from equations proposed by L. Casagrande (Taylor, 1948) and equipotential lines were assumed at a constant slope that varied for each case.

Results were developed for slope angles from 20° to 90° and for any soil strength condition and slope height. The wide range of solutions available from the charts make it very useful. However, this approach was not considered useful for our purposes because of the apparent difficulty in mathematically describing the complex solution charts.

Janbu Solution (Janbu, 1954)

The charts for this simplified solution were based on the Resistance Envelop Method of stability analysis. These charts provide solutions for a wide range of ground water, soil strength, and slope conditions. Significant independent factors affecting stability were grouped to obtain only a few coefficients that were then related to factor of safety and summarized in solution charts. The coefficients used in this method are as follows:

$$\lambda_c = \frac{P_e \tan\phi}{c} \tag{9}$$

where $$P_e = \frac{\gamma H + q - \gamma_w H'_w}{\mu_q \mu'_w} \tag{10}$$

and ϕ = internal angle of friction for soil (degrees)

c = cohesion of soil (F/L^2)

γ = unit weight of soil (F/L^3)

H = slope height (L)

γ_w = unit weight of water (F/L^3)

q = surcharge load above slope (F/L^3)

H_w' = depth of ground water above toe of slope (L)

μ_q = coefficient for surcharge obtained from charts

μ_w' = coefficient for ground water obtained from charts

Using the results from equations (9) and (10) and Janbu's solution chart, a value for another coefficient, N_{cf}, can be obtained. The factor of safety (FS) for slope stability can then be obtained from the following:

$$FS = N_{cf} \frac{c}{P_d} \tag{11}$$

where

$$P_d = \frac{\gamma H + q - \gamma_w H_w}{\mu_q \mu_w \mu_t} \tag{12}$$

and H_w = depth of submergence above toe of slope (L)

μ_w = coefficient for submergence

μ_t = coefficient for tension cracks at top of slope

For the usual conditions found along low volume logging access roads, $H_w = 0$, $q = 0$, and tension cracks at the top of cut and fill slopes can be considered negligible. For these typical field conditions μ_q, μ_t, and μ_w become unity. By grouping equations (9) and (10) and eliminating coefficients equal to unity, the result is:

$$\lambda_{c\phi} = \frac{(\gamma H - \gamma_w H_w')\tan\phi}{\mu_w' c} \tag{13}$$

Substituting equation (12) into (11) and using the above conditions results in:

$$FS = N_{cf}\,\frac{c}{H\gamma} \tag{14}$$

The solution chart and ground water coefficient chart are shown in Figure 2. Results obtained using equations (13) and (4), and the charts in Figure 2 were compared with solutions using the Morgenstern and Price Method employed in the SSTAB1 computer program (Wright, 1974). The results, summarized in Table 1A (Appendix A) indicate that the Janbu Method provides factors of safety consistently higher (up to 50 percent higher) than the results from the more accurate Morgenstern and Price Method. The Janbu Method was not used for this project due to the apparent inaccuracy of results.

Janbu Solution (Janbu, 1967)

Janbu modified the previously described solution (1954) to incorporate the pore pressure ratio, r_u. With this change, equation (13) becomes

$$\lambda_{c\phi} = \frac{(1-r_u)\gamma H \tan\phi}{c} \tag{15}$$

Equation (14) and the solution chart in Figure 2 remain unchanged. Subsequent investigations indicated that this method provided accurate results for slopes with inclinations less than 20 degrees (Cousins, 1978).

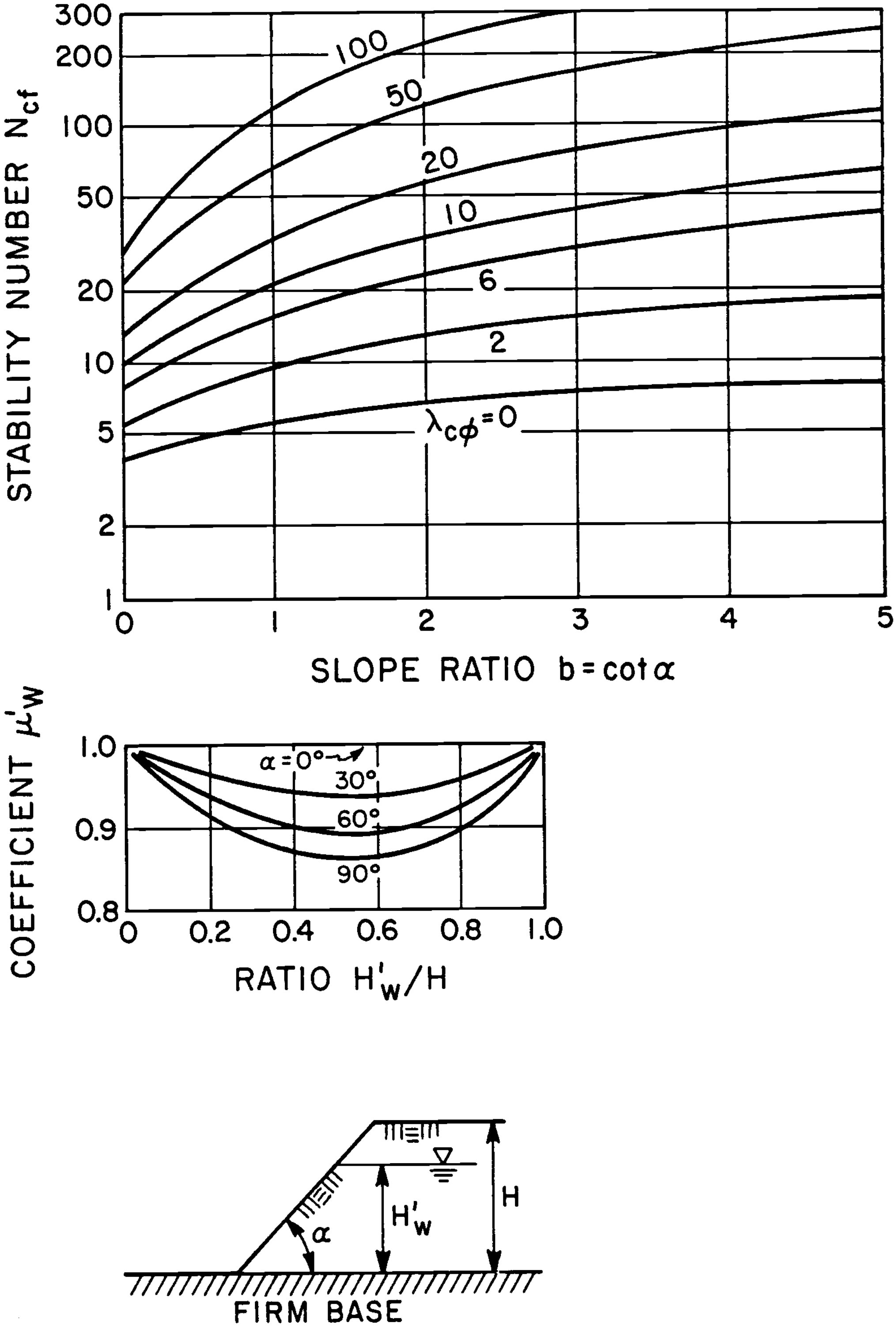


Figure 2. Janbu Solution Charts.

Cousins' Solution (Cousins, 1978)

This solution is based on results obtained using the Friction Circle Method. Solution charts are provided for circular arc failure surfaces that pass through the toe of the slope, or are tangent to a horizontal line at a specified distance below the toe of the slope. Pore pressure variations are addressed by providing solution charts for three values of r_u:0, 0.25, and 0.50. Also provided are solution charts for the radius points of the critical circular arcs for failures through the toe of the slope and for failures tangent to a horizontal line passing through the toe of the slope (Cousins, 1978). Three of Cousins' solution charts are presented in Chapter 4.

Cousins' charts can be used by determining $\lambda_{c\phi}$,

where,

$$\lambda_{c\phi} = \frac{\gamma H \tan\phi}{c}, \tag{16}$$

entering the appropriate chart for a given r_u and circular arc location, and determining the constant N_f knowing $\lambda_{c\phi}$ and the slope angle (α). The factor of safety (FS) for soil strength can be determined from:

$$FS = \frac{N_f c}{H\gamma} \tag{17}$$

All constants are as previously defined. Noting the similarity to the Janbu Methods (1954, 1967), this method was checked for accuracy by comparing Cousins' solutions with solutions obtained with Morgenstern and Price Method employed in the SSTAB1 computer program (Wright, 1974). Results, summarized in Table 1B (Appendix B), indicate the

Cousins Method provides accurate results. The Cousins Solution was considered the most appropriate method for our purposes based on several factors. These were:

1. Solutions cover a wide range of slopes and combinations of slope height and soil strength.
2. Solution accuracy was very good.
3. Solution charts appeared ameable to mathematical description.

CHAPTER 3. FACTORS AFFECTING SLOPE STABILITY

Introduction

The stability of a slope is dependent on the complex interaction between stress states, resulting from slope geometry, ground water conditions, and soil strength characteristics. A review of usual slope geometry and soil strength conditions was made for use in the development of a slope stability model.

Soil Strength

Soil strength characteristics are typically described by a linear relationship known as the Coulomb equation:

$$s = c' + p' \tan\phi' \tag{18}$$

where s = shearing strength of the soil (F/L^2)

c' = soil cohesion (F/L^2)

p' = effective stress applied to the soil normal to the plane of shearing (F/L^2)

ϕ' = angle of internal friction of the soil (degrees)

Results from soil strength tests on forest soils in the Pacific Northwest, in Table 2, indicate that the cohesion of forest soils can vary considerably, whereas the angle of internal friction is generally between 30 and 40 degrees for the sites studied. Other strength considerations not reflected in the c and Φ terms from the tests summarized in Table 2 are increased strength due to "apparent cohesion" and root shearing strength.

Apparent cohesion is a term used to describe the frictional strength of a soil created by capillary tension (negative pore water

Table 2. Summary of Average Soil Strength

Undisturbed Samples		Disturbed Samples		Sample Location
c(psf)	ϕ(degrees)	c(psf)	ϕ(degrees)	
175	36	240	36[1]	S.E. Alaska[2]
380	32	65	33[1]	Olympic Peninsula, Washington[3]
70	38	180	38[1]	Oregon Coast Range[4]
--	--	0	40[5]	Oregon Coast Range[6]

[1] Samples compacted to 90 percent relative compaction according to ASTM D-698.

[2] Results are average of 10 multistage triaxial shear tests (Filz, 1982).

[3] Results for undisturbed samples and compacted samples are average of 7 multistage triaxial shear tests (Alto, 1982).

[4] Results from undisturbed samples and compacted samples are average of 11 multistage triaxial shear tests (Alto, 1982).

[5] Sample compacted to insitu density.

[6] Results are average of 10 triaxial shear tests (Yee, 1975).

pressure). Tensions can reach as high as 2 feet of water during the wet season (Yee, 1975) and undoubtably exceed this value during the dry summer months. Assuming uniform capillary tension and $\phi = 37^o$, the added strength, or apparent cohesion, due to 2 feet of capillary tension would be 95 psf. This amount of added soil strength can greatly increase the stability of a slope. However, capillary tensions may approach zero throughout much of a soil mass during critical wet season periods. Therefore soil strength due to capillary tension is rarely included in slope stability analyses.

Root shearing strength can have an important impact on the stability of forest slopes. Values on the order of 100 psf for cohesion due to root strength in a cedar, hemlock and spruce forest in Southeast Alaska have been reported (Wu and McKinnell, 1979). Roots can have an effect on the stability of a slope when the roots penetrate a potential failure surface. This interaction consists of either roots present in the failure surface or a change in the shape of the failure surface in response to variations in the soil-root strength. Generally, roots may impact the stability of a slope with shallow soils to a greater degree than in deep soil deposits.

Slope Geometry

The range in cut and fill slope inclination is generally between 2:1 and 3/4:1 for road sections in soil deposits and that no road fills are constructed on natural slopes greater than 60 percent (31^o). Slope height is dependent on road width, natural slope angle, and cut or fill slope angle. Road widths are generally about 20 feet.

Generally, the stability of a slope decreases with increased slope height and slope inclination.

Ground Water

Ground water conditions in natural forest slopes are characterized by predominantly unsaturated flow, even during storm events. Field studies discussed more fully in later sections indicate that saturated flow conditions occur predominantly in hillslope depressions or at other points where bedrock conditions concentrate the flow of ground water. Saturated zones of ground water typically develop above contacts between a pervious soil overlying a relatively impervious material such as a clay layer in the soil mantle above or at the soil-bedrock interface.

The stability of a slope is greatly affected by ground water conditions. This results from a decrease in frictional soil strength when pore water pressures increase and the effective soil stresses decrease. Pore water pressures increase with an increase in the degree of saturation and with a rise in the ground water level.

No studies were encountered during the literature review for this project where field observations were made of ground water conditions in typical logging road cut and fill slopes. Cut slopes are believed to experience ground water conditions similar to the natural soil mantle uphill of the cut.

An analysis of ground water conditions in a fill slope was made. This analysis was at a qualitative level due to the complexity of actual ground water conditions in a road cut and fill section.

Sources of moisture for a properly designed fill slope included:

1. Upslope ground water not intercepted by the cut slope and drainage ditch.
2. Precipitation infiltrating directly into the fill.
3. Seepage from the drainage ditch.

This assumes that ground water flow from the bedrock is negligible and that the drainge ditch is properly sized so that water is not diverted onto the road surface from an overflowing drainage ditch during storm events. Precipitation infiltrating directly into the fill provides only small quantities of water to the fill due to the small surface area of the road and fill slope. Assuming that the natural soil beneath the fill slope was not significantly altered by road construction, it appears that the fill slope is, in general, adequately drained. Zero pore pressure conditions were assumed for the fill slope stability analysis in the slope stability model based on the above considerations.

Prellwitz (1983) developed a ground water model based, conceptually, on the decrease in hydraulic conductivity in the natural soil beneath a fill slope due to consolidation of the natural soil resulting from the weight of the fill. In his model, the ground water level rises into the fill slope due to the decreased hydraulic conductivity of the soil beneath the fill forming a ground water mound. This mound is a function of depth of ground water up slope of the road relative to the depth of soil, and the height of the fill relative to the depth of the natural soil.

This concept of decreased hydraulic conductivity beneath a fill slope due to the weight of the fill was investigated. An analysis of the change in void ratio in a natural soil due to the weight of the fill was made using laboratory consolidation test results from forest soil samples taken in the Oregon Coast Range. The consolidation test results indicated a preconsolidation pressure of about 3,000 psf, an initial void ratio of about 1.65, and a compression index of about 0.05 in the overconsolidated zone (Schoenemann, 1983). The change in void ratio (e) is 0.05 and 0.03 for soil one foot and three feet beneath the natural ground surface, respectively, using the above test results and assuming a uniform fill depth of 8 feet, and soil unit weight of 100 pcf.

The change in saturated hydraulic conductivity (k) can be determined using the following proportionality (Terzaghi and Peck, 1969):

$$k \propto e^2 \tag{19}$$

Based on equation (19), the hydraulic conductivity decreases by about 6 percent and 3.5 percent at one foot and three feet beneath the natural ground surface, respectively. These decreases in hydraulic conductivity are not considered great enough to result in the formation of a significant ground water mound within the fill slope.

CHAPTER 4. SLOPE STABILITY MODEL

Introduction

Before a method of stability analysis was developed for the HP 9845B computer, the following constraints were identified which limited certain characteristics of the computer code:

1. The model had to be capable of operating in BASIC computer language on the HP 9845B computer.
2. The model needed to operate rapidly.
3. Results provided by the model had to be reasonably accurate compared with more sophisticated methods of stability analysis.
4. The model could be integrated into the PLANS computer program group.

Based on these constraints, the ideal stability model would have been a simple mathematical expression which provided results as accurate as more sophisticated methods of analysis. This was the goal in the development of this model.

Several tasks were required to transform Cousins' simplified solutions for slope stability into the desired form:

1. Determine if correction factors were required to address discrepencies between the model and field conditions.
2. Extend Cousins' solution charts to include 3/4:1 slopes and $\lambda_{c\phi}$ (see equation (16)) to values greater than 50.
3. Describe mathematically Cousins' solution charts.
4. Correlate ground water conditions with pore pressure ratio (r_u).

Fill Slope Modeling Errors

Cousins' method provides a factor of safety for slope stability for homogeneous, isotropic soil with a uniform pore pressure ratio, and for horizontal top and toe slopes and horizontal bedrock through the toe of the slope. Typical conditions likely to be encountered for a proposed road alignment (Figure 3) deviate from the conditions that are used for the basis of the charts. Actual conditions not addressed in Cousins' method that affect the slope stability of the fill are inclined toe slope and non-circular failure through weak soil layers such as might occur at the fill soil-natural soil interface.

Stability analyses for varying soil strengths and pore pressure ratios were made for a 1½:1 fill slope on a 3:1 natural slope and for a 1:1 fill slope on a 2:1 natural slope using the SSTAB1 computer program. The results from these analyses indicated that the location of the critical failure circles for a fill slope on an inclined ground surface were deeper than the critical failure circles predicted by Cousins' method. Associated with the deeper failure circles was a decrease in slope stability. Table 3 and Figure 4 summarize the results of the stability analyses using SSTAB1 and Cousins' method.

A significant trend indicated by the results was that the change in location of the critical failure circle and the associated decrease in stability diminishes with increasing values of $\lambda_{c\phi}$. Increasing $\lambda_{c\phi}$ values represent a decrease in soil cohesion for a given slope and value. This means that the greatest discrepencies in factor of safety between Cousins' factor of safety and the actual fill slope factor of safety occurred at a factor of safety much greater than 1.0 for the

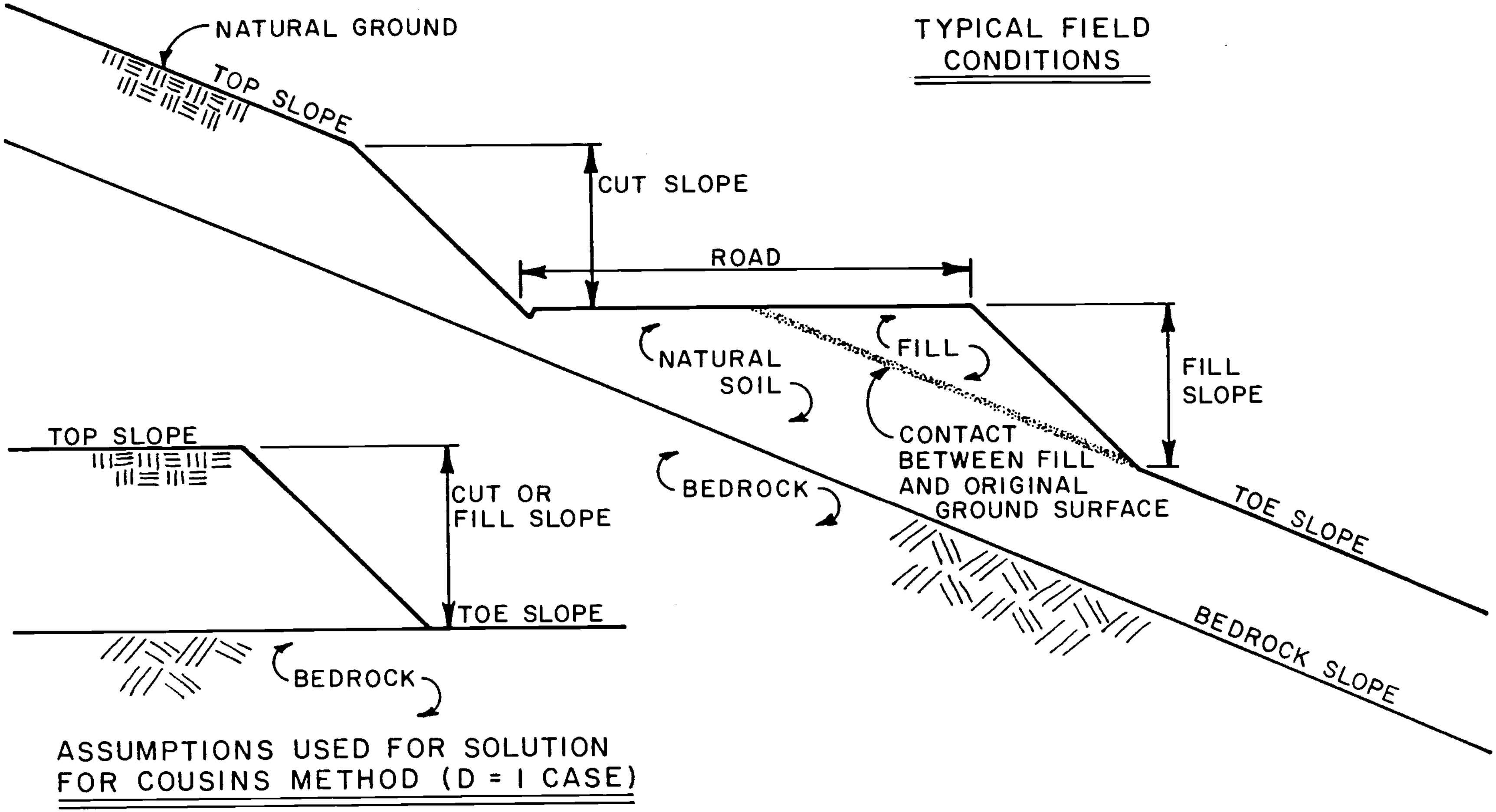


Figure 3. Typical Field Conditions.

Table 3. Summary of Fill Slope Stability Analyses

Case	Natural Slope Angle (degrees)	Fill Slope Angle (degrees)	Slope Height (feet)	$\lambda_{c\phi}$(1)	c (psf)	ϕ (degrees)	r_u	SSTAB1 FS(2)	Cousins FS(3)
1F	18.4	33.7	6.6	0	100	0	0.25	0.97	1.05
2F	18.4	33.7	6.6	2	233	35	0	4.10	4.16
3F	18.4	33.7	6.6	6	77.8	35	0	2.40	2.42
4F	18.4	33.7	6.6	10	46.7	35	0	1.96	2.00
5F	18.4	33.7	6.6	20	23.3	35	0	1.61	1.64
6F	18.4	33.7	6.6	2	23.3	35	0.25	3.61	3.60
7F	18.4	33.7	6.6	6	77.8	35	0.25	1.92	1.92
8F	26.6	45	10	0	100	35	0.25	0.48	0.60
9F	26.6	45	10	2	350	35	0	3.18	3.34
10F	26.6	45	10	4	117	35	0	2.33	2.25
11F	26.6	45	10	6	70	35	0	1.90	1.85
12F	26.6	45	10	10	35	35	0	1.54	1.51
13F	26.6	45	10	2	350	35	0.25	2.73	2.97
14F	26.6	45	10	4	117	35	0.25	1.91	1.88
15F	26.6	45	10	6	70	35	0.25	1.51	1.41

(1)$\lambda_{c\phi}$ defined in equation (16)

(2)Factors of safety (FS) obtained using SSTAB1 computer program.

(3)Factors of safety (FS) obtained from Cousins solution charts.

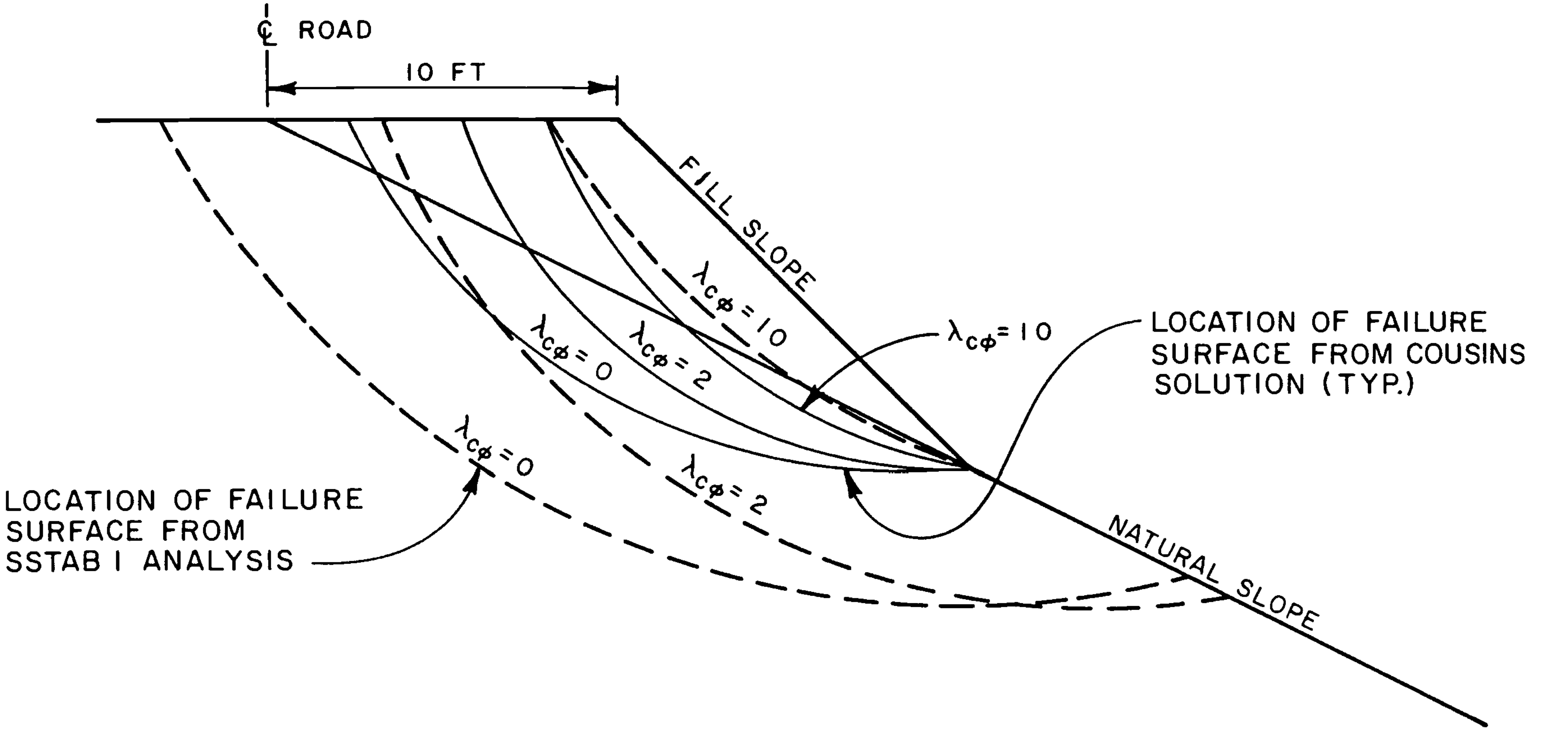


Figure 4. Fill Slope Analysis.

cases considered. Based on these results, no modifications to Cousins' method were made to address actual fill slope conditions.

A condition typically present in fill slopes placed on natural ground is an organic zone at the interface between the fill and natural slope as illustrated in Figure 3. This organic zone results from decomposition of vegetation and debris buried during the filling process. Depending on the inclination of the organic zone, thickness, and shearing strength properties, an analysis of a planar failure along the organic zone may indicate greater instability in the fill than would Cousins' method for circular failure planes. Therefore, the planar failure was incorporated into the stability model as an optional analysis. The development of the equations for this planar failure analysis are summarized in Appendix C.

The factor of safety for a planar failure can be expressed in the following form:

$$FS = \frac{2c \sin \alpha}{\gamma H \sin(\alpha-\beta)\sin\beta} + \frac{\tan \phi}{\tan\beta} \tag{20}$$

where
- c = soil cohesion at the fill-natural soil interface (F/L^2)
- ϕ = soil angle of internal friction at the fill-natural soil interface (degrees)
- γ = unit weight of fill-soil (F/L^3)
- H = vertical height of fill slope (L)
- α = inclination of fill slope (degrees)
- β = inclination of natural ground (degrees)

Cut Slope Modeling Errors

Cousins' method does not address the change in stability due to an inclined top slope and bedrock as illustrated in Figure 3. Stability analyses were made for varying soil conditions, cut slopes, bedrock slopes, and top slopes, using SSTAB1, to determine the change in slope stability for these conditions.

The results from this analysis indicated that the factor of safety for cut slope stability using Cousins' method could deviate from the correct factor of safety by as much as 25 percent for the undrained failure case ($\phi = 0$) and as much as 16 percent for the drained failure case and high factors of safety. The difference in factor of safety between values derived from Cousins' method and SSTAB1 are summarized in Figure 5 and Tables C1 and C2 (Appendix C). The undrained failure case was considered to not reflect actual field conditions. Therefore, the error associated with this condition was of little concern to our purposes.

A review of the results indicates that an inclined top slope tends to decrease stability and an inclined bedrock tends to increase stability. The inclined top slope decreases stability due to the added weight of the soil on the uphill side of the failure mass as shown in Figure 6. An inclined bedrock surface passing through the toe of the slope increases stability by limiting the location of the critical circular arc failure surface. As can be seen by the locations of the critical failure surfaces in Figure 6, the stability of the cut slope is impacted less by top slope and bedrock inclination at higher values of $\lambda_{c\phi}$.

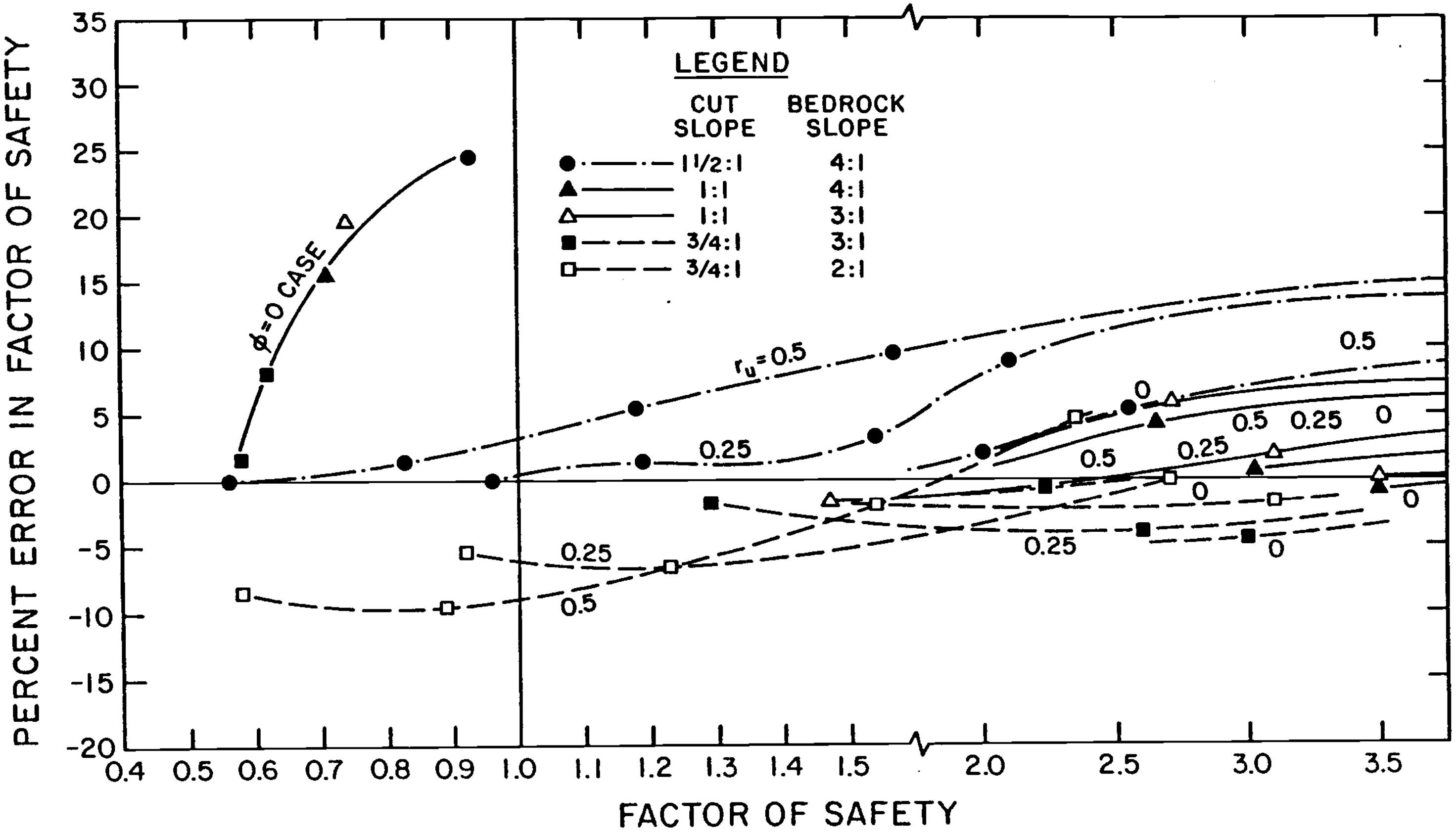


Figure 5. Summary of Results for Cut Slope Analysis.

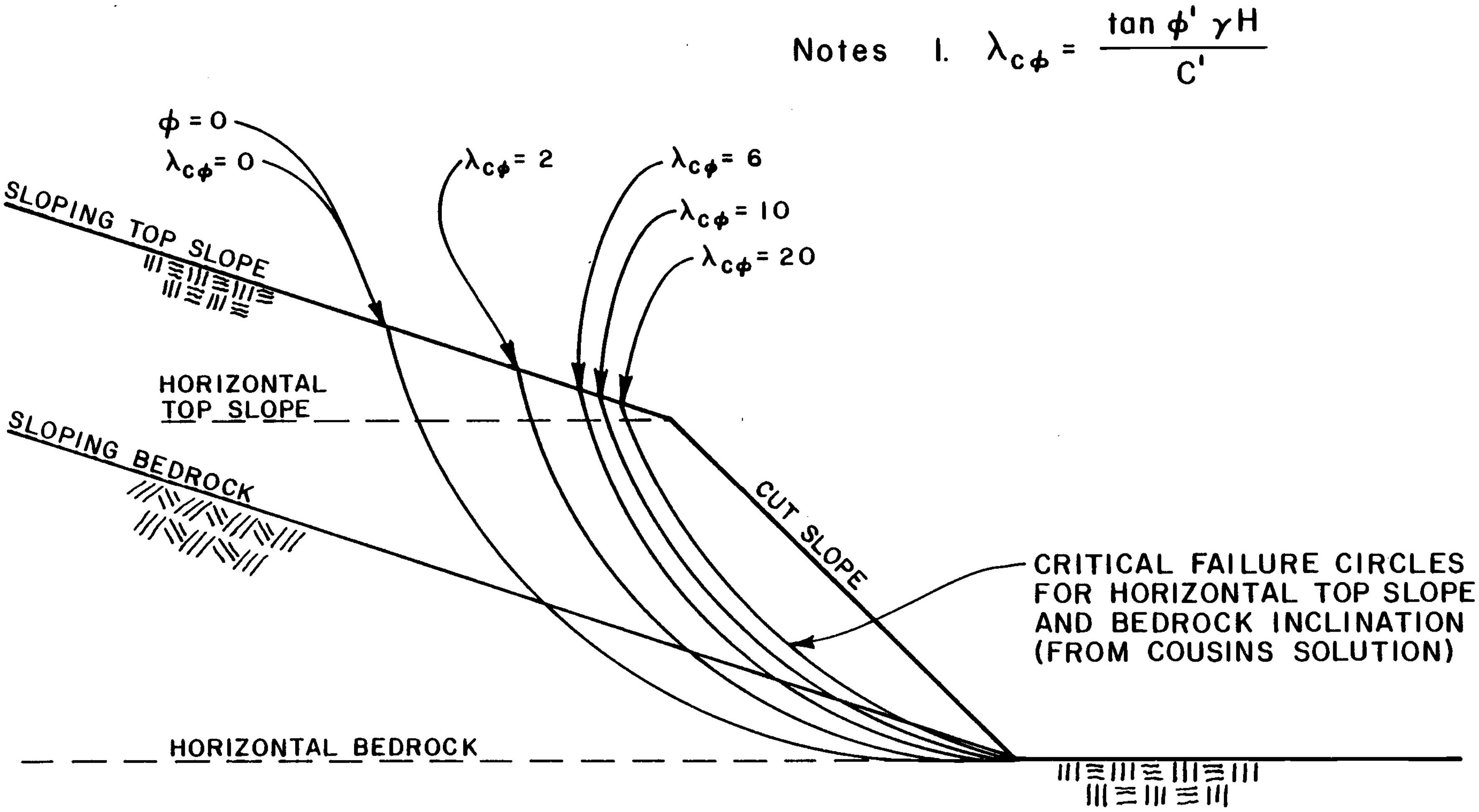


Figure 6. Cut Slope Analysis.

Change in the pore pressure ratio (r_u) also has an impact on the effects of inclined top slope and bedrock. Increasing values of r_u shift the critical circle deeper into the slope for a given slope. This results in a greater difference between actual and Cousins' factors of safety with increasing r_u values as can be seen in Figure 5.

As the cut slope and top slope angles increase, the increased factor of safety due to the inclined bedrock diminishes and the effect of an inclined topslope - decreased stability - becomes more pronounced. The actual factor of safety becomes less than values given by Cousins' method for this case as can be seen in Figure 5.

As shown in Figure 7, most of the error occurs at factors of safety much greater than 1.0. For the 3/4:1 cut slope cases, the error resulting by using Cousins' method is conservative. Based on this analysis, no modification was made to Cousins' method to address the effects of inclined topslope and bedrock.

Extending Cousins' Solution Charts

Cousins' solution charts, as presented in the literature, were developed for slope angles of from 5 to 45 degrees and for $\lambda_{c\phi}$ values from 0 to 50. Cut and fill slopes used for logging roads were believed to vary from between about 2:1 (26^o) and 3/4:1 (53^o). Values for $\lambda_{c\phi}$ were believed to vary from about 0.5 to over 100, depending on slope and soil conditions.

The solution charts were extended to include expected field conditions. This was done by determining the factor of safety for specific cases using SSTAB1, back calculating N_f, and plotting the results on the appropriate solution chart. See Tables 3C (Appendix C) for a summary of the results. Curves were then drawn between known points. The charts were extended for large values of $\lambda_{c\phi}$ for solutions with factors of safety near 1.0 and slope inclination between 35 and 55 degrees.

Manipulation of equations (16) and (17) results in the following relationship:

$$FS = \frac{N_f \tan\phi}{\lambda_{c\phi}} \qquad (21)$$

Plotting this relationship on the solution charts for values of ϕ from 30 to 40 degrees and for FS = 1 indicated the areas on the charts of greatest importance. See Figures 7, 8 and 9 for plots.

<u>Mathematical Description of Cousins' Solution Charts</u>

The solution charts presented in Figure 7, 8 and 9 required mathematical description for incorporation of this method into a computer code. Two methods to describe the three charts were considered: (1) developing an array of values to describe the charts, or (2) developing a group of equations which describe the charts. The latter method was chosen because it would provide the simplest form of description.

Relationships for slope angles of 33.7^{o}, 45.0^{o}, and 53.1^{o} with pore pressure ratios of 0, 0.25, and 0.50 were developed which resulted in nine equations. The form of the equations describing the relationship between N_f and $\lambda_{c\phi}$ was based on the observation that a plot of N_f versus $\lambda_{c\phi}$ on loglog paper was a parabola. Subsequent description of this parabola resulted in the following general relationship:

$$\log_{10}(N_f) = A(\log_{10}\lambda_{c\phi})^B + C \tag{22}$$

where A, B, and C are constants which are dependent on r_u and slope angle (α). Values of these constants are summarized in Table 4C (Appendix C). The A constant defines the general slope of the parabolic curve. The B constant defines the curvature of the parabola, and the C constant defines the value of the logrithm (base 10) of the N_f at $\lambda_{c\phi} = 1.0$. Note that equation (22) is not valid for values of $\lambda_{c\phi}$ less than one. Empirical relationships for the constants as functions of r_u and slope angle (α) were developed. They are as follows:

$$A = (0.31813 - 0.33057\ r_u)^{0.65} - 0.0046(\alpha - 33.69) \tag{23}$$

$$B = (1.206 + 0.129\ r_u)^2 \tag{24}$$

$$C = 1 - 0.2\ r_u - (0.0506/\alpha^{0.5323})(\alpha - 33.69) \tag{25}$$

A graphical comparison between the solution charts and the results from equations (22) through (25) are shown in Figures 7, 8 and 9.

Ground Water Level to Pore Pressure Ratio Correlation

Cousins' solution charts addressed pore water pressure conditions as a function of pore pressure ratio (r_u). As discussed previously, constant values of r_u rarely describe field conditions. One case where a constant r_u accurately describes pore water pressure conditions along a failure plane is for an infinite slope with seepage parallel to the ground surface. The relationship for r_u can easily be

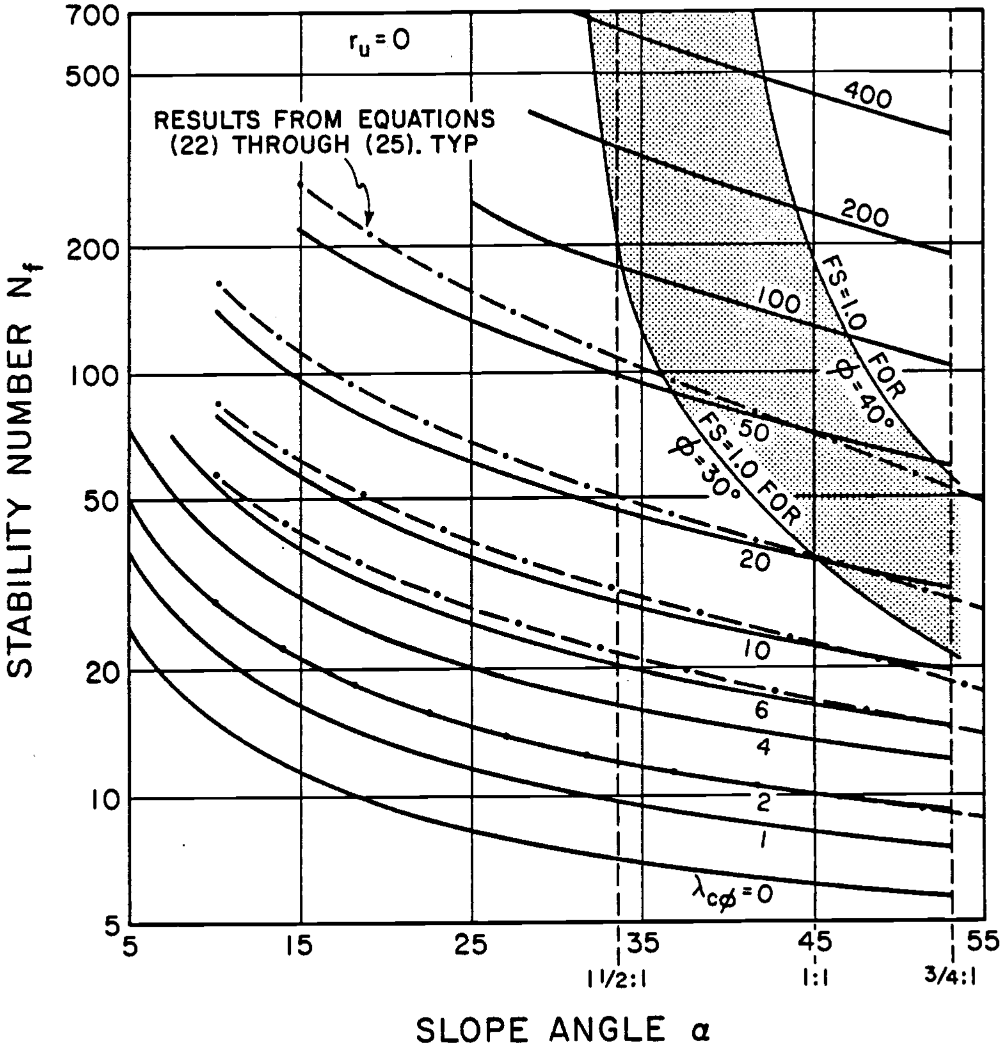


Figure 7. Cousins' Solution Charts (r_u = 0).

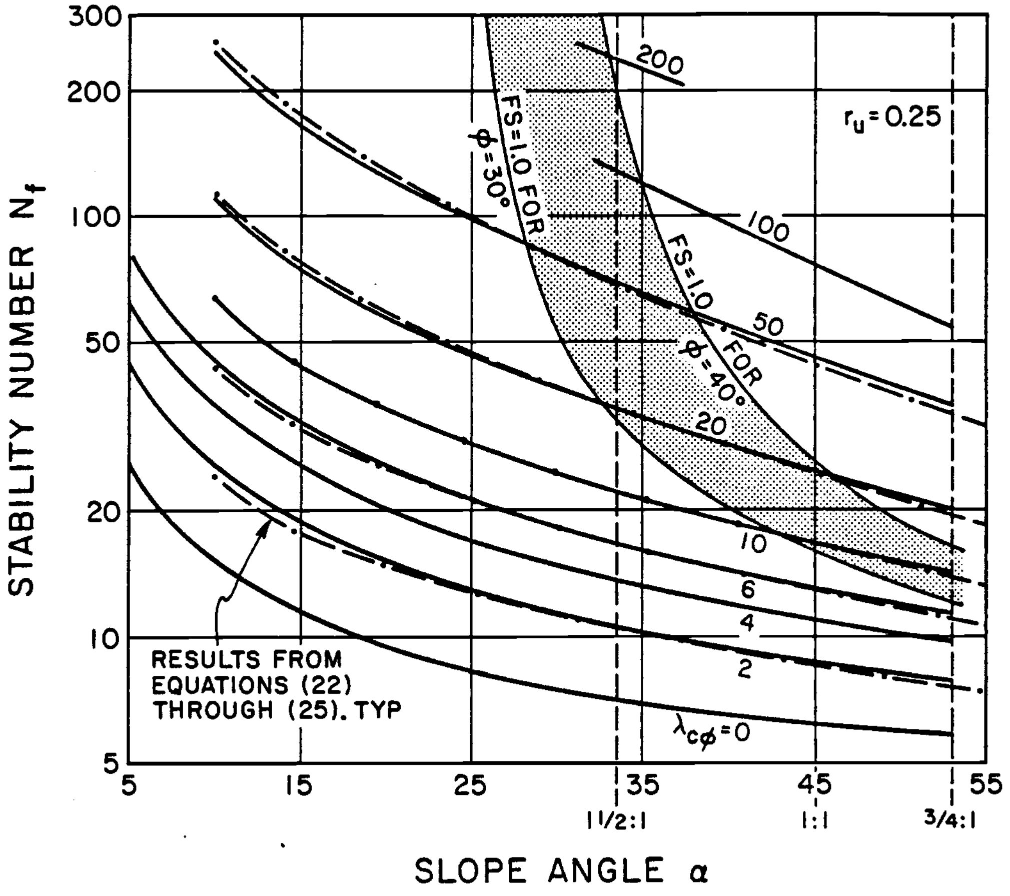


Figure 8. Cousins' Solution Charts (r_u = 0.25).

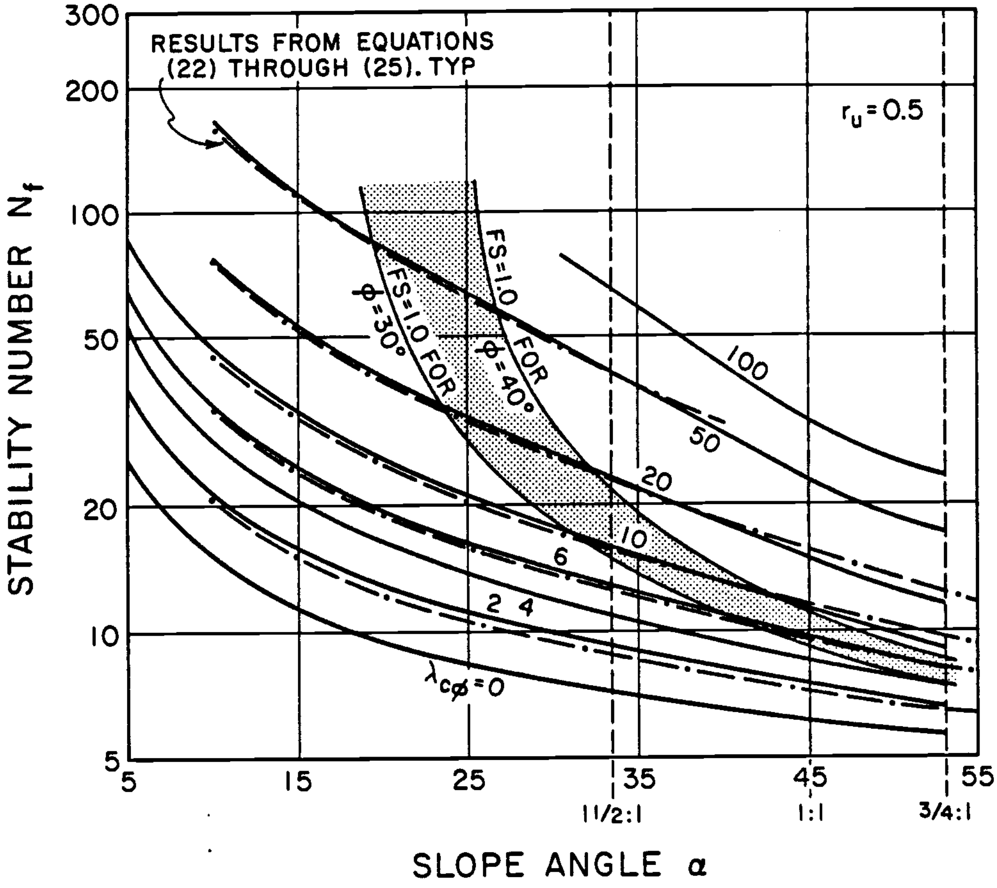


Figure 9. Cousins' Solution Charts (r_u = 0.50).

derived as follows. Figure 10 summarizes the conditions. Pore pressure ratio (r_u) can be described as:

$$r_u = \frac{u}{H\gamma} \tag{26}$$

with the variables defined in equation (7). Pore water pressure (u) can be expressed as:

$$u = \gamma_w H'_w \cos^2\alpha \tag{27}$$

with H'_w and α as shown in Figure 10. Substituting into equation (26) results in:

$$r_u = \frac{\gamma_w H'_w}{H_s} \cos^2\alpha \tag{28}$$

Janbu (1954) presented a method for including ground water level (H'_w) into his simplified method of analysis for circular arc failure surfaces. Janbu (1967) modified his earlier method to address ground water conditions using the pore pressure ratio (r_u). Equating relationships from the two methods (equations (13) and (15)) results in the following relationship:

$$r_u = 1 - \frac{1}{\mu'_w} + \frac{\gamma w(H'_w/H)}{\mu'_w} \tag{29}$$

where the variables are as previously defined in equation (10).

The ground water correction factor (μ'_w) was expressed mathematically to allow incorporation of Janbu's relationship (equation (29)) into a computer program code. The relationship is:

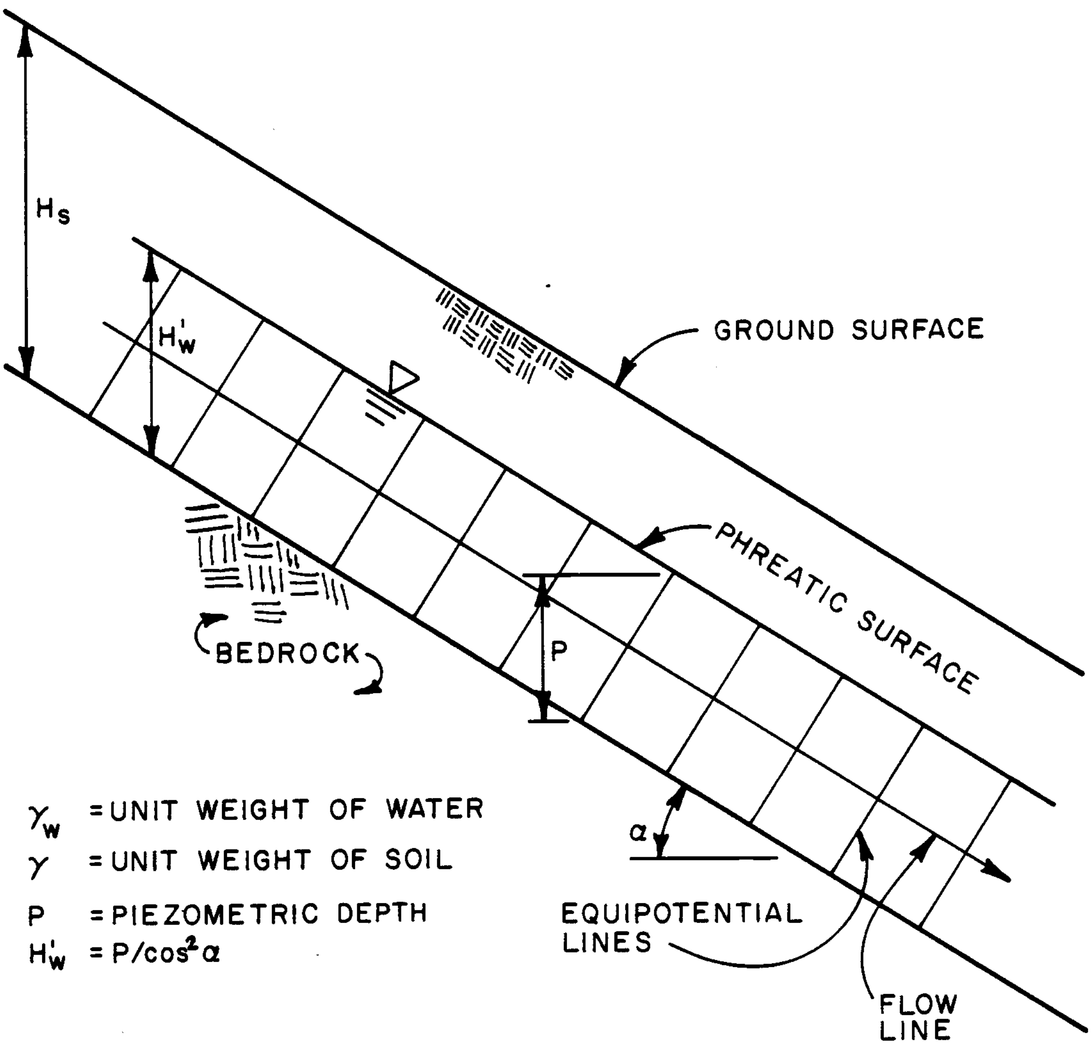


Figure 10. Infinite Slope Ground Water Conditions.

$$\mu_W' = 1 + (.1 + \frac{(\alpha-45)}{1125}) \sin (180 + (1 + \frac{H_W'}{H})) \tag{30}$$

Equations (29) and (30) were used in conjunction with Cousins' solution (equations (22) through (25)) to determine the factor of safety for stability for several cases. These results were compared to results obtained with the SSTAB1 program. Comparisons were generally poor as shown in Table 2C (Appendix C). The error was believed to have been from several sources. These were:

1. Janbu's equation (29), which relates ground water levels to pore pressure ratios, was intended only as an approximation.
2. The error difference in FS associated with inclined bedrock and top slope as previously discussed.
3. Janbu's relationship (equation (29)) was based on the piezometric line and phreatic surface being coincident and horizontal rather than the presumed field conditions, shown in Figure 11, used for the analysis.

Janbu's expression for r_u was considered useful because it included the important parameters in the correct form. An empirical relationship was developed to be incorporated into Janbu's expression for r_u which provided more accurate values for factor of safety. This was accomplished by back calculating the piezometric levels which would result in the same factor of safety as provided by the SSTAB1 program for many cases. An expression was then developed which described the relationship between the piezometric levels used in analyses using the SSTAB1 program and the piezometric levels required to provide the same factors of safety using Cousins' method and

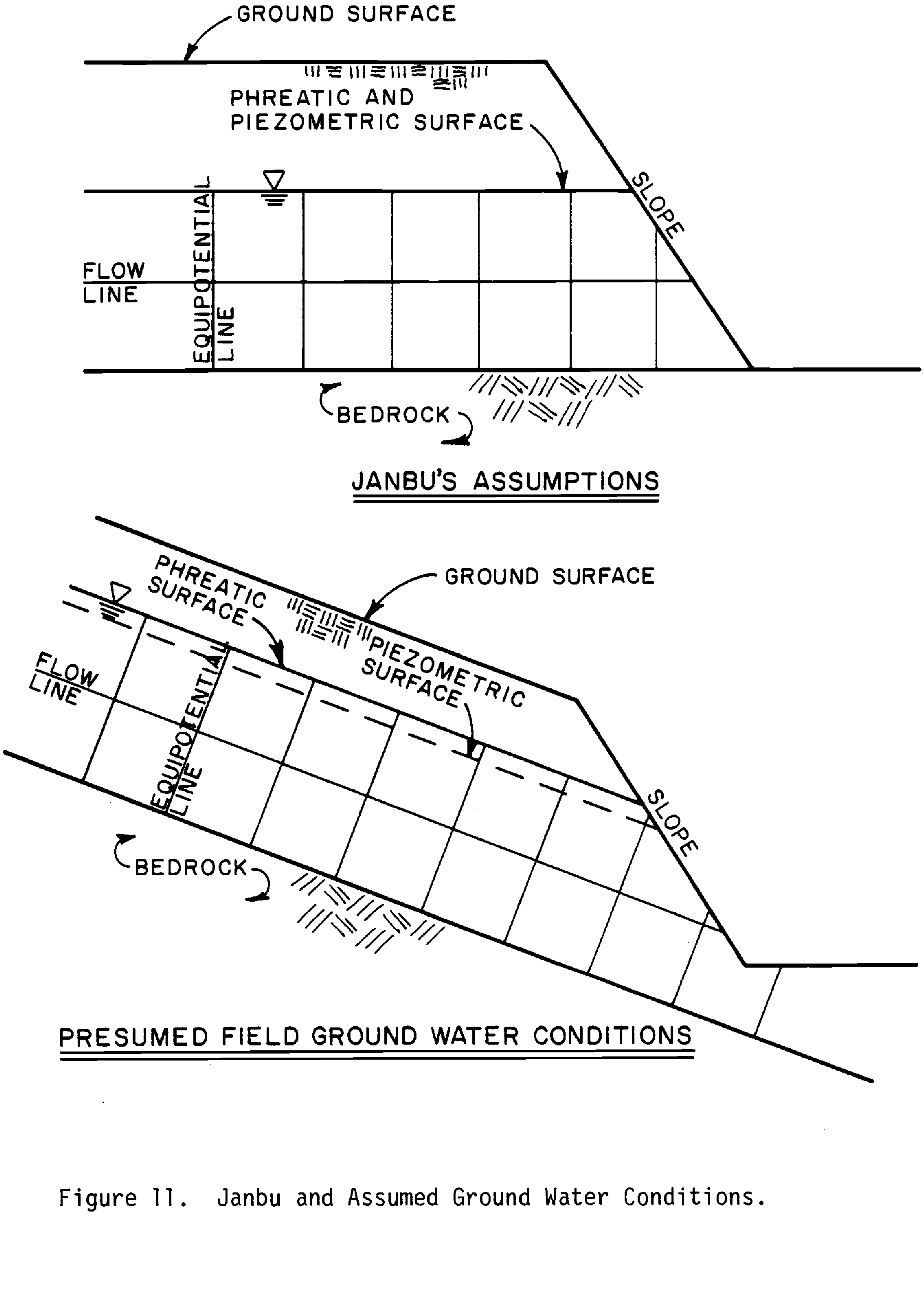


Figure 11. Janbu and Assumed Ground Water Conditions.

Janbu's r_u expression. It was found that this correction factor (C_w) could be described approximately as a function of ground water level and $\lambda_{c\phi}$. The correction factor was found to approximately fit a straight line on semi-logrithmic paper for a given relative ground water depth (H_w'/H). The form of the relationship is as follows:

$$C_w = L + M \log(\lambda_{c\phi}) \tag{31}$$

where values of L and M are as shown in Table 4.

Table 4. Summary of Ground Water Correction Coefficients

H_w'/H	L	M	Coefficient of correlation, r^2
0.25	1.057	1.233	0.839
0.50	1.250	0.772	0.757
0.75	1.353	0.386	0.906
1.0	1.442	0.078	0.350

Equations were developed which described L and M as functions of H_w'/H. These are as follows:

$$L = 1.438 + 0.633 \log_{10}(H_w'/H) \qquad r^2 = 1.00 \tag{32}$$

$$M = 0.129 - 1.899 \log_{10}(H_w'/H) \qquad r^2 = 0.994 \tag{33}$$

Combining equations (31), (32), and (33) results in the following:

$$C_w = 1.438+0.633 \log_{10}(H_w'/H)+(0.129-1.899 \log_{10}(H_w'/H))\log_{10}(\lambda_{c\phi}) \tag{34}$$

Values of C_w for varying ground water levels and $\lambda_{c\phi}$ are summarized in Figure 2C (Appendix C). Note that the above equation is invalid for values of $H_w'/H = 0$.

The corrected r_u is the product of r_u (equation (29)) and C_w (equation (34)). The expression that relates r_u to ground water level based on equations (29), (30), and (34) indicates that r_u is a function of the ground water level relative to the slope height, the net weight of the soil and water, the slope angle, and $\lambda_{c\phi}$. Values of r_u are summarized in Figure 3C (Appendix C) for varying conditions.

Accuracy of Slope Stability Model

A computer program in BASIC language for the HP9845B computer, was written which used the equations describing Cousins' solution charts (equations (22) through (25)), Janbu's ground water level to r_u equation (equations (29) and (30)) and the expression for the ground water correction factor (equation (34)). The program was titled RSTAB, for Road Stability program, and is discussed more fully in a subsequent section. Factors of safety were determined using RSTAB and SSTAB1 computer programs. A comparison of these results is summarized in Figure 12.

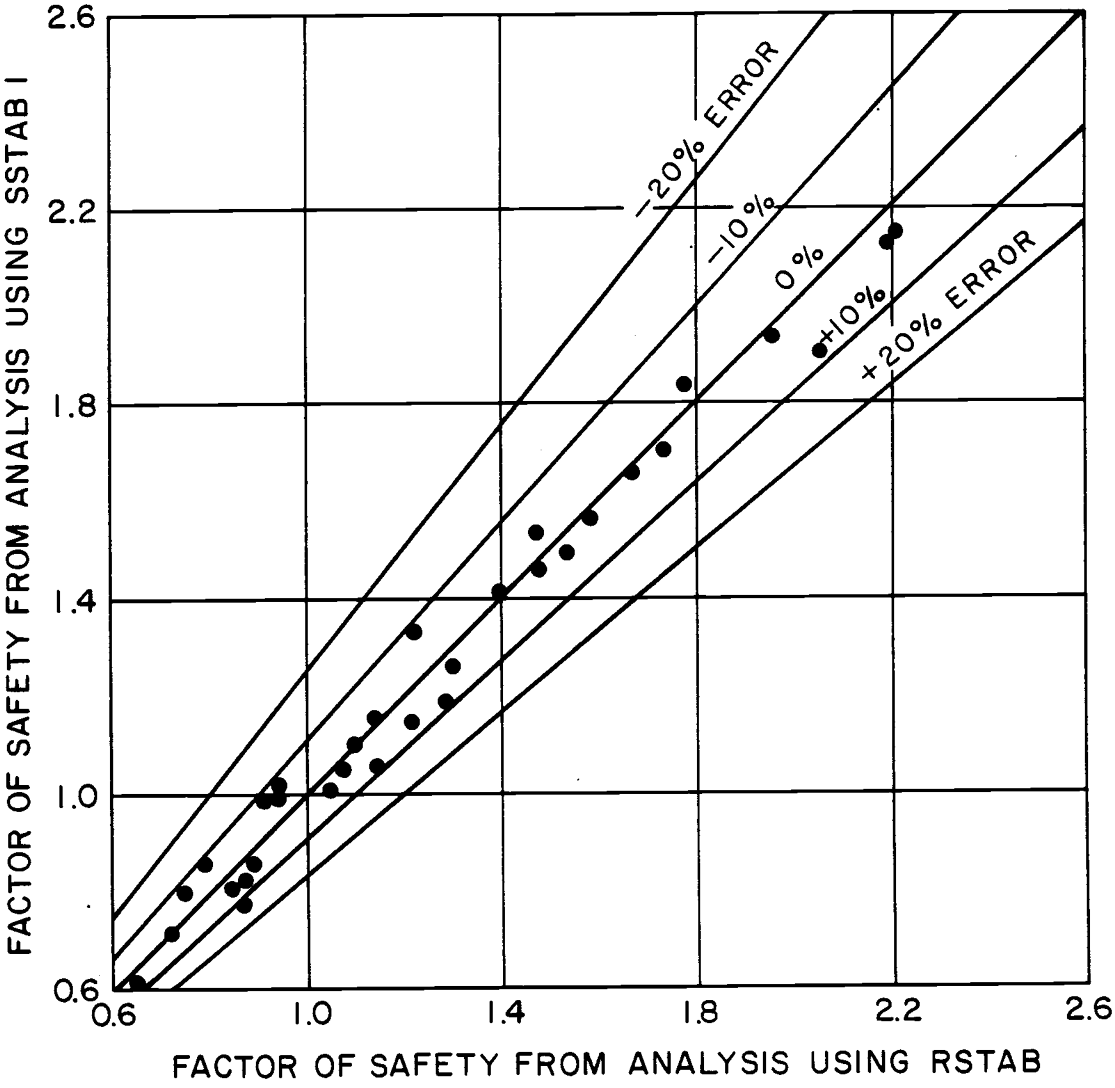


Figure 12. Accuracy of RSTAB Solution.

CHAPTER 5. RSTAB COMPUTER PROGRAM

Introduction

The purpose of the RSTAB computer program is to provide the factor of safety for slope stability for the cut and fill slopes associated with the construction of logging roads in Northwest forests. RSTAB determines the factor of safety for the cut and fill slopes based on a circular arc failure surface passing through the toe of the slopes. If requested, RSTAB computes the factor of safety for a planar failure surface along the fill/natural soil contact. A listing of the code is included in Figure 1D (Appendix D).

Input

Input parameters include slope geometry, soil properties and ground water level. Slope geometry parameters include the height and inclination of the natural ground surface. The bedrock and natural ground surface were assumed parallel for this analysis. The program limits the cut and fill slope inclinations to between 25 degrees and 55 degrees (between about 2:1 and 3/4:1 slopes). This is because the accuracy of results were not verified beyond the indicated range. However, this limitation is not expected to reduce the usefulness of RSTAB. A full bench cut was assumed for natural slopes steeper than 31 degrees (60 percent) to match the operating procedures of the ROAD program. Therefore, fill slope geometry and soil properties are not requested from the program user for natural slopes steeper than 31 degrees.

Soil properties requested from the program user include soil unit weight and effective stress strength parameters, c' and ϕ'. Values for each of these parameters are requested for the cut and fill slope soils. Effective stress strength parameters are requested from the user for the soil along the fill/natural soil contact if the user has requested the planar surface analysis. Values for all soil properties must be greater than zero. This limitation excludes the stability analysis for the undrained failure case (ϕ=o) and the cohesionless soil case (c=0). However, choosing low values of ϕ or c would provide results very similar to these two cases.

The ground water level above the bedrock surface is requested from the user. This value could be predicted from a ground water model such as presented herein or by Burroughs and Thomas (1981), or from engineering judgement. The ground water level is assumed to be equal to zero in the fill slope for all cases as previously discussed. However, the RSTAB program is written such that a ground water level in the fill could easily be incorporated into the analysis.

Input variable names used in RSTAB and brief definitions are summarized in Table 1D (Appendix D). Input variable names are summarized graphically in Figure 13.

Computations

Computation of the factor of safety commences upon completion of defining the input parameters. Computations proceed in several steps. These steps are:

1. If requested, the factor of safety for stability of the fill is calculated, assuming a planar failure surface along the

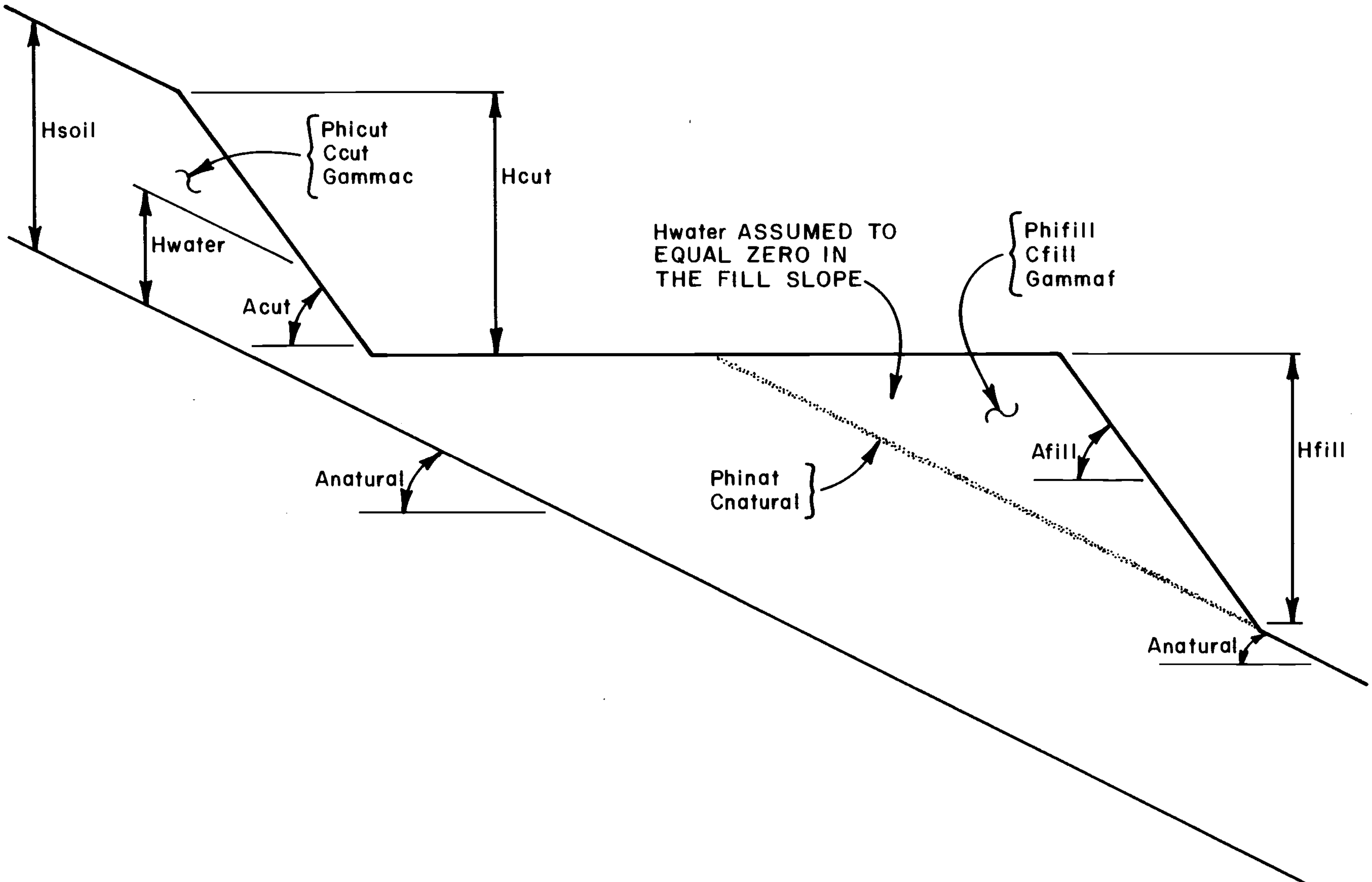


Figure 13. RSTAB Input Variable.

fill/natural soil contact. Equation (20) is used for this analysis.

2. Changes are made to the cut slope height and ground water depth to obtain the appropriate values for the three possible cases. Case 1 has the cut slope extending through the soil mantle and into the bedrock, as shown in Figure 14. The height of the cut slope is reduced to reflect only the height of the cut within the soil. This new height is calculated using the following equation:

$$\text{Corrected Hcut} = \text{Hsoil} \left[\frac{1+\tan(\text{Anatural})/\tan(\text{Acut})}{1-\tan(\text{Anatural})/\tan(\text{Acut})}\right] \quad (35)$$

The variables are defined in Figure 14. The input ground water value is correct for this case. Case 2 has the cut slope extending through the soil mantle, to the bedrock, as shown in Figure 14. No changes in slope height or ground water depth are required for this case. Case 3 has the soil mantle extending below the cut slope, as shown in Figure 14. The ground water depth is reduced to reflect only the depth of ground water within the cut slope. The ground water depth is calculated from the following equation:

$$\text{Corrected Hwater} = \text{Hwater} + \text{Hequiv} - \text{Hsoil} \quad (36)$$

where,

$$\text{Hequiv} = \text{Hcut}\ \left(1 - \frac{\tan(\text{Anatural})}{\tan(\text{Acut})}\right) \quad (37)$$

and the remaining variables are defined in Figure 14. No correction to the cut slope height is required for this case.

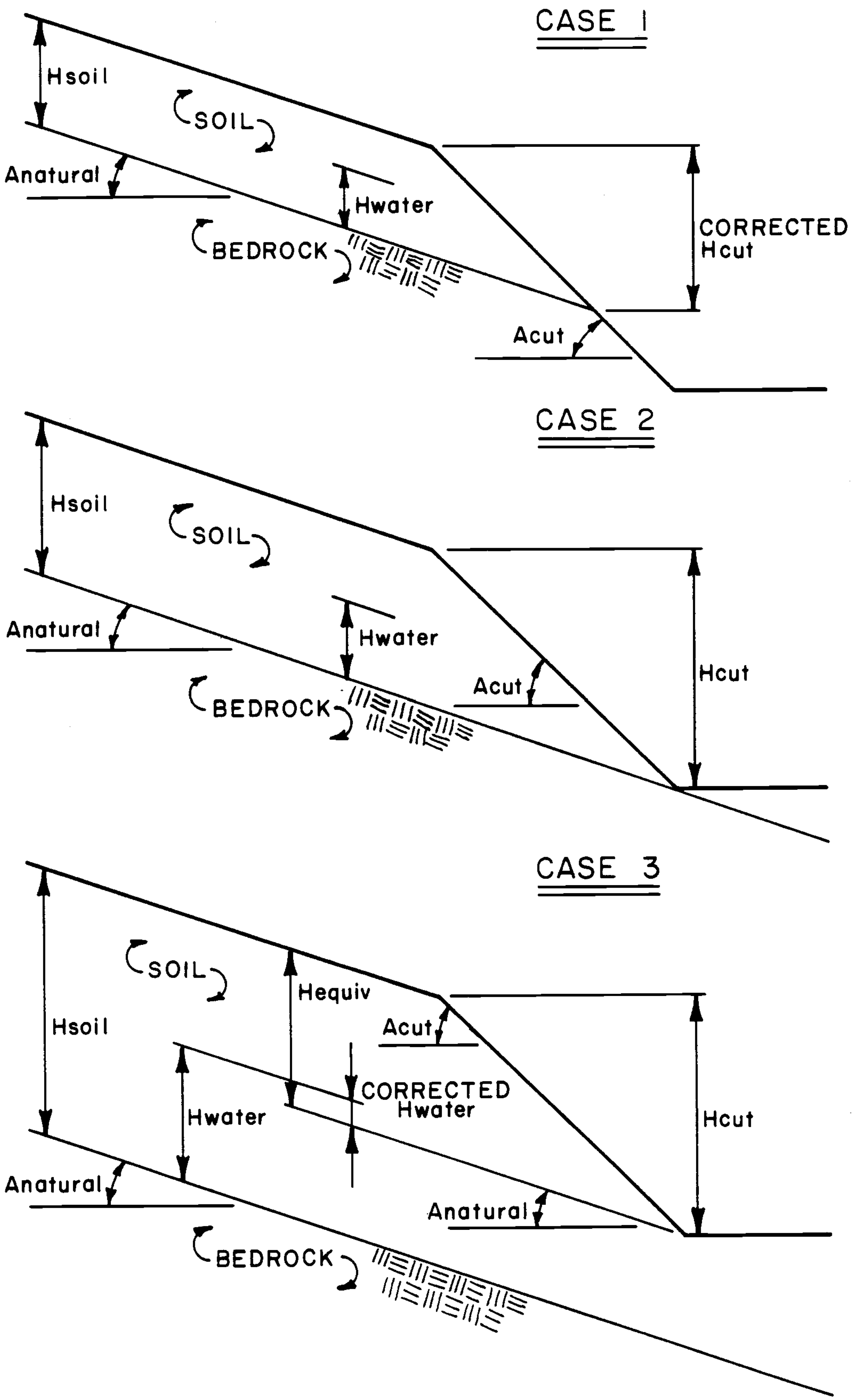


Figure 14. Cut Slope Conditions in RSTAB.

3. Calculate the pore pressure ratio (r_u) for the cut slope using equations (29), (30), and (34). The pore pressure ratio for the fill slope equals zero because the ground water level in the fill slope is assumed equal to zero.
4. Calculate the factor of safety for the cut slope and the fill slope using equations (22) through (25).
5. Output the results.

Variable names used for computations in RSTAB and brief definitions are summarized in Table 2D (Appendix D). Implementation of the RSTAB program for use with the PLANS program group is discussed in a subsequent section.

Implementation of the RSTAB Program

Three considerations must be addressed for implementation of the RSTAB program. These are:

1. Methods of inputting data to operate the RSTAB program
2. Operation of the RSTAB program within the PLANS program group,
3. Methods of outputting results from the RSTAB program.

Each of these considerations are discussed in the following sections.

Input. The input required to operate the RSTAB program includes slope geometry, soil properties and ground water level, as discussed previously. Certain slope geometry parameters are generated for use in the ROAD program and can be re-used in the RSTAB program. These include cut and fill slope inclination and height. The bedrock and natural ground inclination, assumed to be the same, can be obtained for any point along a proposed road alignment from the digital terrain model. The soil depth can be specified in several ways. These include:

1. Define a single value of soil depth for a given digital terrain model.
2. Define a single value of soil depth for a each road alignment.
3. Separate a given road alignment into lengths which correspond to different soil depths.
4. Separate a given digital terrain model into areas which correspond to different soil depths.

The most appropriate method of inputting soil depth is dependent on the quality of soil depth data and on the desired quality of the stability analysis. The method discussed in 1. would provide the lowest quality results but would require the least effort to implement into PLANS and for the program user to operate.

The soil properties (soil strength and unit weight) and ground water level can be specified in a manner similar to the soil depth. Again, the most appropriate method for inputting these parameters is dependent on the quality of the input data base and the desired quality of the RSTAB program results. Based on our understanding of Forest Service needs and the operation of PLANS, it appears that the most effective method of developing the required RSTAB input values is as follows:

1. Develop a proposed road alignment in the usual manner using ROAD.
2. Separate the road alignment into lengths with constant values of soil depth, soil strength, soil unit weight, and ground water depth defined for each length.
3. Obtain cut and fill slope inclination and height from ROAD program for road cross-sections to be analyzed.
4. Determine bedrock and natural ground inclination from the digital terrain model for road cross-sections to be analyzed.

Operation. A criterion, based on the desired use of the RSTAB program, must be developed regarding the locations along a proposed road alignment where a stability analysis is to be made. The locations could be user-defined or at evenly spaced intervals determined by a computer program.

Output. The output from the RSTAB program consists of a factor of safety for stability for the cut and fill slopes for a given road cross section. The results can be expressed in several ways. These include:

1. Tabulating the factors of safety for each cross-section.
2. Performing a statistical analysis of the factors of safety for each road alignment.
3. Identifying only the cross-section with factors of safety less than a prescribed value such as 1.0.

The most effective method of prescribing the output will be determined by the desired quality of analysis and the skills of the user who must interpret the results.

CHAPTER 6. FACTORS INFLUENCING GROUND WATER

Introduction

The pore water pressure in a slope is an important factor affecting the stability of a road cut or fill slope. The determination of the distribution of pore water pressure within a slope must be based on knowledge about the presence of ground water and an understanding of the ground water flow system. For the forests of the northwestern United States, the following factors are considered to have a significant impact on ground water flow in the soil mantle:

1. Precipitation events
2. Vegetation
3. Topography
4. Subsurface Conditions

A simple ground water model was developed considering the above factors to determine the feasibility of predicting design ground water levels in road cut slopes for use with the road stability computer program. A discussion of each of the above factors and development of the ground water model is presented.

Precipitation

Precipitation is considered the primary source of water for the ground water flow system on hillside slopes based on several studies in the Coast and Cascade Range Mountains (Ranken, 1974; Yee, 1976; Harr, 1973; Pierson, 1980).

As is typical of many natural events, precipitation occurs on an irregular basis with varying intensity and duration. Even within a given precipitation event, the precipitation rate and duration vary considerably between locations. However, probable recurrance intervals for average precipitation rates with given durations have been identified using statistical methods. A summary of published results for the Oregon Coast Range and Western Cascades is presented in Figure 15.

Precipitation may fall in the form of either snow or rain with rain being the predominant form in the Coast Range and at the lower elevations of the Cascade Range. Rain infiltrates or flows overland immediately upon impacting the ground, whereas snow may be stored on the vegetation and ground surface for long periods of time before melting and infiltrating. A combination of moderate rainfall and rapid snow melt may generate quantities of water available for ground water flow similar to quantities produced by very large precipitation events. Inflow into the ground water flow system was considered to be solely from rain precipitation events for modeling purposes.

Vegetation

Vegetation intercepts a certain amount of precipitation during each rainfall event. The quantity intercepted depends primarily on antecedent conditions, precipitation rate and duration, and species and stature of vegetation (Ovington, 1954; Voight, 1960). Wet antecedent conditions results in nearly full vegetation interception storage, therefore, more precipitation reaches the ground surface during a given storm event with wet rather than dry antecedent

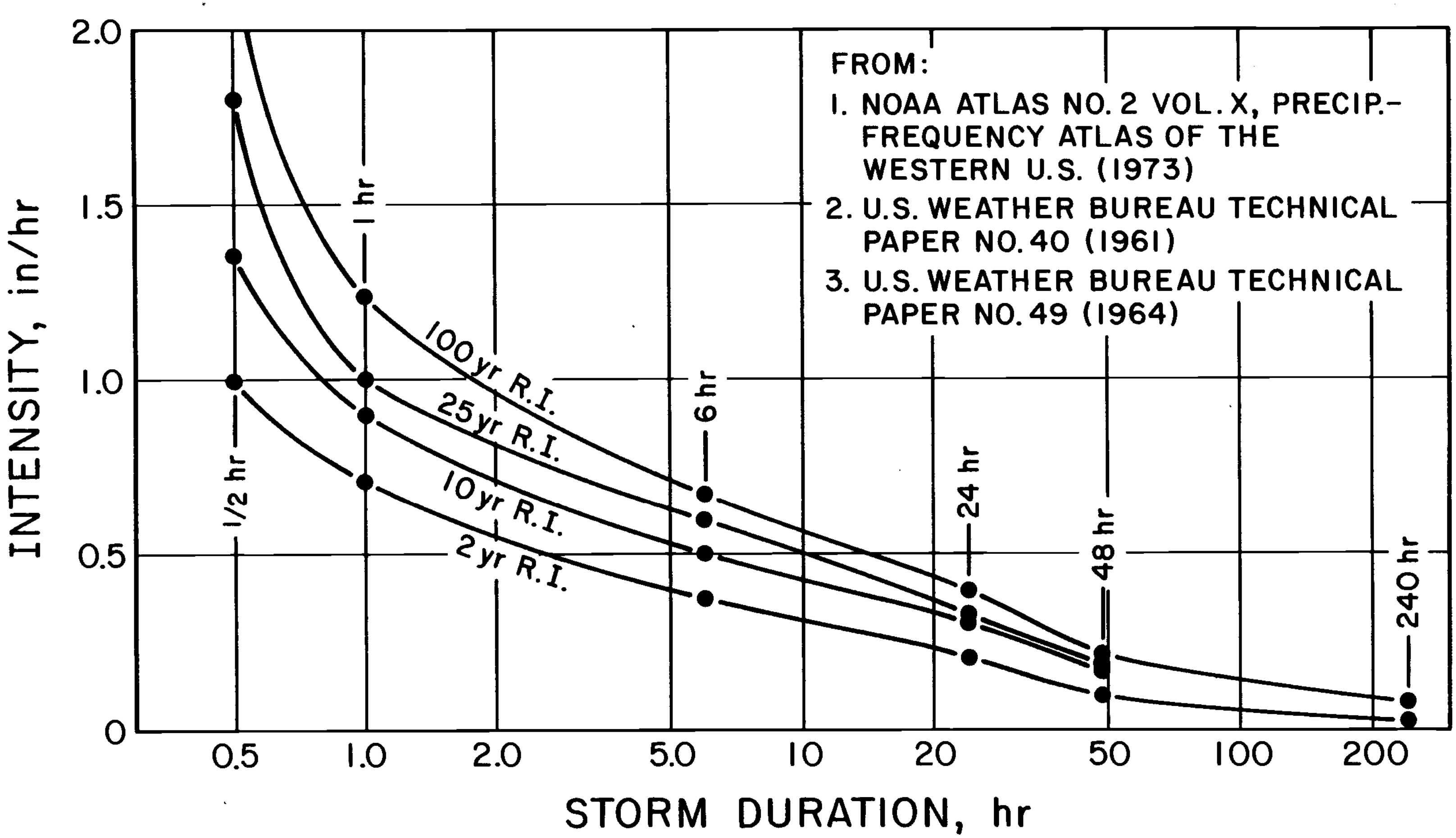


Figure 15. Precipitation Intensity-Duration Chart.

conditions. For small storm events with a low precipitation rate, short duration, and dry antecedent conditions as much as 80 percent of the rainfall is intercepted by the forest vegetation. Very little of the total quantity of rainfall is retained by the vegetation for large storm events with moderate to high precipitation rates and long duration. A smaller portion of rainfall will be retained by a deciduous forest than by a coniferous forest for a given rainfall event.

Based on these studies, it appears that for large storm events, of concern when analyzing the critical ground water conditions, vegetation has little effect on the quantity of rainfall reaching the ground surface. Vegetation was considered to have no effect on the quantity, intensity, or duration of the design storm event, for modeling purposes.

Topography

Surface topography can play a significant role in controlling the distribution of ground water. This is because surface topography affects the distribution of precipitation and controls the flow of overland runoff.

On a regional scale, topography can play a dominant role in precipitation distribution. Oregon provides a good example of this with the high annual precipitation in the Coast Range, much lower precipitation in the Willamette Valley, and high precipitation in the western slope of the Cascade Range.

Topography has must less influence on precipitation distribution on a local scale than it does on a regional scale. Certain slope aspects may experience greater quantities of precipitation than other slope aspects. However, slope aspect was not addressed in the ground water model because it was considered to have a relatively minor impact on the distribution of ground water. Surface topography controls only the overland flow component of precipitation reaching the ground surface. Overland flow has not been observed in Coast or Cascade Range soils (Dyrness, 1969; Ranken, 1974; Yee, 1975; Harr, 1977; Pierson, 1980) due to the high infiltration rate of the natural soils. Therefore, surface topography alone has little effect on the distribution of ground water.

The flow of water is controlled by capillary and gravitational forces (Kirby, 1978) after surface water enters the ground. The potential energy associated with the gravitation force is much larger than the energy associated with capillary tension for the usual hillslope case where soil depths are shallow and constant compared to the size of hillslope features, and the soils overly relatively impervious bedrock. Therefore, subsurface topograhic conditions control the distribution of ground water on hillslopes (Anderson and Burt, 1978). Because the subsurface features, such as contacts between soil horizons and soil and bedrock, are generally parallel to the surface topography, the surface topography can be used in the determination of the distribution of ground water.

The ground water model incorporates topography in two ways. First, by considering the inclination of the slope of interest and, second, by considering the area and shape of the surface drainage area above the point of interest. The ground surface and bedrock surface are considered parallel in the model.

Subsurface Conditions

Soils of interest are located in the western forests of Oregon and Washington and the forests of southeast Alaska. Several studies have been done on soils located in the H.J. Andrews Experimental Forest on the lower western slopes of the Cascade Range (Dyrness, 1969; Ranken, 1974; Harr, 1977). The soils encountered in these studies consisted typically of about 2 inches of organic debris overlying about 5 feet of gravelly clay to gravelly, clayey sandy silt. Underlying the gravelly soil was weathered basalt or volcanic breccia. The climate, vegetation, geology, soils, and general hydrologic properties of these study areas were considered typical of the Western Cascades (Rothacher et al., 1967).

The soils studied in the Coast Range were located on the lower western slopes of Marys Peak (Yee, 1975) and in the Perkins Creek Watershed (Pierson, 1980). The soils encountered in these studies were less cohesive than the Cascade Range soils studied. They consisted of about 3 inches of organic debris overlying about 6 feet of sandy gravelly silt. The silt was underlain by weathered sandstone or igneous bedrock. These soils are considered typical for large portions of the Coast Range (Yee, 1975).

The Coast Range soils are similar to soils encountered in soil studies in Southeast Alaska (Filz, 1981; Sidle and Swanston, 1982) which are generally non-plastic gravelly sandy silts that extend to depths of 1 to 3 feet.

Saturated hydraulic conductivity. Several studies determined hydraulic properties of soils in the Coast Range (Yee, 1977; Pierson, 1980) and Cascade Range (Dyrness, 1969; Ranken, 1974; Harr, 1977). Soil properties determined by the investigators included horizontal and vertical saturated hydraulic conductivities, bulk dry densities, and average unsaturated hydraulic conductivity relationships as a function of depth. Figure 16 summarizes average values of hydraulic conductivity with depth for Western Cascade and Coast Range soils.

The saturated hydraulic conductivity test results indicate that saturated conductivity decreased with depth and for the sites studied, the Western Cascade soils had a greater saturated conductivity than the Coast Range soils. The differences in conductivity can be attributed to the different soil types represented in the investigations.

Saturated hydraulic conductivity is dominated by flow through macropores in the soil matrix near the ground surface. Studies done on sandy silt forest soil in northeastern United States indicate that saturated hydraulic conductivity decreases with increasing depth as does the number of macropores. The number of macropores decreases with depth because the formation processes at or near the ground surface. Macropores are formed by plants, insects, burrowing animals, and by dessication and freezing cycles. Once formed, macropores persist longer in cohesive soils than non-cohesive soils (Aubertine 1971). The differences in the persistence of macropores is a likely

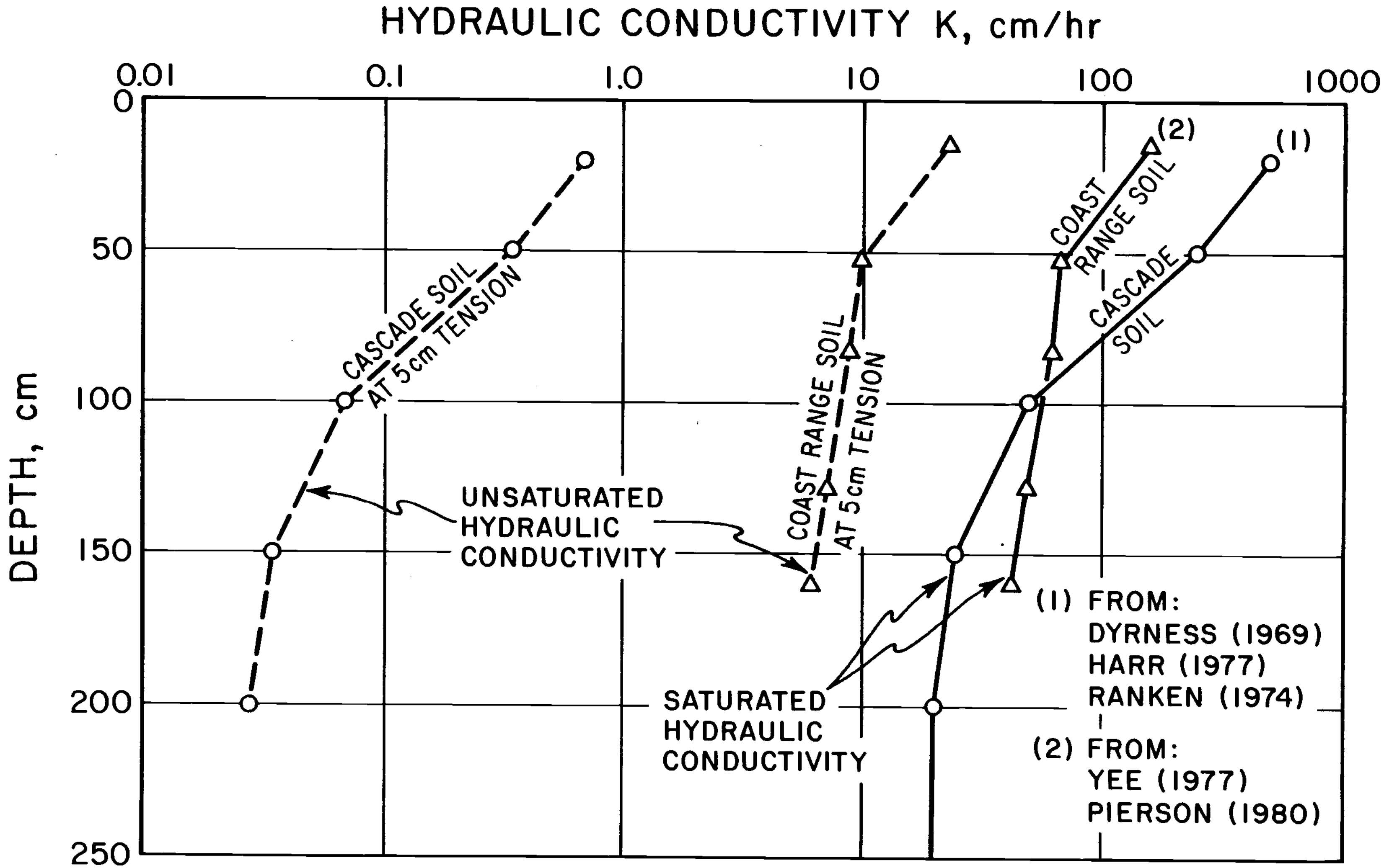


Figure 16. Summary of Hydraulic Conductivities.

explanation for the greater saturated hydraulic conductivity in the cohesive Cascade soils than in the less cohesive Coast Range soils for the upper 100 cm.

Unsaturated hydraulic conductivity. Several researchers have investigated the unsaturated hydraulic conductivity characteristics of Coast Range (Yee, 1976) and Cascade Range (Harr, 1973; Ranken, 1974) soils. All unsaturated hydraulic conductivity relationships were determined from laboratory desorption test results (Laliberte et al., 1968).

Laliberte et al. (1968) found that the hydraulic conductivity of a soil at any capillary tension could be described by two measured parameters P_b and λ. The P_b parameter is called the bubbling pressure and represents the minimum capillary pressure on the drainage cycle where air is continuous in the soil. The λ parameter is a dimensionless constant which is characteristic of a given soil. A soil with uniform pore sizes would have a high λ value whereas a soil with a wide range of pore sizs would have a low λ value. The laboratory test results indicate λ values of from about 0.01 to 0.1 for the Coast Range soils (Yee, 1976) and from about 0.1 to 0.4 for the Cascade Range soils (Harr, 1973). Values for P_b were also determined. The relationships describing the change in unsaturated permeability with respect to capillary tension are (Laliberte et al., 1968):

$$K_{unsat} = K_{sat}\left(\frac{P_b}{P_c}\right)^n \qquad (38)$$

where $n = 2 + 3\lambda$ (39)

and K_{unsat} = unsaturated hydraulic conductivity (L/T)

K_{sat} = saturated hydraulic conductivity (L/T)

P_c = any capillary pressure less than or equal to P_b (L)

P_b = bubbling pressure (L)

λ = dimensionless constant.

For the soil tested, equation (38) becomes:

$$K_{unsat} = K_{sat} \left(\frac{2.0(\mathrm{cm})}{P_c(\mathrm{cm})}\right)^{2.1} \text{, Coast Range} \tag{40}$$

$$K_{unsat} = K_{sat} \left(\frac{0.4(\mathrm{cm})}{P_c(\mathrm{cm})}\right)^{2.6} \text{, Cascade Range} \tag{41}$$

Based on the equations (40) and (41), one can see that for a given capillary tension (P_c) and saturated hydraulic conductivity (K_{sat}) the unsaturated hydraulic conductivity (K_{unsat}) is much greater for the tested Coast Range soils than the Cascade Range soils. See Figure 16 for a graphical comparison. The unsaturated hydraulic conductivity (K_{unsat}) for the Coast Range soils is considered to be high compared to typical sands, silts, and clays (Yee, 1976). Both Coast and Cascade Range soils have a great capacity to transmit water in the unsaturated condition (Yee, 1976; Harr, 1973).

No studies were identified that determined the hydraulic conductivity of the bedrock. However, based on field observations of piezometric response in a soil mantle due to rainfall (Harr, 1973; Ranken, 1974; Yee, 1976; Pierson, 1980; Sidle and Swanston, 1982; Weyman, 1973), it appears that the hydraulic conductivity of the bedrock is negligible relative to the conductivities of the soil mantle.

The saturated hydraulic conductivity of the soil zone expected to saturate - typically at the soil/rock contact - is a required input parameter in the model. The hydraulic conductivity of the saturated zone is assumed to be constant with depth. The bedrock underlying the soil mantle is assumed to be impermeable for modeling purposes. Soil-bedrock interflow, often an important slope stability consideration, is assumed negligible for purposes of modeling.

Infiltration rate. No studies were identified where maximum infiltration rates were determined for undisturbed forest soils in the northwest United States. However, it is known that soil infiltration rates are great enough that overland flow is an extremely rare event in undisturbed forest soils (Whipkey, 1965; Harr, 1973; Yee, 1976). A soil infiltration study in the Southeastern United States on undisturbed forest soil indicated that maximum infiltration rates of from two to three inches per hour were typical for natural silty forest soils (Arend, 1941). For modeling, it was assumed that all rainfall reaching the ground infiltrated into the ground and no overland flow occurred.

Ground Water Conditions

Numerous investigators have studied the ground water conditions in hillslopes during precipitation events in western Oregon forests (Ranken, 1974; Yee, 1975; Harr, 1977; Pierson, 1980; Burroughs and Thomas, 1981) and Southeast Alaska forests (Swanston, 1967; Sidle and

Swanston, 1982). Other studies have been performed on hillslopes in England with a geologic setting similar to much of the Oregon Coast Range, a residual soil mantle underlain by sandstone bedrock (Weyman, 1973; Anderson and Burt, 1977).

An understanding of the basic principles governing ground water flow is required for this discussion. Darcy's Law describes the energy-flow relationship. For saturated soil conditions this is:

$$q = kiA \tag{42}$$

where, q = the volume rate of flow (L^3/T)

k = the coefficient of saturated hydraulic conductivity (L/T)

i = the hydraulic gradient which represents the change in energy potential per unit distance (L/L)

A = the area over which saturated flow occurs (L^2)

The energy potential (H) is usually defined as $H = Z + P$ for saturated flow in soil where Z is the distance from the point of interest to a reference plane and P is the energy potential due to the water pressure. Potentials due to kinetic energy, and temperature and osmotic gradients are generally small and considered negligible. The water pressure term, P, becomes negative in unsaturated soil due to capillary pressures and can be redefined as capillary pressure, P_c. With this adaption, and interpreting the hydraulic conductivity as the unsaturated hydraulic conductivity, $K(P_c)$, Darcy's Law can be applied to unsaturated flow conditions (Hillel, 1971). Based on this relationship, it can be shown that ground water in the unsaturated condition is drawn from a wetter (lower capillary tension) to a drier (higher capillary tension) location as well as vertically downward by gravitational forces.

Capillary tensions near the ground surface may become greater than gravitational forces due to evaporation and plant uptake during dry periods which results in upward flow of the unsaturated ground water. Capillary tensions are much lower during wet periods and unsaturated flow becomes controlled primarily by gravitational forces.

During a storm event, rain water infiltrates into the surface soil due to a downward hydraulic gradient caused by gravitational and capillary forces. A portion of the infiltrating water may enter non-capillary pores and move rapidly to greater soil depths or downslope. The extent of non-capillary pore water movement is determined by the degree of interconnection of the non-capillary pores. Field studies have not indicated a significant amount of flow in non-capillary pores (Weyman, 1973; Yee, 1975; Harr, 1977). The soil profile in Yee's study had less decrease in hydraulic conductivity with depth than the soil profile in Harr's study and Yee also observed less unsaturated downslope flow than did Harr. A theoretical model developed by Zaslavsky and Rogoski (1969) agrees well with these field observations.

Zaslavsky and Rogoski's theoretical model states that the direction of unsaturated flow is a function of the degree of anisotropy of the soil and of the slope angle. This can be expressed as:

$$\tan\beta = \frac{\tan\alpha}{N} \tag{43}$$

where, β = the angle between the flow line and the hillslope surface (degrees)

N = the degree of anisotropy between soil layers (K_1/K_2)

α = the angle between horizontal and the contact between layers as shown in Figure 17 (degrees).

Based on the field observations and Zaslavsky and Rogoski's model, it appears that ground water flow lines can extend tens of feet downslope before reaching the bedrock surface. This downslope movement is of little consequence at mid-height on a planar slope because all flow lines are parallel, but can have a great impact on ground water distribution where there is a break in the slope. Figure 18 illustrates the convergence of the unsaturated flow lines in an idealized hillslope depression with constant capillary tension. Constant capillary tension approximates actual conditions during a storm event with wet antecedent conditions due to the low capillary tensions at high soil water content. Base on the Zaslavsky and Rogoski model, one would expect to observe a much greater piezometric rise at breaks in a slope than within a planar slope for a given storm event. Results from field studies agree with this observation. Piezometric rise on planar slopes have been observed to be typically less than 10 cm above bedrock for moderate storms (Ranken, 1974; Yee, 1975; Harr, 1977). Piezometric rise in hollows for similar sized storms were observed to be between 25 and 150 cm, depending on the upslope drainage area (Swanston, 1967; Anderson and Burt, 1978; Pierson, 1980; Sidle and Swanston, 1982; Burroughs and Thomas, 1981).

Pierson (1980) and Burroughs and Thomas (1981) have shown that piezometric rise in hollows is a function of the upslope drainage area. However, Pierson's results suggest that there may be a maximum

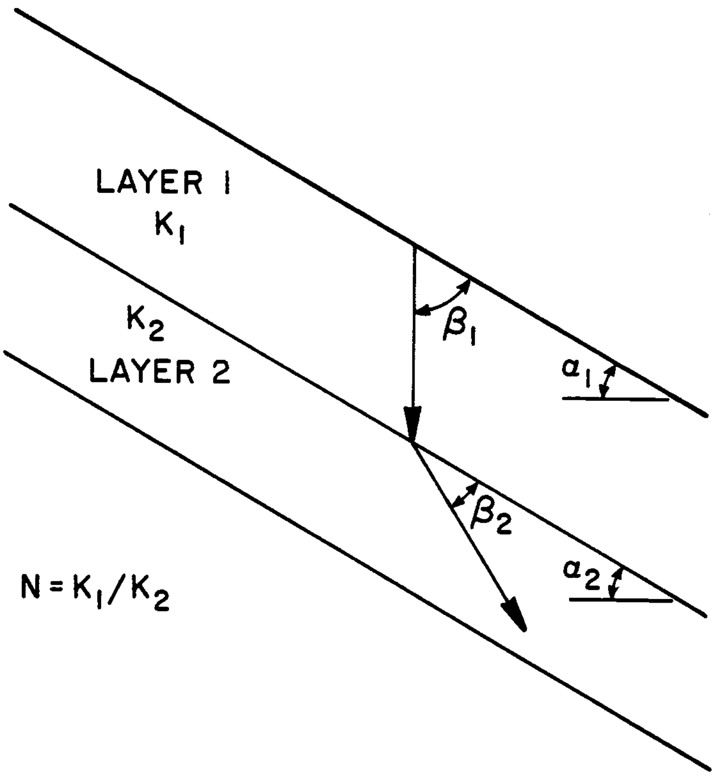


Figure 17. Ground Water Flow Through Anisotropic Soils.

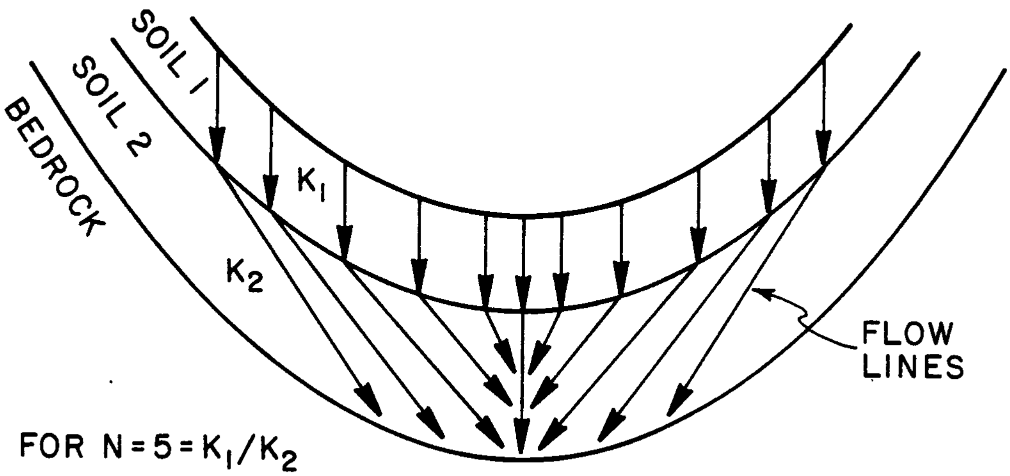


Figure 18. Convergent Ground Water Flow in a Hillslope Depression.

contributing drainage area for some cases. Pierson found no increase in piezometric rise for a given storm with increasing drainage area for areas greater than about 5 x 10^3 sq. meters (1.2 acres) in two separate hollows. This condition was not observed by Burroughs and Thomas (1981) for drainage areas as large as 4.2 acres.

After a storm event, ground water continues to flow in response to gravitational and capillary forces. Theoretically, the unsaturated flow eventually becomes parallel to the bedrock surface, downhill unsaturated flow prevails and soil water content decreases with increased height above bedrock (Weyman, 1973). Field results, however, have not shown the parallel unsaturated flow predicted by Weyman (Harr, 1977), possibly due to the great length of time required for this condition to develop.

Drainage between storm events results in wetter antecedent conditions at the bottom of a hillslope than at the top. This condition coupled with convergent unsaturated flow lines in hollows, at the base of slopes, seems a likely explanation for the rapid response and large rise in piezometric levels in hollows.

For modeling purposes, ground water conditions were assumed to be a function of long-term precipitation and topography. This assumption is based on the fact that the movement of ground water is very slow - on the order of centimeters per hour. Therefore, stated in a simplistic manner, precipitation entering the ground travels only a short distance and contributes to the ground water level only near the point where it entered the ground on a short term basis. The effects of topography come into play as the water travels downslope for long periods of time.

Modifications to this model were made to address the long term convergent downslope flow into hillslope hollows.

CHAPTER 7. GROUND WATER MODELING

Introduction

Natural hillslope ground water flow systems are extremely complex due to highly variable subsurface hydrologic conditions. Methods of modelling this system include deterministic models such as developed by Hodge and Freeze (1977), empirical models that describe observed conditions (Weyman, 1973; Harr, 1977; Pierson, 1980; Burroughs and Thomas, 1981), or models developed by a combination of deterministic and empirical methods.

The Hodge and Freeze model, requiring the use of large computers, and similar computer models were considered too complex for our purposes. These models generally deal only with saturated soil conditions. Of the empirical models, only the Burroughs and Thomas model was developed for predictive purposes. A simple ground water model was developed for this project which attempts to describe field observations by incorporating significant physical parameters rather than empirically derived coefficients.

Burroughs and Thomas Model

Piezometric response of ground water in hillslope depressions was related to site characteristics and precipitation in eight separate clearcut areas. Sensitivity analyses showed that the most significant variables were drainage area, average 24 hr. precipitation, antecedent precipitation, and slope gradient. Variables not addressed in this model included subsurface hydrologic properties, vegetation, and topographic features. Assuming that the excluded variables play a significant role in piezometric response, this model can be expected to provide valid results only for the specific subsurface conditions, vegetation, and topographic features for which the model was developed. Because subsurface conditions, vegetation, and topography vary considerably in Northwest forests, the usefulness of this model is restricted.

Simple Ground Water Model

A simple ground water model was developed for possible use in conjunction with the simplified slope stability model also developed for this project. Fundamental relationships observed in field studies and discussed previously were expressed mathematically to arrive at a model that is ameable to computer programming and predicts reasonable ground water levels for a variety of conditions.

The key assumptions for this model are as follows:

1. Subsurface conditions consist of homogeneous soil overlying an impervious barrier (bedrock).
2. The ground surface topography is parallel to the underlying bedrock surface topography.
3. Overland flow occurs only when the soil mantle is completely saturated.
4. Relative piezometric response is dependent on topography and long term precipitation conditions.
5. Unsaturated flow is vertically downward except in hillslope depressions where it has a downslope component.

6. The downslope component of unsaturated flow is dependent on the hydrologic anisotropy of the soil.
7. Saturated flow is parallel to the ground surface.
8. Saturated downslope flow can be considered to be negligible for short periods of time.
9. The model is in a steady state condition.
10. The piezometric level and phreatic surface are coincident.

Hillslope ground water level. Based on the listed simplifying assumptions, a relationship between the hillslope ground water level above bedrock, long term precipitation, and topography can be developed by employing the concept of continuity. Using the variables defined in Figure 19, the total inflow (q_t) into a soil element can be described by:

$$q_t = q_{in} + \Delta q \tag{44}$$

where according to Darcy's Law (equation (42))

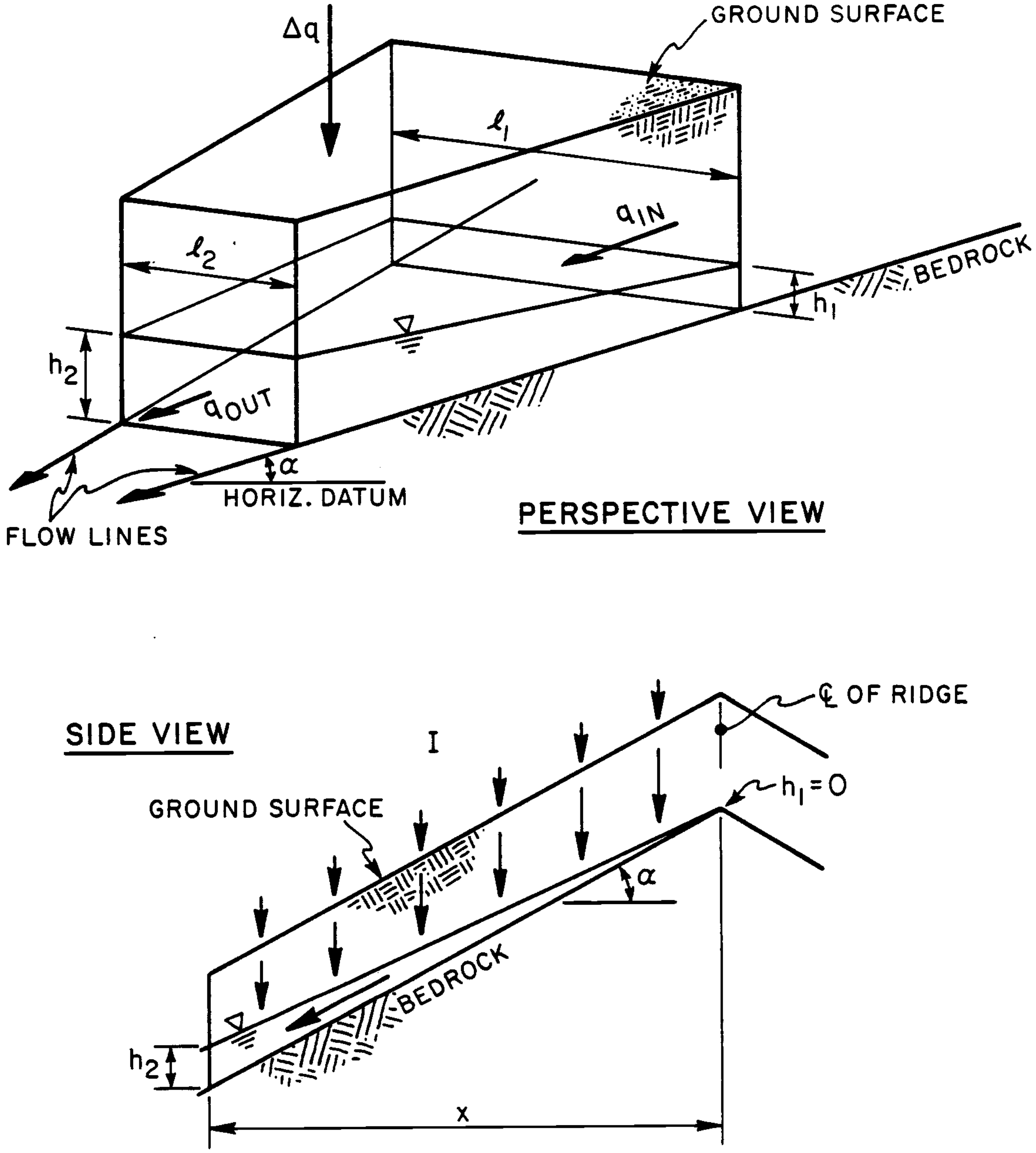
$$q_{in} = kiA_{in} \tag{45}$$

The rate of infiltrating precipitation (Δq) can be related to the ground surface area and the rate of rainfall (I) as follows:

$$\Delta q = I\left(\frac{\ell_1 + \ell_2}{2}\right)X \tag{46}$$

By substituting $A_{in} = h_1\ell_1$ into equation (45), and substituting equations (45) and (46) into equation (44)

$$q_t = kih_1\ell_1 + I\left(\frac{\ell_1 + \ell_2}{2}\right)X \tag{47}$$



k = THE SATURATED HYDRAULIC CONDUCTIVITY IN THE SATURATED ZONE (L/T)

I = THE LONG TERM AVERAGE PRECIPITATION RATE (L/T)

Figure 19. Definition of Variables for Simple Ground Water Model.

The outflow (q_{out}) from a soil element can be described as:

$$q_{out} = ki\ h_2 \ell_2 \tag{48}$$

If one assumes a steady state condition, there is no change in ground water stored in the soil element, therefore,

$$q_{out} = q_t \tag{49}$$

By substitution of equations (47) and (48) into equation (49) and with $i = \sin\alpha$

$$k \sin\alpha\ h_2 \ell_2 = k \sin\alpha\ h_1 \ell_1 + I\ \left(\frac{\ell_1 + \ell_2}{2}\right) X \tag{50}$$

By rearranging to solve for h_2

$$h_2 = \frac{h_1 \ell_1}{\ell_2} + \frac{I}{2K \sin\alpha} \left(\frac{\ell_1 + \ell_2}{\ell_2}\right) X \tag{51}$$

Note that equation (51) is dimensionally correct. Therefore, any consistent set of units may be used to determine h_2.

The ground water level can be determined at any point on a hillslope by using equation (51). The methodology to determine antecedent ground water levels is as follows:

1. Define the drainage area above the point of interest from a topographic contour map.
2. Break the drainage area into trapezoidal subareas with lines that are perpendicular to the axis of the drainage area.
3. Define the upslope subarea widths (h_1), downslope subarea widths (h_2), the length of each subarea along the axis of the drainage area (X), and the average slope of the ground surface in each subarea (α).

4. Solve for the ground water level (h_2) for the upper most subarea by assuming $h_1 = 0$ and using equation (51).
5. Continue solving for h_2 for each subarea by substituting the h_2 from the previous subarea for h_1 for the present subarea. Continue until h_2 is calculated for the point of interest. See an example of this procedure in Appendix E.

An important consideration when employing this model is determining the width, ℓ_2, at the point of interest. The most appropriate choice of width, ℓ_2, for varying topographic conditions is not known. Empirical studies may be required to determine widths, ℓ_2, which give the most correct results. However, general guidelines are presented which seem appropriate for varying topographic conditions.

For divergent slopes, a drainage basin width at the point of interest should be great enough so that the drainage area extends to the top of the slope but should not be greater than the width of the topographic feature. For planar slopes, the choice of basin width at the point of interest has no effect on the results. For convergent slopes, a drainage basin width should be chosen which reflects the estimated actual width of the saturated zone in the hillslope depression.

The precipitation rate (I) appearing in equation (51) should be the average long term precipitation. A reasonable approach for determining the antecedent precipitation period is based, conceptually, on the method used to determine the time of concentration (t_c) for use in the evaluation of the Rational Formula (Chow, 1964). The time of concentration is defined as the time required for the surface runoff from the most remote part of a drainage basin to reach the point under consideration. For a uniform, long duration rainfall rate this is the time at which steady state conditions are reached (Chow, 1964).

An estimate of the time required for the steady state conditions to be achieved can be made by calculating the time for saturated ground water to flow from the most remote part of a drainage basin to the point under consideration. The time can be estimated by calculating the velocity of the saturated flow using Darcy's Law and dividing the maximum distance to be traveled by the velocity. The velocity of the saturated flow can be expressed as follows:

$$V = \frac{q}{A_v} \tag{52}$$

where, V = the effective fluid velocity in the direction of the hydraulic gradient (L/T)

q = the volume rate of flow from area A (L^3/T)

A_v = the area of the void space in the soil matrix within area A (L^2)

The area of the void space (A_v) can be expressed in terms of porosity (n) as follows:

$$n = \frac{A_v}{A} \tag{53}$$

The hydraulic gradient (i) can be expressed as $\sin\alpha$. Rewritting equation (42) as $v = ki$ and substituting to solve for the time of concentration for saturated ground water flow (t_c'):

$$t_c' = \frac{nL}{k \sin\alpha} \tag{54}$$

where, n = the soil porosity

L = the distance from the most remote part of the drainage basin to the point under consideration along the flow path (L)

k = the saturated hydraulic conductivity (L/T)

α = the angle of the slope in the drainage area (degrees)

Equation (54) is expected to provide results that are higher than the appropriate time of concentration because of the significant downslope direction of the unsaturated flow lines. However, the long term precipitation rate that is determined from t_c' is not particularly sensitive to changes in t_c' due to the great length of time involved. Additional refinement of this parameter (t_c') may require empirical correlations.

The value for average long term precipitation (I) can be obtained from charts such as Figure 15, after the period (t_c') has been determined. To provide a more convenient determination of I, one month design storms can probably be used until additional refinements can be made to the procedure.

Hillslope depression ground water level. Observed hillslope depression ground water response is much greater than would be predicted by the hillslope ground water model. This is believed to be because the hillslope model does not address the effect of downslope unsaturated flow. The effects of downslope unsaturated flow can be addressed by employing relationships developed by Zaslavsky and Rogoski (1969). Variables used for the development of the hillslope depression correction factor (D) are summarized with the idealized hillslope depression in Figure 20.

As shown in Figure 20, the correction factor (D) can be approximated by the ratio of the width of the zone of influence (W_T) divided by the estimated width of the saturated zone (W_S). From trigonometric relationships W_S can be related to the height of ground water determined from the hillslope ground water model (h_2). The relationship is:

$$W_S = \frac{2h_2}{\tan\alpha} \tag{55}$$

Rearranging equation (43) to solve for β results in:

$$\beta = \arctan\left(\frac{\tan\alpha}{N}\right) \tag{56}$$

Using equations (55) and (56) and solving the trigonometric relationships, the equation for W_T is:

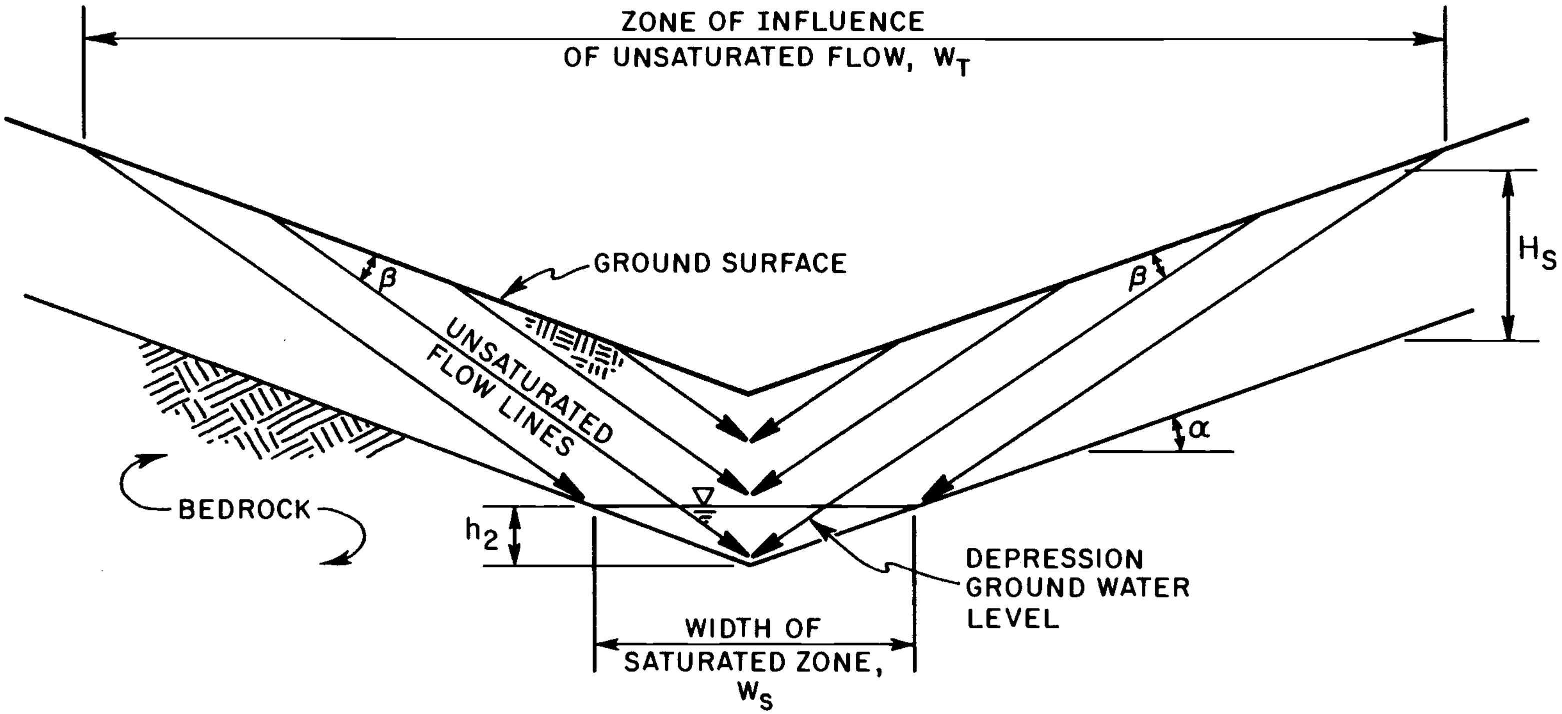
$$W_T = \frac{2H_S}{\tan(\alpha + \beta) - \tan\alpha} + \frac{2h_2'}{\tan\alpha} \tag{57}$$

Assuming that the correction factor (D) can be expressed as:

$$D = \frac{W_T}{W_S} \tag{58}$$

Substituting equations (55) and (57) into equation (58) and rearranging terms results in:

$$D = 1 + \frac{H_S \tan\alpha}{h_2'[\tan(\alpha+\beta) - \tan\alpha]} \tag{59}$$



h_2 = GROUND WATER LEVEL PREDICTED BY EQUATION (51)

Figure 20. Definition of Terms for Hillslope Depression Correction.

By definition, the corrected ground water level (h_2') is related to the correction factor as follows:

$$h_2' = Dh_2 \tag{60}$$

where, h_2 is the ground water in the hillslope depression as predicted by equation (51).

Substituting equation (60) into (59) and rearranging terms results the following quadratic equation:

$$0 = D^2 - D - \frac{H_s \tan\alpha}{h_2[\tan(\alpha+\beta) - \tan\alpha]} \tag{61}$$

Solving for the positive root of equation (61) results in:

$$D = 1/2 + \left(1/4 + \frac{H_s \tan\alpha}{h_2(\tan(\alpha+\beta) - \tan\alpha)}\right)^{1/2} \tag{62}$$

Careful observation of equation (62) shows that an increase in soil anisotropy (N), soil depth (H_s), or slope angle (α) results in an increase in the correction factor (D). An increase in h_2 causes D to decrease. See Figures 1E, 2E, and 3E (APPENDIX E) for plots of D as a function of the variables.

Additional improvement of equation (62) would require empirical correlation with field observations. The corrected hillslope depression ground water level (h_2') can be determined from equation (60). An example problem in APPENDIX E illustrates the use of the simple ground water model.

<u>Implementation of the Simple Ground Water Model</u>

Implementation of the simple ground water model is not considered to be warranted until comparisons can be made with field observations from several hydrogeologic settings. Results from field observations could guide the development of a rational method for choosing a value for ℓ_2 (equation (48)) which has a profound effect on the predicted ground water level. Results from field observations could also be used to verify the general form of the relationship describing the hillslope depression correction (equation (56)).

REFERENCES

Alto, J.V. 1982. Engineering properties of Oregon and Washington Coast Range soils. M.S. thesis, Oregon State University,

Anderson, M.G. and Burt, T.P. 1978. The role of topography in controlling throughflow generation. Earth Surfaces Processes 3:331-344.

Arend, J.L. 1941. Infiltration as affected by the forest floor. Soil Science Society of America Proceedings 6:430-435.

Aubertine, G.M. 1971. Nature and extent of macropores in forest soils and their influence on subsurface water movement. USFS NE Forest Experiment Station, Research Paper NE-192.

Bell, J.M. 1966. Dimensionless parameters for homogeneous slopes. Soil Mechanics and Foundation Division, ASCE 92(5):51-65.

Bishop, A.W. and Morgenstern, N.R. 1960. Stability coefficients for earth slopes. Geotechnique 10:129-150.

Burroughs, E.R. and Thomas, B.R. 1982. A model of ground water response to precipitation in the Oregon Coast Range. USFS Intermountain Forest and Range Experiment Station Research Paper (unpublished).

Chen, W.F. 1969. Soil mechanics and theorems of limit analysis. Journal of Soil Mechanics and Foundations Division, ASCE 95(2):493-518.

Chen, W.F. and Giger, M.W. 1971. Limit analysis of stability of slopes. Journal of Soil Mechanics and Foundations Division, ASCE 97(1):19-26.

Chow, V.T. 1964. Handbook of Applied Hydrology. McGraw-Hill, New York, NY.

Cousins, B.F. 1978. Stability charts for simple earth slopes. Journal of the Geotechnical Engineering Division, ASCE 104(2):267-279.

Dunn, I.S., Anderson, L.R., and Kiefer, F.W. 1980. Fundamentals of Geotechnical Analysis. John Wiley and Sons, New York, NY.

Dyrness, C.T. 1969. Hydrologic properties of soils on three small watersheds in the Western Cascades of Oregon. USDA For. Serv., Pacific Northwest Forest Range Experiment Station, Research Note PNW-111.

Filz, G. 1982. Engineering properties of Southeast Alaskan forest soils. M.S. thesis, Oregon State University, Corvallis.

Harr, R.D. 1977. Water flux in a soil and subsoil on a steep forested slope. Journal of Hydrology 33:37-58.

Hillel, D. 1971. Soil and Water Physical Principles and Processes. Academic Press, New York, NY.

Hodge, R.A.L. and Freeze, R.A. 1977. Groundwater flow systems and slope stability. Canadian Geotechnical Journal 14:466-476.

Hoeke, E. and Bray, J.W. 1977. Rock Slope Mechanics, Second Edition. Institute of Mineral and Metal, London, England.

Hovland, H.J. 1977. Three-dimensional slope stability analysis method. J. Geotechnical Engineering Division, ASCE 103(9):971-987.

Hunter, J.H. and Schuster, R.L. 1968. Stability of simple cuttings in normally consolidated clays. Geotechinque 18:372-378.

Janbu, N. 1954. Stability analysis of slopes with dimensionless parameters. Harvard Soil Mechanics Series No. 46, Harvard University.

Janbu, N. 1967. (Discussion of) Dimensionless parameters for homogeneous earth slopes by J.M. Bell. J. Soil Mechanics and Foundation Division ASCE 93(6):367-374.

Kirkby, M.J. 1978. Hillslope Hydrology. John Wiley and Sons, New York, NY.

Laliberte, G.E., Brooks, R.H., and Corey, A.T. 1968. Permeability calculated from desaturation data. J. Irrigation and Drainage Division, ASCE 94(1):57-71.

Lo, K.Y. 1965. Stability of slopes in anisotropic soils. J. Soil Mechanics and Foundations Division, ASCE 91(4):85-106.

Morgenstern, N.R. and Price, V.E. 1965. The analysis of the stability of general slip surfaces. Géotechnique 15:79-93.

National Oceanic and Atmospheric Administration, National Weather Service. 1973. Precipitation - Frequence Atlas of the Western United States. NOAA Atlas 2(X).

Ovington, J.D. 1954. A comparison of rainfall in different woodlands. Forestry 27:41-53.

Pierson, T.C. 1980. Peizometric response to rainstorms in forested hillslope drainage depressions. New Zealand Journal Hydrology 19(1):1-10.

Prellwitz, R.W. 1983. Preliminary Instructions for "SSICH". Unpublished.

Prellwitz, R.W., Howard, T.R., and Wilson, W.D. 1983. Landslide analysis concepts for management of forest lands on residual and colluvial soils. Presented as the 62nd annual meeting of the Transportation Research Board, January, 1983.

Ranken, D.W. 1974. Hydrologic properties of soil on a steep forested slope. M.S. thesis, Oregon State University, Corvallis.

Rothacher, J., Dyrness, C.T., and Fredriksen, R.L. 1967. Hydrologic and related characteristics of three small watersheds in the Oregon Cascades. USDA For. Serv., Pacific Northwest Forest Range Experiment Station. 54 p.

Schoeneman, M. 1983. Unpublished laboratory test results for samples obtained in the central Oregon Coast Range.

Sidle, R.C. and Swanston, D.N. 1982. Analysis of a small debris slide in coastal Alaska. Canadian Geotechnical Journal 19(2):167-174.

Sowers, G.B. and Sowers, G.F. 1970. Introductory Soil Mechanics and Foundations, Third Edition. MacMillan Publishing Co., Inc. New York, NY.

Spencer, E. 1967. A method of anlaysis of the stability of the embankments assuming parallel inter-slice forces. Geotechnique 17:11-26.

Swanson, F.J., Swanson, M.M., and Woods, C. 1981. Analysis of debris-avalanche erosion in steep forested lands; an example from Mapleton, Oregon. Symposium on Erosion and Sediment Transport in Pacific Rim Steepland, International Association of Hydrologic Sciences. Christchurch, NZ.

Swanston, D.N. 1967. Soil-water piezometry in a southeast Alaska landslide area. USDA For. Serv., Pacific Northwest Forest Range Experiment Station, Research Note PNW-68.

Taylor, D.W. 1948. Fundamentals of Soil Mechanics. John Wiley and Sons, New York, NY.

Terzaghi, K. and Peck, R.B. 1967. Soil Mechanics in Engineering Practice, Second Edition. John Wiley and Sons, New York, NY.

Twito, R., and Mifflin, R.W. 1982. PLANS: A preliminary logging analysis system. USDA For. Serv., Pacific Northwest Forest Range Experiment Station (Preliminary Draft).

United States Weather Bureau. 1961. Rainfall frequency atlas of the United States for durations from 30 minutes to 24 hours and return periods from 1 to 100 years. Technical Paper No. 40.

United States Weather Bureau. 1964. Two to ten-day precipitation for return periods of 2 to 100 years in the contiguous United States. Technical Paper No. 49.

Voight, G.K. 1960. Distribution of rainfall under forest stands. Forest Science 6:2-10.

Weyman, D.R. 1973. Measurements of downslope flow of water in soil. J. Hydrology 20:267-288.

Whipkey, R.Z. 1965. Subsurface storm flow on forested slopes. Bulletin of the International Association of Science Hydrology 10(2):74-85.

Winterkorn, H.F. and Fang, H.Y. 1975. Foundation Engineering Handbook. Van Nostrand Reinhold Co., New York, NY.

Wright, S.G. 1974. SSTAB1 - A general computer program for slope stability analysis. Research Report No. GE-74-1. Dept. Civil Engineering, University of Texas, Austin.

Wright, S.G., Kulhawy, F.H., and Duncan, J.M. 1973. Accuracy of equilibrium slope stability analysis. J. Soil Mechanics and Foundations Engineering Division, ASCE 99(10):783-793.

Wu, T.H. and McKinnell, W.P. 1979. Strength of tree roots and landslides on Prince of Wales Island, Alaska. Canadian Geotechnical Journal 16:19-33.

Yee, C.S. 1975. Soil and hydrologic factors affecting the stability of natural slopes in the Oregon Coast Range. Ph.D. thesis, Oregon State University, Corvallis.

Zaslavsky, D. and Rogoski, A.S. 1969. Hydrologic and morphologic implications of anisotropy and infiltration in soil profile development. Soil Science Society of America Proceedings 33:594-599.

APPENDICES

APPENDIX A

Janbu Solution

Table 1A. Comparison of Factors of Safety[1]

Case No.	C (psf)	ϕ (degrees)	γ (pcf)	H (ft)	α (degrees)	β (degrees)	H_w' (ft)	SSTAB1[2] FS	Janbu (1954)[2] FS
1	40	40	100	10	33.7	14.0	0	1.90	1.88
2	40	40	100	10	33.7	14.0	1.56	1.49	1.64
3	40	40	100	10	33.7	14.0	3.13	1.06	1.52
4	40	40	100	10	33.7	14.0	4.69	0.86	1.20
5	40	40	100	10	33.7	14.0	6.25	0.72	0.88
6	100	38	100	10	33.7	14.0	3.13	1.83	1.85
7	10	30	100	10	33.7	14.0	1.56	0.80	1.00
8	100	38	100	10	33.7	14.0	6.25	1.33	1.32
9	50	38	100	10	45.0	18.4	0	1.46	1.35
10	50	38	100	10	45.0	18.4	1.67	1.15	1.25
11	50	38	100	10	45.0	18.4	3.33	0.81	1.10
12	50	38	100	10	45.0	18.4	5.0	0.61	1.00
13	150	36	100	10	45.0	18.4	0	2.17	2.03
14	150	36	100	10	45.0	18.4	1.67	1.94	1.95
15	150	36	100	10	45.0	18.4	3.33	1.66	1.74
16	150	36	100	10	45.0	18.4	5.00	1.41	1.62
17	25	36	100	10	45.0	18.4	1.67	0.86	0.95
18	200	35	100	10	53.1	21.8	0	2.13	2.00
19	200	35	100	10	53.1	21.8	3.5	1.70	1.90
20	100	40	100	10	53.1	21.8	3.5	1.10	1.39
21	100	40	100	10	53.1	21.8	5.25	0.78	1.20
22	50	40	100	10	53.1	21.8	1.75	1.02	1.12
23	80	40	100	10	53.1	21.8	3.50	1.00	1.18
24	200	37	100	20	45.0	18.4	3.33	1.56	1.60
25	200	37	100	20	45.0	18.4	6.67	1.26	1.40
26	200	37	100	20	45.0	18.4	10.0	1.01	1.30
27	400	37	100	20	45.0	18.4	13.3	1.53	1.80
28	200	37	100	20	45.0	18.4	13.3	0.82	1.10
29	100	38	100	20	53.1	21.8	3.5	0.99	1.05
30	200	38	100	20	53.1	21.8	7.0	1.05	1.30
31	300	42	100	20	53.1	21.8	10.5	1.15	1.50
32	400	42	100	20	53.1	21.8	14.0	1.19	1.60

(1)Variables as defined in text. β is inclination of bedrock surface which passes through the toe of the slope.

(2)Factor of safety from SSTAB1 computer program and Janbu (1954) solution.

APPENDIX B

Cousins Solution

Table 1B. Comparison of Factors of Safety.

Case No.	c (psf)	ϕ (degrees)	γ (pcf)	H (ft)	$\alpha^{(1)}$ (degrees)	r_u	SSTAB1 FS	Cousins FS
1	200	0	100	10	33.7	0.0	1.42	1.40
2	350	35	100	10	33.7	0.25	3.69	3.68
3	70	35	100	10	33.7	0.25	1.53	1.54
4	14	35	100	10	33.7	0.50	0.59	0.56

(1)α = slope inclination. All other variables as defined in text.

APPENDIX C

Development of RSTAB

Analysis of Planar Failure in Fill

Note: zero pore water pressure assumed for analysis

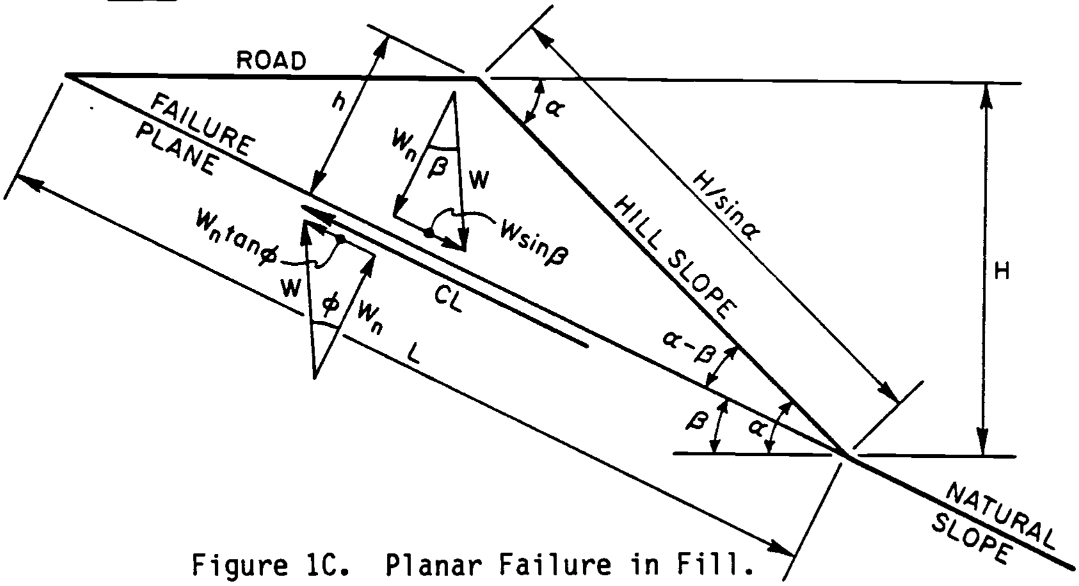


Figure 1C. Planar Failure in Fill.

W = total weight of fill (F)

W_n = normal force on failure plane due to weight of fill (F)

ϕ = maximum angle of internal friction of soil in failure plane (degrees)

c = cohesive strength of soil in failure plane (F/L^2)

γ = unit weight of soil in fill (F/L^3)

From Figure 1c it can be seen that:

$$W = \frac{1}{2} hL\gamma \qquad (1c)$$

From geometry:

$$h = \frac{H}{\sin\alpha} \sin(\alpha-\beta) \qquad (2c)$$

and,

$$L = \frac{H}{\sin\beta} \qquad (3c)$$

By substitution of equation (3c) and (2c) into (1c) and rearranging terms,

$$W = \frac{1}{2} H^2 \frac{\sin(\alpha-\beta)\gamma}{\sin\alpha \sin\beta} \qquad (4c)$$

The forces tending to cause failure of the fill are:

$$\text{forces causing} = W\sin\beta \tag{5c}$$

and the forces tending to resist failure are:

$$\text{forces resisting} = CL + W\cos\beta\tan\phi \tag{6c}$$

when,

$$W_n = W\cos\beta$$

If the factor of safety for stability (FS) is

$$FS = \frac{\text{forces resisting failure}}{\text{forces causing failure}} \tag{7c}$$

then by substitution and rearranging terms

$$FS = \frac{cL}{W\sin\beta} + \frac{\tan\phi}{\tan\beta} \tag{8c}$$

by again substituting and rearranging terms

$$FS = \frac{2c\sin\alpha}{\gamma H\sin(\alpha-\beta)\sin\beta} + \frac{\tan\phi}{\tan\beta} \tag{9c}$$

Table 1C. Change in Factor of Safety with Inclined Bedrock and Top Slope.

Case	$\lambda_{c\phi}$	r_u	Cut Slope Inclination	Bedrock Slope Inclination	SSTAB1 FS	Cousins FS	Error(%)
1	0	0.25	1½:1	4:1	0.93	0.7	+24.7
2	2	0.25	1½:1	4:1	4.38	3.68	+16.0
3	6	0.25	1½:1	4:1	2.11	1.92	+ 9.0
4	10	0.25	1½:1	4:1	1.59	1.54	+ 3.1
5	2	0.0	1½:1	4:1	4.82	4.20	+12.9
6	6	0.0	1½:1	4:1	2.52	2.39	+ 5.2
7	10	0.0	1½:1	4:1	2.00	1.96	+ 2.0
8	2	0.5	1½:1	4:1	3.92	3.29	+16.1
9	6	0.5	1½:1	4:1	1.66	1.50	+19.6
10	10	0.5	1½:1	4:1	1.18	1.11	+ 5.6
11	0	0.25	1:1	4:1	0.71	0.60	+15.5
12	2	0.0	1:1	4:1	3.47	3.50	- 0.9
13	2	0.25	1:1	4:1	3.06	3.04	+ 0.7
14	2	0.5	1:1	4:1	2.66	2.55	+ 4.1
15	0	0.25	1:1	3:1	0.74	0.60	+18.9
16	2	0.0	1:1	3:1	3.50	3.50	0
17	2	0.25	1:1	3:1	3.10	3.04	+ 1.9
18	2	0.5	1:1	3:1	2.71	2.55	+ 5.9
19	6	0.25	1:1	3:1	1.47	1.50	- 1.4
20	0	0.25	3/4:1	3:1	0.58	0.57	+ 1.7
21	2	0.0	3/4:1	3:1	3.01	3.15	- 4.7
22	2	0.25	3/4:1	3:1	2.60	2.70	- 3.9
23	2	0.50	3/4:1	3:1	2.23	2.24	- 0.5
24	6	0.25	3/4:1	3:1	1.29	1.31	- 1.6
25	0	0.25	3/4:1	2:1	0.62	0.57	+ 8.1
26	2	0.0	3/4:1	2:1	3.10	3.15	- 1.6
27	2	0.25	3/4:1	2:1	2.70	2.70	0.0
28	2	0.50	3/4:1	2:1	2.35	2.24	+ 4.7
29	6	0.0	3/4:1	2:1	1.60	1.63	- 1.9
30	6	0.25	3/4:1	2:1	1.23	1.31	- 6.5
31	6	0.50	3/4:1	2:1	0.87	0.95	- 9.3
32	10	0.25	3/4:1	2:1	0.92	0.97	- 5.3
33	20	0.50	1½:1	4:1	0.82	0.81	+ 1.44
34	20	0.25	1½:1	4:1	1.21	1.19	+ 1.62
35	10	0.50	3/4:1	2:1	0.58	0.63	- 8.84

Table 2C. Comparison of FS Between SSTAB1 and Cousins' Solution Using Janbu's H_w' to r_u Relationship.

Case	$\lambda_{c\phi}$	H (ft)	H_w'/H	Cut Slope Inclination	Bedrock Slope Inclination	SSTAB1 FS	Model[(1)] FS	Error %
1	20.9	10	0	1½:1	4:1	1.90	1.94	+ 2.10
2	20.9	10	0.25	1½:1	4:1	1.49	1.74	+16.8
3	20.9	10	0.50	1½:1	4:1	1.06	1.44	+35.8
4	20.9	10	0.75	1½:1	4:1	0.86	1.07	+24.4
5	29.9	10	1.00	1½:1	4:1	0.72	0.69	- 7.2
6	7.8	10	0.50	1½:1	4:1	1.83	1.92	+ 4.9
7	57.7	10	0.25	1½:1	4:1	0.80	0.96	+20.0
8	7.8	10	1.00	1½:1	4:1	1.33	1.20	- 9.8
9	15.6	10	0	1:1	3:1	1.46	1.44	- 1.4
10	15.6	10	0.25	1:1	3:1	1.15	1.30	+13.0
11	15.6	10	0.50	1:1	3:1	0.81	1.07	+32.1
12	15.6	10	0.75	1:1	3:1	0.61	0.78	+27.9
13	4.8	10	0	1:1	3:1	2.17	2.12	- 2.3
14	4.8	10	0.25	1:1	3:1	1.94	1.98	+ 2.1
15	4.8	10	0.50	1:1	3:1	1.66	1.77	+ 6.6
16	4.8	10	0.75	1:1	3:1	1.41	1.46	+ 3.5
17	3.5	10	0	3/4:1	2:1	2.13	2.19	+ 2.8
18	3.5	10	0.50	3/4:1	2:1	1.70	1.70	0
19	8.4	10	0.50	3/4:1	2:1	1.10	1.21	+10.0
20	8.4	10	0.75	3/4:1	2:1	0.78	0.88	+12.8
21	16.8	10	0.25	3/4:1	2:1	1.02	1.15	+12.7
22	10.4	10	0.50	3/4:1	2:1	1.00	1.08	+ 8.0

(1)FS for Cousins' solution using Janbu's H_w' to r_u relationship.

Table 3C. Extending Cousins' Solution Charts

Case	$\lambda_{c\phi}$	c' (psf)	ϕ'	r_u	Inclination	Slope Height (ft)	FS	N_f
1	0	100	0	0.25	3/4:1	10	0.648	6.48
2	10	83.9	40	0.25	3/4:1	10	1.168	13.92
3	50	16.8	40	0.25	3/4:1	10	0.571	34.03
4a	10	83.9	40	0.0	3/4:1	10	1.616	19.26
5a	10	83.9	40	0.25	3/4:1	10	1.168	13.92
6a	10	83.9	40	0.50	3/4:1	10	0.757	9.02
4	10	17.3	30	0.0	3/4:1	3	1.128	19.54
5	10	17.3	30	0.25	3/4:1	3	0.826	14.30
6	10	17.3	30	0.50	3/4:1	3	no run	no run
7	50	50.3	40	0.0	3/4:1	30	0.990	58.99
8	50	50.3	40	0.25	3/4:1	30	0.572	34.08
9	50	50.3	40	0.50	3/4:1	30	0.260	15.49
10	100	16.7	40	0.0	1.5:1	20	1.486	117.0
11	100	16.7	40	0.25	1.5:1	20	1.023	122.0
12	200	8.4	40	0.0	1.5:1	20	1.407	335.0
13	200	8.4	40	0.25	1.5:1	20	0.952	227.0
14	400	4.2	40	0.0	1.5:1	20	1.355	646.0
15	100	16.7	40	0.0	1:1	20	1.060	126.0
16	200	8.4	40	0.0	1:1	20	0.984	235.0
17	400	4.2	40	0.0	1:1	20	0.955	446.0
18	100	16.7	40	0.0	3/4:1	20	0.842	100.0
19	200	8.4	40	0.0	3/4:1	20	0.787	188.0

Note: Bulk unit soil weight, γ, is 100 pcf

Table 4C. Summary of Equations Describing Counsins' Solution Charts

Slope Inclination α (degrees)	r_u	A	B	C
33.69	0	0.4651	1.433	0.9912
45.00	0	0.4240	1.483	0.9085
53.13	0	0.4193	1.445	0.8663
33.69	0.25	0.3930	1.535	0.9494
45.00	0.25	0.3403	1.540	0.8751
53.13	0.25	0.3231	1.460	0.8261
33.69	0.50	0.2773	1.670	0.9269
45.00	0.50	0.2402	1.460	0.8129
53.13	0.50	0.1785	1.730	0.7853

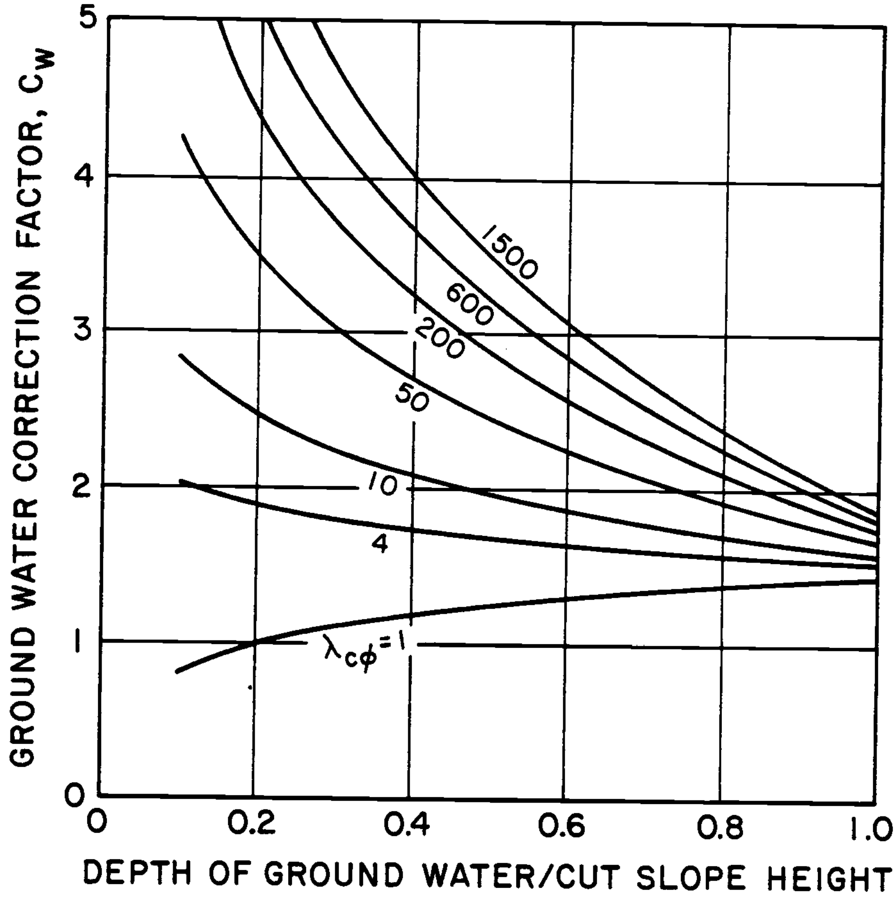


Figure 2C. Ground Water Correction Factor.

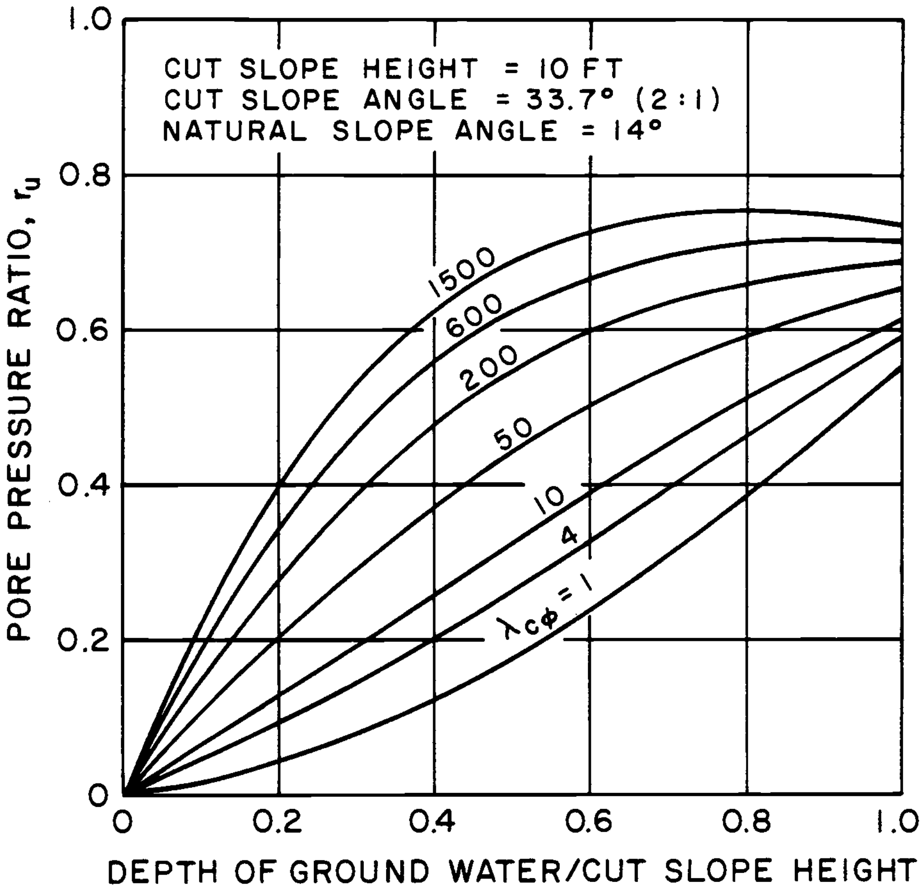


Figure 3C. Ground Water Depth Versus Pore Pressure Ratio.

APPENDIX D

RSTAB

Table 1D. Summary of RSTAB Input Variables.

Name	Definition
Acut	Inclination of cut slope (degrees) Values limited to between 25 and 55 degrees
Afill	Inclination of fill slope (degrees) Values limited to between 25 and 55 degrees
Anatural	Inclination of the natural ground and bedrock surface (degrees)
Ccut	Effective stress soil cohesion in the cut slope (psf)
Cfill	Effective stress soil cohesion in the fill slope (psf)
Cnatural	Effective stress soil cohesion along the fill/natural soil contact (psf)
Gammac	Unit weight of soil in the cut slope (psf)
Gammaf	Unit weight of soil in the fill slope (psf)
Hcut	Vertical height of the cut slope (feet)
Hfill	Vertical height of the fill slope (feet)
Hsoil	Vertical depth of soil (feet)
Hwater	Vertical depth of the groundwater piezometric surface above the bedrock surface (feet)
Phicut	Effective stress angle of internal friction of the soil in the cut slope (degrees)
Phifill	Effective stress angle of internal friction of the soil in the fill slope (degrees)
Phinat	Effective stress angle of internal friction of the soil along the fill/natural soil contact (degrees)

Table 2D. Summary of RSTAB Computation Variables.

Name	Definition
A1	Value of equation (23) for cut slope
A2	Value of equation (23) for fill slope
B1	Value of equation (24) for cut slope
B2	Value of equation (24) for fill slope
C1	Value of equation (25) for cut slope
C2	Value of equation (25) for fill slope
C	Ground water corection factor from equation (34)
Fcut	Factor of safety for cut slope assuming a circular arc failure surface
Ffill	Factor of safety for fill slope assuming a circular arc failure surface
Fplane	Factor of safety for fill slope assuming a planar failure surface along the fill/natural soil contact
Hequiv	The vertical depth of soil above the toe of the cut slope. Equal to the value of equation (37)
Hnet	Equal to the lesser value of either Hsoil or Hequiv
Lambda	Value of $\lambda_{c\phi}$ (equation (16)) for cut slope
Lambdaf	Value of $\lambda_{c\phi}$ (equation (16)) for fill slope
Nfc	Value of equation (22) for cut slope
Nff	Value of equation (22) for fill slope
Mu	Janbu groundwater correction factor (μ_w') from equation (30)
Ru	Pore pressure ratio (r_u) for cut slope from equation (28)

```
10    ! ************************ RSTAB (12-21-83)********************************
20    ! THIS PROGRAM CALCULATES FACTOR OF SAFETY FOR ROAD CUT AND FILL SLOPES
30    ! FOR SLOPES BETWEEN 3:1 AND .75:1 USING 1 EQUATION TO DESCRIBE THREE
40    ! OF COUSINS' SLOPE STABILITY CHARTS(D=1) AND USING A MODIFIED VERSION
50    ! OF JANBU'S GROUND WATER CORRECTION FACTOR .
60    ! ************************************************************ INPUT DATA
70    INPUT "ENTER CUT SLOPE HEIGHT(FEET)",Hcut
80    INPUT "ENTER CUT SLOPE ANGLE(DEGREES)",Acut
90    IF Acut>55 THEN 120
100   IF Acut<25 THEN 150
110   GOTO 180
120   INPUT " ENTER A CUT SLOPE ANGLE LESS THAN OR EQUAL TO 55 DEGREES",Acut
130   INPUT "ENTER THE NEW CUT SLOPE HEIGHT",Hcut
140   GOTO 90
150   INPUT "ENTER A CUT SLOPE ANGLE GREATER THAN OR EQUAL TO 25 DEGREES",Acut
160   INPUT "ENTER THE NEW CUT SLOPE HEIGHT",Hcut
170   GOTO 90
180   INPUT "ENTER CUT SLOPE SOIL DATA,C(PSF),PHI(DEGREES),UNIT WEIGHT(PCF)",Ccut,Phicut,Gammac
190   INPUT "ENTER NATURAL SOIL DEPTH(FEET)",Hsoil
200   INPUT "ENTER GROUND WATER DEPTH IN CUT SLOPE (FEET)",Hwater
210   Hwater1=Hwater
220   INPUT "ENTER NATURAL SLOPE ANGLE(DEGREES)",Anatural
230   IF Anatural>30.966 THEN 430
240   INPUT "ENTER FILL SLOPE HEIGHT(FEET)",Hfill
250   INPUT "ENTER FILL SLOPE ANGLE(DEGREES)",Afill
260   IF Afill>55 THEN 290
270   IF Afill<25 THEN 320
280   GOTO 350
290   INPUT "ENTER A FILL SLOPE ANGLE LESS THAN OR EQUAL TO 55 DEGREES",Afill
300   INPUT "ENTER THE NEW FILL SLOPE HEIGHT",Hfill
310   GOTO 260
320   INPUT "ENTER A FILL SLOPE ANGLE GREATER THAN OR EQUAL TO 25 DEGREES",Afill
330   INPUT "ENTER THE NEW FILL SLOPE HEIGHT",Hfill
340   GOTO 260
350   INPUT "ENTER FILL SLOPE SOIL DATA,C(PSF),PHI(DEGREES),UNIT WEIGHT(PCF)",Cfill,Phifill,Gammaf
360   DEG
370   INPUT "WOULD YOU LIKE TO DO A PLANAR FILL SLOPE ANALYSIS (Y or N)",Zz$
380   IF Zz$[1,1]="N" THEN 440
390   INPUT "DATA FOR SOIL @ FILL/NATURAL SOIL CONTACT,C(PSF),PHI(DEGREES)",Cnat,Phinat
400   ! ********CALCULATE FS FOR PLANAR FAILURE ALONG FILL/NATURAL SOIL CONTACT
410   K1=Gammaf*Hfill*SIN(Afill-Anatural)*SIN(Anatural)  ! EQUATION (20)
420   Fplane=2*Cnat*SIN(Phinat)/K1+TAN(Phinat)/TAN(Anatural)  ! EQUATION (20)
430   ! ****************CALCULATE TRUE CUT SLOPE HEIGHT AND GROUND WATER LEVEL
440   IF Hwater>Hsoil THEN Hwater=Hsoil
450   Hequiv=Hcut*(1-TAN(Anatural)/TAN(Acut))              ! EQUATION (37)
460   IF Hequiv>Hsoil THEN 480
470   IF Hequiv<=Hsoil THEN 510
480   Hcut=Hsoil*(1+TAN(Anatural)/TAN(Acut)/(1-TAN(Anatural)/TAN(Acut)))!EQ(35)
490   Hnet=Hsoil
500   GOTO 550
510   Hwater=Hwater+Hequiv-Hsoil                           ! EQUATION (36)
520   IF Hwater<0 THEN Hwater=0
530   Hnet=Hequiv
540   ! *********************************************CALCULATE Lambda AND Lambdaf
550   Lambda=Gammac*Hcut*TAN(Phicut)/Ccut                  ! EQUATION (16)
560   IF Lambda<=1 THEN Lambda=1
570   IF Anatural>30.966 THEN 610
580   Lambdaf=Gammaf*Hfill*TAN(Phifill)/Cfill              ! EQUATION (16)
590   IF Lambdaf<=1 THEN Lambdaf=1
```

Figure 1D. Listing of RSTAB.

```
600  ! *************************CALCULATE GROUND WATER CORRECTION FOR CUT SLOPE
610  IF Hwater=0 THEN GOTO 650
620  C=1.438+.633*LGT(Hwater/Hnet)+(.129-1.899*LGT(Hwater/Hnet))*LGT(Lambda)
630  Hwater=Hwater*C                                     ! EQUATION (34)
640  ! ******************** CALCULATE Ru FOR CUT SLOPE FROM GROUND WATER DEPTH
650  Mu=1+(.1+(Afill-45.0)/1125)*SIN(180.0*(1+Hwater/Hcut)) ! EQUATION (30)
660  Ru=1-1/Mu+62.4*Hwater/Hcut/(Gammac*Mu)              ! EQUATION (28)
670  Ruf=0
680  ! **********************************************CALCULATE Nf FOR CUT SLOPE
690  A1=(.31813-.33057*Ru)^.65-.0046*(Acut-33.69)        ! EQUATION (23)
700  B1=(1.206+.129*Ru)^2                                ! EQUATION (24)
710  C1=1-.2*Ru-.0506/Acut^.5323*(Acut-33.69)            ! EQUATION (25)
720  Nfc=10^(A1*LGT(Lambda)^B1+C1)                       ! EQUATION (22)
730  ! **********************************CALCULATE FACTOR OF SAFETY FOR CUT SLOPE
740  Fcut=Nfc*Ccut/(Gammac*Hcut)                         ! EQUATION (17)
750  ! ********************************************CALCULATE Nf FOR FOR FILL SLOPE
760  IF Anatural>30.966 THEN 840
770  A2=(.31813-.33057*Ruf)^.65-.0046*(Afill-33.69)      ! EQUATION (23)
780  B2=(1.206+.129*Ruf)^2                               ! EQUATION (24)
790  C2=1-.2*Ruf-.0506/Afill^.5323*(Afill-33.69)         ! EQUATION (25)
800  Nff=10^(A2*LGT(Lambdaf)^B2+C2)                      ! EQUATION (22)
810  ! *********************************CALCULATE FACTOR OF SAFETY FOR FILL SLOPE
820  Ffill=Nff*Cfill/(Gammaf*Hfill)                      ! EQUATION (17)
830  ! *********************************************OUTPUT DATA AND FACTOR OF SAFETY
840  FIXED 2
850  PRINT "*************************INPUT DATA*******************************"
860  PRINT "CUT SLOPE HEIGHT :                        ";Hcut;"FEET"
870  PRINT "CUT SLOPE ANGLE :                         ";Acut;"DEGREES"
880  PRINT "CUT SLOPE SOIL COHESION :                 ";Ccut;"PSF"
890  PRINT "CUT SLOPE SOIL FRICTION ANGLE :           ";Phicut;"DEGREES"
900  PRINT "CUT SLOPE SOIL UNIT WEIGHT :              ";Gammac;"PCF"
910  PRINT "NATURAL SOIL DEPTH :                      ";Hsoil;"FEET"
920  PRINT "GROUND WATER DEPTH IN CUT SLOPE :         ";Hwater1;"FEET"
930  PRINT "NATURAL SLOPE ANGLE :                     ";Anatural;"DEGREES"
940  IF Anatural>30.966 THEN 1030
950  PRINT "FILL SLOPE HEIGHT :                       ";Hfill;"FEET"
960  PRINT "FILL SLOPE ANGLE :                        ";Afill;"DEGREES"
970  PRINT "FILL SLOPE SOIL COHESION :                ";Cfill;"PSF"
980  PRINT "FILL SLOPE SOIL FRICTION ANGLE :          ";Phifill;"DEGREES"
990  PRINT "FILL SLOPE SOIL UNIT WEIGHT :             ";Gammaf;"PCF"
1000 IF Zz$[1,1]="N" THEN 1030
1010 PRINT "FILL/NATURAL SOIL COHESION :              ";Cnat;"PSF"
1020 PRINT "FILL/NATURAL SOIL FRICTION ANGLE :        ";Phinat;"DEGREES"
1030 PRINT "***********************OUTPUT DATA*******************************"
1040 PRINT "FACTOR OF SAFETY FOR CUT SLOPE :            ";Fcut
1050 IF Anatural>30.966 THEN 1070
1060 GOTO 1090
1070 PRINT "NO FILL SLOPE ANALYSIS FOR NAT SLOPES > 30.966 DEGREES"
1080 GOTO 1120
1090 PRINT "FACTOR OF SAFETY FOR FILL SLOPE :           ";Ffill
1100 IF Zz$="N" THEN 1120
1110 PRINT "FACTOR OF SAFETY FOR PLANAR FAILURE IN FILL:";Fplane
1120 END
```

Figure 1D. Listing of RSTAB (continued).

APPENDIX E

Ground Water Model

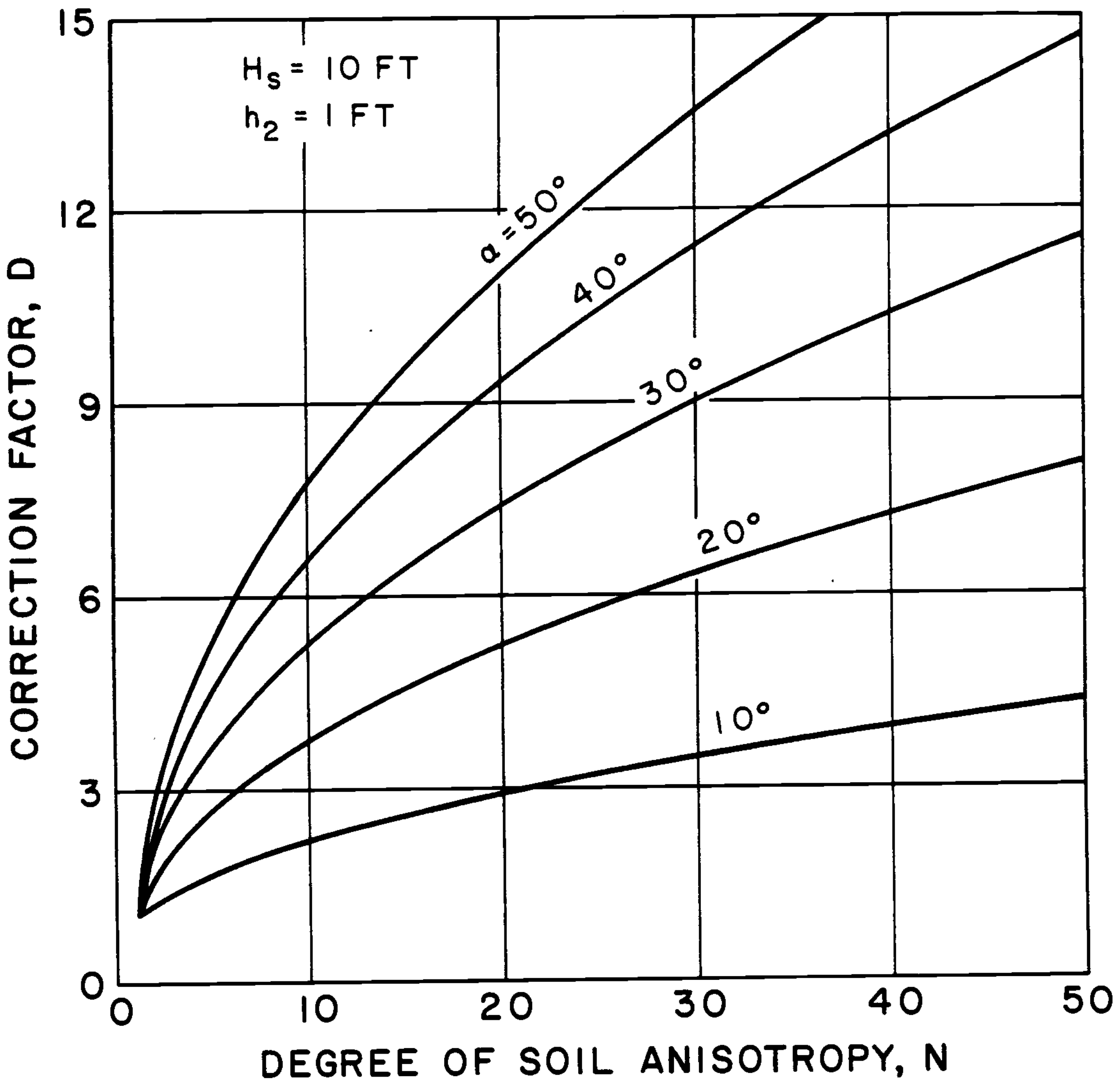


Figure 1E. Hillslope Depression Correction Factor.

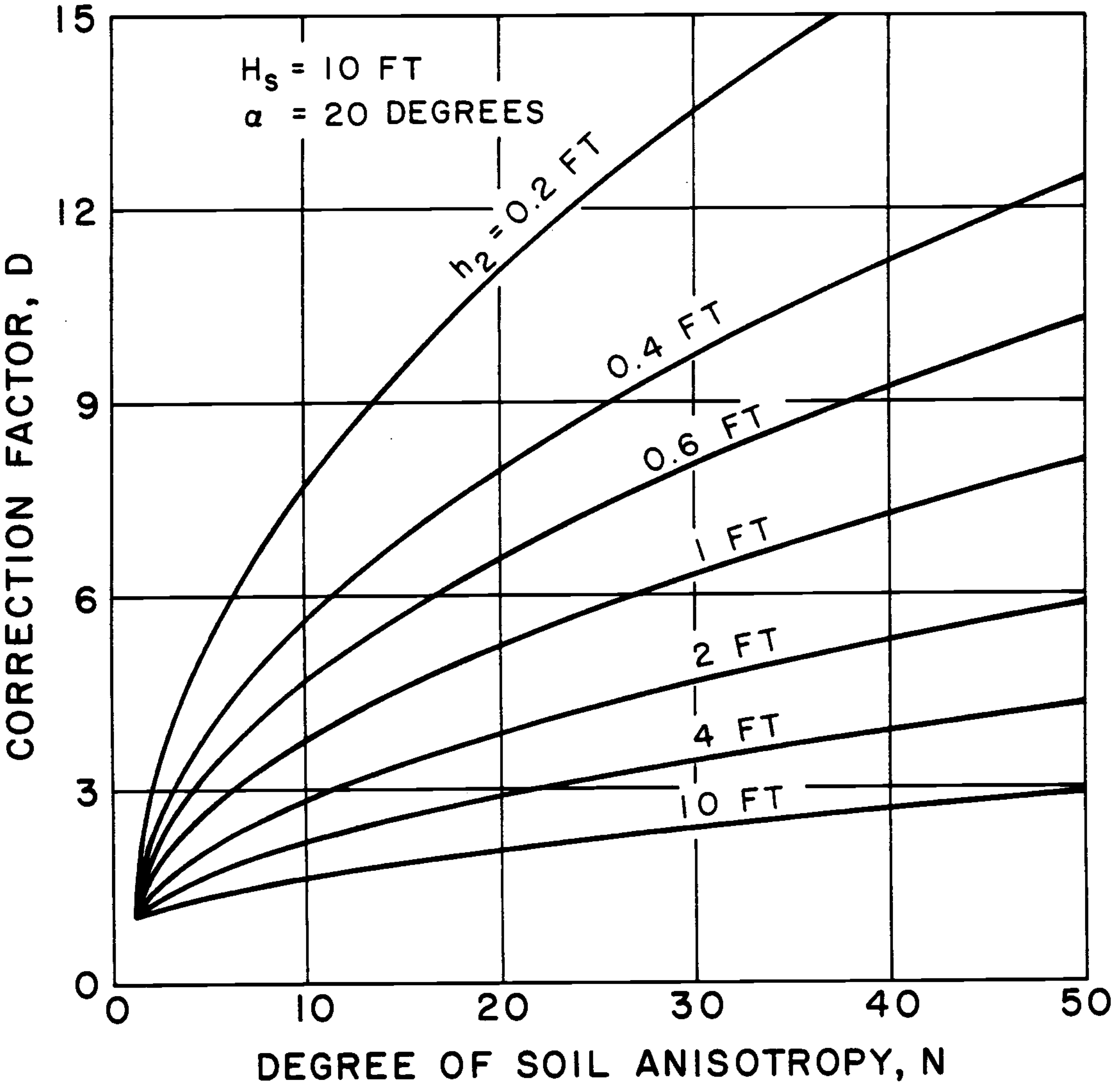


Figure 2E. Hillslope Depression Correction Factor.

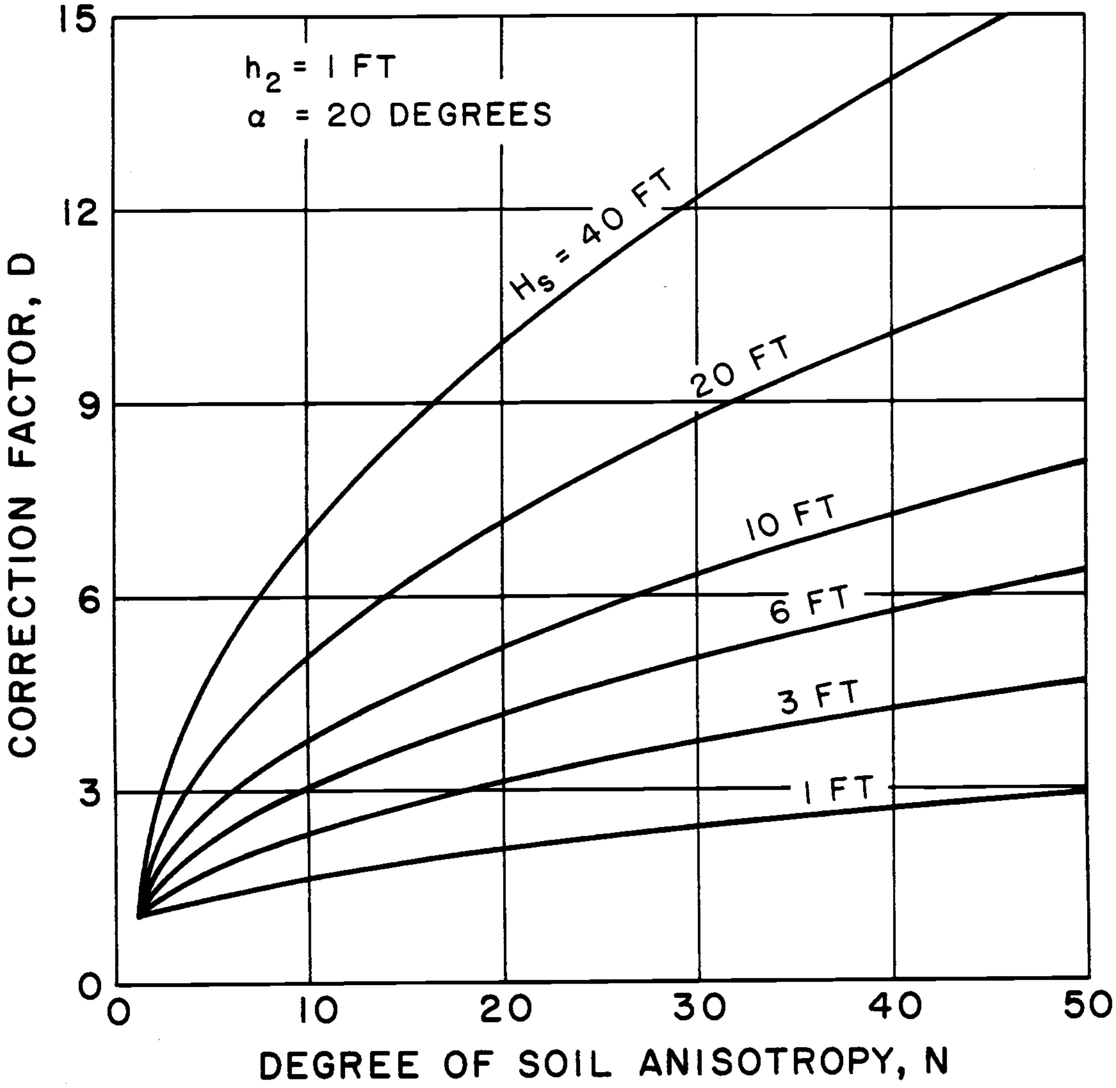


Figure 3E. Hillslope Depression Correction Factor.

Example Problem Using the Simple Ground Water Model

As shown in Figure 4E, the drainage area begins at the boundary of the drainage basin and converges into a hillslope depression at the bottom. The width of the drainage area at the base was arbitrarily chosen to be 40 feet. The boundaries of the basin were then drawn, beginning at the bottom of the drainage area, perpendicular to the elevation contours. The drainage area was idealized as a serie of quadrangular shapes as shown in Figure 5E. The quadrangles were then idealized as symmetric trapezoids as shown in Figure 6E. The trapezoids shown in Figure 6E were used in the determination of the ground water level as follows:

Assume field reconnaisance indicated an average of 8 feet of soil overlying bedrock. Assume soil tests indicated the following properties:

a. the saturated hydraulic conductivity of the soil near the bedrock is 8 inches/hour

b. the saturated hydraulic conductivity decreases by a factor of 20 between the ground surface and the soil near the bedrock. Therefore K_{ave} = 80 in/hr

c. the average porosity of the soil is 60%.

1. Determine the time of concentration, t_c'

From equation (54) $t_c' = \frac{nL}{k\sin\alpha}$

where, $n = 0.60$

L = 384 ft + 220 ft + 200 ft = 804 ft

k = 8 inches/hour = 16 ft/day

α = arctan ((2260-1800)/804) = 47 degrees

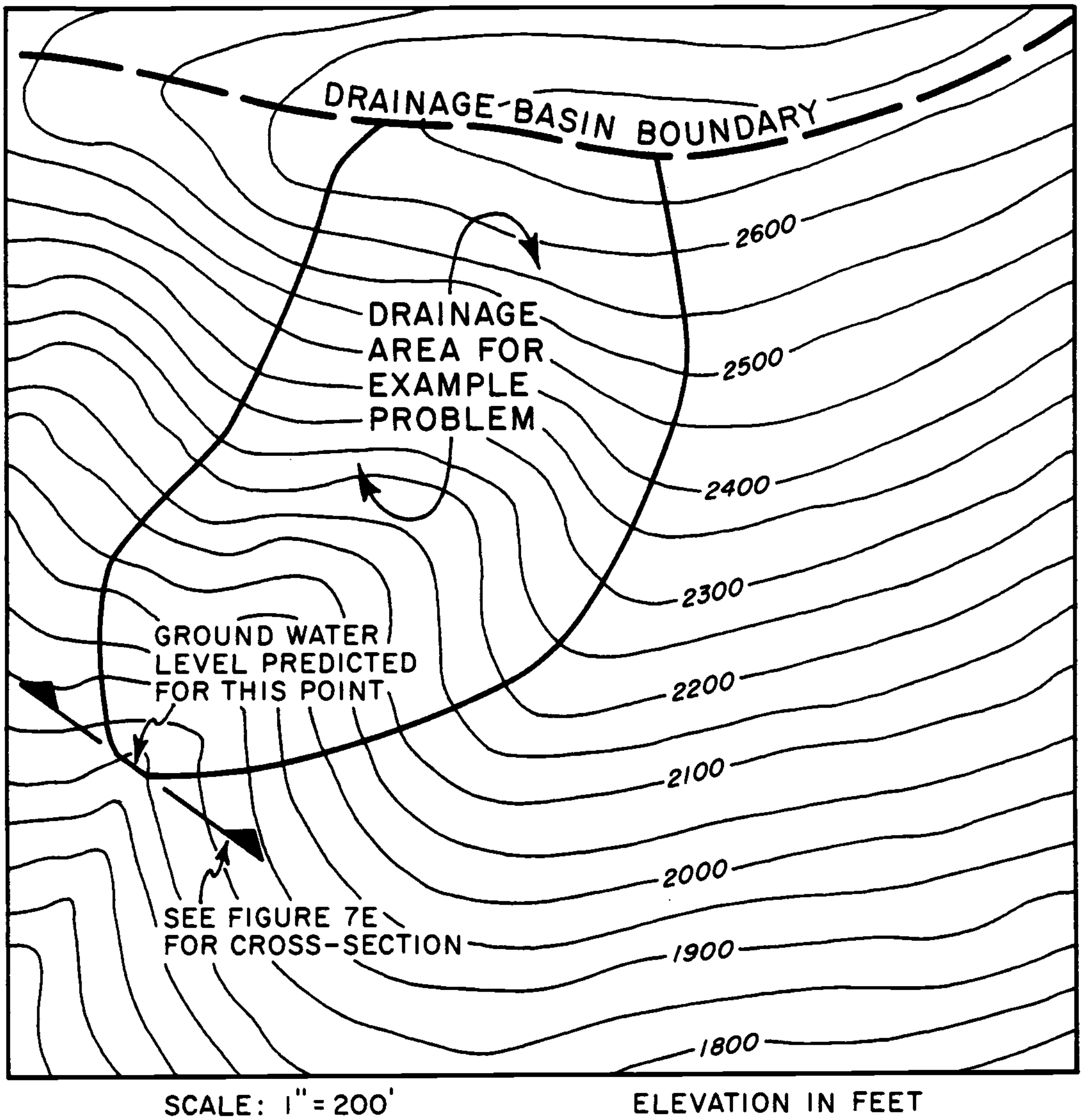


Figure 4E. Drainage Area for Example Problem.

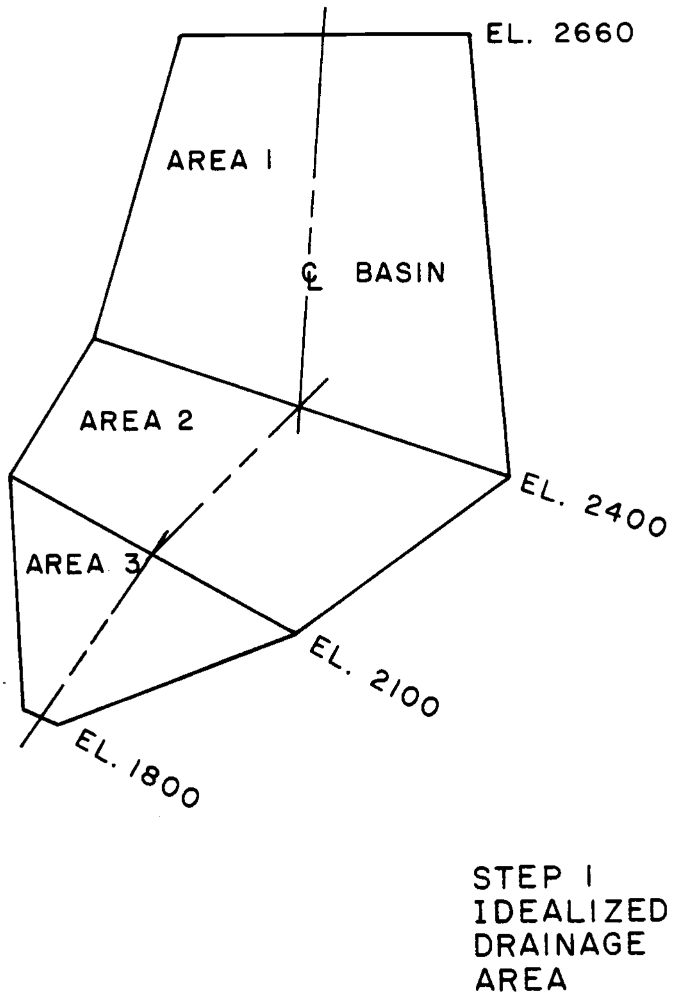


Figure 5E. Idealized Drainage Area-Step 1.

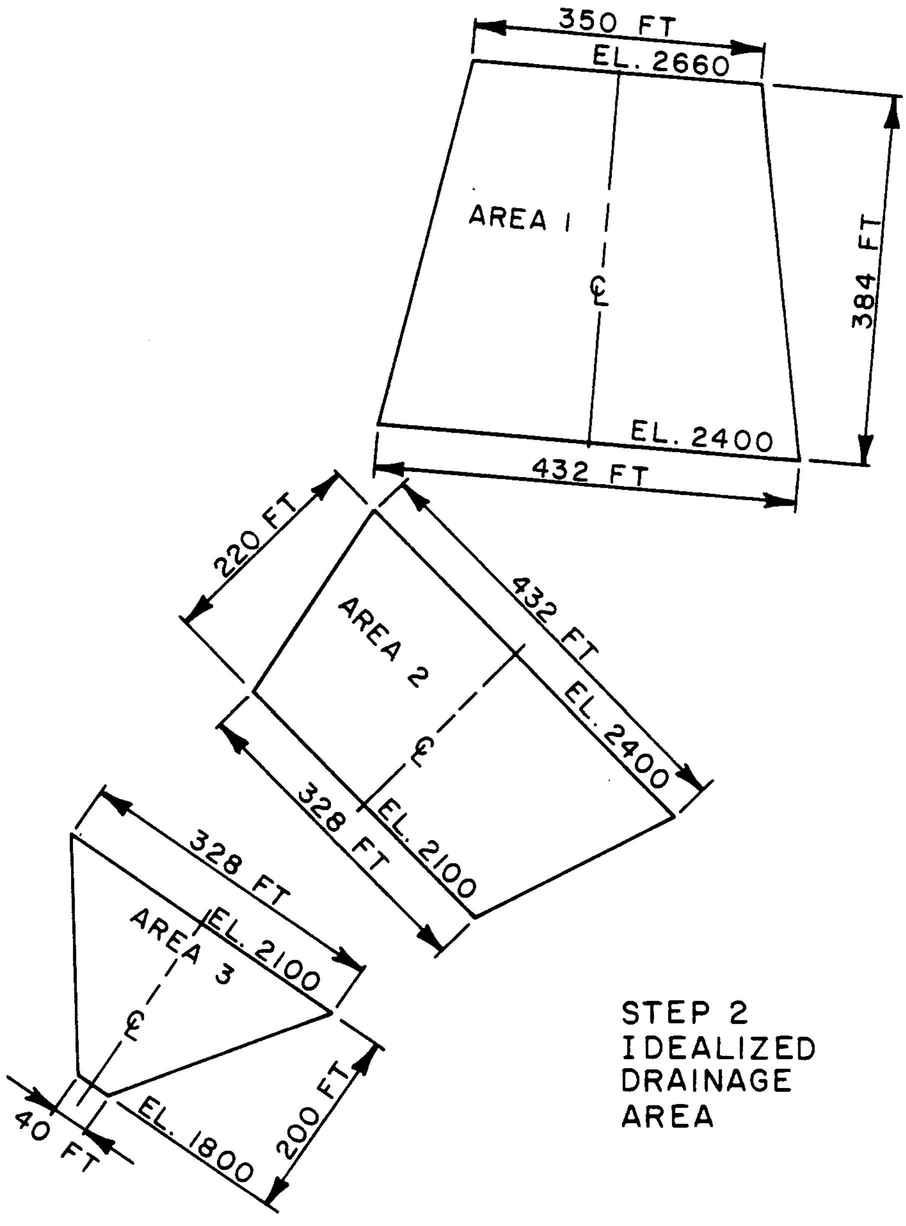


Figure 6E. Idealized Drainage Area - Step 2.

then,

$$t_c' = \frac{(0.60)(804\ \text{ft})}{(16\ \text{ft/day})(\sin 47^o)} = 41\ \text{days}$$

2. Determine the long-term precipitation rate

Using t_c' = 41 days and extrapolating from Figure 15,

I = 0.05 inches/hour for a 100 year recurrance interval

3. Determine groundwater level at the lowest end of Area 1 (Figure 6E).

Using equation (51) $h_2 = \frac{h_1 \ell_1}{\ell_2} + \frac{I}{2K \sin\alpha} \left(\frac{\ell_1 + \ell_2}{\ell_2}\right) X$

where, h_1 = 0 at the drainage basin boundary

ℓ_1 = 350 ft

ℓ_2 = 432 ft

I = 0.03 inches/hour

K = 8 inches/hour

α = arctan ((2660-2400)/384) = 34 degrees

X = 384 ft

then,

$$h_2 = \frac{(0)(350\text{ft})}{432\text{ft}} + \frac{(0.03\text{in/hr})}{(2)(80\ \text{in/hr})(\sin 34^o)} \left(\frac{350\text{ft}+432\text{ft}}{432\text{ft}}\right)(384\text{ft})$$

and h_2 = 0.23 ft

4. Determine ground water level at the lower end of Area 2 (Figure 6E).

Using equation (51), h_1 = 0.23ft,

then,

$$h_2 = \frac{(0.23\text{ft})(432\text{ft})}{(328\text{ft})} + \frac{(0.03\text{in/hr})}{(2)(80\text{in/hr})(\sin 54^o)} \left(\frac{432\text{ft} + 328\text{ft}}{328\text{ft}}\right)(220\text{ft})$$

h_2 = 0.30 ft + 0.12 ft = 0.42 ft.

5. Determine groundwater level at the lower end of Area 3 (Figure 6E).

Using equation (51),

$$h_2 = \frac{(0.42\text{ft})(328\text{ft})}{40\text{ft}} + \frac{(0.03\text{in/hr})}{(2)(80\text{in/hr})(\sin 56^{\circ})}\left(\frac{328\text{ft} + 40\text{ft}}{40\text{ft}}\right)(200\text{ft})$$

$$h_2 = 3.44\text{ ft} + 0.42\text{ ft} = 3.86\text{ ft}$$

6. Determine the hillslope depression factor, D

Using equation (56) $\beta = \arctan\left(\frac{1}{N\tan\alpha}\right)$

where, α = 45 degrees from Figure 7E

$N = 20$

$$\beta = \arctan\left(\frac{1}{(20)\tan 45^{\circ}}\right) = 2.86\text{ degrees}$$

Using equation (62)

$$D = \tfrac{1}{2} + \tfrac{1}{4}\left(\frac{H_s \tan\alpha}{h_2(\tan(\alpha+\beta) - \tan\alpha)}\right)^{1/2}$$

where, H_s = 8 feet

α = 45 degrees

β = 2.86 degrees

h_2 = 3.86 ft

$$D = \tfrac{1}{2} + \tfrac{1}{4} + \left(\frac{(8\text{feet})(\tan 45^{\circ})}{(3.86\text{ft})(\tan(45^{\circ}+2.86^{\circ})-\tan 45^{\circ})}\right)^{1/2}$$

$$D = 4.97$$

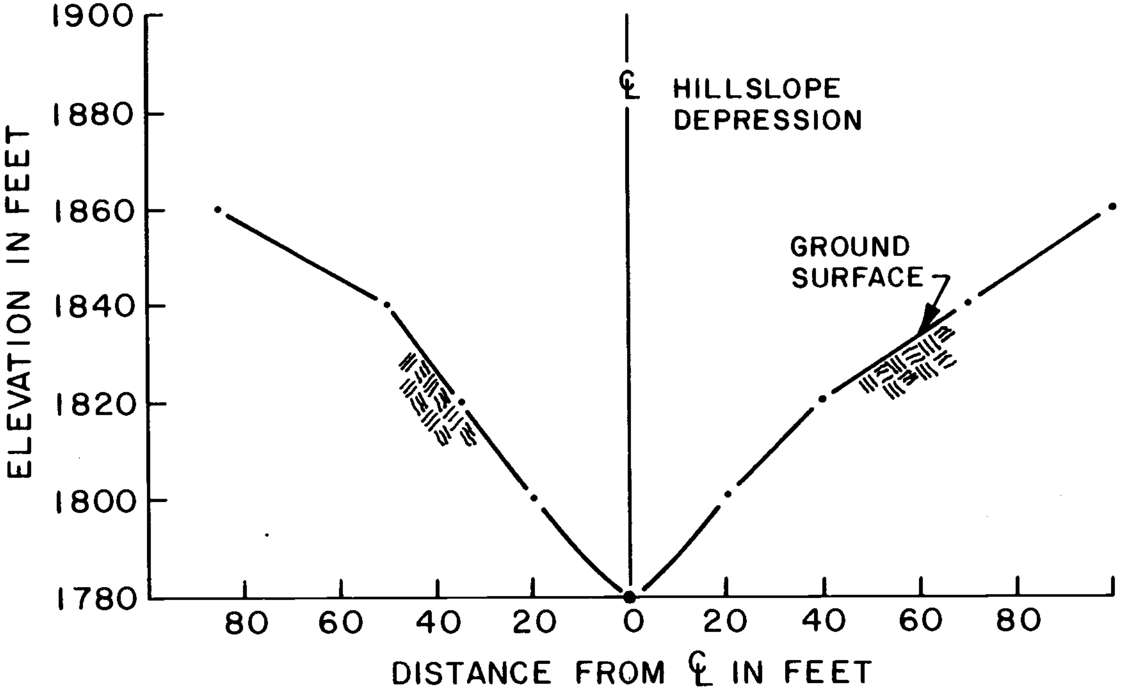


Figure 7E. Cross Section of Hillslope Depression.

7. Determine the corrected groundwater level for Case 1.

Using equation (56) $h_2' = h_2 D$

then,

$$h_2' = (3.86\text{ft})(4.97) = 19.2 \text{ feet}$$

The value of h_2' is greater than the soil depth, therefore, the soil is saturated to the ground surface and some overland flow is likely in the hillslope depression. Note that an even greater value of h_2' would have been predicted if a more realistic width had been chosen for the saturated zone at the bottom of the drainage area. Say, 16 feet rather than the 40 foot width used in the problem.