To the Graduate Council:

I am submitting herewith a thesis written by James Daniel Cowan entitled "Analysis of Landslides and Slope Stability in Coal Surface Mining." I recommend that it be accepted in partial fulfillment of the requirements for the degree of Master of Engineering, with a major in Civil Engineering.

R. Dan Marks, Major Professor

We have read this thesis
and recommend its acceptance:

Accepted for the Council:

Vice Chancellor
Graduate Studies and Research

ANALYSIS OF LANDSLIDES AND SLOPE STABILITY IN COAL SURFACE MINING

A Thesis

Presented for the

Master of Science

Degree

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James Daniel Cowan

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ABSTRACT

This investigation reviewed the literature pertaining to methods of slope stability analysis developed for civil engineering works. Particular emphasis was placed on those methods of analysis which are applicable to coal surface mine overburden waste fills typical of contour mining in the mountainous Appalachian region.

Using Bishop's Simplified Method of Slices and soil perameters determined by laboratory triaxial and direct shear strength tests, three existing landslides were analyzed to establish the reliability of the analysis procedure in predicting slope failures. After establishing the reliability of the analysis procedure, alternate cross sections were analyzed to evaluate surface mining fill bench regulations, and to determine the effects of various factors on stability of large overburden waste fills. The investigation determined that cross sections other than those specified by current Tennessee Surface Mine Regulations can result in higher factors of safety against sliding while possibly reducing coal production costs.

It was also determined that for large overburden waste fills two potential failure surfaces exist with nearly equal factors of safety. Those two failure surfaces correspond to: 1) the interface between the fill and foundation colluvium; and 2) the interface between the colluvium and intact rock. The controlling failure surface is determined by the relative strength between the waste fill material and the colluvium, and by the size of the fill. The colluvium-rock potential failure plane controlled for most large fills investigated. When the colluvium-rock

failure plane controlled, the depth of the colluvium layer was an important factor in determining the stability of the fill. A deeper colluvium layer resulted in a less stable fill.

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CHAPTER I

INTRODUCTION

Surface mined coal has long been a source of inexpensive energy in the Appalachian region. However, many environmental costs associated with surface mining of coal were not included as a direct cost of the coal. With the tremendous increases in demand for energy and the dwindling supply of oil and natural gas, for the next several years coal is likely to increase as a source for supplying that demand. Because of the heightening concern for preserving environmental quality, there is a need for evaluating mining procedures with two conflicting requirements of maximizing production and minimizing environmental degradation in mind.

Two types of surface mining are utilized in Tennessee for recovery of coal resources. The first type is referred to as area surface mining. Area surface mining is utilized predominantly in the southern portion of the Cumberland Plateau which consists of gently rolling or flat terrain. Contour surface mining is utilized predominantly in northern mountainous portions of the Cumberland Plateau known locally as the Cumberland Mountains. According to Tennessee surface mining regulations, all surface mining on slopes steeper than 15^{o} is defined as contour mining. Contour mining involves removal of overburden from coal seams which outcrop and continue along a given contour or elevation on a mountain.

One of the major problems associated with coal surface mining in the mountainous Appalachian region is disposal or storage of large

volumes of overburden waste material (often referred to as "spoil") generated during the mining process. Prior to development of surface mining and environmental regulations, dumping of all overburden onto the mountain slope below the coal seam elevation was common practice. Spoil piles placed on steep mountain slopes by dumping or loose storage have been a source of siltation, acid water runoff, and landslides. Unstable spoil piles have defied attempts to revegetate and reclaim many surface mined areas. Consequently, siltation problems continue to exist many years after termination of active mining.

State regulations have been promulgated to lessen the environmental effects of surface mining. Several different approaches have been attempted in regulating the size and location of overburden storage areas in order to assure their stability. Some different approaches include: limiting total bench width (including the width of the cut portion); limiting the percent of overburden removed which can be placed in a slope fill; limiting the steepness of natural slope upon which overburden can be placed; and limiting the distance to which a fill bench can extend down a slope from the coal seam elevation. Surface mine regulations which became effective July 1, 1975 in Tennessee have adopted a combination of these approaches. Fill benches are restricted by limiting overburden storage to less than 28^{o} natural slopes. On slopes less than 28^{o}, overburden may be placed down the slope a maximum of fifty feet measured from the location of the coal seam outcrop.

Prior to July 1, 1975, overburden was allowed to extend a maximum of 125 feet down the slope from the coal seam outcrop on less than 28^{o} natural slopes. Current regulations and those effective immediately prior

to July 1, 1975 limited the angle of the fill slope to 35^{o} which is referred to as the "natural angle of repose" of spoil.

Numerous fill slope failures occurred in surface mines which were in compliance with regulations effective prior to July 1, 1975. Since July 1, 1975 and the adoption of stricter fill bench constraints, the percentage of failures has markedly decreased. However, no analysis was made to determine whether current regulations are overconservative resulting in excessive costs, or whether an alternative grading configuration could yield comparable or greater stability with less mining cost.

By applying methods of slope stability analysis developed for civil engineering works, this research attempts to study mechanisms involved in spoil slope failures in order that current regulations and mining procedures can be qualitatively evaluated. Specifically, three surface mines which have experienced slope failures in the constructed overburden fill benches were studied to determine causes of failures, and to establish reliability of the analysis procedure in predicting failures. Once the reliability of the analysis procedure was established, different factors involved in stability were varied and the slopes reanalyzed to determine the effect on stability. In this manner, an optimum design for fill bench construction was obtained. The optimum design was then compared with existing surface mine regulations. Results of this research indicate that grading configurations other than those specified in the current surface mining regulation can lead to increased slope stability with probable increased economy of production and reduced environmental degradation in the form of reduced siltation. The study also points out the need for increased application of existing technologies developed in the various branches of engineering to surface mining problems.

CHAPTER II

LITERATURE REVIEW

I. INTRODUCTION

Stability analyses of natural and man-made earth slopes are often associated with many simplifying assumptions resulting from complexities associated with geological conditions and complexities. In the field of soil mechanics or geotechnical engineering, the degree of complexity and variability of materials is greater than in many other fields of engineering where quality of materials of construction is often closely controlled during production. Uncertainties involved in stability analyses are accounted for by factors of safety. The magnitude of factors of safety utilized depends upon the degree of uncertainties involved, and upon consequences which would result from failure of the structure.

In analysis of the stability of earth slopes, an idealized central cross section of unit thickness is normally assumed. The investigation is reduced to a two-dimensional plane strain problem. The effects of end restraints are commonly assumed negligible. Since the effects of end restraints would tend to prevent sliding, these assumptions are conservative. The degree to which neglecting side forces is conservative depends upon the ratio of crest width of a slope to the downslope length and depth of the critical failure surface.

The soil is usually considered to be homogeneous in directions normal to the cross section. Strength parameters and pore pressures distributions are commonly assigned to the cross section based on a combination of in-situ and/or laboratory testing. Sampling and testing methods used should be designed to reproduce as closely as possible stress conditions which exist in the field situation. The degree to which strength parameters and pore pressures assigned to the cross section approximate those existing in the field depends upon extent of sampling, homogeneity of the soil mass, duplication of in-situ stress conditions, loading rates, and proper choice of shear strength test methods. The degree to which the uncertainties can be quantitatively and qualitatively evaluated dictates the magnitude of the factor of safety required. The complexity of the problem dictates that some conditions must be ignored or assumed negligible, however, it is never advantageous to ignore a condition because the condition is not known to exist (Parcher 1968)*.

Most currently popular methods of slope stability analysis employ the assumption of limit equilibrium. The soil is assumed to be in a state of plastic equilibrium. A "free body diagram" of a soil mass, bounded by the surface of the slope on top and by some assumed failure or critical shear surface on the bottom, is analysed using equations of statics. The factor of safety in such an analysis is commonly defined as the ratio of available shear strength of the soil to the shear resistance required to maintain equilibrium. Therefore, at any point on a potential shear surface (Bishop 1955)

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*The surname of the author with year of the publication in parentheses refer to those entries in the List of References.

$$s = \frac{1}{F_s} (c + \sigma \tan \phi).$$

where s is the shear strength mobilized. The shear strength parameters are those defined by the Mohr-Coulomb shear strength theory, where c is cohesion, ϕ is the angle of internal friction, and σ is the normal stress on the plane of failure. The factor of safety, F_s, is the factor of safety with respect to shear strength. The shear strength of the soil at a point along the failure surface is dependent upon the normal stress which exists at that particular point. Determination of normal stresses along the failure surface is a statically indeterminant problem. In order to mathematically determine normal stresses on the failure surface, a stress analysis involving stress-strain characteristics of the soil would be required. Such analyses, which have been attempted using theories of elasticity and plasticity, are highly desirable (Jurgenson 1934, Bishop 1952). However, at the present state of the art, complexities and variabilities associated with most geological profiles require simplifying assumptions which render such an analysis little, if any, improvement over currently popular methods using limit equilibrium. A distinct disadvantage of stress-deformation methods is the increase in number of mathematical computations required (Parcher 1968).

The alternative to a rigorous stress-strain analysis is to make simplifying assumptions regarding the stresses in the soil mass. In methods of analysis which consider the soil mass as a unit, an assumption is made regarding distribution of normal stresses along the failure surface.

In methods of analysis which divide the soil mass above the shear surface into a number of slices, an assumption is made regarding forces which act between slices. Assumptions involving "interslice forces," allow normal forces on the failure surface to be calculated using equations of statics.

In limit equilibrium methods of analysis, a number of potential shear surfaces are analyzed to determine the factor of safety against shear failure. The shear surface which produces the minimum factor of safety is considered to be the critical failure surface.

In the early development of stability analysis, failure surfaces were assumed to be planar or series of plane surfaces. This type analysis is often referred to as the Wedge Method. The Wedge Method is still applicable in certain stability problems where potential failure surfaces are dictated by specific planes of weakness. However, the observation of many slices with curved failure surfaces resulted in development of stability analysis procedures which considered circular arc failure surface.

Slope stability analysis procedures which assume circular arc shaped failure surfaces are known as Swedish Methods. Swedish Methods can be divided into two procedural groups. The first procedure assumes that the soil mass above the failure surfaces acts as a mass unit. The conditions of static equilibrium are applied to the entire soil mass bounded by the failure surface. The second procedure assumes the soil mass to be divided into a number of slices. Conditions of static equilibrium are applied to the individual slices and then summed for the entire mass. More recently, the

slices procedure has been extended to failure surfaces which have no restrictions placed on their shape. The methods are referred to as Generalized Method of Slices.

It should be noted that there has been an extremely vast amount of literature published on slope stability analysis procedures and their evaluation. The purpose of this review is not to present a comprehensive critique. However, major developments and the most widely used methods of analysis are reviewed. Particular emphasis is placed on those methods which are most applicable to the analysis of landslides and slope failures studied throughout this research.

II. SLOPES STABILITY ANALYSIS PROCEDURES

Wedge Method

The Wedge Method of analysis is commonly attributed to Culmann (1866). The method was initially developed to determine earth pressures on retaining wall structures. In the Wedge Method, potential failure surfaces are assumed to be planar or a combination of two or more intersecting planes. In the most simple case of the single plane failure surface, forces involved in stability are weight of the soil wedge, cohesion of the soil, and the resultant of the normal force and the force produced by the internal friction of the soil. Unit cohesion, c, is assumed to act parallel to the planar failure surface. The resultant of the normal force and the force produced by internal friction of the soil act at an angle ϕ to a line perpendicular to the plane. The values of cohesion, c, and the internal friction, ϕ, are those associated with the Mohr-Coulomb

failure model. By applying the laws of static equilibrium, the critical slope of the failure plane and the corresponding cohesion required for stability can be determined.

The natural occurrence of a plane failure surface in a nearly homogeneous material is very improbable. Observation of many failed slopes indicate that the failure surfaces are more nearly approximated by curved surfaces. In a homogeneous material, the Wedge Method approximates reality in cohesive soils only in cases of vertical or near vertical slopes. For flatter slopes in homogeneous material the failure surfaces are likely to be more curved. The assumption of a plane failure where a curved surface exists leads to large errors on the unsafe side. For practical purposes, the Wedge Method is restricted to those cases involving nonhomogeneous material where failure surfaces are determined by the presence of specific planes of weakness.

The Wedge Method has been utilized in stability analysis of nonhomogeneous material such as those encountered in sloping core earth dams (Sulton and Seed 1967) where plane surfaces of failure are dictated by the non-homogeniety of the soil mass. In such cases, the failure surface is approximated by two or more intersecting failure planes. The Wedge Method has also been utilized in analyzing the stability of surface mine spoil banks (Huang 1977), where it was assumed that the failure surface coincides with the surface of the natural slope which forms a foundation for the spoil bank. Huang concluded that plane failure surfaces are possible in cases where material comprising the surface of the natural slope is lower strength than the spoil in the fill bench. Plane failure surfaces

are most likely in cases where the natural slope is steep. For cases involving potential plane failure surfaces, the maximum allowable height of the slope is strongly dependent upon the base width of the spoil bank, and only slightly dependent on the degree of slope of the spoil bank.

ϕ - Circle Method

The development of the ϕ - Circle Method of slope stability analysis is normally credited to W. Fellenius (1927) and H. Krey (1932). The friction circle is sometime referred to as the Krey-Circle especially in problems involving earth pressures behind retaining structures. A paper by Taylor (1937) promoted utilization of the ϕ - Circle Method by providing a mathematical rather than graphical solution to the problem. The ϕ - Circle Method, as used by Taylor, assumes a potential failure surfaces of circular arch shape of radius, R. (See Figure 1.) The Soil strength parameters, represented by a value of cohesion, c, and internal friction, ϕ, are constant over the entire length of the potential shear surface. In other words, the soil is assumed homogenious through out.

The ϕ - Circle analysis is used to calculate a factor of safety with respect to cohesion only. The frictional resistance of the soil is assumed to be fully developed along the entire length of the potential shear surface. This assumption sets the factor of safety with respect to internal friction at unity; that is:

$$F_{\phi} = \frac{\tan \phi}{\tan \phi_1} = 1$$

where ϕ_1 is the developed angle of internal friction. The factor of safety with respect to cohesion is defined as:

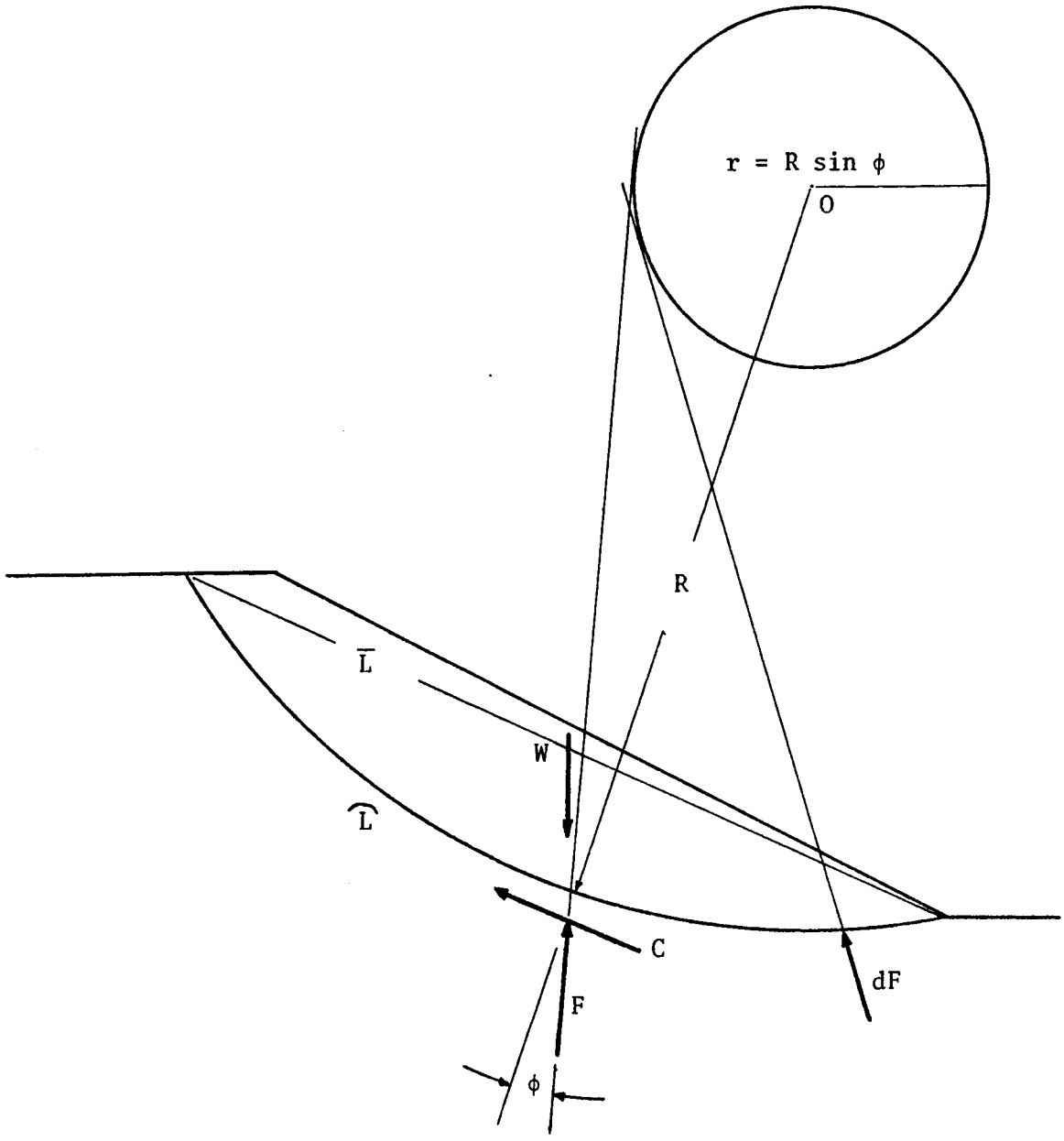


Figure 1. Diagram of ϕ - Circle Method (Taylor 1937).

$$F_c = \frac{c}{c_1}$$

where c_1 is the unit cohesion required to maintain equilibrium in the soil mass.

The frictional resistance developed by the soil mass is a distributed force over the entire shear surface. Since the magnitude of the frictional resistance at a point is a function of the normal stress at that point, the distribution of the frictional resistance will be the same as the distribution of normal stress if the angle of internal friction is assumed constant. As was stated previously, the determination of the distribution of normal stresses is a statically indeterminant problem. However, if the resultant of the normal force and the frictional resistance is designated as F, it is known that the elementary reactions dF are inclined at an angle ϕ to a line normal to the shear surface. Since the shear surface is assumed to be a circular arc with center at 0 and radius R, all elementary reactions, dF, will be tangent to a small circle centered at 0 with radius

$$r = R \sin \phi$$

The small circle is known as the ϕ - circle. Since any two elemental forces, dF, intersect just outside the ϕ - circle, the resultant, F, must be tangent to a circle slightly larger than the ϕ - circle. The resultant, F, is commonly assumed to be tangent to the ϕ - circle which has the effect of using a slightly conservative value for ϕ. Taylor (1937), by assuming that the distribution of the elementary

resultants, dF, along the shear surface was zero at the end points of the shear surface and varied sinusoidally between, developed a correction for the radius of the ϕ - circle. Comparing results using the corrected radius with results using the radius of the ϕ - circle as initially defined, Taylor demonstrated that the assumption that the resultant, F, is tangent to the ϕ - circle as initially defined is sufficiently accurate.

The other forces involved in the ϕ - circle analysis are the weight of the soil mass, W, and the resultant cohesion, C. The weight of the soil is assumed to act vertically through the center of gravity of mass. The resultant cohesion is:

$$C = c_1 \overline{L}$$

where $\overline{L}$ is the chord length of the shear surface. The resultant cohesion acts parallel to the chord at a distance

$$x = \frac{R\overset{\frown}{L}}{\overline{L}}$$

from the center of rotation, O, where $\overset{\frown}{L}$ is the arc length of the shear surface. Since the soil mass is assumed in equilibrium, the resultant, F, must act through the intersection of W and C which is known.

By analyzing a large number of slopes and tabulating and plotting the results, Taylor developed relationships between the slope angle, i, the angle of internal friction, ϕ, and a Stability Number, $\frac{c}{F_c \gamma H}$, where γ is the unit weight of the soil and H is the

height of the slope. Increasing values of the Stability Number indicate decreasing stability. Parcher (1968) suggests the ratio would be more logically called the instability number. Terzaghi and Peck (1967) define a stability factor which is the reciprocal of Taylor's Stability Number. The stability factor therefore increases numerically with increasing stability of the slope. From his investigation using the ϕ - Circle Method, Taylor determined that for most cases of ϕ greater than zero the most critical circle involved a circle passing through the toe of the slope. For those few cases of $\phi > o$ where the critical circle passed below the toe of the slope, the Stability Number for the critical circle was extremely close to the Stability Number for the most critical toe circle. For the special case where $\phi = 0$ and c is constant with depth, the critical circle will pass through the toe of the slope for all slopes steeper than 53^{o} (i.e. for $i > 53^{o}$). This finding is consistent with work previously done by Fellenius (1927).

For slopes less than 53^{o} steep, ($\phi = 0$, c is constant with depth) the most critical circle extends to an infinite depth and has an infinite radius. However, since c cannot be constant to an infinite depth, the depth of the most critical circle depends on some confining layer of higher strength. The most critical circle becomes tangent to that confining layer. Such circles are known as midpoint circles because their centers are located above the midpoint of the slope. If the depth to a confining layer is small enough, the critical circle can intersect the slope above the toe, in which case the critical circle is referred to as a slope circle.

In addition to developing relationships between Stability Number, slope angle, and angle of internal friction, Taylor related depth to a confining layer, by means of a Depth Factor, to slope angle and Stability Number for the case of $\phi = 0$.

Intrinsic in the ϕ - Circle Method of analysis, the internal friction and cohesion parameters of shear strength of the soil must be separated. As a result, the factor of safety calculated is a factor of safety with respect to cohesion, F_c. However, the factor of safety with respect to both cohesion and internal friction, which is the factor of safety with respect to shear strength F_s, can be determined using an iterative procedure which results in $F_c = F_\phi$. The critical surface will not be the same as that found when F_ϕ is set at unity (Parcher 1967).

The ϕ - Circle Method is limited to the assumption of a homogeneous soil mass. The value for internal friction and cohesion must be assumed constant over the entire failure circle. In cases where the cross section is stratified with layers of varying strength properties, only average values of shear strength parameters may be used in the analysis.

An assumption regarding the distribution of normal stresses is required to allow the use of statics. The assumption made using the corrected ϕ - circle radius is that the normal stress is zero at the endpoints of the shear surface and varies sinusoidally between the ends.

The ϕ - Circle Method accounts for pore pressures within the soil mass by assuming stresses are total stresses which include pore

pressures. Effects of seepage pressures on the soil mass can only be accounted for using graphical methods including the construction of a flow net.

The surface of the slope must be represented by a straight line. The method does not take into consideration irregular surfaces. The ϕ - Circle Method is restricted to analysis of potential failure surfaces with circular arc shapes. In cases where critical failure surfaces vary significantly from the assumed circular arc shape, an alternate method of analysis must be undertaken.

Method of Slices

Fellenius (1927, 1936) developed a method of analysis using the Swedish circular arc in which the soil mass above the potential failure arc was divided into a number of slices. In this procedure, Fellenius graphically constructed a force polygon including the forces of weight, cohesion and the resultant of the normal force and internal friction. The weight of the individual slices are assumed to act vertically downward through the center of gravity of the slice. The magnitude of the cohesion was assumed known and assumed to act tangent to the arc at the midpoint of the slice. The resultant of the normal force and internal friction for each slice was assumed to act at an angle ϕ to a line perpendicular to the failure surface at the midpoint of each slice. Trial solutions were attempted varying the magnitude of ϕ angles until the force polygon closed. An alternative to this approach was to assign a specific ϕ angle and vary the magnitude of the cohesion until the polygon closed.

Whereas the ϕ - Circle Method, as previously described, is restricted to constant values of cohesion and internal friction along the entire shear surface, the Method of Slices permits the utilization of different values for c and ϕ on each slice if so desired.

Bishop (1955) extended the Slices Method by including in the analysis the effect of forces between slices, i.e. "interslice forces." The method developed, currently known as the Bishop Method of Slices, analyzes each slice with respect to the forces shown in Figure 2.

The symbols in Figure 2 on page 18 are defined as follows:

X_n, X_{n+1} - the vertical shear forces on sections n and n+1 respectively

E_n, E_{n+1} - the resultants of the total horizontal forces, including the effect of seepage forces if present

W - the total weight of the slice of soil

P - the total normal force acting on the base of the slice (includes pore pressure and effective stresses)

S - the shear force on the base of the slice

h - the height of the slice

b - the width of the slice

ℓ - the length BC

α - the angle between BC and the horizontal

X - the horizontal distance of the slice from the center of rotation 0.

The force W is assumed to act vertically through the center of the slice. The normal force, P, acts perpendicular to the base of the

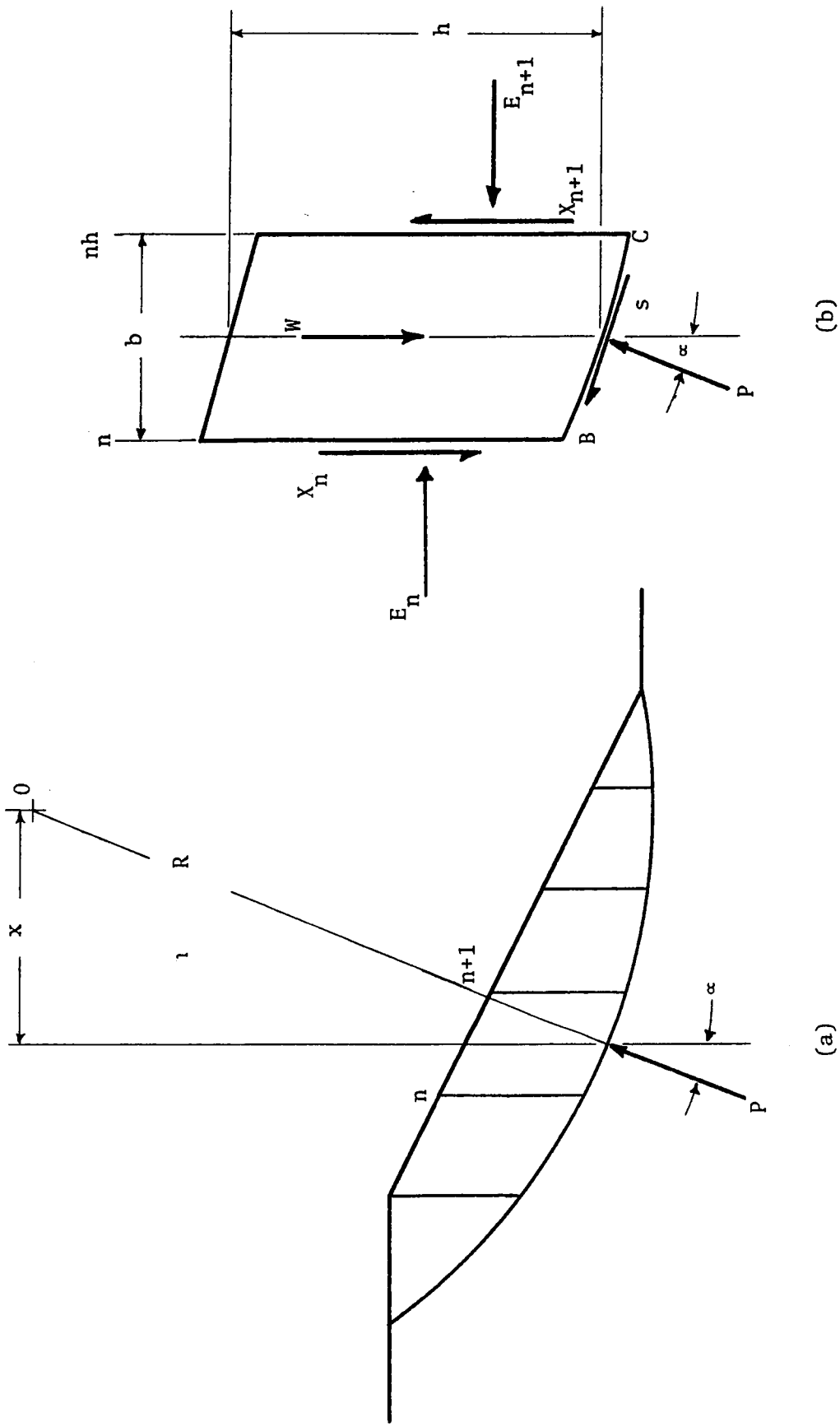


Figure 2. Diagram of Slices Method (Bishop 1955).

slice at its midpoint. The shear force, S, is a combination of the cohesion and internal friction components of shear resistance required to maintain equilibrium, which acts tangent to the base of the slice at its midpoint.

The method of slices defines the factor of safety as:

$$F_s = \frac{c + \bar{\sigma} \tan \phi}{c_1 + \bar{\sigma} \tan \phi_1}$$

and assumes that F_s is equal for each slice. In the equation above the subscript 1 indicates values required to establish equilibrium. The term $\bar{\sigma}$ is the effective normal stress where $\bar{\sigma} = (\sigma - \mu)$ and μ is the excess pore pressure. Prior to Bishop's developments, Krey (1926), and Terzaghi (1929) developed a similar method of analysis in which the resultants of the interslice forces were assumed to be equal and opposite therefore cancelled each other. The condition of equal and opposite resultants is strictly true only in the case of a plane slip surface with constant angle of internal friction. This method, known as the Ordinary Method of Slices is still widely used due to the ease of calculations. Bishop (1955) points out that the value of F_s using the Ordinary Method of Slices is conservative when compared to the Bishop Method of Slices. The degree to which the Ordinary Method is conservative is not constant but is greater for conditions involving deep failure arcs or where the center angle of the arc is large. This should be expected since conditions involving larger central arc angles result in greater deviation from the assumption of a plane slip surface.

Using the assumptions stated for the Ordinary Method of Slices, and summing forces in a direction normal to the shear surface at the midpoint of each slice, the expression for F_s can be written as:

$$F_s = \frac{1}{\Sigma W \sin \alpha} \cdot \Sigma\{c_1 \ell + (W \cos \alpha - \mu\ell) \tan \phi_1\}$$

In the rigorous Bishop Method, where the interslice forces are not ignored, the expression for F_s can be determined by summing forces vertically so that:

$$F_s = \frac{1}{\Sigma W \sin \alpha} \cdot \Sigma \left[\{c_1 b + ((W - \mu b) + (X_n - X_{n+1})) \tan \phi_1\} \cdot \frac{\sec \alpha}{1 + \frac{\tan \phi_1 \tan \alpha}{F_s}} \right]$$

For a derivation of these expressions the reader should refer to the original paper by Bishop (1955).

Values for the interslice forces cannot be determined using methods of statics. However, if no external forces are present and the slope is stable, it is known that

$$\Sigma (X_n - X_{n+1}) = 0$$

and

$$\Sigma (E_n - E_{n+1}) = 0.$$

Bishop's rigorous method involves a lengthly process of determining F_s using values of $(X_n - X_{n+1})$ which satisfy both force and moment conditions of equilibrium. Initial values of F_s are picked by assuming

$(X_n - X_{n+1}) = 0$. The values of $(X_n - X_{n+1})$ are then adjusted to satisfy the condition that $\Sigma\ (E_n - E_{n+1}) = 0$.

Since it is desirable to investigate a large number of circles to determine which is the most critical, Bishop proposed that in many cases the factor of safety given by assuming $(X_n - X_{n+1}) = 0$ is sufficiently accurate. This method which is known as Bishop's Simplified Method, in effect, assumes that the interslice forces are horizontal. This assumption satisfies the conditions of moment equilibrium but not force equilibrium.

The Bishop Method, both the rigorous and simplified procedure, is applicable for analysis of slopes with vertically varying soil types and strength parameters. The analysis is capable of including the effects of pore pressure and seepage pressures. The rigorous method satisfies both force and moment conditions of equilibrium; however, the procedure is very lengthly which could limit the number of failure arcs investigated. The simplified method satisfies conditions of moment equilibrium but not force equilibrium. The simplified method greatly reduces time required for analyzing each circle; therefore, allowing a significantly larger number of circles to be analyzed to determine the most critical. A comparison of factors of safety given by the rigorous and simplified method indicates that the simplified method gives equivalent or slightly smaller values of F_s over a wide range of conditions of slope angle, shear strength, and pore pressure. The variation in F_s has been determined to be 6% or less (Wright, et. el., 1973).

Spencer (1967) extended Bishop's Method of Slices by assuming that the resultant of the interslice forces are parallel. Spencer's method satisfies both force and moment conditions of equilibrium with less time required for computation when compared with Bishop's rigorous method. According to Spencer, the error involved in the Bishop Simplified Method is conservative and is normally less than 1% but is always less than 4%. Spencer accredits the accuracy of the Bishop Simplified Method, which does not satisfy all conditions of statics, to the fact that the factor of safety is relatively insensitive to the slope of the interslice forces. The error involved in Bishop's Simplified Method is greatest under a combination of conditions involving steep slopes, high pore pressures, high angle of internal friction and a low value for the parameter $c/\alpha H$.

The Bishop Method is a limit equilibrium method. It is restricted to the investigation of circular failure arcs. The soil is assumed homogeneous in directions normal to the cross section. The method assumes that the factor of safety is constant along the shear surface and is therefore the same for each slice. The factor of safety, F_s, is calculated with respect to shear strength, which means F_c and F_ϕ are equal. The problem of static indeterminacy is overcome by assuming horizontal interslice forces.

The method of slices has also been applied to limit equilibrium methods of analysis in which composite or general failure surfaces were investigated. The first such analysis was developed by N. Janbu (1954). Janbu's Method for General Slip Surfaces utilizes the same assumption employed in Bishop's Simplified Method, i.e.,

$(X_n - X_{n+1}) = 0$. The primary difference between the two methods is that in Bishop's Method, moments are taken about a central location which is the center of the circular arc; whereas in Janbu's Method moments are taken about the midpoint of the base of each slice.

Janbu suggests that where more accuracy than obtained by assuming $(X_n - X_{n+1}) = 0$ is desired, an assumption can be made regarding the "line of thrust" of the interslice forces. The line of thrust is the line passing through the points of action of the interslice forces. The assumption made by Janbu is that the line of thrust passes through or slightly above the lower one-third of each slice. Neither of the assumptions used in Janbu's analysis necessarily satisfy the condition of overall moment equilibrium.

Morgenstern and Price (1965) developed a method of limit equilibirum analysis which is applicable to general slip surfaces and satisfies both moment and force conditions of equilibrium. In order to make the problem statically determinant, Morgenstern and Price assume a functional relationship between the horizontal, E, and vertical, X, interslice forces. The relationship between E and X is not necessarily constant but is a function of the location of the slice. It was determined that as long as the relationship is reasonable the value for the calculated factor of safety is relatively insensitive to variations in the relationships.

A comparison of the Morgenstern and Price Method with the Bishop Simplified Method indicated that the two methods gave approximately similar results when a circular arc is analyzed.

The Morgenstern and Price Method gave values of factor of safety slightly higher or slightly lower than the Bishop Simplified Method depending upon the relationship chosen between the horizontal and vertical interslice forces.

III. THE INFLUENCE OF SOIL PARAMETERS ON STABILITY ANALYSES

The determination of soil strength parameters, pore pressure distributions, stress conditions, and special geological conditions existing in a slope is as important to the stability analysis as the selection of the proper method of analysis. These factors can best be determined using a combination of laboratory and field tests and field observations. The triaxial test, because of its versatility, is frequently used in determining the soil shear strength properties and pore pressure coefficients to be input into slope stability analysis. In its various forms, the triaxial test can approximate various stress and drainage conditions existing in field situations. Depending on the existing conditions, stability problems can be analyzed using either a total stress or effective stress parameters.

Total Stress Analysis

The total stress analysis, often referred to as the $\phi = 0$ analysis, is acceptable in short term stability problems involving rapid loading conditions on saturated soil. The portion of soil strength resulting from internal friction is dependent upon the effective normal stress, $\bar{\sigma}$, which is equal to the total normal stress, σ, less the pore pressure, μ. Under rapid loading conditions no

volume change (i.e. no drainage) is allowed; therefore, excess pore pressure is induced which results in the soil acting as a purely cohesive material with an angle of internal friction equal to zero. Such a stress situation can be duplicated in the unconsolidated, undrained (Q-Test) triaxial test. An unconfined compression or a field vane shear test can also be used to determine the undrained strength.

According to Skemption (1948), the $\phi = 0$ analysis is applicable to short term slope stability problems on saturated material which exhibits sufficiently low permeability to prevent immediate drainage. These conditions are frequently met during and immediately after the construction of a cut slope or placement of small fills. Conditions are not met on natural slopes which have experienced no recent change in stress.

It should be pointed out that no natural soil has a true angle of internal friction equal to zero. However, it has been widely demonstrated that under certain stress conditions, an assumption of $\phi = 0$ can be satisfactorily used to determine the stability of a slope in the undrained condition.

Effective Stress Analysis

In an effective stress analysis the shear strength parameters assigned to the soil are those which exist independent of pore pressure. In this type analysis, it is necessary to determine the pore pressure which will exist in the slope. Two types of triaxial tests can be used to determine the effective shear strength parameters of a soil. An undrained test can be employed in which the pore pressure existing in

a saturated soil under stress is measured. Alternately, a drained test can be used in which additional stress is added at a rate slow enough to assure that no excess pore pressures are present at the time of failure of the sample. A great disadvantage of the drained test is the time required to perform the test since in clays of low permeability the dissipation of pore pressure in the sample may require days. For this reason the undrained test with pore pressure measurement is most commonly used for design purposes.

In addition to determining the shear strength parameters for the effective stress analysis, the increased pore pressure induced in a soil by increasing stress must also be determined. Skemption (1954) introduced the A and B pore pressure parameters defined as

$$\Delta\mu = B \{\Delta\mu_3 + A (\Delta\sigma_1 - \Delta\sigma_3)\},$$

where $\Delta\mu$ is the change in pore pressure due to the change in the major principal stress, $\Delta\sigma_1$, and the change in the minor principal stress, $\Delta\sigma_3$. The A and B pore pressure parameters can be determined in the triaxial test and used to predict the pore pressures which will exist in the actual slope. Alternatively, or in addition to determining pore pressure parameters, piezometers can be installed in the actual slope to measure and monitor the existing pore pressures. The effective stress analysis can be applied to any type loading condition, that is either long term or short term since the pore pressures involved are known and can be taken into consideration.

A comparison of total stress and effective stress analysis can be made only in the undrained condition. For an undrained slope on

the verge of failure both methods will result in a factor of safety of unity (Bishop and Bjerram, 1960). However, the location of the failure circle is dependent upon the angle of internal friction of the soil. Therefore the two methods of analysis do not predict indentical failure surfaces. Since in the effective stress method the value for angle of internal friction approximates the actual value, the effective stress analysis predicts the actual failure surface more accurately. For slopes which are not on the verge of failure ($F_s > 1$) the two methods will not generally predict equivalent values for the factor of safety even if the definition of the factor of safety is equivalent.

IV. FACTORS OF SAFETY

Several terms have been used to define the factor of safety against sliding. It should be pointed out that the numerical value of the factor of safety is dependent upon the definition of the term in all cases except at incipient failure, in which case, the factor of safety should equal unity regardless of definition.

The most universally accepted definition of factor of safety is with respect to shear strength given by

$$F_s = \frac{S}{\tau} = \frac{c + \bar{\sigma} \tan \phi}{c_1 + \bar{\sigma} \tan \phi_1}$$

where S is the shearing strength of the soil and τ is the shearing stress. Because the assumptions of a circular failure arc has been widely used in stability analysis, an equivalent method of defining

F_s as the ratio of resisting moment to driving moment has common usage. These definitions of factor of safety assume the failure of a slope is the result of a decrease in the shear strength of the soil.

As was brought out in describing the ϕ - Circle Method, the factor of safety with respect to cohesion, $F_c = c/c_1$, is calculated separate from the factor of safety with respect to internal friction, $F_\phi = \tan \phi/\tan \phi_1$, which was assigned the value of unity. In the slices methods described, the factor of safety is defined with respect to shear strength in which case F_c is equal to F_ϕ. Parcher (1968) points out that the critical circle determined by the different assumptions is not the same. In a purely cohesive soil the critical circle is located at an infinite depth. The frictional resistance is the factor which determines the depth at which the critical circle will be located in a soil exhibiting both cohesion and internal friction. Since at failure, full frictional resistance is developed, the ϕ - Circle analysis which assumes the full development of the component of frictional resistance gives a more accurate estimate of the critical circle than those methods which assume only partial development of the frictional resistance. However a factor of safety with respect to cohesion as calculated using the ϕ - Circle Method can result in a false evaluation of the stability of the slope.

There has been much controversy as to which is the best definition for factor of safety. In choosing the method of analysis and the definition of the factor of safety to be used for a particular problem one must realize that the true factor of safety against sliding depends upon the element which actually causes the slope to fail. In his

General Report to the Fifth International Conference on Soil Mechanics and Foundation Engineering, D. H. Trollope (1961) states:

> Too much importance should not, however be attached to the "exact" calculation of factors of safety at this stage. The intrinsic errors in these methods of analysis preclude calculation of a "correct" value and the selection of a factor for design purposes still demands careful engineering judgment.

V. SUMMARY

Many variables and complexities are involved in slope stability problems. Assumptions and simplifications are made in all methods of stability analysis in order to render the problem manageable. In choosing a particular procedure for analyzing a slope, an engineer should always be aware of what assumptions are made in that procedure and what effects those assumptions will have on the factor of safety calculated. The shape of failure surfaces analyzed can be varied by using the various procedures. The critical failure surface is that surface which produces the lowest factor of safety. Since all other potential failure surfaces produce higher factors of safety, any method of analysis which does not analyze the critical surface results in a factor of safety on the unsafe side. Many of the methods discussed yield very similar results when analyzing a particular type problem. In such a case, it is easy to justify using the simpliest method. However, when analyzing a different type problem the same methods may yield widely varying results. In this case if assumptions of one method are too restrictive, a more rigorous and lengthly method may be necessary to obtain required accuracy.

CHAPTER III

SITE SELECTION AND DESCRIPTION

I. INTRODUCTION

Slope failures in spoil banks of contour surface mining in mountainous terrain can be grouped into two broad categories. The first group includes slope failures in spoil banks in which the entire failure surface lies within the fill material placed on the slope during the mining process. Although this type failure is quite common, the failure arc is normally very shallow and such a slide therefore involves only small quantities of materials. The presence of a shallow failure arc indicates that shear strength of spoil material results primarily from internal friction of the material and that cohesion contributes only slightly to strength development. Drnevich et al. (1976) substantiate this observation by triaxial testing mine spoils to determine shear strength. Shallow failures occur when outslope steepness of the spoil bank exceeds a value which has been referred to as the "angle of repose" of spoil. In Tennessee, surface mine regulations have been written using the angle of repose concept and limiting outslope steepness to 35^{o}.

Shallow failure arcs involve small quantities of material, which do not slide great distances but usually merely slump until a more stable slope angle is attained. Damage done by such a slide is normally not extensive. The primary concern is that vegetation which has been established on the slope is damaged. Corrective procedures are usually

limited to reseeding with appropriate grasses and legumes and, if necessary, replanting tree seedlings.

The second group of slope failures includes those in which failure surfaces lie partly within the fill material placed during the mining process and extend down into the natural slope which serves as a foundation for the fill bench. Failures of this type are not as common as those included in the first group. However, damage done by a single failure of the second type often exceeds the damage caused by a number of the smaller failures. Slides involving foundation material are associated with deep failure arcs which include large quantities of material that move considerable distances downslope. Such slides extend into the previously undisturbed forest or woodland causing extensive environmental damage primarily in the form of increased sediment load in streams.

This investigation includes a detailed study of three slope failures which fall within the second group. The three slope failures chosen for study are typical failures of that group. Since no pertinent information is to be gained by naming the mining companies involved, the sites will be designated as A, B. and C. The three slides studied have many factors in common, however the triggering cause of failure in each case is believed to be different. Locations of each site and the existing conditions prior to failure are discussed below.

Site A

Site A is located in the southeast portion of Scott County on U. S. Geologic Survey (U.S.G.A) 7.5 minute quadrangle number N3607.5-W8422.5

at 36^{o} 46' 30" latitude, 84^{o} 23' 30" longitude on the Tennessee rectangular coordinate system. Coal which was mined at the site is from the Jellico seam at approximately 1520 feet elevation. The Jellico coal is the top unit of the Pennsylvanian age Slatestone rock group which consists of alternating sandstone and shale units with some coal. A few feet below the Jellico coal lies the Newcomb sandstone which is a unit averaging 20 feet in thickness. The Jellico coal itself lies within an unnamed shale unit (Luther 1959).

During the premine planning stage of operations, an existing natural bench, previously used as an access trail, was noted within 125 feet downslopes from the coal cut crop. At that time (Fall, 1974), the Tennessee surface mine regulations allowed spoil to be placed below the coal outcrop up to 125 feet measured down the slope perpendicular to the coal cropline. The natural bench, which paralleled the coal seam within the allowable spoil distance was believed to be made up of relatively intact weathered rock. Mining plans included using the natural bench as a buttress upon which to place the downslope toe of the spoil fill bench. Existing slopes between the coal elevation and the natural bench were approximately 24^{o}.

In preparation for spoil placement, trees and brush were cleared and windrowed below the proposed toe of the outslope fill. A single cut was taken on the coal seam back to a highwall height of 35 to 40 feet producing a cut bench 75 to 80 feet wide. Spoil from the cut was pushed over the outslope to produce the fill bench with a downslope toe on the natural bench. This mining process proceeded until the mine extended approximately 3600 feet along the contour, with none of the spoil graded back toward the highwall.

During a wet late fall of 1974 and early winter of 1975 a series of slides occurred in the spoil bank and underlying natural bench. Three of the slides extended into Straight Fork Creek approximately 400 feet downslope from the coal cropline. When the failures occurred, it was learned that rather than intact rock, the natural bench, which was used as a buttress, actually consisted of a colluvial deposit of sandstone boulders in a sandy-clay matrix existing in a metastable condition above the residium and intact rock.

An approximate cross section immediately before failure is shown in Figure 3, page 34. A photograph of the resulting series of slides is shown in Figure 4, page 35.

Site B

Site B is also located in Scott County Tennessee on U.S.G.S. 7.5 minute quadrangle number N3615-W8430 at 36^{o} 15' 45" latitude, 84^{o} 31' 45" longitude. Mining at Site involved the Windrock coal seam at approximately 2,240 feet elevation and the Big Mary coal seam which lies stratigraphically 20 to 25 feet above the Windrock coal. Both coal seams were mined on one highwall in a multiple seam operation. The Windrock coal forms the top stratigraphic unit of the Graves Gap group. A flinty underclay typically lies immediately below the Windrock coal. An unnamed shale interval lies between the Windrock and Big Mary coal seams. The shale interval and Big Mary coal lie at the bottom of the Redock Mountain group (Luther 1959).

Trees and brush were cleared ahead of mining operations and in preparation for spoil storage on the slopes. The multiple seam mining

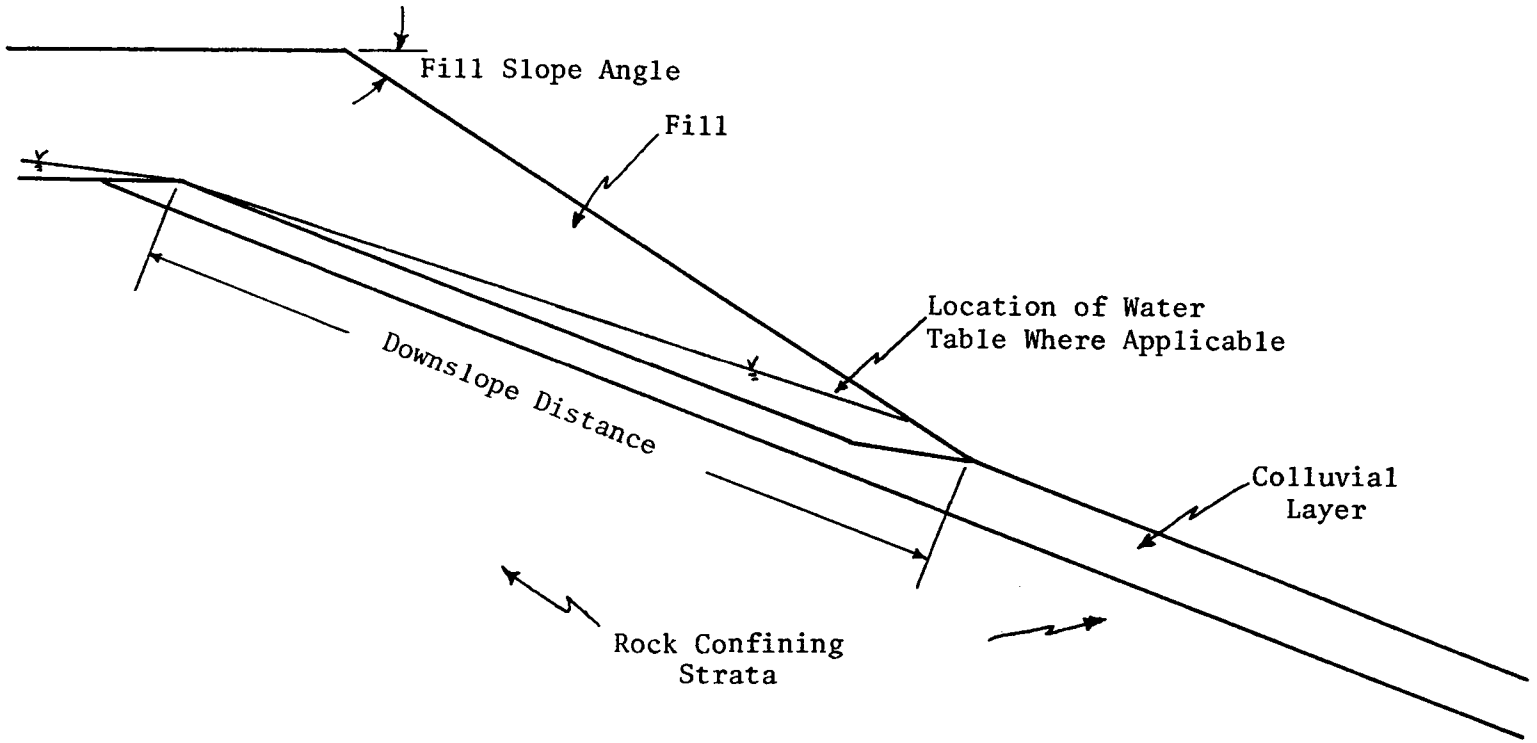


Figure 3. Typical Cross Section of the Fill Bench at Site A.
Scale 1 inch = 30 feet

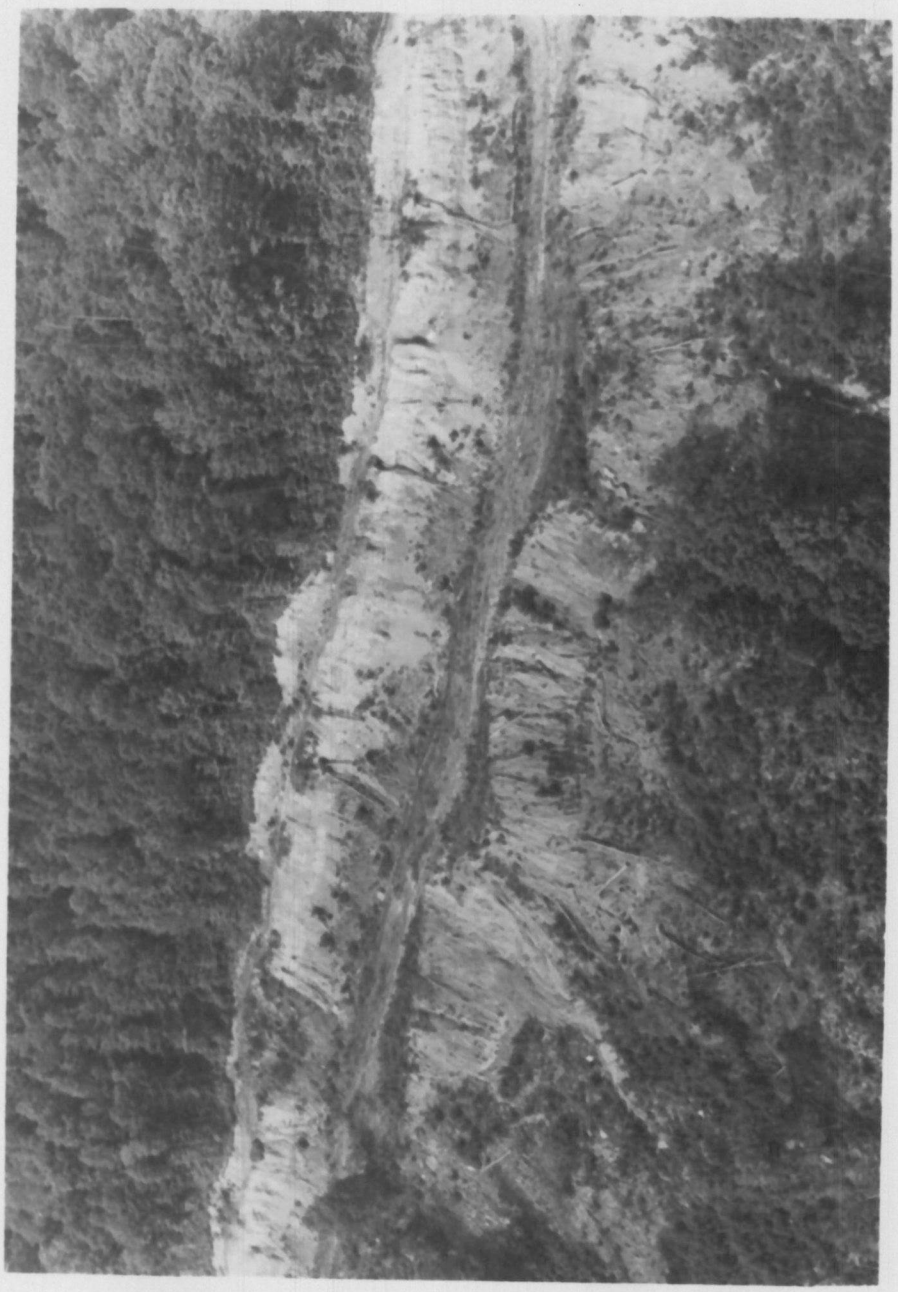


Figure 4. Photograph of Slides at Site A.

process involved an initial narrow cut (approximately 40 feet wide) on the lower Windrock seam. Overburden from the initial cut was pushed over the outslope until the toe of the fill extended 125 feet downslope below the coal level. Natural slopes below the Windrock seam ranged from 22 to 25°. A cut was then made on the Big Mary seam with overburden being piled onto the cut and fill bench produced by the initial Windrock cut. A final cut was then made on the Windrock seam. Overburden from this last cut was hauled with trucks along contour to fill a previous final cut in the Windrock seam.

The depth of unconsolidated material, colluvium and residium, was determined during initial benching to be 10 to 15 feet. It was also noted during the mining process that the shale and coal strata were gently rolling and not horizontal. Locations of low points in the strata were associated with natural wet weather springs and seepage.

Mining began in the early spring of 1975 at which time the fill bench was built. The slope remained stable for approximately 10 months into the winter of 1976. During the winter of 1976 four major slope failures occurred. A simplified typical cross section of the fill bench before failure is shown in Figure 5, page 37. Photographs of the failed slopes are shown in Figure 6, pages 38 and 39.

Site C

Site C is located in the central portion of Fentress County Tennessee on U.S.G.S. 7.5 minute quadrangle number N3622.5-W8500 at 36° 25' 45" latitude, 85° 00' 15" longitude. The Lower Wilder coal seam at 1540 feet elevation was mined at the site. The Lower Wilder coal is located in the lower part of the Signal Point shale

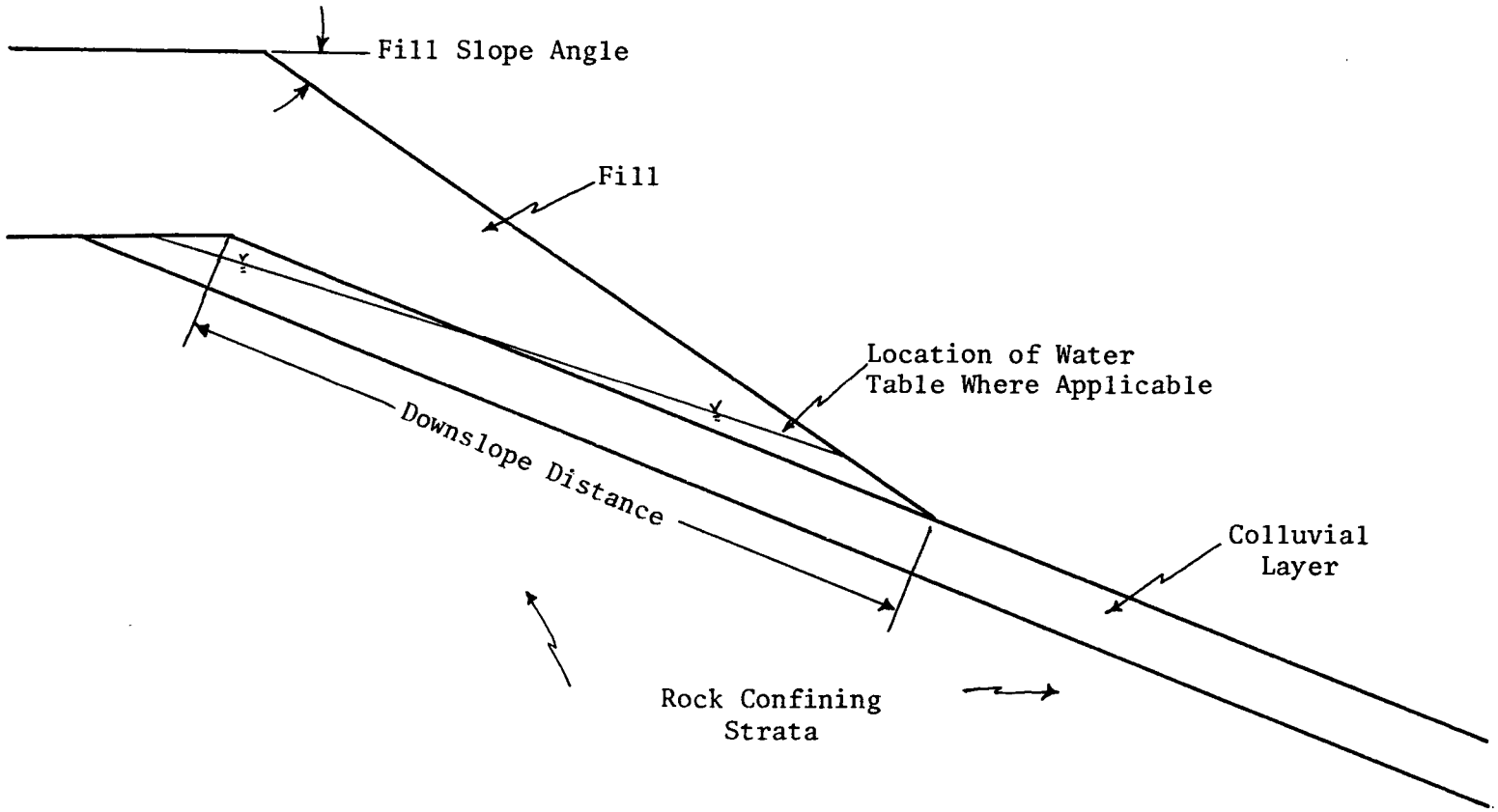


Figure 5. Typical Cross Section of the Fill Bench at Site B.
Scale 1 inch = 30 feet

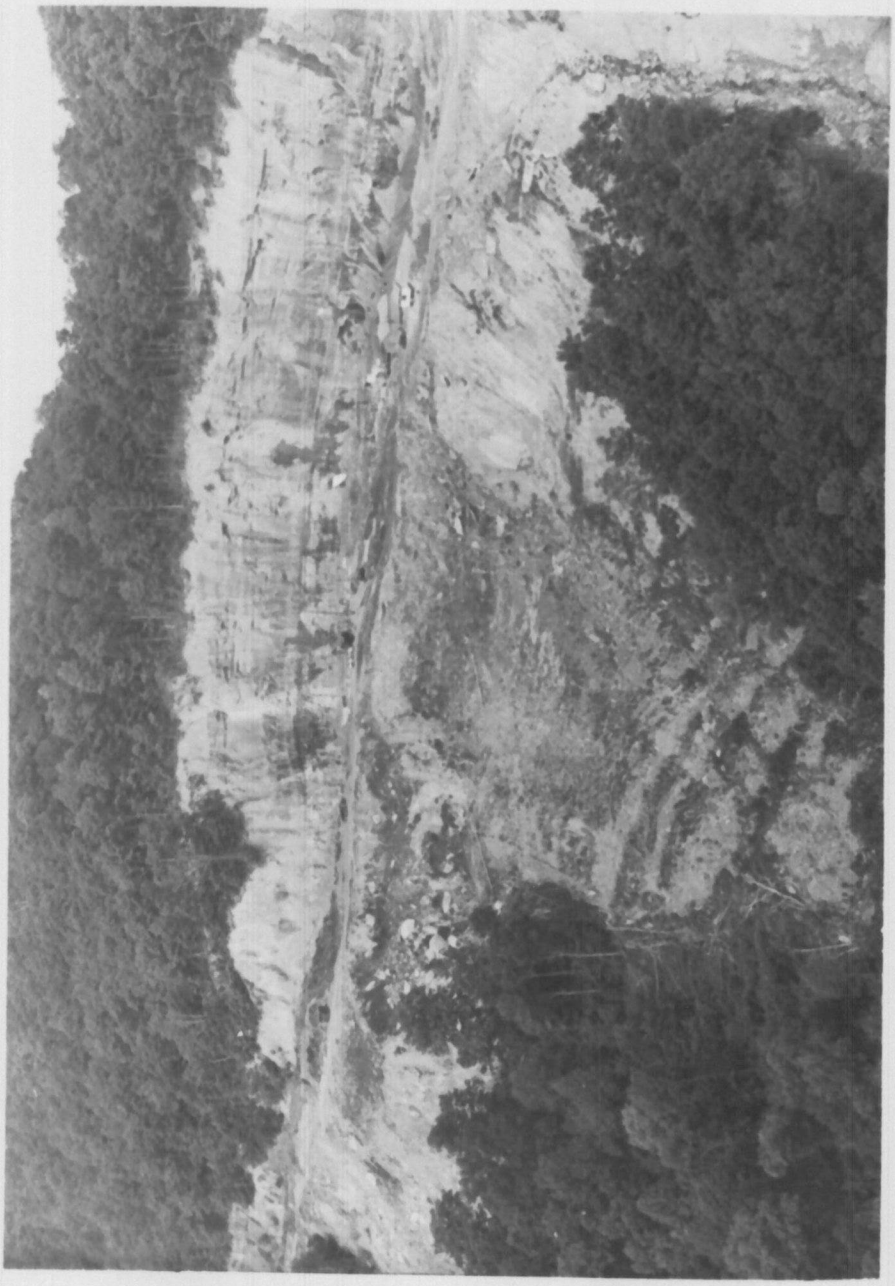


Figure 6. Photographs of Slides at Site B.

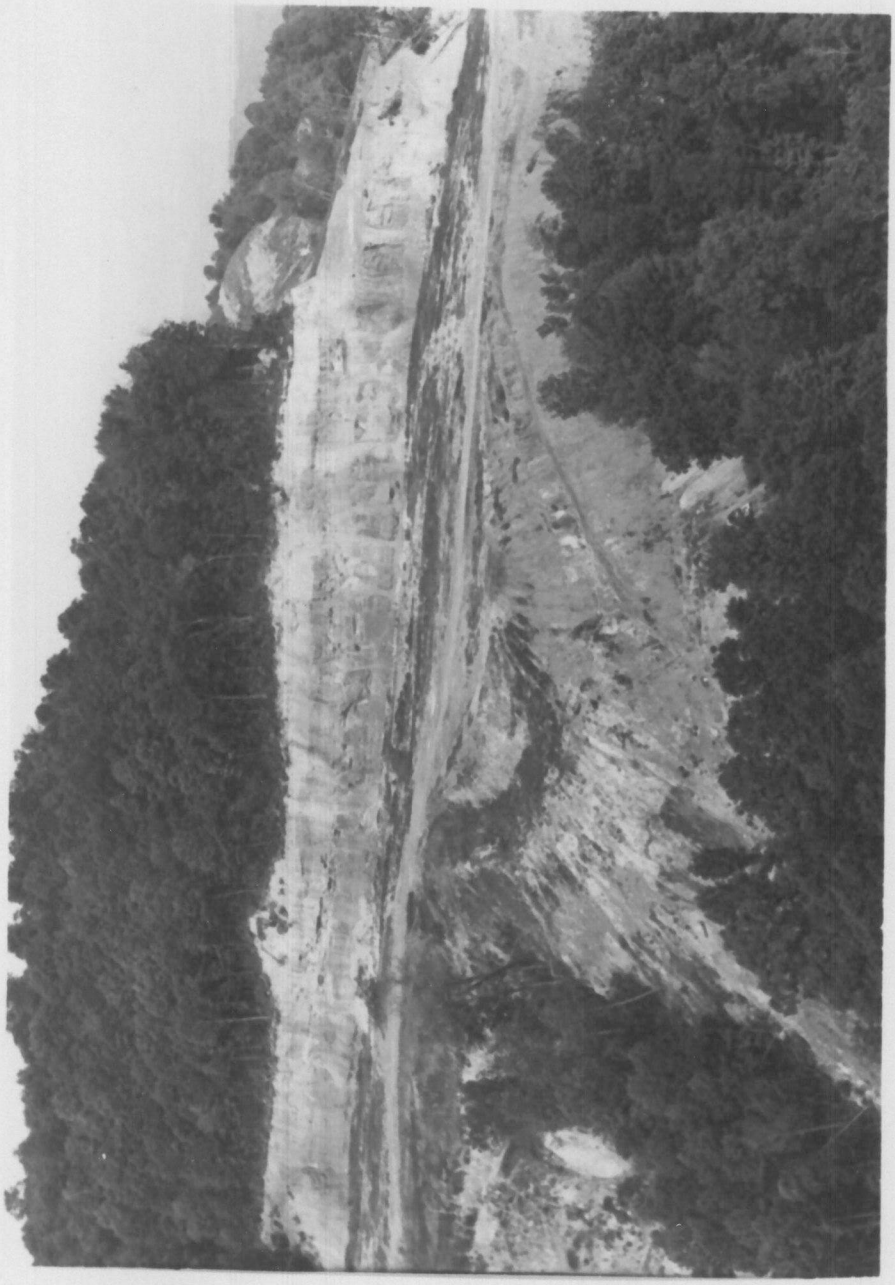


Figure 6. (Continued)

which is a part of the Gizzard rock group. Sandstone and conglomerate units which are present in other areas where the Gizzard group is located are missing in this area. Areas in which the sandstones and conglomerates are missing and only shale units and coal remain are referred to as the Fentress formation (Luther 1959).

A majority of the natural slopes of Site C were steeper than the 28^{o} maximum allowable for slope spoil storage according to the Tennessee surface mine regulations effective in fall 1975 when the area was mined. As a result of limitations on slope storage, an alternative storage area was needed. On the northwest end of the site, natural slopes were approximately 24^{o} for a distance of 50 feet down the slope below the coal cropline. Farther than 50 feet below the cropline, the natural slope tapered to constant 15^{o} for several hundred feet. The mine operator requested and was granted permission to use the gently sloping areas as an alternative spoil storage area.

During the mining process, spoil was hauled to the storage area with scrapers and pushed into place with bulldozers. This method of fill placement continued for six weeks at which time the fill and underlying material experienced failure. From observation of the failed slope, it was apparent that a deep rotational slump had occurred. Such a rotational slump indicates that to a significant depth, the material comprising the gently sloping area consisted of unconsolidated colluvium and/or residium. An approximate cross section existing immediately before failure is shown in Figure 7, page 41. A photograph of the slope after failure is shown in Figure 8, page 42.

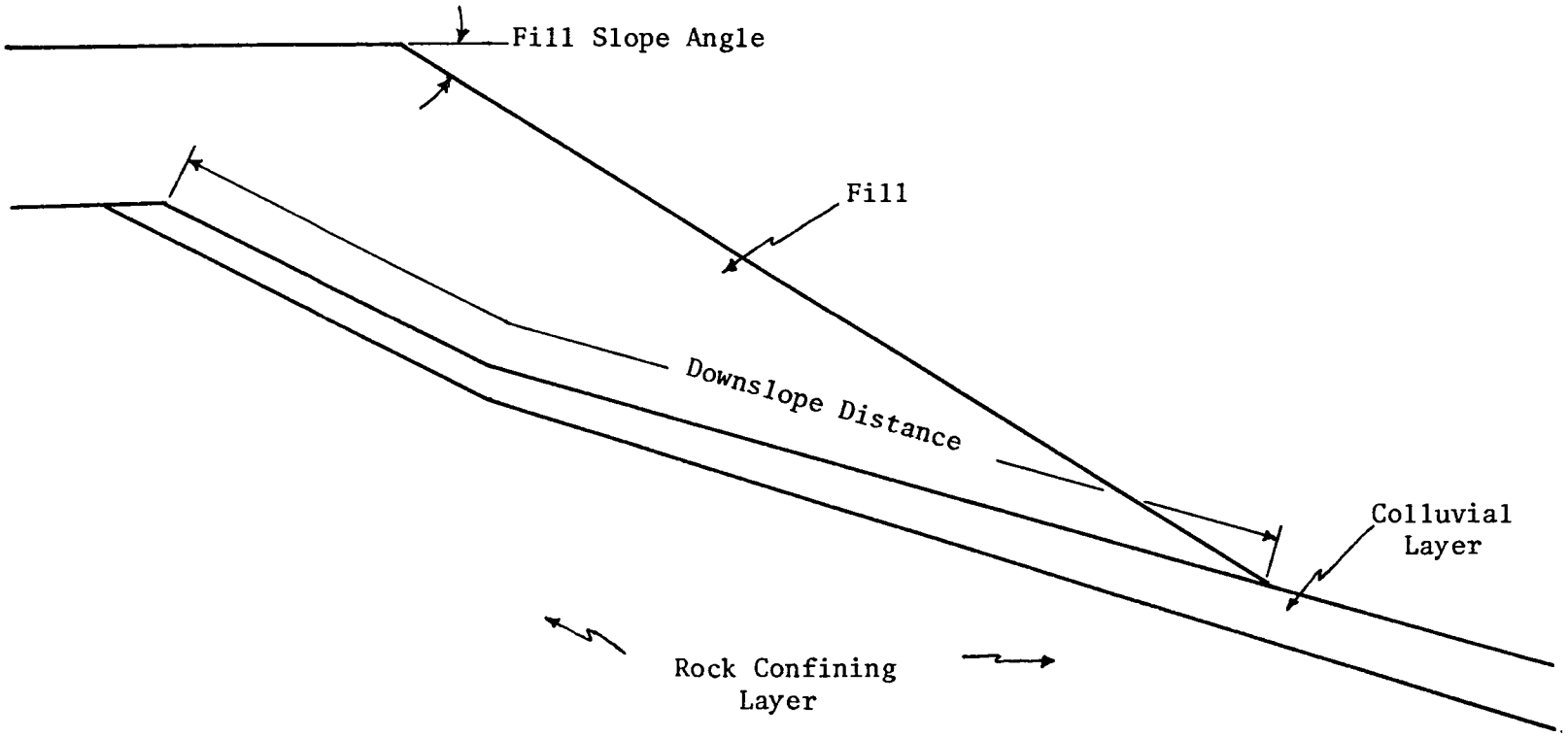


Figure 7. Cross Section of the Fill at Site C.
Scale 1 inch = 30 feet

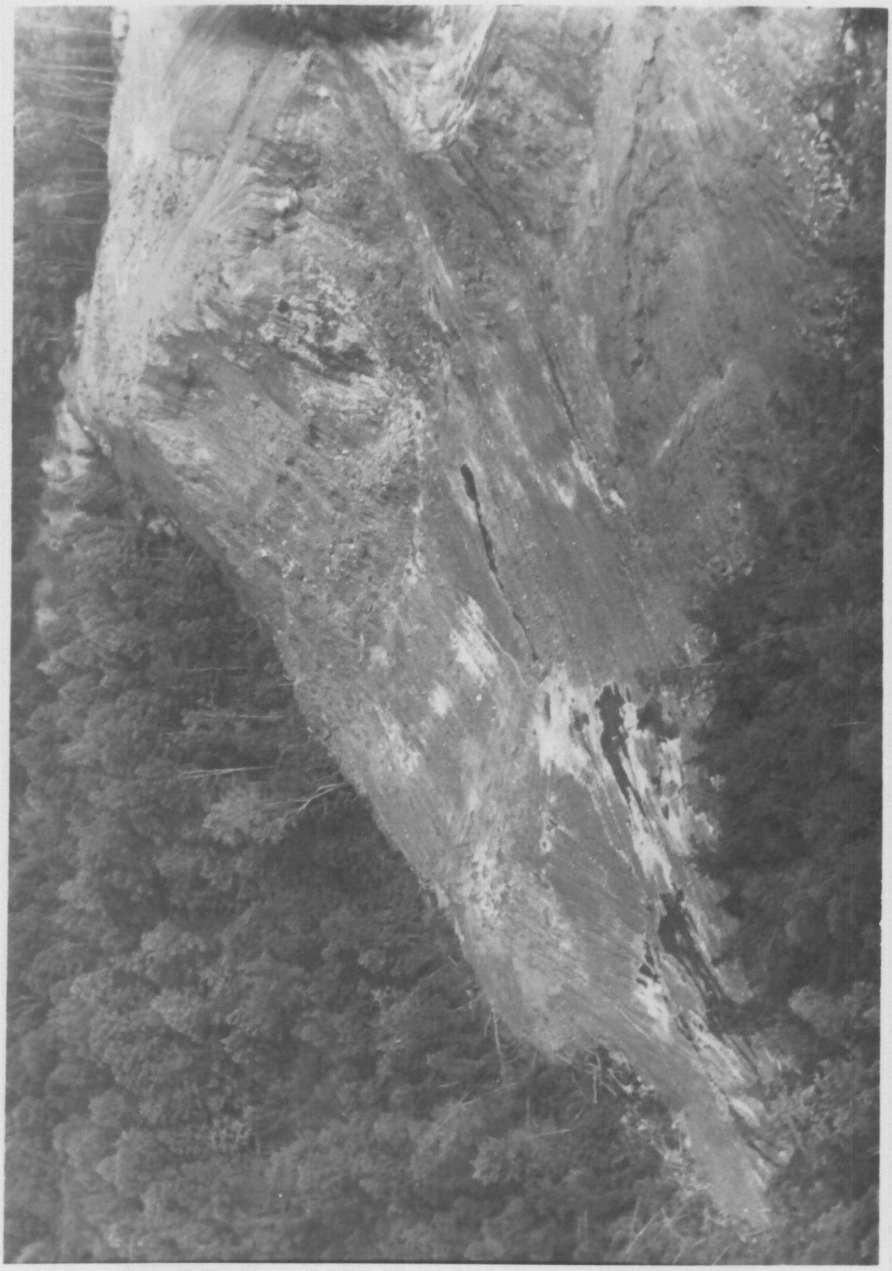


Figure 8. Photograph of Slide at Site C.

Reconstruction of Pre-failure Cross Section

Since all three sites came under intensive study after slope failure occurred, direct measurement of cross section shape immediately prior to failure was not possible. Several sources of information have been incorporated to provide a reasonably accurate estimate of the pre-failure cross sections. Extensive interviews were conducted with the respective mine superintendants, and Tennessee Division of Surface Mining Inspectors responsible for assuring compliance with the Tennessee surface mine regulations at the various sites. Premine plans and maps supplied additional information regarding proposed mine methods and spoil placement locations. Aerial photographs of surface mines in Tennessee are made annually by the Tennessee Valley Authority. Stero-graphic copies of these photographs taken before and after mining, along with U.S.G.S. 7.5 minute topographic quadrangle maps provided valuable information. In addition to the above information, the author, who at the time was employed by the Tennessee Division of Surface Mining, is personally familiar with, and frequently visited, each of the mine operations involved.

CHAPTER IV

PROCEDURES OF SOIL TESTING AND SLOPE STABILITY ANALYSIS

I. INTRODUCTION

An analytical investigation of slope stability requires the determination of several factors describing materials which comprise the slope. Included in these factors are: effective unit weight of the soil mass; Mohr-Coulomb strength parameters c and ϕ, utilized to describe the shear strength of the soil; and the location of existing seepage line or water table elevation.

Materials comprising failed slopes studied in this investigation can be divided into three types. At some depth beneath all slides was intact rock which did not undergo failure but served as a lower confining limit on the depth of failure surfaces. Above the intact rock, colluvium-residuum soil consisting of unconsolidated gravity deposited and inplace weathered material serve as a foundation for the fill bench. The fill bench consists of blasted shale and sandstone rock along with unconsolidated overburden which originally covered the coal seam.

Field and laboratory tests were utilized in determining the physical and engineering properties of the materials. Results of these tests were compared with values obtained from other research on similar materials in the general area.

Sampling and Testing

Since the fill material consists of a heterogeneous mixture of blasted rock and unconsolidated overburden placed with no specific compaction, undisturbed sampling of the waste fill material was considered unwarranted relative to difficulties encountered in sampling this type material. Representative disturbed samples were obtained for laboratory testing. In place densities of the fill material were determined by the sand cone method (ASTM D 1556). Disturbed samples were molded at moisture contents and densities consistent with those determined in the field by the sand cone density test. Remolded samples were tested in the unconsolidated undrained condition using triaxial and direct shear equipment. Results of the tests indicate that shear strength of waste fill material resulted primarily from internal friction. These materials exhibit very little cohesion. Shear strength parameters for spoil materials obtained from various sites were found to be very consistent. Since there was very little variation in the shear strength parameters, the values in Table I which represent an average of the test result, were utilized for all waste fill materials.

TABLE I

PROPERTIES OF MINE WASTE FILL

Fill Material (all sites)	
dry unit weight, γ	84 pcf
moisture content, w	13 percent
internal friction, ϕ	21 degrees
cohesion, c	155 psf

These values compare very closely with those presented by Drnevich et al. (1976) in research on mine overburden waste from the same general area. Drnevich et al. report values of internal friction angle, ϕ, of 33^{o} and cohesion C equal to 200 psf. The angle of internal friction of 33^{o} reported by Drnevich et al. is possibly representative of surface mine waste containing significant quantities of sandstone as opposed to larger percentages of quickly weathering shale investigated in this research. Samples tested by Drnevich et al. were also compacted to higher densities than those determined from field testing and utilized in this study.

Since the colluvium-residuum had undergone natural placement and subsequent natural consolidation, the need for undisturbed sampling was recognized. Material comprising the slopes under study had undergone failure and therefore was disturbed. Representative undisturbed samples of the material within the failed slope were taken immediately adjacent to the existing slides. Samples, 10 to 12 inches long, were obtained from a depth of approximately two feet below the surface. The samples were obtained using a hand operated driving hammer manufactured by Soil Test Inc. and 2.8 inch diameter Shelby tubes. The amount of consolidation resulting from driving the tubes was noted for each sample and was consistently less than one inch. Shear strength of the undisturbed samples was determined using a series of unconsolidated-undrained, staged triaxial tests. A summary of the test results showing parameters used in analysis is given in Table II.

TABLE II

PROPERTIES OF IN-SITU COLLUVIUM/RESIDUUM SOIL

Colluvium - Residium		
Site A		
	dry unit weight, γ	113.4 pcf
	moisture content, w	19.0 percent
	internal friction, ϕ	5.0 degree
	cohesion, c.	720 psf
Site B		
	dry unit weight, γ	113.5 pcf
	moisture content, w.	18.0 percent
	internal friction, ϕ	8.0 degrees
	cohesion, c.	780 psf
Site C		
	dry unit weight, γ	113.7 pcf
	moisture content, w	17.8 percent
	internal friction, ϕ	7.0 degrees
	cohesion, c.	740 psf

Since the slopes were investigated after failure, the use of field measuring devices such as piezometers to determine the presence of water table or seepage pressures was not possible. A knowledge of each individual site was necessary to be able to estimate the possibility of a water table and its location.

Method of Stability Analysis

Several factors entered into the decision regarding which of the many stability analysis procedures should be utilized in the analysis procedure. A very important factor in choosing the method was shape of the failure surface. As was stated, each of the failures studied herein involved a relatively deep seated failure surface which included foundation material below the fill. The failure surfaces could be closely approximated by a circular arc and therefore lend themselves to analysis by one of the Swedish Methods. Since the slopes were made up of layered materials with different properties, the assumption of a homogeneous cross section as required by the ϕ - Circle Method was not suitable. A method of slices was chosen to allow consideration of various layers of material. Bishop's Simplified Method of Slices was used because it is sufficiently accurate without requiring extensive calculations.

The University of Tennessee IBM 360/60 computer was employed for the calculations utilizing a program of Bishop's Simplified Method written by R. M. Leary (1970). A listing of the program is given in the Appendix. The computer program is capable of analyzing a layered cross section containing a horizontal or inclined water table. A number of circles with varying radii and center location were systematically analyzed until the critical circle corresponding to the minimum factor of safety was found.

In addition to analyzing the cross sections illustrated in Chapter III, the initial cross sections were varied to determine the effect which various factors have on slope stability. Factors which were investigated included: fill slope angle, benches in the fill slope,

height of fill, distance downslope to which the fill extended, depth of colluvium-residuum layer and effect of water table and seepage forces within the slope.

CHAPTER V

DISCUSSION OF RESULTS

I. INTRODUCTION

In the study of landslide or slope failure mechanisms, post-failure analysis has two distinct advantages over the analysis of stable slopes. First, the factor of safety of failed slopes is known to have been unity at the time of the failure. This fact can be an important tool in recreating the events leading to failure. Although there is no absolute assurance that values assigned to all the various parameters involved in a stability analysis are correct merely because the resulting factor of safety calculated is unity, one can be assured that at least one, possibly more of the parameters, or the analysis procedure is incorrect if the factor of safety of failed slope varies significantly above or below unity. A variance in factor of safety greater than 10% (that is $\pm$ 0.1) was considered significant, in this study.

The second advantage realized in studying post-failure slopes is that the location and shape of the failure surface can be observed. Knowledge of the shape of the failure surface is particularly helpful in determining the analysis procedure which should be used in investigating the slope (i.e. whether circular arc, wedge, or composite failure surfaces should be analyzed). If stability analysis indicates a critical failure surface located other than in position observed, it can be inferred that the cross section analyzed is not entirely consistent with the

conditions existing in the actual slope. It should also be pointed out that the factor of safety corresponding to that critical failure arc should be approximately unity. Knowledge of these two factors allow rigid control over the variables involved in analysis.

Because of the close similarities of sites A and B, in that both fill benches were limited to 125 feet downslope as specified in Tennessee surface mining regulations in effect prior to July 1, 1975, results of the investigation of those sites will be presented together. Site C, which involved a designated additional storage area, will be discussed separately.

Sites A and B

The cross section of site A as it was constructed is illustrated in Figure 3, page 34, of Chapter III. Bishop's Simplified Method of Slices was used to locate the critical failure surface and determine the factor of safety for that cross section. The stability analysis yielded a factor of safety equal to 1.009 which indicates that the outslope, as constructed, existed in a metastable condition. The critical circle corresponding to that factor of safety extends to the full depth of the colluvium layer which comprised the natural berm and foundation for the fill bench. The calculated critical surface correlates well with the location of the actual failure surface as observed on the site.

Original mining plans for site A included constructing the fill bench, as shown in the cross-section, for temporary spoil storage. During reclamation phase of mining, which was to follow closely behind removal of the coal, the size of the fill bench was to be reduced by removing the upper portion of the fill and placing the material into the pit from

which the coal was removed. However, soon after the fill bench was constructed in late fall, December rains began and prevented the regrading process. Rains resulted in increasing the unit weight of the spoil material and produced a situation in the foundation material conducive to liquefaction. At one time water was ponded temporarily in the pit behind the fill bench producing seepage forces in the foundation material. The slope, as constructed existed in a metstable condition. The effects of the rain on the slope were sufficient to cause a series of slope failures resulting from liquefaction.

The cross section of site B as it was constructed is illustrated in Figure 5, page 37, of Chapter III. Stability analysis of that cross section by Bishop's Simplified Method yields a factor of safety of 0.914 with corresponding critical circle extending the full depth of the fill material but not into the colluvium which serves as a foundation for the fill. The slope as constructed existed approximately 10 months without failure. Although a true factor of safety of 0.914 indicates that the slope would have failed immediately, the reader is reminded that present methods of stability analysis do not permit calculating the exact factor of safety. A calculated factor of safety of 0.914 is sufficiently close to unity to indicate that the slope existed in a metstable to unstable condition immediately after construction.

A primary difference between the fill benches at site A and site B is that part of the fill at site A was intended to be temporary whereas all the fill at site B was intended as permanent. The fill at site B was constructed higher than at site A because overburden from a higher seam in the multiple seam operation was pushed onto the fill bench of

the lower seam. This process allowed construction of a higher fill without pushing material up hill.

Although no water was ever ponded behind the fill bench at site B, it is believed that water from wet weather springs saturated the toe of the fill and colluvium foundation material cuasing a slight reduction in shear strength and a corresponding reduction in factor of safety and relocation of failure surface sufficient to cause a slope failure involving the foundation material. The slope was able to exist during the spring, summer and fall because of the time required for saturation during the drier months. The observed failure surface at the site extended into the colluvial foundation material.

The purpose of discussion thus far has been to establish the fact that the failures which occurred at sites A and B can be closely predicted using the parameters and cross sections as given. Establishing thus fact, it is desirable to investigate the effects of different variables on the stability of slopes. Tables III and IV summarize the various cross sections derived from sites A and B respectively which were analyzed. Resulting factors of safety and location of the corresponding failure arc are also given. The cross sections as constructed for sites A and B, numbered 1A and 1B respectively in the tables are illustrated in Chapter III.

For most cross sections in Tables III and IV factors of safety corresponding to two possible failure arcs are given. One factor of safety corresponds to a failure arc which extends the full depth into the colluvial layer to the colluvial-rock interface. The other factor

TABLE III

SUMMARY OF RESULTS AT SITE A

Cross Section No.	Fill Slope Angle Degrees	Downslope Distance (ft.)	Colluvium Depth and Internal Friction	Depth of Critical Circle	Water Table	Factor of Safety
1A	33	125	5ft, $\phi=5^0$	Colluvium-Rock Interface	No	1.009
				Fill-Colluvium Interface		1.013
2A	33	125	10ft, $\phi=5^o$	Colluvium-Rock Interface	No	0.948
3A	33	125	5ft, $\phi=0^o$	Colluvium-Rock Interface	No	0.868
4A	33	125	10ft, $\phi=0^o$	Colluvium-Rock Interface	No	0.774
5A	35	125	5ft, $\phi=5^0$	Colluvium-Rock Interface	Yes	0.912
				Fill-Colluvium Interface		1.045
6A	33	125	5ft, $\phi=5^o$	Colluvium-Rock Interface	Yes	0.934
				Fill-Rock Interface		0.973
7A	30	125	5ft, $\phi=5^o$	Fill-Colluvium Interface	Yes	1.083
				Colluvium-Rock Interface		1.121
8A	35	100	5ft, $\phi=5^o$	Within Fill	Yes	0.944
				Colluvium-Rock Interface		1.200
9A	33	100	5ft, $\phi=5^o$	Fill-Colluvium Interface	Yes	1.057
				Colluvium-Rock Interface		> 1.350
10A	30	100	5ft, $\phi=5^o$	Fill-Colluvium Interface	Yes	1.213
				Colluvium-Rock Interface		1.286
11A	35	50	5ft, $\phi=5^o$	Within Fill	Yes	1.183
12A	33	50	5ft, $\phi=5^o$	Within Fill	Yes	1.279

TABLE IV

SUMMARY OF RESULTS AT SITE B

Cross Section No.	Fill Slope Angle Degrees	Downslope Distance (ft.)	Location of Critical Circle	Water Table	Factor of Safety
18	35	125	Fill-Colluvium Interface Colluvium-Rock Interface	No	0.914 0.982
2B	33	125	Fill-Colluvium Interface Colluvium-Rock Interface	No	1.016 1.061
3B	30	125	Fill-Colluvium Interface Colluvium-Rock Interface	No	1.178 1.223
4B	35	125	Colluvium-Rock Interface Fill-Colluvium Interface	Yes	0.958 0.958
5B	33	125	Colluvium-Rock Interface Fill-Colluvium Interface	Yes	1.021 1.091
6B	30	125	Colluvium-Rock Interface Fill-Colluvium Interface	Yes	1.171 1.344
7B	35	100	Colluvium-Rock Interfacw Fill-Colluvium Interface	Yes	0.967 1.014
8B	33	100	Within Fill Colluvium-Rock Interface	Yes	1.077 > 1.200
9B	30	100	Within Fill	No Effect	1.241
10B	35	50	Within Fill	No Effect	1.100
11B	33	50	Within Fill	No Effect	1.160
12B	30	50	Within Fill	No Effect	1.322

of safety corresponds to a failure arc which extends the full depth of the fill material to the fill colluvium interface. It can be seen that, particularly for the steeper fill slopes (33^{o} and 35^{o}), the difference in the two factors of safety is very small (in most cases less than 0.05), indicating two potential failure surfaces exist. The relative strength between the fill material and colluvium would dictate which failure surface would control. Lower strength fill material would tend to promote failure along the fill-colluvium interface whereas lower strength foundation material would promote failure along the colluvium-rock interface.

In addition to the relative strengths of the foundation and fill materials, the angle of the fill slope and the distance downslope to which the fill extends are important in determining whether the colluvium-rock interface or the fill-colluvium interface is most critical with respect to slope stability. Steeper, longer slopes, because of their increased size and weight tend to produce deeper failure arcs. Fills extending shorter distances downslope tend to produce shallow failure arcs.

It has been suggested by Huang (1977) that failures along the fill-colluvium interface should be analyzed using a wedge method. The author agrees that the wedge method is very applicable but would also like to point out that methods utilizing a circular arc (such as the Bishop Method utilized in this study) are capable of analysis surfaces closely approximating the surface analyzed by the wedge method. The approximation is obtained when a circular arc of very large radius is analyzed. It was found in this study that the critical surfaces corresponding to the

fill-colluvium depth and the colluvium-rock depth were characterized by arcs of large radius which parallel the confining layer.

The addition of a water table to the various cross sections had an interesting effect in that it resulted in very little change in the factor of safety, and in two cases (cross section nos. 1B-4B and 2B-5B) resulted in a slight increase in the factor of safety. The reason for the seemingly unusual effect is that, because of the restraints of the boundary layers, the water table as placed in the cross sections have a very steep downward gradient. The resulting seepage forces produced have a large component in the vertical downward direction and a small component in the horizontal direction. The vertical component of the seepage force has the mechanical effect of increasing the weight of the material at the toe of fill therefore offsetting the effect of the horizontal component of the seepage forces. It is by no means proposed here that the presence of water within the slope is beneficial to stability. It is however proposed that instability resulting from water is probably not a result of seepage forces but a result of the effects of water on shear strength and the suspectibility to liquefaction of the colluvial layer. As can be seen in Table III, cross sections 3A and 4A, the result of assigning $\phi = 0$ to the colluvial layer, as would occur in the case of liquefaction, is a greatly reduced factor of safety.

It is apparent from the fact that the slopes failed that the design procedure (which consisted of complying with the Tennessee surface mine regulations in effect at that time) was not adequate. Current Tennessee surface mining regulations have become more restrictive than

those in effect at the time of construction of the fills studied herein. Current regulations allow fill benches to extend 50 feet downslope as opposed to the 125 feet limit in effect at the time of construction of the fill benches at sites A and B. Fill slopes inclined at 35^{o} maximum are currently permitted. Information in Tables III and IV indicate that fills constructed according to current restrictions would have been stable at either site. The least stable fill being cross section no. 10B with a 35^{o} fill slope and a factor of safety of 1.100.

An important question to be considered is; are the current regulations the best design, considering economics of production, slope stability, and other environmental effects. From a standpoint of slope stability, a more desirable grading configuration which restricts the fill slope to 30^{o} but allows a greater downslope spoil distance of 100 feet results in a factor of safety of 1.213 and 1.241 for sites A and B respectively. Such a grading configuration allows more spoil storage area and more working room for the mining operation thereby reducing production costs. The reduced fill slope angle allows greater ease of establishing vegetation therefore reducing harmful siltation.

It is felt that a factor of safety of 1.100 resulting from the current regulations is too low for design of permanent slopes even in a low risk area usually affected by surface mine operations. A factor of safety of 1.200 should be the minimum allowed for the design of permanent fill benches in low risk areas. For temporary fill benches a design factor of safety of 1.100 would be acceptable. A temporary fill being one which will be reduced in size during the reclamation and

regrading process. The alternative grading configuration stated above would provide the suggested factors of safety.

Site C

In mining areas where natural slopes are predominantly steeper than 28^{o} or other factors restrict fill bench construction, there is often a need for additional overburden storage area. At site C a storage area was selected which provided a natural slope of 16^{o} at the base of the fill. The storage area was cleared of trees and brush prior to placement of the mine wastefill. The fill was constructed in layers from the bottom upward, by hauling overburden to the storage area in earthmoving scrapers. After approximately six weeks of mining and fill placement, the fill and underlying colluvial foundation material experienced shear failure. The failure occurred prior to planned storage area configuration. Mining and fill construction took place during a dry fall season, and inspection of the slope shortly after failure revealed no seepage, indicating that water was not a factor in the failure. From inspection, it was apparent that the failure involved a rotational slump involving the foundation material.

Analysis of the fill cross section as it existed just prior to failure (Figure 7, page 41, Chapter III) results in a factor of safety of 0.984 with a corresponding failure arc extending to the fill depth of the colluvium layer.

A summary of cross sections derived from site C and results of analysis of those cross sections are given in Table V. Cross section no. 1C which corresponds to the fill just prior to failure, is illustrated in Chapter III.

TABLE V

SUMMARY OF RESULTS AT SITE C

Cross Section No.	Fill Slope Angle Degrees	Downslope Distance (ft.)	Colluvium Depth* and Internal Friction	Depth of Critical Circle	Factor of Safety
1C	33	175	7ft, $\phi=7^{o}$	Colluvium-Rock Interface	0.984
				Fill-Colluvium Interface	1.012
2C	35	175	7ft, $\phi=7^{o}$	Colluvium-Rock Interface	0.870
				Fill-Colluvium Interface	0.911
3C	33	175	10ft, $\phi=7^{o}$	Colluvium-Rock Interface	0.927
4C	35	175	10ft, $\phi=7^{o}$	Colluvium-Rock Interface	0.864
5C	33	175	7ft, $\phi=0^{o}$	Colluvium-Rock Interface	0.759
6C	30	175	7ft, $\phi=7^{o}$	Fill-Colluvium Interface	1.089
				Colluvium-Rock Interface	1.114
7C	28	175	7ft, $\phi=7^{o}$	Fill-Colluvium Interface	1.206
				Colluvium-Rock Interface	1.279
8C	Terraced	175	7ft, $\phi=7^{o}$	Colluvium-Rock Interface	1.353
				Fill-Colluvium Interface	1.385
9C	Terraced	175	10ft, $\phi=7^{o}$	Colluvium-Rock Interface	1.301
10C	Terraced	175	14ft, $\phi=7^{o}$	Colluvium-Rock Interface	1.245

*at 125 Feet Downslope

It can be seen from Table V that, as with sites A and B, two potential failure arcs with nearly equal factors of safety exist for the steeper slopes. Again, one potential failure arc corresponds to the colluvium-rock interface. The other corresponds to the fill-colluvium interface.

An important factor involved in the stability of a large fill such as the one at site C is the depth of the unconsolidated colluviul layer. When the critical circle depth corresponds to the colluvium-rock interface, which includes all cases in Table V except the 28^{o} and 30^{o} fill slopes and most of the cases in Tables III and IV where the slopes are longer than 100 feet and 33^{o} or steeper, increasing the depth of the colluvial layer results in a decrease in factor of safety.

It might be expected that a $\phi = 0^{o}$ analysis as previously discussed would be applicable to the loading situation involved in this fill construction. However, when such an analysis was attempted, the resulting factor of safety was 0.759 which is significantly below unity. If the $\phi = 0^{o}$ condition were applicable the fill would have experienced failure before it reached the size corresponding to the cross section analyzed. Similar results (Table III cross sections 3A and 4A) occurred for cross sections of site A analyzed using the $\phi = 0^{o}$ condition.

Three alternative design cross sections were analyzed. The effect of slope reduction in the form of a 28^{o} and 30^{o} fill slopes was evaluated. The resulting factors of safety were 1.206 for the 28^{o} fill slope and 1.089 for the 30^{o} fill slope. In both cases the critical failure arc did not include the colluvial layer. A terraced fill slope

was also evaluated. For the terraced slope, the critical circle did extend to the base of the colluvial layer and therefore the stability would be dependent upon the depth of unconsolidated colluvium. However, significantly higher factors of safety were obtained for the terraced slope even with increased depth of colluvium. The terraced slope also has the advantage of allowing better control of surface water.

For areas in which large volumes of overburden are concentrated into a large fill, it is felt that a higher design factor of safety than required for small slope fill benches is needed. Such fills should be designed with a minimum factor of safety of 1.250 or 1.300. The requirement of a higher factor of safety is justified because of the increased potential for damage should failure occur. Because of the complexities involved in the larger fills, it is difficult to write regulations which apply to every situation and do not infringe upon the ability and responsibility of engineering judgement. Regulation of engineering design leads to either gross over design which is uneconomical, or possible under design with resulting failures. The technology for proper individual design of overburden storage fills exists and should be employed.

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

I. CONCLUSIONS

This study was undertaken to determine the mechanisms involved in slope stability of overburden fill benches and storage areas. The following conclusions can be derived from the investigation.

1) Analysis of failed slopes in this study revealed the presence of two potential failure surfaces. One corresponds to the fill-colluvium interface; the other corresponds to the colluvium-rock interface. The presence of two potential surfaces indicates the metastable or critical condition of mine waste fill slopes.

2) The difference in factors of safety for the two failure surfaces is very small, particularly for fills extending more than 100 feet downslope with steep fill slopes. The relative strength of the colluvium and fill is important in dictating which surface is critical.

3) Seepage forces resulting from steeply inclined water tables produce little detriment to short-term slope stability of sites investigated because the horizontal component of the seepage force which tends to produce instability is offset by the large vertical component of the seepage force which tends to increase stability.

4) The detremental effects of water within the slopes are a result of decreased shear strength and increased potential for liquefaction of the slope material.

5) The depth of the colluvium layer upon which the fill is placed has a significant effect on slope stability when the colluvium-rock interface potential failure surface controls. Increasing depth of the colluvial layer results in decreasing factor of safety.

6) The $\phi = 0^{o}$ analysis can result in overly conservative factors of safety when applied to the stability analysis of mine waste fills.

7) Terracing the slope of a large fill is more effective with respect to increasing stability than reduction of fill slope angle. However for terraced slopes investigated, the colluvium-rock interface potential failure surface controlled. The stability of such slopes is therefore dependent upon the depth of the colluvial layer. For the reduced slopes (30^{o} and less) the fill-colluvium interface potential failure surface controlled. The stability of those slopes is not dependent upon the depth of the colluvium layer.

II. RECOMMENDATIONS

Much knowledge, developed in various disciplines of engineering, has direct applicability to problems encountered in surface mining of coal. There is an urgent need for applying the existing knowledge to surface mining to minimize environmental degradation and at the same time increase productivity. There is also a need for continuing research to refine existing theories and develop new technologies. The following recommendations can be made based upon the results of this research.

1) Tennessee surface mine regulations should adopt, possibly on a trial basis, an alternative grading configuration for fill benches consisting of 100 feet maximum downslope limitation and 30^{o} maximum fill slope angle.

2) Tennessee surface mine regulations should require or encourage individual design of all overburden storage sites larger than regulated fill benches.

3) The minimum factor of safety against slope failure for design of the overburden storage sites should be 1.25 or 1.30.

Additional research is needed in the following areas.

1) This study was limited to the analysis of circular arc failure surfaces. Additional information can be gained by comparing these results with methods of analysis utilizing the wedge and general or composite failure surfaces.

2) Additional triaxial and direct-shear testing on both foundation and overburden materials within the coal producing region is needed to allow broader application of the principals observed herein.

3) Field instrumented overburden storage sites are needed to extend the knowledge of the behavior of the large fills.

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APPENDIX

COMPUTERIZED ANALYSIS OF THE
STABILITY OF EARTH SLOPES

BY R. M. LEARY

```
      DIMENSION SOILHL(50),SOILVL(50),SOILHR(50),SOILVR(50)
      DIMENSION WEIGHT(50),COHES(50),PHI(50),WATRHL(10)
      DIMENSION WATRVL(10),WATRHR(10),WATRVR(10),BET(10)
      DIMENSION XSEC(100),ASEC(100),NSOIL(20),RFS(20)
      DIMENSION VFS(10),CRAD(100),CHOR(100),CVER(100)
      DIMENSION XRAD(20),HFS(20)
C  READ  INITIALIZATION DATA
  126 READ(1,2,END=125) NSL,NWL,INPUT,IALLFS,ITER,NUMCEN
    2 FORMAT(6I2)
C  READ SOIL DATA AND CONVERT PHI TO RADIANS
      DO 4 I=1,NSL
      READ (1,3) SOILHL(I),SOILVL(I),SOILHR(I),SOILVR(I),WEI
     1GHT(I),PHI(I),COHES(I)
    3 FORMAT(7F10.4)
    4 PHI(I)=PHI(I)/57.29578
C  READ WATER TABLE DATA AND COMPUTE
C  SLOPE ANGLE OF EACH LINE
      DO 5 I=1,NWL
      READ (1,3) WATRHL(I),WATRVL(I),WATRHR(I),WATRVR(I)
    5 BET(I)=ATAN((WATRVL(I)-WATRVR(I))/(WATRHR(I)-WATRHL(I)
     1))
C   PRINT OR DO NOT PRINT INPUT DATA DEPENDING
C   ON OPTION INPUT
      IF(INPUT) 6,6,8
    6 WRITE (3,7)
    7 FORMAT(24X,24HINPUT DATA NOT REQUESTED,//)
      GO TO 14
    8 WRITE (3,9)
    9 FORMAT(31X,10HINPUT DATA,//,10X,10HSOIL LINES,/,5
     1H LINE,3X,6HSOILHL,3X,6HSOILVL,3X,6HSOILHR,3X,6
     2HSOILVR,3X,6HWEIGHT,3X,3HPHI,3X,5HCOHES,/,9X,4(2HFT,7X
     3),3HPCF,4X,3HDEG,4X,3HPSF,/)
      DO 10 I=1,NSL
      DPHI=PHI(I)*57.29578
   10 WRITE (3,11) I,SOILHL(I),SOILVL(I),SOILHR(I),SOILVR(I)
     1,WEIGHT(I),DPHI,COHES(I)
   11 FORMAT(2X,I3,5F9.2,F6.1,F9.2)
      WRITE (3,12)
   12 FORMAT(//,10X,17HWATER TABLE LINES,/,5H LINE,2X,6
     2HWATRHL,3X,6HWATRVL,3X,6HWATRHR,3X,6HWATVR,/,9X,4(2
     2HFT ,7X),/)
      DO 13 I=1,NWL
```

```
   13 WRITE(3,11) I, WATRHL(I),WATRVL(I),WATRHR(I),WATRVR(I)
C  LOOP FOR MULTIPLE STARTING CENTERS
   14 DO 124 ICEN=1,NUMCEN
C  READ STARTING CENTER DATA
      READ (1,3) HORZCN,VERTCL,RADIUS,RADINC,GRID
C  PRINT SEARCH INFORMATION
      WRITE (3,15) RADINC,GRID
   15 FORMAT(//,10X,19HRADIUS INCREMENT IS,F5.1,5X,17
     2HGRID INCREMENT IS,F5.1)
C  PRINT OR DO NOT PRINT HEADINGS DEPENDING ON OPTION IALLFS
      IF(IALLFS) 18,18,16
   16 WRITE (3,17)
   17 FORMAT(//,2X,6HSEPMON,3X,6HDRVMOM,3X,6HPHIMOM,3X,6
     1HCHSMON,3X,20HHORZCN VERTCL RADIUS,/,2X,4(6HFT-KIP,3X
     2),3(4H  FT,3X),12HFSNYS  FSBIS,/)
   18 NUMRAD=0
      NUMLOC=0
      IVER=0
   19 INDEX=0
      IHOR=0
   20 IRAC = 0
      NUMLOC=NUMLOC+1
   21 K=0
      NUMRAD=NUMRAD+1
C  COMPUTE INTERSECTIONS OF CIRCLE WITH SOIL STRATA LINES
      DO 32 I=1,NSL
      A=(SOILVR(I)-SOILVL(I))/(SOILHR(I)-SOILHL(I))
      B=SOILVL(I)-VERTCL-SOILHL(I)*A
      C=B*B+HORZCN*HORZCN-RADIUS*RADIUS
      D=(A*B-HORZON)*(A*B-HORZON)-(A*A+1)*C
      IF(D) 32,22,22
   22 X1=(HORZON-A*B-SQRT(D))/(A*A+1)
      IF (X1-(HORZON-RADIUS))151,152,152
  151 X1=HORZON-RADIUS
  152 X2=(HORZON-A*B+SQRT(D))/(A*A+1)
C  STORE INTERSECTIONS AND DISCONTINUITIES IN XSEC
      IF(SOILHL(I)-X1) 23,25,25
   23 IF(SOILHR(I)-X1)27,24,24
   24 K=K+1
      XSEC(K)=X1
      GO TO 27
   25 IF(SOILHL(I)-X2)26,26,27
   26 K=K+1
      XSEC(K)=SOILHL(I)
   27 IF(SOILHR(I)-X2)28,28,30
   28 IF(SOILHR(I)-X1)32,29,29
   29 K=K+1
      XSEC(K)=SOILHR(I)
      GO TO 32
   30 IF(SOILHL(I)-X2) 31,31,32
   31 K=K+1
```

```
      XSEC(K)=X2
   32 CONTINUE
C  K IS THE NUMBER OF ELEMENTS IN XSEC
C  DELETE DUPLICATE ELEMENTS FROM XSEC
      JJ = 0
  215 K1=K-1
      DO 35 I=1,K1
      K = K - JJ
      K1 = K1 - JJ
      JJ = 0
      N=I+1
      DO 35 J=N,K
      IF(XSEC(I)-XSEC(J-JJ))35,33,35
   33 DO 34 L=J,K1
   34 XSEC(L-JJ) = XSEC(L-JJ+1)
      JJ = JJ + 1
   35 CONTINUE
C  ARANGE ELEMENTS OF XSEC IN ORDER OF
C  MAGNITUDE AND STORE IN ASEC
C  K IS NOW THE NUMBER OF ELEMENTS IN ASEC
      DO 40 I=1,K
   36 DO 39 J=I,K
      IF(I-J) 37,39,37
   37 IF(XSEC(I)-XSEC(J)) 39,38,38
   38 B=XSEC(J)
      XSEC(J)=XSEC(I)
      XSEC(I)=B
   39 CONTINUE
   40 ASEC(I)=XSEC(I)
      IF (K-1)163,163,164
  163 NUMRAD=NUMRAD-1
      GO TO 97
C  INITIALIZE COMPONENTS OF NEW YORK STATE FACTOR OF SAFETY
  164 SEPMOM=0.
      CHSMOM=0.
      PHIMOM=0.
      DRVMOM=0.
      L=1
C  LOOP FOR NUMBER OF SEGMENTS K1
      DO 62 I=1,K1
C  DETERMINE WIDTH OF ALL SLICES IN SEGMENT I
      IF((ASEC(I+1)-ASEC(I))-2.) 41,41,42
   41 NSLIC=1
      GO TO 47
   42 IF((ASEC(I+1)-ASEC(I))-(.1*RADIUS)) 43,43,44
   43 NSLIC=5
      GO TO 47
   44 IF((ASEC(I+1)-ASEC(I))-(.5*RADIUS)) 45,45,46
   45 NSLIC=10
      GO TO 47
   46 NSLIC=20
```

```
   47 ASLIC=NSLIC
C  NSLIC IS THE NUMBER OF SLICES IN SEGMENT I
      WIDTH=(ASEC(I+1)-ASEC(I))/ASLIC
      XS=ASEC(I)-WIDTH/2.
C  LOOP FOR SLICES IN SEGMENT
      DO 61 J=1,NSLIC
C  XS IS THE X-COORDINATE OF TEH CENTERLINE OF THE SLICE
      XS=XS+WIDTH
C  YC IS THE Y-COORDINATE OF THE CIRCLE AT XS
      YC=VERTCL-SQRT((RADIUS**2)-((XS-HORZON)*(XS-HORZON)))
      YCI=YC
   48 IF(XS-WATRHR(L)) 50,50,49
   49 L=L+1
      GO TO 48
C  YW IS THE Y-COORDINATE OF THE WATER TABLE AT XS
   50 YW=WATRVL(L)-(XS-WATRHL(L))*TAN(BET(L))
      IF(YW-YC) 51,51,52
C  SEEP IS THE SEEPAGE FORCE
   51 SEEP=0.
      GO TO 53
   52 SEEP=WIDTH*(YW-YC)*62.4*SIN(BET(L))
   53 SEPMOM=SEPMOM+SEEP*(SIN(BET(L))*(HORZON-XS)+COS(BET(L)
     1)*(VERTCL-YC-(YW-YC)/2.))
C  COMPUTE SLICE WHICH IS THE WEIGHT OF SLICE
      SLICE=0.
      N=0
      M=NSL
   54 YS=SOILVL(M)-((XS-SOILHL(M))*(SOILVL(M)-SOILVR(M))/(SO
     1ILHR(M)-SOILHL(M)))
      IF(YS-YC) 58,58,55
   55 IF(XS-SOILHL(M)) 58,58,56
   56 IF(SOILHR(M)-XS) 58,58,57
   57 N=N+1
      NSOIL(N)=M
      SLICE=SLICE+WEIGHT(M)*WIDTH*(YS-YC)
C  YS IS THE Y-COORDINATE OF SOIL LINE M AT SLICE CENTERLINE
      YC=YS
   58 IF(M-1) 60,60,59
   59 M=M-1
      GO TO 54
   60 DRVMOM=DRVMOM+SLICE*(HORZON-XS)
C  NSOIL(1) IS THE NUMBER OF THE SOIL LINE
C  WHICH DESCRIBES THE SOIL
C  PROPERTIES AT THE BOTTOM OF THE SLICE
      PHIMOM=PHIMOM+SLICE*TAN(PHI(NSOIL(1)))*(VERTCL-YCI)
   61 CHSMOM=CHSMOM+COHES(NSOIL(1))*WIDTH*(RADIUS/(VERTCL-YC
     1I))*RADIUS
   62 CONTINUE
C  FSOLD IS THE OLD FACTOR OF SAFETY FOR BISHOP ITERATION
      FSNYS=(PHIMOM+CHSMOM)/(DRVMOM+SEPMOM)
      FSOLD=FSNYS+.2
```

```
C   IF FSNYS IS LESS THAN 0.60, TERMINATE PROGRAM
      IF(FSNYS-.6) 63,65,65
   63 WRITE(3,64)
   64 FORMAT(/,5X,33HA NEW YORK STATE FACTOR OF SAFETY,/,12X
     ,18HOF LESS THAN 0.600,/,11X,20HHAS BEEN ENCOUNTERED)
      GO TO 125
C   INITIALIZE COMPONENTS OF BISHOP SAFETY FACTOR
   65 PARTA=0.
      PARTB=0.
      PARTC=0.
      L=1
C   LOOP FOR NUMBER OF SEGMENTS K1
      DO 87 I=1,K1
C   DETERMINE WIDTH OF ALL SLICES IN SEGMENT I
      IF((ASEC(I+1)-ASEC(I))-2.) 66,66,67
   66 NSLIC=1
      GO TO 72
   67 IF((ASEC(I+1)-ASEC(I))-(.1*RADIUS)) 68,68,69
   68 NSLIC=5
      GO TO 72
   69 IF((ASEC(I+1)-ASEC(I))-(.5*RADIUS)) 70,70,71
   70 NSLIC=10
      GO TO 72
   71 NSLIC=20
   72 ASLIC=NSLIC
      WIDTH=(ASEC(I+1)-ASEC(I))/ASLIC
      XS=ASEC(I)-WIDTH/2.
C   LOOP FOR SLICES IN SEGMENT
      DO 86 J=1,NSLIC
      XS=XS+WIDTH
      YC=VERTCL-SQRT((RADIUS*RADIUS)-((XS-HORZON)*(XS-HORZON
     1)))
      YCI=YC
   73 IF(XS-WATRHR(L)) 75,75,74
   74 L=L+1
      GO TO 73
   75 YW=WATRVL(L)-(XS-WATRHL(L))*TAN(BET(L))
      IF(YW-YC) 76,76,77
   76 SEEP=0.
      GO TO 78
   77 SEEP=WIDTH*(YW-YC)*62.4*SIN(BET(L))
   78 SLICE=0.
      N=0
      M=NSL
   79 YS=SOILVL(M)-((XS-SOILHL(M))*(SOILVL(M)-SOILVR(M))
     1/(SOILHR(M)-SOILHL(M)))
      IF(YS-YC) 83,83,80
   80 IF(XS-SOILHL(M)) 83,83,81
   81 IF(SOILHR(M)-XS) 83,83,82
   82 N=N+1
      NSOIL(N)=M
```

```
      SLICE=SLICE+WEIGHT(M)*WIDTH*(YS-YC)
      YC=YS
   83 IF(M-1) 85,85,84
   84 M=M-1
      GO TO 79
   85 ALPHA=ATAN((HORZCN-XS)/(VERTCL-YCI))
      PARTA=PARTA+(SLICE+SEEP*SIN(BET(L)))*SIN(ALPHA)+SEEP*C
     1OS(BET(L))*(VERTCL-YCI-(YW-YCI)/2.)/RADIUS
      PARTB=PARTB+(COHES(NSOIL(1))*WIDTH+(SLICE+SEEP*SIN(BET
     1(L)))*TAN(PHI(NSOIL(1))))/(FSOLD*COS(ALPHA)+TAN(PHI(NS
     2OIL(1)))*SIN(ALPHA))
   86 PARTC=PARTC+(COHES(NSOIL(1))*WIDTH+(SLICE+SEEP*SIN(BET
     1(L)))*TAN(PHI(NSOIL(1))))*TAN(PHI(NSOIL(1)))*SIN(ALPHA
     2)/((FSOLD*COS(ALPHA)+TAN(PHI(NSOIL(1)))*SIN(ALPHA))**2
     3)
   87 CONTINUE
      FSBIS=FSOLD*(1.-(PARTA-PARTB)/(PARTA-PARTC))
C  IF FSBIS IS NOT NEARLY EQUAL TO FSOLD, REPLACE FSOLD WITH
C  FSBIS AND COMPUTE A NEW FSBIS
      IF(ABS(FSBIS-FSOLD)-.001) 89,89,88
   88 FSOLD=FSBIS
      GO TO 65
C  SEARCH USING FSNYS OR FSBIS DEPENDING ON OPTION ITER
   89 IF(ITER) 90,90,91
   90 IRAD=IRAD+1
      RFS(IRAD)=FSNYS
      GO TO 92
   91 IRAD=IRAD+1
      RFS(IRAD)=FSBIS
C  PRINT OR DO NOT PRINT FACTORS OF SAFETY
C  DEPENDING ON OPTION IALLFS
   92 IF(IALLFS) 95,95,93
C     CONVERT MOMENT COMPONENTS OF FSNYS TO FT-KIPS
   93 SEPMOM=SEPMOM*.001
      DRVMOM=DRVMOM*.001
      CHSMOM=CHSMOM*.001
      PHIMOM=PHIMOM*.001
      WRITE(3,94)SEPMOM,DRVMOM,PHIMOM,CHSMOM,HORZON,VERTCL,R
     1ADIUS,FSNYS,FSBIS
   94 FORMAT(4F9.0,3F7.1,2F7.3)
C  IF ONLY ONE FS HAS BEEN COMPUTED AT THIS LOCATION,
C  INCREASE RADIUS BY RADINC AND COMPUTE NEW FS
   95 IF(IRAD-1) 97,97,96
C  IF FS IS LESS THAN PREVIOUS FS, INCREASE RADIUS BY RIDINC
C  NEW FS.  IF GREATER, CHANGE CIRCLE CENTER LOCATION
C  AND COMPUTE NEW FS.  IF GREATER, CHANGE
C  CIRCLE CENTER LOCATION
   96 IF(RFS(IRAD)-RFS(IRAD-1)) 97,97,98
   97 RADIUS=RADIUS+RADINC
      GO TO 21
   98 IHOR=IHOR+1
```

```
      XRAD(IHOR)=RADIUS-RADINC
C  HFS IS THE MINIMUM  FS COMPUTED AT
C  A CIRCLE CENTER LOCATICN
      HFS(IHOR)=RFS(IRAD-1)
C  IF ONLY ONE LOCATICN HAS BEEN USED,
C  INCREASE HORIZCN BY GRID AND COMPUTE NEW HFS
      IF(IHOR-1) 99,99,100
   99 RADIUS=RADIUS-RADINC*2.
      HORZCN=HORZCN+GRID
      GO TO 20
C  IF HFS AT NEW LOCATIPN IS LESS THAN HFS AT PREVIOUS
C  LCCATICN.CONTINUE IN SAME HORIZONTAL DIRECTION. IF
C  GREATER, AND MORE THAN TWO LOCATIONS HAVE BEEN TRIED ON
C  SAME HCRIZONTAL LINE, INCREASE VERTICLE BY GRID AND
C  CCMPUTE MINIMUM HFS CN NEW HORIZONTAL LINE.  IF GREATER
C  AND CNLY TWO LOCATIONS HAVE BEEN TRIED, PROCEED TOWARD
C  LEFT CN SAME HORIZCNTAL LINES
  100 IF(HFS(IHOR)-HFS(IHCR-1)) 101,101,102
C  IF INDEX IS ZERO, SEARCH IS PROCEEDING TO RIGHT, IF
C  PCSITIVE, SEARCH IS PROCEEDING TO LEFT ON HORIZONTAL LINE
C  PRESENTLY BEING SEARCHED
  101 IF(INDEX) 99,99,104
  102 IF(IHOR-2) 103,103,106
  103 INDEX=1
      HORZCN=HORZCN-GRID*2.
      RADIUS=RADIUS-RADINC*2.
      GC TC 20
  104 IF(HFS(IHOR)-HFS(1)) 105,105,109
  105 INDEX=INDEX+1
      HORZCN=HORZCN-GRID
      RADIUS=RADIUS-RADINC*2.
      GC TC 20
  106 IF(INDEX) 107,107,109
  107 IVER=IVER+1
      CRAD(IVER)=XRAD(IHOR-1)
      CHCR(IVER)=HORZCN-GRID
      CVER(IVER)=VERTCL
C  VFS IS THE MINIMUM HFS ON HORIZONTAL LINE
      VFS(IVER)=HFS(IHCR-1)
      RADIUS=RADIUS-RADINC*2.+GRID
      HORZCN=HORZCN-GRID
      VERTCL=VERTCL+GRID
  108 IF(IVER-1) 19,19,112
  109 IVER=IVER+1
      CHCR(IVER)=HORZCN+GRID
      CVER(IVER)=VERTCL
      RADIUS=RADIUS-RADINC*2.+GRID
      HORZCN=HORZCN+GRID
      VERTCL=VERTCL+GRID
      IF(INDEX-1) 110,110,111
  110 VFS(IVER)=HFS(1)
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      CRAD(IVER)=XRAD(1)
      GO TO 108
  111 VFS(IVER)=HFS(IHOR-1)
      CRAD(IVER)=XRAD(IHOR-1)
      IF(IVER-1) 19,19,112
  112 IF(VFS(IVER)-VFS(IVER-1)) 19,19,113
C  PRINT OUT WHICH FS HAS BEEN USED IN SEARCH, MINIMUM
C  SAFETY FACTOR, AND LOCATION AND RADIUS OF CRITICAL CIRCLE
  113 IF(ITER) 114,114,116
  114 WRITE(3,115)VFS(IVER-1)
  115 FORMAT(//,5X,36HSEARCH HAS USED NYS FACTOR OF SAFETY,/
     1,5X,31HTHE MINIMUM FACTOR OF SAFETY IS,F6.3)
      GO TO 118
  116 WRITE(3,117) VFS(IVER-1)
  117 FORMAT(//,5X,25HSEARCH HAS USED BISHOP FS,/,5X,17
     2HTHE MINIMUM FS IS, F6.3)
  118 WRITE(3,121)CHOR(IVER-1),CVER(IVER-1),CRAD(IVER-1)
  121 FORMAT(5X,38HCRITICAL CIRCLE CENTER COORDINATES ARE,/,
     21CX,10HHORIZON IS ,F6.2,10X,10HVERTCL IS ,F6.1,/,20X,1
     30HRADIUS IS ,F6.1)
C  PRINT OUT NUMBER OF SAFETY FACTORS COMPUTED
C  AND NUMBER OF  LOCATIONS
  122 WRITE(3,123) NUMRAD,NUMLOC
  123 FORMAT(/,5X,I5,3X,24HFS HAVE BEEN COMPUTED AT,I5,3X,9
     2HLOCATIONS)
  124 CONTINUE
      GO TO 126
  125 STOP
      END
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VITA

James Daniel Cowan was born in Brownsville, Tennessee on October 28, 1952. He attended elementary and high school in Bells, Tennessee until graduation in May 1970. In September of the same year, he entered The University of Tennessee at Knoxville and studied civil engineering for four years completing his requirements for a Bachelor of Science degree in Civil Engineering in December 1974. While working part-time toward a graduate degree, he accepted a position with the State of Tennessee, Division of Surface Mining and worked until September of 1976, at which time he accepted a research assistantship at the University of Tennessee at Knoxville and received his Master of Science degree in Civil Engineering in June 1977. He is an Associate Member of the American Society of Civil Engineers.