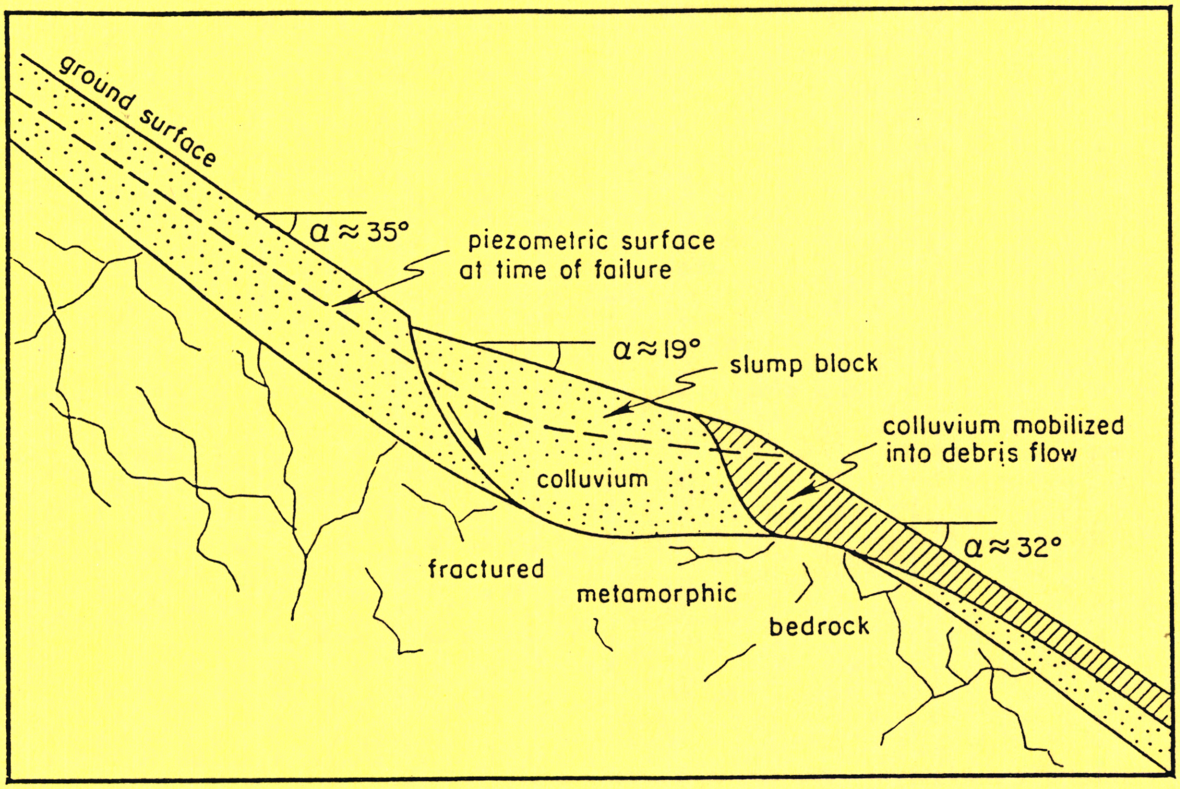
A PROBABILISTIC INVESTIGATION OF SLOPE STABILITY IN THE WASATCH RANGE, DAVIS COUNTY, UTAH



by
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ABSTRACT

A Probabilistic Investigation of Slope Stability in the Wasatch Range, Davis County, Utah (May 1991)

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Disastrous debris flows occurred in Davis County, Utah, in 1983 and 1984. The large number of debris flows was a result of heavy autumn rainfall, deep snow packs, and rapid spring snow melt. Colluvium in three drainage basins was analyzed and variations within the colluvium were taken into account in attempting to understand the mechanics of the initiation of landslides that mobilized into debris flows.

Data from the field, laboratory, and literature were used to perform a probabilistic factor of safety analysis using a U. S. Forest Service model, LISA (Level 1 Stability Analysis). For a given set of eleven parameters, LISA calculates a mean, median, maximum and minimum factor of safety, and a probability of failure. A deterministic model, dLISA, was used to determine the ratio of saturated thickness to total thickness of colluvial cover (ground-water ratio) necessary to induce failure for ranges of slope angle and soil depth representative of the three drainage basins. In this aspect of the study it was determined that at initiation (factor of safety = 1.0) the ground-water ratio was greater than 1.0. If the ground-water ratio exceeds 1.0, this may imply that artesian pressures exist in the colluvial cover. If artesian pressures existed at failure, then it is concluded that ground water flow is not always parallel to the ground surface and an infinite slope is not an accurate model for the slopes in Davis County, Utah.

ACKNOWLEDGMENTS

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Dr. Christopher C. Mathewson, chairman of my committee, played a key role in this study, as well as going beyond the call of duty in providing guidance and support in my professional development as an Engineering Geologist. I greatly acknowledge my committee members, Dr. J. R. Giardino and Dr. W. A. Dunlap for providing assistance, insight, and instruction. Robyn Skelton, a graduate student in geology at Texas A&M University, was conducting field research in Davis County at the same time I was, and her assistance, astuteness, and companionship were very valuable. Souren Ala, former geology graduate student at Texas A&M University (now in the Santa Ana, California office of Woodward-Clyde), and Kevin Coleman, a geology graduate student at Texas A&M University, provided perceptive discussions.

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Without the support of my family this thesis and my education could not have been completed. My parents, the late W. E. (Bill) Eblen Jr., and Mary Ann (Eblen) Marshall, taught me the value of an education and to always do my very best. My father was exceptional in sparking my interest in geology and pursuit of a career that I truly enjoy. Most importantly, he emphasized the value of always taking advantage of an opportunity to help someone. My step-father, John Marshall, was a trusted friend through it all. My sister, Nanette E. Moody, and brother, W. E. (Bill) Eblen III, were always supportive and interested.

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INTRODUCTION

Debris Flows

Natural hazards demand attention when there are risks of loss of property or life. Hazards, naturally occurring or human induced, are processes that have the potential to cause damage or injury. Risks are the exposure of something of value or humans to potential injury as a result of the hazards (Keaton, 1988).

One natural hazard that exists in mountainous terrain is a debris flow. Risks are associated with this hazard when people choose to develop areas at potentially dangerous locations. One such location is at the mouth of canyons through which debris flows move, or in the runout zone in front of the canyon.

Engineering geologists who deal with home site development in hazardous areas face the possibility of being criminally charged with negligence. Negligence is the omission to do something which an ordinarily wise person would do, or the doing of something which a wise person would not have done under similar circumstances (Schuster, 1978).

Debris flows are very dangerous and destructive natural hazards. The source of debris for debris flows is formed as a result of the weathering process. Weathering of underlying bedrock produces an

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This thesis follows the style of The Bulletin of the Association of Engineering Geologists overlying mantle of unconsolidated debris. As the debris is moved downslope as a result of gravity and deposited, it becomes, by definition, colluvium (Deere and Patton, 1971). Bates and Jackson (1984) define colluvium as unconsolidated gravity deposits above bedrock.

Problem Scope

Debris flows are a slope development process and are initiated when the overlying debris on a slope becomes saturated from a surface or subsurface source of water. During 1983 and 1984 numerous debris flows occurred along the Wasatch Front causing millions of dollars worth of damage. The three drainage basins that were studied are Lightning, Steed, and Centerville Canyons located in Davis County, Utah (Figure 1). Excessive pore pressures were developed in the colluvium due to heavy autumn rainfall, melting of large winter snowpacks, extended cool springs followed by a sudden and sustained temperature increase in the spring (Wieczorek et al., 1989).

Colluvium covers the slopes and is deposited as fans along the Wasatch Front. In order to more fully understand the debris flow process, it is important to understand how the properties of the colluvium and the geologic, hydrologic, and geomorphic conditions affect the stability of slopes in the area, and their importance in the initiation process.

Campbell (1975) studied debris flow initiation in the Santa Monica Mountains, California, in 1969. He (1975) attributed debris flow initiation solely to a temporary perched water table over bedrock, as a result of an intense rainstorm. Additional sources of pore pressures, that may have been present were ignored. Johnson (1987) discusses additional sources

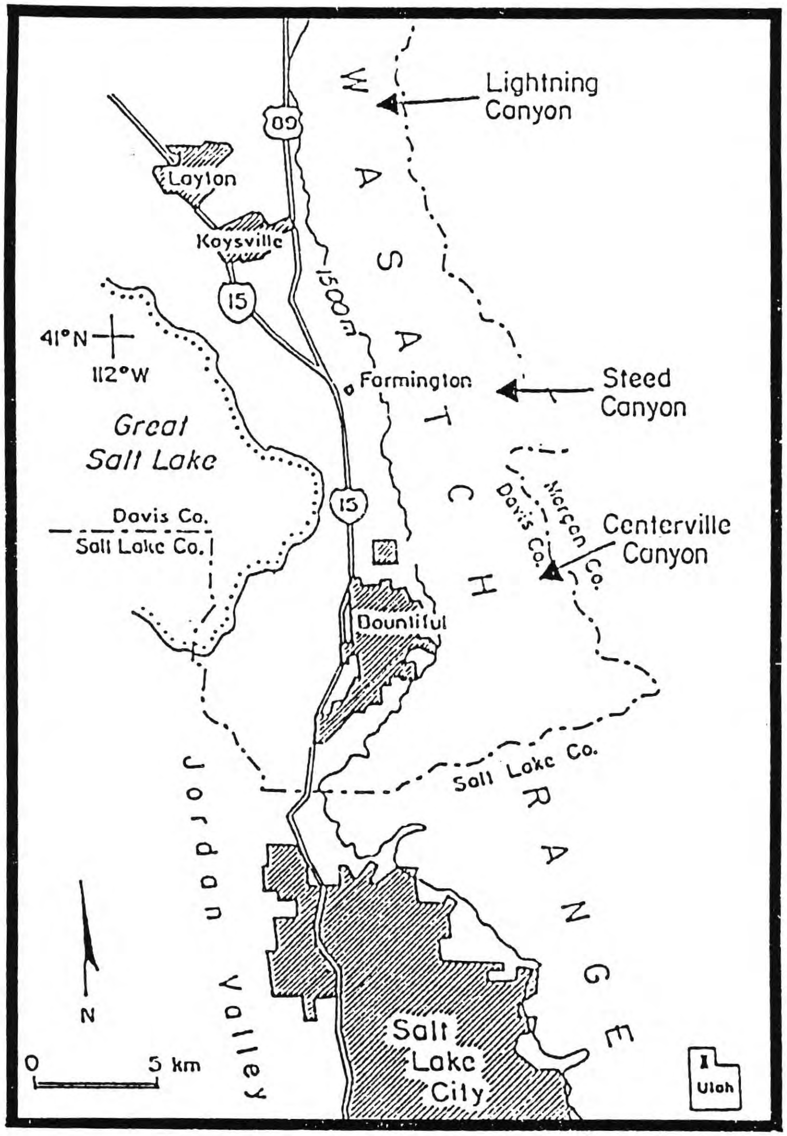


Figure 1. Location of Lightning, Steed, and Centerville Canyons in Davis County, Utah (modified from Ala, 1990).

of pore pressure development which will be discussed later. Mathewson and Santi (1987) reported "pop out" failures in Davis County, Utah. If the debris flow initiation potential of an area is to be understood, the initiation process has to be understood.

Purpose of Study

The purpose of this study is to test the applicability of using two U. S. Forest Service slope stability models in the Wasatch Range, Davis County, Utah, in order to understand debris flow initiation potential. LISA (Level I Stability Analysis), a U. S. Forest Service probabilistic, slope stability model, and a deterministic model, dLISA, will be used in this study. The applicability of the two models will be established as follows:

1) Establish parametric values for LISA and dLISA, based on data collection in the laboratory, field, and literature.

2) Perform a dLISA sensitivity analysis of input parameters.

3) Compare LISA stability values with empirical observations.

4) Back calculate input parameters to establish the geologic, hydrologic, and geomorphic conditions necessary to induce future debris flows.

PREVIOUS WORK

The first established white settlements began in Utah in the 1800's. Irrigation of farmland began in Utah in July of 1847. With the coming of the white settlements, many changes took place on watersheds in Utah. These changes included overgrazing the land and fire. Both damaged the land and significantly reduced infiltration.

Debris floods were a serious problem in the the Farmington-Centerville canyon areas in 1923 and 1930. Damages during the 1923-30 period totalled approximately $1,000,000. A commission was appointed by the governor of Utah to study the debris flood situation. Examinations of the colluvium after the 1930 debris flows showed that the rainfall had penetrated only 1 to 2 in. (Bailey et al., 1947). Rainfall had penetrated 6 to 10 in. in more densely vegetated areas. It was evident that the debris flows were initiated because the field capacity of the soil was reached and, then water was added to the colluvium at a faster rate than it could drain, causing sheet wash erosion.

The Civilian Conservation Corps constructed contour trenches and revegetated drainage basins as a result of the 1923-30 debris flows and therefore, increased the infiltration into the colluvium and reduced the sheetwash erosion potential (Bailey et al., 1947).

Some of the 1983 and 1984 debris flows appear to have been initiated by a different mechanism than the 1923-30 debris flows. Mathewson and others (1990) reported that there is a bedrock reservoir in which water is discharged from the fractured, metamorphic bedrock into the overlying colluvium. Excess recharge of a fully charged bedrock aquifer allows the pore pressure to build until the colluvium fails as a "blowout" and is mobilized into a debris flow (Mathewson et al., 1990).

The Steed Canyon landslide has received attention because it still exists as a detached block on a natural slope. Debris flows were initiated from the toe of the detached block in 1983 and 1984. Monteith (1988) analyzed the stability of the Steed Canyon landslide, and determined that the geotechnical factors that contribute to the slope stability of Steed Canyon are slope geometry, shear strength parameters, and oscillating ground-water levels.

Adjacent to the failed "east" swale of Steed Canyon is a stable "west" swale that did not fail. The soil conditions of the east and west swales vary drastically over short distances (Brooks,1986). In the borings that Brooks drilled in Steed Canyon a relatively cohesive layer was usually found immediately above bedrock in the east and west swales. This cohesive layer may have formed as a result of either residual or pedogenic processes.

Keaton (1988) developed a probabilistic model to evaluate hazards that are associated with alluvial fan sedimentation in Davis County, Utah. Keaton concluded that most of the canyons which yielded large volumes of sediment in 1983 and 1984 had no recorded sediment discharge in historic time. Small volumes of sediment were discharged from canyons that had discharged large volumes in the 1920's and 1930's. Consequently, it appears that some amount of time is needed for debris to accumulate in the stream channels (Keaton, 1988). Santi (1988) states that the amount of colluvium that exits the mouth of a canyon during a debris flow depends not so much on the size of an original slope failure, but on the amount of debris in the channel through which the flow moves. After colluvium in the channel is removed, slabs of colluvium along the channel margins become less stable because of removed toe support. Given the appropriate climatic conditions the potential exists for the side slopes to provide the source of debris for future disastrous debris flows (Figure 2).

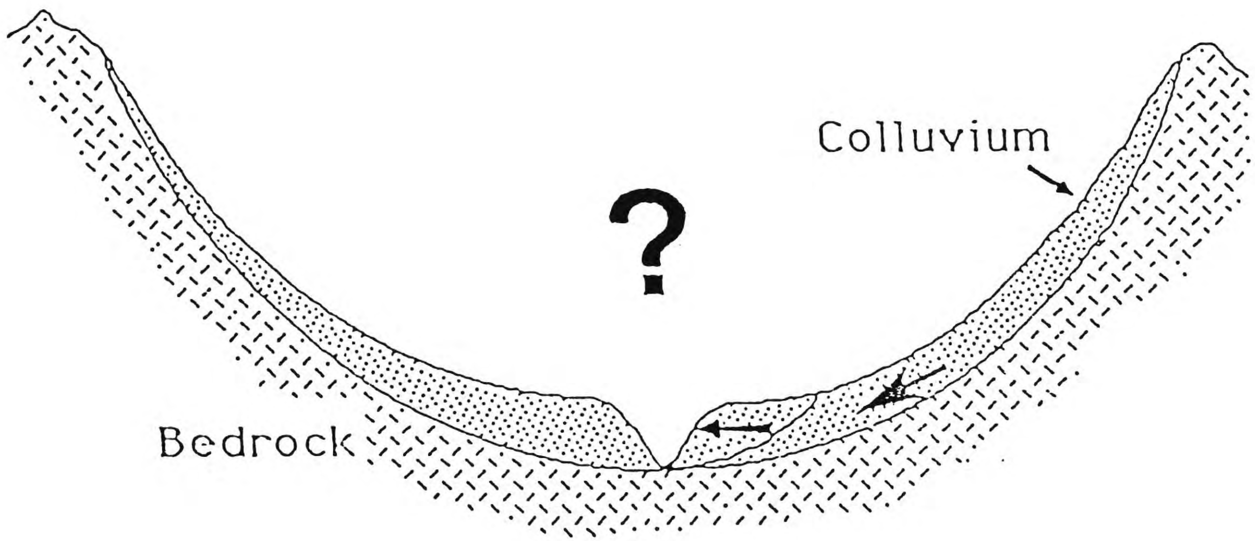


Figure 2. Cross-section showing side slope failures as a result of 1983 and 1984 debris flows removing toe support (from Mathewson and Keaton, 1990).

STUDY AREA

Geology

The Farmington Canyon Complex underlies the study area. This complex consists of highly fractured and deformed Precambrian metasediments (Figure 3). Lithologically, the area is composed of schist, gneiss, amphibolite, and pegmatite dikes. Ala (1990) determined that the orientation of fractures in the bedrock in the area was random, but fracture intersections had preferred orientations. The trends of the fracture intersection lines are believed to be the direction of maximum bedrock permeability of the region.

Hydrogeology

Montieth (1988) attempted to establish a ground-water distribution in Steed Canyon, but encountered problems with both electric and pneumatic piezometers. Five of the piezometers were destroyed by lightning (Montieth, 1988). Numerous springs and seeps were observed in the study area in 1988 and 1989 (Ala, 1990; Coleman 1990; and Skelton, 1990). Mathewson and Santi (1987) report "wet" debris scars, which also indicates a bedrock reservoir. Discharge from one of the springs of the area was measured by Ala (1990) and found to have a low specific yield. Specific yield is defined as the volume of water that an unconfined aquifer releases from storage per unit surface area of aquifer, per unit decline in the water table (Freeze and Cherry, 1979). One of the

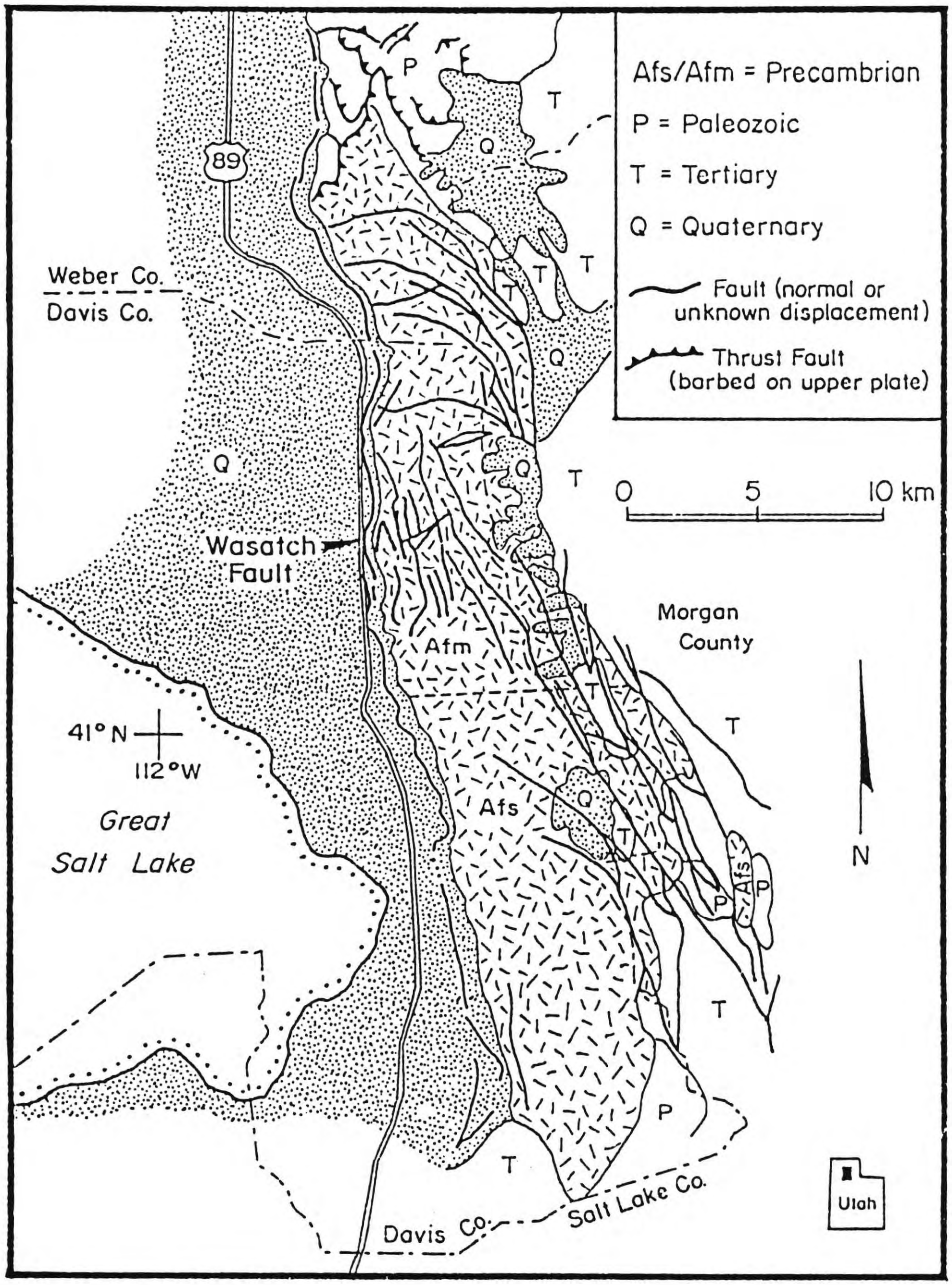


Figure 3. Geologic map of the Precambrian Farmington Canyon Complex (from Ala, 1990).

implications of a bedrock reservoir with a low specific yield is during aquifer recharge pore pressures can be developed more quickly than if the reservoir was in a more porous unit. Some of the discharge locations of the springs and seeps may be occurring above low permeability rock units such as pegmatites. Skelton (1990) made numerous observations of springs in close association with pegmatite bodies. These discharge points may provide the artesian pressures necessary to cause "pop out" failures as described by Mathewson and Santi (1987) and Johnson, (1987). Pack (1984) observed artesian pore pressures in the summer of 1983, at 1983 debris flow initiation sites. Piezometers were installed in Davis County, Utah, during June of 1983, adjacent to debris flow scar areas that had formed earlier 1983.

Geomorphology

Lightning, Steed, and Centerville Canyons are characterized by steep upper mountain slopes, upper mountain swales, and more subdued topography along the alluvial fans along the Wasatch Front. On the westward edge, the Wasatch fault forms a generally linear north-south boundary. The linear character of the boundary is an indication that uplift of the mountain block is at a rate greater than erosion can significantly reshape it (Keaton, 1988).

Lightning, Steed, and Centerville Canyons were the three drainage basins of the study area. Table 1 shows morphological data for the three drainage basins.

Table 1. Morphological data for the three drainage basins of interest in Davis County, Utah (after Keaton, 1988).

Basin	Order*	Area (mi^2)	Length (ft)	Relief (ft)	Relief Ratio
Lightning	2	0.21	6299	2260	0.36
Steed	4	2.59	16,499	4708	0.29
Centerville	4	3.15	21,001	4235	0.20

* The stream order was calculated according to Strahler (1952).

Even though Lightning Canyon is the shortest canyon in length, it possesses the greatest potential to transport debris from head to mouth based on the relief ratio. The relief ratio is calculated by dividing the basin relief by the basin length. If a debris flow enters the main drainage channel of a basin, it is generally on a greater slope in Lightning Canyon than in Centerville Canyon. If we consider other parameters in the debris flow transport potential through the basins, then the ordering according to increasing transport potential is not as simple.

A factor to be considered in the ordering process is the amount of debris that is in the channel. Lightning Canyon was the site of a debris flow that failed on May 14, 1984, and deposited its debris at the mouth of the canyon (Santi, 1988). Observations made in July-August of 1989, confirm that the highest order channel was cleared of debris. Steed Canyon was not cleared of debris in 1983 and 1984, but it did fail as a hyperconcentrated sediment flood. The channel of Steed Canyon contains great quantities of levee deposits and deposits from side slope failures, and these deposits have become somewhat stabilized by vegetation. Centerville Canyon contains a greater amount of levee deposits and denser vegetation in its highest order drainage channel (fourth), than Steed Canyon. Therefore, ordering of the debris flow transport potential for Lightning, Steed, and Centerville Canyons from lowest to highest results in Table 2. It should be emphasized that this ordering is strictly qualitative, and based on the assumption that the debris has entered the highest order drainage channel of each canyon. Based on the observations from the three drainage basins of Lightning, Steed, and Centerville Canyons, the relief ratio (see Table 1) is the most critical parameter that determines the transport potential for a drainage basin.

Table 2. Debris flow transport potential for Lightning, Steed, and Centerville Canyons.

Canyon	Transport Potential
Lightning	high
Steed	intermediate
Centerville	low

RESEARCH METHODS

Field Work

Sample Location

Colluvium samples were collected from Lightning, Steed, and Centerville Canyons (Figure 4a, 4b, 5). The samples were placed in plastic sample bags and transported back to the laboratory. All samples were disturbed, and the *in situ* moisture content was not preserved. The majority of the samples were taken from colluvium that was exposed in the side slopes of the drainage channels, of the drainage basins.

Colluvium Thickness

Colluvium thickness measurements were made in the drainage basins by measuring the thickness of colluvium exposed in the side slopes of the drainage channels. Figure 6 is a diagrammatic sketch of how the field thickness measurements were made. A collapsible level rod, hand level, Brunton compass, and a K&E range finder were all used to field survey the thickness of the exposed colluvium. In the field α, eye height, and the hypotenuse distance were all measured, where α is an acute angle of a right triangle. The eye height was measured with the collapsible level rod and hand level. The angle α was measured with a Brunton compass, and the hypotenuse distance was measured with the K&E range finder. Colluvium thickness can be calculated by the equation:

$$\text{colluvium thickness} = \sin\alpha \times (\text{hypotenuse distance}) + \text{eye height}.$$

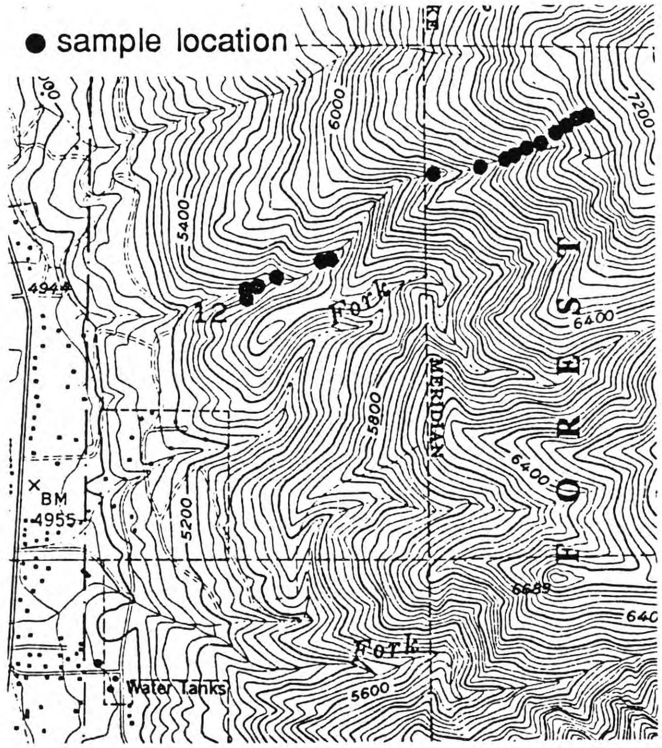


Figure 4a. Sample locations in Lightning Canyon. Map taken from the Kaysville Quadrangle (1955), Davis County, Utah.

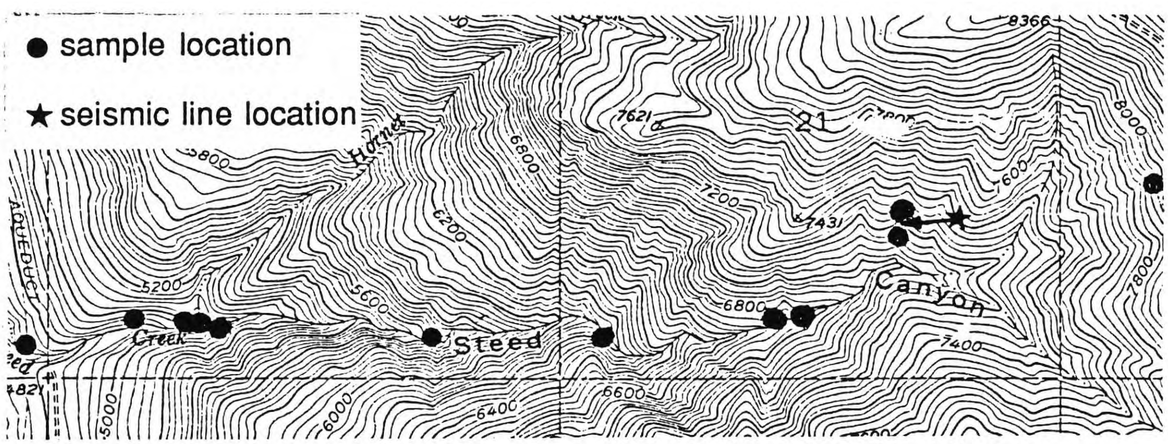


Figure 4b. Sample locations and seismic line locations in Steed Canyon. Map taken from the Bountiful Peak Quadrangle (1952), Davis County, Utah.

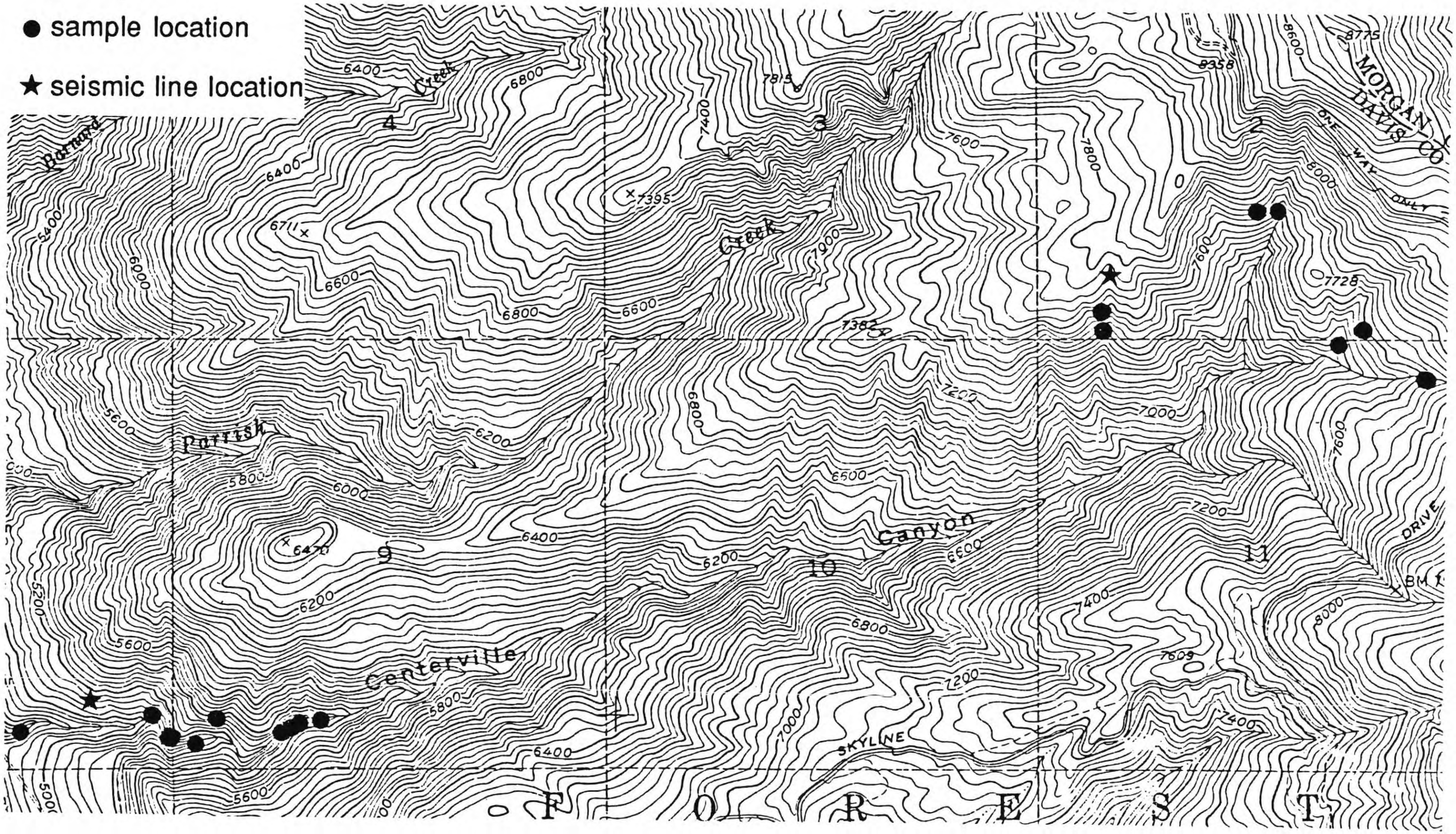


Figure 5. Sample locations and seismic line locations in Centerville Canyon. Map taken from the Bountiful Peak Quadrangle (1952), Davis County, Utah.

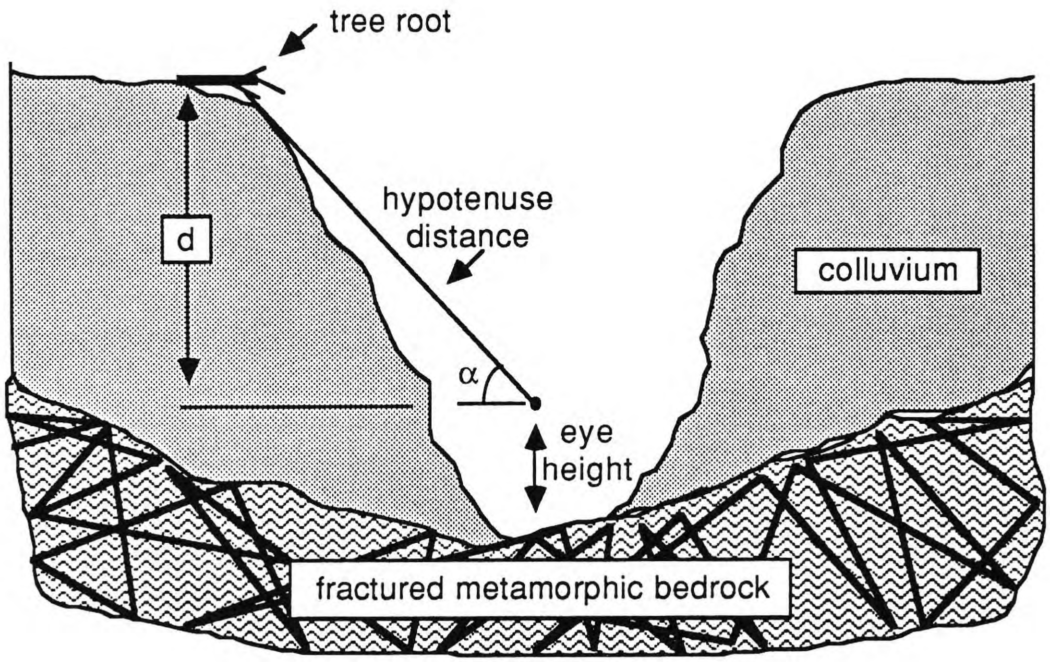


Figure 6. Sketch diagramming the methodology of colluvium thickness field measurements.

Seismic Refraction

In addition to field measurements of colluvium, single channel seismic lines were run in the study area in order to determine the thickness of colluvium. Velocities and depths to the underlying refracting layer were calculated according to Telford and others (1976).

Laboratory Work

Direct Shear

Consolidated, drained (CD), direct shear tests were performed on a representative sample from each of the three drainage basins. The samples were sheared at a rate of 0.02 mm/min with applied loads of 5.16, 10.10, and 15.03 psi. An attempt was made to compact the samples before shearing to *in situ* densities.

Grain Size

Grain size analysis was carried out on 9 samples each from Lightning, Steed, and Centerville Canyons. The samples were analyzed according to the sieve analysis procedures described in Lambe (1951).

Level I Stability Analysis (LISA)

Model Explanation

LISA (Level I Stability Analysis) is a U. S. Forest Service model. LISA uses a Monte Carlo simulation to determine the probability of failure in a given region. Each parameter of LISA is input as a range of values, in order to account for that parameter's variation throughout its drainage basin. The probability of failure is the number of factor of safety values less than 1, divided by the total number of Monte Carlo iterations. LISA allows the user to vary the number of iterations between one and one-thousand.

An infinite slope equation is used in LISA. The equation LISA uses is shown below:

$$FS = \frac{Cr + \tau}{\sin\alpha \cos\alpha \, [q_0 + \gamma(D + D_w) + \gamma_{sat} D_w]}$$

where FS = factor of safety

Cr = tree root cohesion, psf

τ = soil shear strength, psf

α = ground slope, degrees

q_0 = tree surcharge, psf

γ = moist soil unit weight, pcf

γ_{sat} = saturated soil unit weight, pcf

D = total soil thickness, ft

D_w = saturated soil thickness, ft

$\frac{Dw}{D}$ = ground-water ratio.

$\sigma'_n = \cos^2 \alpha[q_0 + \gamma(D - D_w) + (\gamma_{sat} - \gamma_w)D_w]$ = effective normal stress.

The infinite slope equation is used in LISA, because of its ease of use in the Monte Carlo simulation. The LISA assumes that the failure plane and the ground water surface are parallel and that the failure plane never intersects the ground surface. Obviously, if failure has occurred at a particular location, then the values for shear strength need to be values that actually exist along the failure plane.

Failures that occurred in the Wasatch Range, Davis County, Utah, in 1983 and 1984 were initiated in drainage basin channels and swales. In these areas, during snowmelt and rainfall events, ground-water flow is not always parallel to the ground surface. When heterogeneities are encountered by the ground water, then sharp fluctuations in piezometric pressure can occur due to spatial variation in hydraulic conductivities (Pack, 1984). The spatial arrangement of hydraulic conductivities can produce a locally confined ground-water condition, which cannot be modelled accurately using LISA, because LISA assumes that the ground-water flow is always parallel to the ground surface.

dLISA

A separate executable program from LISA, dLISA, also uses the infinite slope equation. dLISA is a deterministic slope stability model, and can be used for back calculations, as well as performing sensitivity analyses of input parameters. The parameters that can be solved for in dLISA are soil depth, surface slope, root cohesion, ground-water ratio, internal friction angle, soil cohesion, and factor of safety. Additional parameters of dLISA that can not be solved for are tree surcharge, dry density, moisture content, and specific gravity. Tables of solutions can be obtained for the solvable parameters, by varying one of the parameters over a specified range and solving for another parameter of interest. The parameters that are not being solved for or varied are input as constants. Soil moisture conditions are printed out as a table of dry, moist, and saturated unit weight, and moisture content of soil above the water table.

Sensitivity Analysis

One of the first steps in conducting a probabilistic slope stability analysis, is to determine which stability parameters cause the greatest relative change in the factor of safety. A sensitivity analysis is valuable, because it allows the user to determine where research dollars should be spent collecting parameter information.

dLISA was used to conduct the sensitivity analysis. dLISA solves for a single factor of safety, after single parameter values are input. Parameters of the sensitivity analysis can be ordered according to dLISA's sensitivity to their change, by plotting the percent change of each parameter vs. the percent change in factor of safety. The infinite slope equation is "most sensitive" to the parameter that, for a given percent value change of the parameter, causes the greatest percent factor of safety change.

Defining Failure Conditions

In order to perform an adequate slope stability analysis it is imperative to define the conditions under which failure will occur in the future. An analysis such as this was performed for the study area. The way to approach such a problem is to study each parameter at failure. This is done by varying the factor of safety over a range, including one, and solving for the parameter in question. Input for all other parameters should depict failure conditions. These conditions may be different from those present in the field at the time of data collection. For example, in modelling conditions at failure in the three drainage basins in Davis County, Utah, the moisture content of colluvium was assumed to be at the liquid limit at failure. This moisture content is obviously greater than what existed in the colluvium at the time of data acquisition.

Here, it is important to emphasize the fact that LISA and dLISA assume that artesian conditions do not exist, but Montieth (1988) determined, by using the computer program UTEXAS2, that artesian pressures were necessary to cause failure of the landslide block in Steed Canyon. As stated previously, Pack (1984) monitored artesian pressures in Davis County, Utah, at 1983 debris flow initiation sites. The sites were monitored after the failures occurred, in order to understand site specific conditions that may have contributed to initiation. Obviously, LISA and dLISA will have limited use in accurately defining failure conditions if ground-water flow is not parallel to the ground surface.

Statistics

Basics

Statistics can be defined as the theory of information, in which information is obtained by experimentation or by sampling, and it is employed to make inferences about a larger set of measurements called a population (Ott, 1988). A population is the set of all measurements of interest to the sample collector. A sample is any subset of measurements selected from the population. The primary objective of statistics is to make inferences about the population, based on information obtained from the sample. It is crucial in sample collecting, to collect samples that are truly representative of the population. If the sample is representative of the population, then an evaluation of the "goodness" of the original inference can be made.

Central tendency and variability are two common numerical descriptive measures. Three measures of the central tendency are the mode, median, and mean. The mode is defined as the measurement of a set of measurements that occurs most frequently. The median is defined as the central value, when the measurements are arranged from highest to lowest. In other words, half of the measurements are larger than the median, and half of the measurements are smaller than the median. The mean is the numerical average of a set of measurements.

One measure of the variability is the p^{th} percentile which is, for a set of measurements arranged in order of magnitude, that value that has at most p percent of the measurements below it and at most (100-p) percent above it. Another measure of the variability is the deviation, defined as

$y-y_m$ for a measure y, where y_m is the sample mean and y is an individual measurement from the sample. A function of the deviation, which also measures variability of a sample, is the variance. Variance is defined as the sum of the squared deviations divided by n-1. For a sample with n measurements of y_1, y_2, y_3....., y_n with a mean of y_m the variance is:

$$\frac{\Sigma_i (y_i - y_m)^2}{(n-1)}.$$

Standard deviation is also a measure of the variability, and is defined as the positive square root of the variance for a set of measurements.

Analysis of Variance

Analysis of variance is a statistical approach to a problem, that allows a statement to be made about the amount of difference that exists between sample means. For example, assume independent random samples are selected from three different populations of political affiliation. Suppose inferences are to be made about the differences in incomes among the three populations. Based on the sample means, inferences could be made about the population means. The differences in sample means may, or may not be different enough to state that the populations are different. The procedure involved to test the amount of difference is called an analysis of variance.

The test statistic used to test the equality of sample means is

$$F = \frac{SB^2}{SW^2}$$

where SB^2 = the measure of the variability between sample means

SW^2 = the measure of the variability within sample means.

The hypothesis that is being tested with the test statistic F, is the null hypothesis (H_0). The null hypothesis (H_0) states, that the sample means are equal and also, assumed to be equal to a grand, or overall population mean. The overall population mean would be a mean of all the sample measurements collected. If F from the analysis of variance table is less than the appropriate F from a F-Distribution table, at between and within error degrees of freedom, then the null hypothesis is rejected and the alternate hypothesis (H_A) holds. The null hypothesis is rejected, based on insufficient data to accept it. The F-Distribution value can be determined from any statistics text.

The alternate hypothesis states, that at least one of the sample means is different from the rest. When the alternate hypothesis is accepted, this implies that at least one of the sample means is from a different population. The analysis of variance is always performed with a confidence interval, suitable for the given situation. For a detailed description of the analysis of variance procedure see Ott (1988).

When an F test is completed, then the study can be summarized in an analysis of variance (ANOVA) table. Table 3 shows an example of an ANOVA table.

For Table 3: $TSS = \Sigma y_{ij}^2 - \frac{G^2}{n}$ (total sum of squares)

$$SSB = \frac{\Sigma T_i^2}{n_i} - \frac{G^2}{n} \text{ (between sum of squares)}$$

$$SSW = TSS - SSB \text{ (within sum of squares)}$$

Table 3. Analysis of variance table.

Source	Sum of Squares	Degrees of Freedom	Mean Square	F Test
Between	SSB	t-1	SB^2	SB^2/SW^2
Within	SSW	n-t	SW^2	
Totals	TSS	n-1		

$SB^2 = SSB/(t - 1)$

$SW^2 = SSW/(n - t)$

i = rows of data

j = columns of data

G = sum of all sample observations

n = the total sample number

t = number of populations

T_i = sum of sample measurements from population.

After completing the analysis of variance table, then a least significance difference test can be performed to determine which sample mean is different, at a given confidence level. An analysis of variance is used to determine if population means are different and a least significance difference determines which population mean is different. For this study the three population means compared were the percent fines for grain size samples from Lightning, Steed, and Centerville Canyons. The null hypothesis was that the mean percent fines from Lightning, Steed, and Centerville Canyons were equal. An analysis of variance and a least significant difference test will be performed later in Statistical Comparison.

RESULTS

Field Work

Colluvium Thickness

Figures 7a and 7b are colluvium thickness measurements made in Lightning and Centerville Canyons. Figure 8 shows measured thickness and boring thickness taken by Brooks (1986) in Steed Canyon. S1-S8 are Steed measurements and B1-B13 are Brooks' (1986) borings. Brooks' (1986) borings generally yield thinner colluvium than the measurements that were made in this study, because Brooks' (1986) borings were from an upper mountain swale where the colluvium is thinner. The thickness measurements made in this study were taken, where the colluvium is thickest in the drainage basin, along the highest order drainage channels.

Figure 9 shows the average colluvium thickness of all three drainage basins, based on all field thickness measurements, Brooks' (1986) borings, and seismic refraction measurements. The average thickness measurements indicate the thickness increases from most northern drainage basin (Lightning) to most southern drainage basin (Centerville).

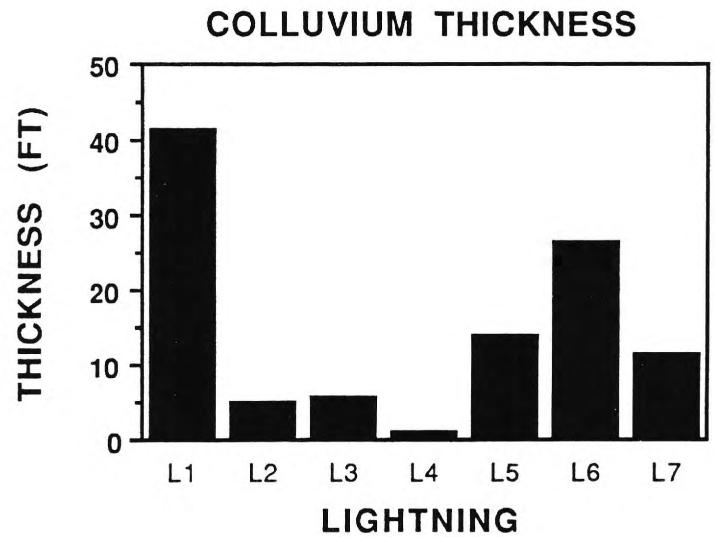


Figure 7a. Colluvium thickness field measurements for Lightning Canyon.

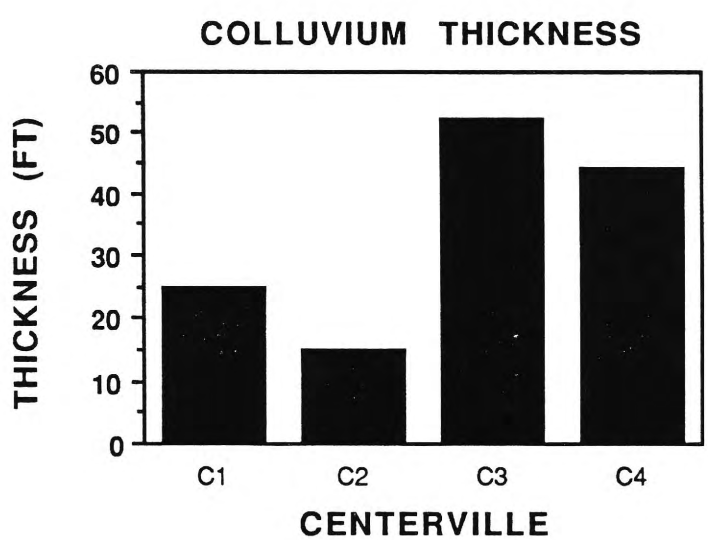


Figure 7b. Colluvium thickness field measurements for Centerville Canyon.

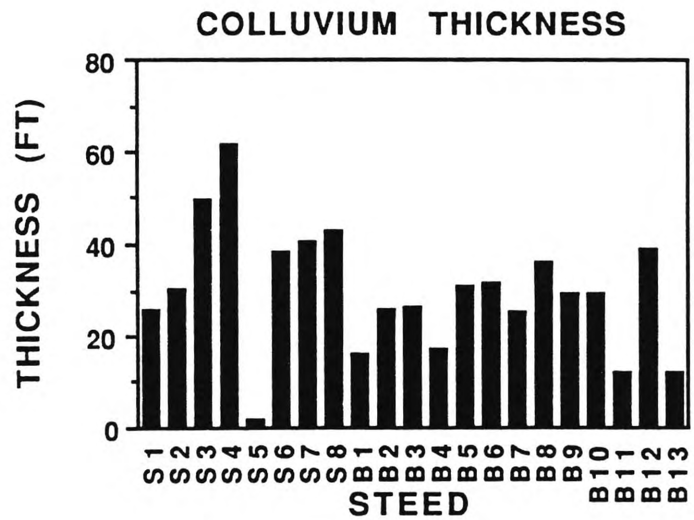


Figure 8. Colluvium thickness field measurements and Brooks' (1986) boring depth to bedrock.

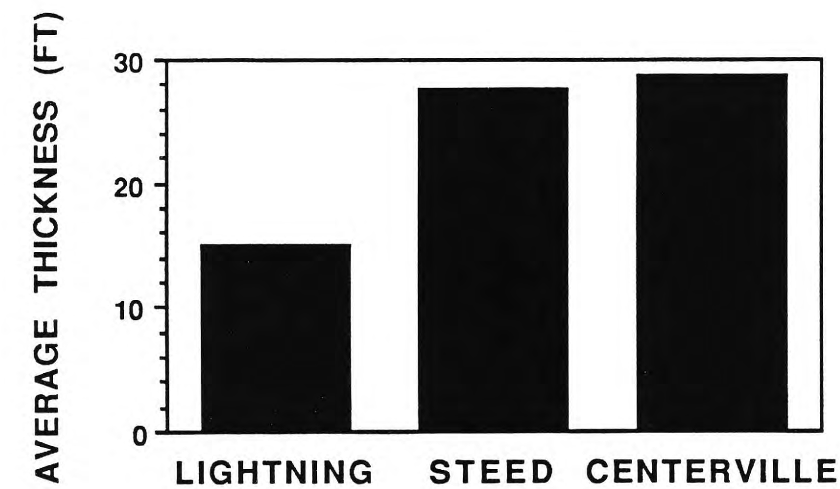


Figure 9. Average thickness of colluvium for Lightning, Steed, and Centerville Canyons.

It should be emphasized and stated again that the thickness measurements represent some of the thickest colluvium deposits in the drainage basins. Initiation locations of debris flows occur in areas of much thinner colluvium deposits, on steeper slopes. The thick colluvium deposits in the side slopes of the drainage channels are sites of numerous surficial debris slides. These debris slides may have increased in numbers since 1983 and 1984, along the side slopes of drainage channels that had debris removed in 1983 and 1984.

Seismic Refraction

Thickness values of Table 4 were computed based on data that were gathered from a single channel seismic refraction survey, that was carried out at locations in Steed and Centerville Canyons. The thicknesses were calculated according to Telford and others (1976). Appendix A contains Travel Time versus Distance plots for Steed and

Centerville Canyons. The reverse line of Centerville, S40E was not interpreted, because there was not a strong velocity contrast found along the line, probably implying the base of colluvium was never found along this line. Because the reverse depth was not determinable, the dip of the refracting surface was not calculated along this bearing.

Table 4. Depth to first refractor in selected locations of Steed and Centerville Canyons.

Seismic Line Bearing	Forward Thickness (ft)	Reverse Thickness (ft)
Centerville, N40W	19.98	8.11
Centerville, S40E	33.10	------
Centerville, N65W	35.67	25.80
Steed, S80W	10.86	5.94

Laboratory Work

Direct Shear

The results of the direct shear tests are presented in Figures 10a, 10b, and 10c. Cohesion and ø angles are presented in Table 5.

Table 5. Shear strength data of three samples from Lightning, Steed, and Centerville Canyons.

Canyon	Cohesion (psi)	ø
Lightning	0	35.5^0
Steed	1.3	37.7^0
Centerville	1.2	35.1^0

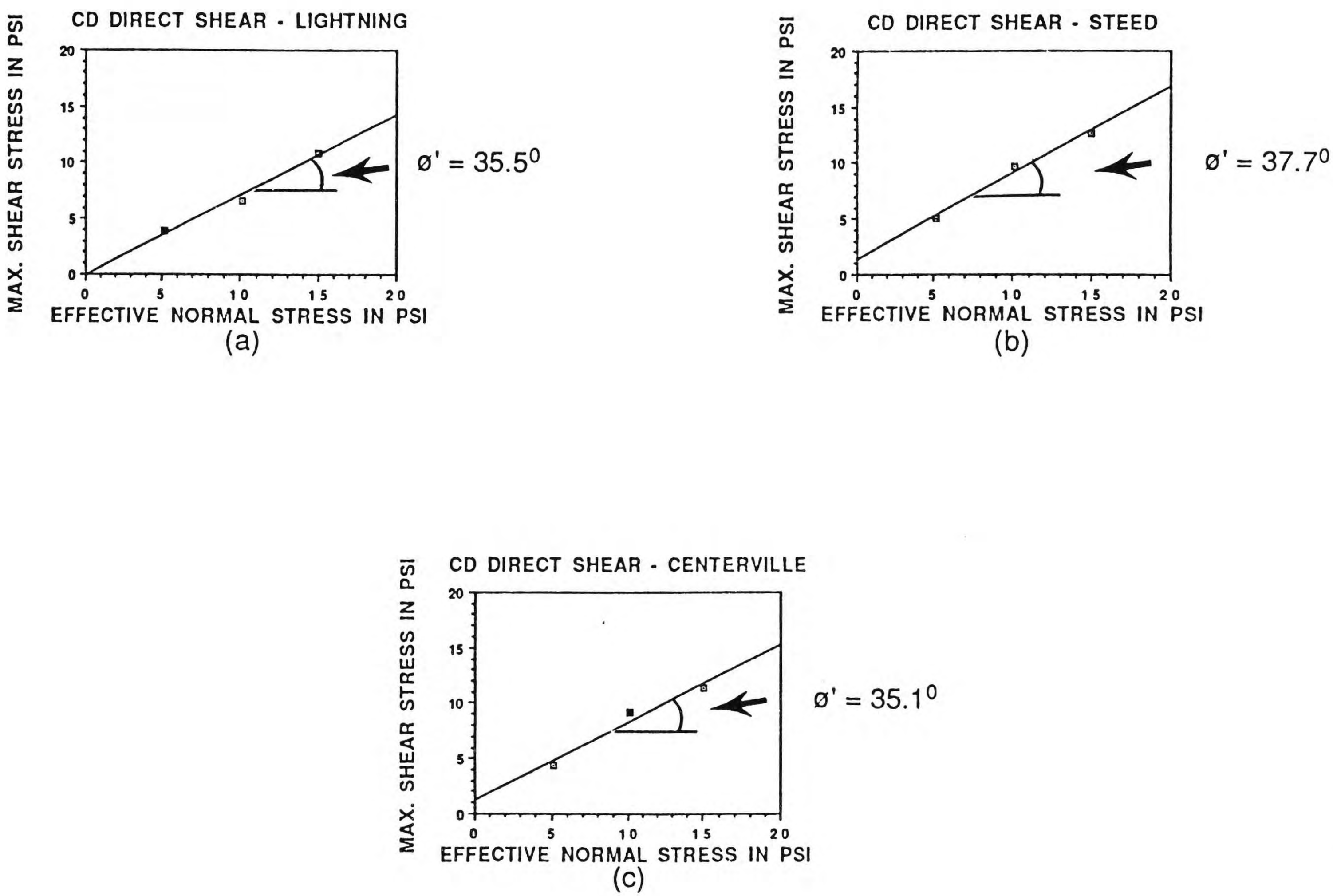


Figure 10. Plot of maximum shear stress vs. effective normal stress. For plot a) $S = \sigma' \tan 35.5^0$, b) $S = 1.3$ psi $+ \sigma' \tan 37.7^0$, and c) $S = 1.2$ psi $+ \sigma' \tan 35.1^0$. S = shear strength and σ' = effective normal stress.

Grain Size

After each of the individual 27 samples were analyzed, an average percent gravel, sand, and fines was calculated for the three drainage basins (Figure 11). The average grain sizes were then statistically compared by performing an analysis of variance on the percent fines of each canyon.

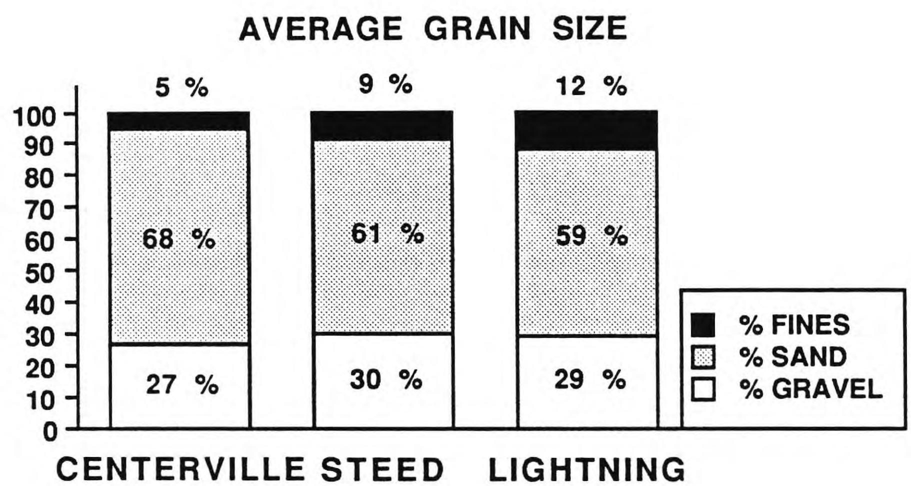


Figure 11. Average grain sizes for Lightning, Steed, and Centerville Canyon samples. The grain sizes were classified according to the Unified Soil Classification System (USCS).

Statistical Comparison

Table 6 contains the percent fines of 27 samples from Lightning, Steed, and Centerville Canyons. Grain sizes were classified according to the Unified Soil Classification System. The samples of Table 6 are random samples from the field, which implies that no sample preference was used in collecting the samples.

Table 7 is an Analysis of Variance table for the percent fines of colluvium samples from the three populations of Table 6. From Table 7, the F calculated is compared to an F value from an F distribution plot at 2 and 24 degrees of freedom, as described previously. The F value from the distribution plot was 3.40, which implies rejection of the null hypothesis and acceptance of the alternate hypothesis which states that at least one of the population means is different. This implies that the samples of Lightning, Steed, and Centerville Canyons cannot be assumed to be from one grand population, on a 95 percent confidence interval.

After determining that at least one of the population means is different, then a procedure, proposed by Tukey in 1953, can be used to determine which mean is different. The Tukey test defines the amount of difference that is necessary to state that two populations are different (least significant difference). Tukey's W procedure (Ott, 1988) was followed to calculate a W = 4.52 at a 95 percent confidence level. Therefore, it can be stated that the probability of randomly selecting nine samples each from Lightning, Steed, and Centerville Canyons, that yield mean percent fines which differ more than 4.52 percent, is less than 5 times in 100.

Table 6. Percent fines for colluvium samples from Lightning, Steed, and Centerville Canyons.

Sample	Lightning % fines	Steed % fines	Centerville % fines
1	9.52	17.50	0.92
2	11.19	5.19	2.65
3	19.06	3.84	10.40
4	14.04	6.14	2.12
5	11.78	6.83	9.82
6	1.79	7.83	4.42
7	11.7	16.21	4.01
8	9.25	7.48	10.65
9	18.76	8.94	3.36
mean	11.89	8.88	5.37

Table 7. Analysis of variance of the percent fines for samples from Lightning, Steed, and Centerville Canyons.

Source	Sum of Squares	Degrees of Freedom	Mean Square	F Test
Between	191.993	2	95.997	4.445
Within	518.323	24	21.597	
Total	710.32	26		

Based on the Tukey test value of 4.52, at a confidence level of 95 percent, Lightning and Steed Canyon samples are assumed to be from the same population. Similarly, Steed and Centerville Canyon samples are assumed to be from the same population, but Lightning and Centerville Canyon samples are assumed to be from two different populations of grain sizes. There appears to be a gradational north to south decrease in grain size of the colluvium from the study area. This decrease in grain size is possibly controlled by lithologic variations, but a confirmation of this is beyond the scope of this study.

Stability Analysis

Sensitivity Analysis

Figure 12 is a sensitivity analysis plot for the dLISA variables. The dLISA parameters can be ordered according to dLISA's sensitivity, from most sensitive to least sensitive as follows: 1) soil depth, 2) surface slope, 3) root cohesion, 4) soil cohesion, 5) ground-water ratio, and 6) internal friction angle. Changes in tree surcharge, dry density, and moisture content had no effect on the factor of safety. The sensitivity analysis will be discussed further in Discussion of Results. Appendix B contains the sensitivity analysis graphs and sensitivity analysis data sheets.

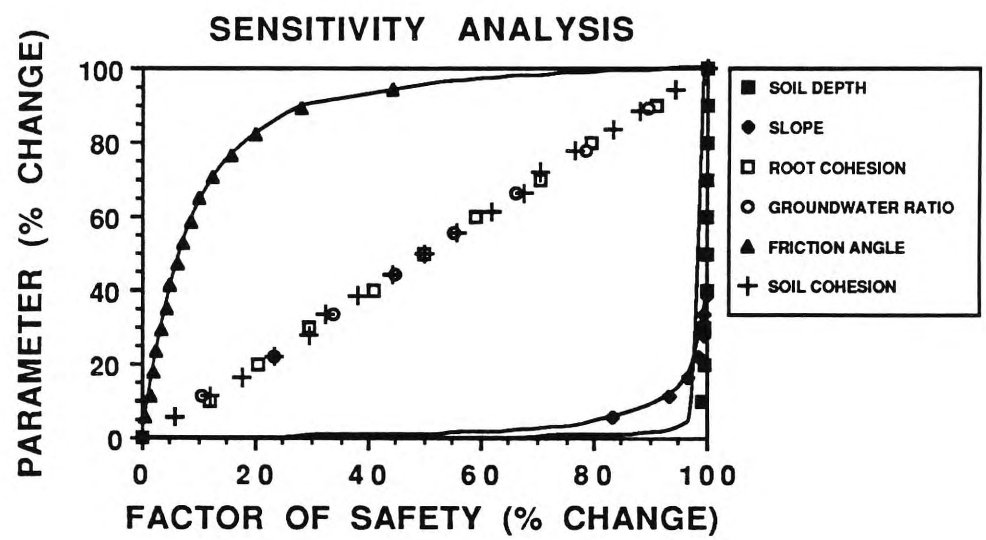


Figure 12. Sensitivity analysis plot for the variables that can be solved for using dLISA.

In situ Factor of Safety

Slope stability analyses have no real value, if the data used are not representative of *in situ* conditions. LISA, for example, only performs an analysis based on the parameter values the user inputs. If these values are not modelling field conditions, then the analysis becomes only a game of numbers. Because LISA operates using a Monte Carlo simulation, it is unable to distinguish unlikely combinations of conditions during its iterations. For example, assume we are modelling a drainage basin with a range of soil depth and surface slope equal to 0 to 50 ft and 5 to 40 degrees, respectively. During 1000 iterations, it is feasible to assume that at least one of the iterations will choose 50 ft of soil on a 40 degree slope. Certainly, this condition is unstable, but it is also a condition that probably does not exist in the field. To create this as a possible condition for LISA to select is of no use to anyone. The solution to such a problem is to split the model condition in such a manner that the thinnest soils are on the steepest slopes. For example, the soil depth and surface slope could be split into three model conditions as listed below: 1) soil depth = 0 to 10 ft with surface slope = 30 to 40 degrees; 2) soil depth = 10 to 25 ft with surface slope = 20 to 30 degrees; and 3) soil depth = 25 to 40 ft with surface slope = 5 to 20 degrees. By choosing three new conditions of soil depth and surface slope, LISA is forced to choose situations that are geologically and geomorphically reasonable. Table 8 shows five different

Table 8. Five soil depth and surface slope conditions modelled using LISA.

Condition	Soil Depth (ft)	Surface Slope (deg.)	Ground-water Ratio (D_W/D)	Factor of Safety (mean)	Probability of Failure
1	0 -40	10 - 40	0 - 0.5	1.85	0.019
2	10 - 40	5 - 25	0 - 0.5	3.47	0.000
3	0 - 10	25 - 40	0.9 - 1.0	1.37	0.145
4	5 - 10	25 - 32.5	0.9 - 1.0	1.21	0.018
5	0 - 5	32.5 - 40	0.9 - 1.0	2.09	0.000

combinations of soil depths and surface slopes that were tested. Data for Table 8 were based on measurements made in Steed Canyon. Although conditions 1 and 2 were modelled using reasonable values for the entire drainage basin, particular combinations of values are unlikely. The primary concern in modelling debris flows is the ability to model the condition present during debris flow initiation. Debris flows of 1983 and 1984 were initiated on steep slopes with thin colluvium. Conditions 4 and 5 are probably the most accurate representation. As a geologist, field data that is collected needs to be applied in a way that is utilitarian to the setting. This is accomplished by understanding the magnitude of each piece of data to the overall project.

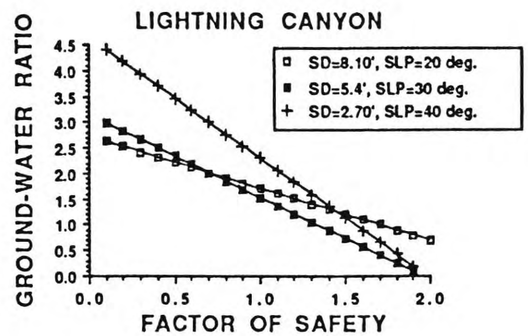
Defining Failure Conditions

Using dLISA forces an assumption that artesian pressures do not exist, because as stated earlier, the maximum ground-water ratio value that can be input is unity. For artesian conditions, the ground-water ratio would be greater than one. If artesian pressures are not considered, then what ground-water level should be modelled? Three phreatic surfaces were used to define threshold values at failure. Montieth (1988) analyzed the stability of a detached landslide block in Steed Canyon. Both artesian and phreatic conditions were considered, although the two ground-water conditions had to be examined independent of one another. For a representative failure plane of the detached landslide block, it was determined that if a phreatic surface was representative of the ground-water condition, failures further down slope would have occurred before the necessary modelled ground-water height was achieved.

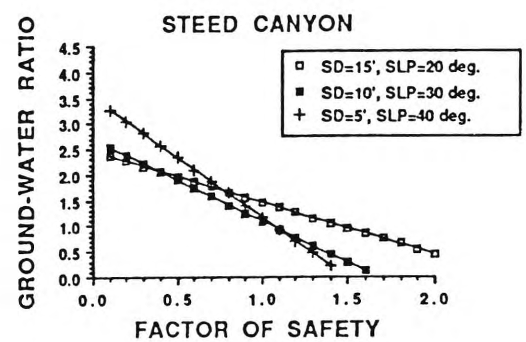
Tables 9, 10, and 11 are the results from determining the threshold values at which failure will occur, if each parameter is analyzed, as all other parameters are held constant at conditions that existed at failure. Table 12 is a compilation of parametric constants that were used in determining the threshold values. The soil depth of 5 ft on a 32^0 is representative of what was observed in Steed Canyon at the head of a debris flow scar. Recall, data from Table 4 indicates colluvium thickness varies between 5 ft and 10 ft, along a seismic refraction line perpendicular to the debris flow channel scar in the "east" swale. The slope stability of the "east" swale was studied by Montieth (1988). The friction angle and soil cohesion were based on consolidated, drained, direct shear tests that were performed on three representative colluvium samples. Values of dry density, from Das (1985) for a loose-angular, silty sand and a dense-angular, silty sand were used as the basis for extrapolating a dry density of 118 pcf. A moisture content of 20 percent was assumed to be the maximum *in situ* moisture content at failure. This moisture content is representative of liquid limit values from Keaton (1988) and Santi (1988).

It is interesting to note that the three combinations of soil thicknesses and slopes of tables 9, 10, 11 are greater and contrary to what was observed in the field. Field observations yielded the thinnest colluvium on the steepest slopes. Based on Table 11, if the ground-water ratio is one and the slope is 43.81 degrees, then all slopes with soil depths less than 9.62 ft. are stable. Similarly, if the ground-water ratio is one and the soil depth is 9.62 ft., then all slopes less than 43.81 degrees are stable. According to the data of tables 9, 10, 11 all the slopes of the study area are stable. If they are stable, then why did they fail in 1983 and 1984 causing 22 of Utah's 29 counties to be declared national disaster areas (Anderson et al., 1985)? A discussion of this is included in the Discussion of Results.

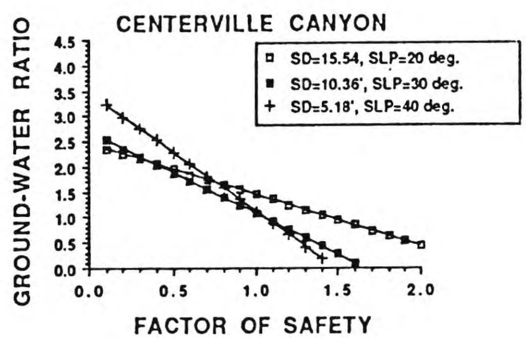
After determining the threshold values using dLISA at ground-water ratios of 0.5, 0.75, and 1.00, correlation between the ground-water ratio and the factor of safety was established at three different soil depth and slope conditions for Lightning, Steed, and Centerville Canyons (Figure 13a, 13b, 13c).



(a)



(b)



(c)

Figure 13. Factor of safety vs. ground-water ratio at three soil depths (SD) and surface slopes (SLP) for a) Lightning, b) Steed, and c) Centerville Canyons.

Table 9. Parameter threshold values at a ground-water ratio of 0.5.

FAILURE CONDITIONS	
Parameter	Threshold Value
soil depth (ft.)	85.29
surface slope (deg)	57.57
root cohesion (psf)	solution impossible
soil cohesion (psf)	11.14
friction angle (deg)	17.06
Factor of Safety = 1	
Ground-water ratio = 0.5	

Table 10. Parameter threshold values at a ground-water ratio of 0.75.

FAILURE CONDITIONS	
Parameter	Threshold Value
soil depth (ft.)	17.28
slope (deg.)	49.78
root cohesion (psf)	solution impossible
soil cohesion (psf)	54.97
friction angle (deg.)	19.82
Factor of Safety = 1	
Ground-water ratio = 0.75	

Table 11. Parameter threshold values at a ground-water ratio of 1.0.

FAILURE CONDITIONS	
Parameter	Threshold Value
soil depth (ft.)	9.62
slope (deg.)	43.81
root cohesion (psf)	solution impossible
soil cohesion (psf)	98.79
friction angle (deg.)	23.60
Factor of Safety = 1	
Ground-water ratio = 1.0	

Table 13 displays nine soil thicknesses and slope conditions that were assumed for Lightning, Steed, and Centerville Canyons. Appendix C contains the plots of Ground-water Ratio vs. Factor of Safety for the conditions of Table 13. In all nine situations, the ground-water ratio is greater than one, which indicates that artesian pressures are present at initiation. The soil depths and slopes used in Table 13 were designed based on thickness measurements, colluvium side slope thickness measurements and seismic refraction, and field observations and measurements of slopes.

Table 12. Parametric constants used in determining the threshold values.

PARAMETRIC CONSTANTS	
soil depth (ft.)	5
slope (deg.)	32
tree surcharge (psf)	0
root cohesion (psf)	0
friction angle (deg.)	38
soil cohesion (psf)	190
dry density (pcf)	118
moisture content (%)	20
specific gravity	2.66

Table 13. dLISA ground-water ratios at a FS = 1 for different soil depths and surface slopes

Lightning			Steed			Centerville		
soil depth (ft)	slope (deg)	ground -water ratio	soil depth (ft)	slope (deg)	ground -water ratio	soil depth (ft)	slope (deg)	ground -water ratio
8.10	20	1.71	15	20	1.46	15.54	20	1.45
5.40	30	1.53	10	30	1.09	10.36	30	1.07
2.70	40	2.30	5	40	1.17	5.18	40	1.12

More data were collected and available for Steed Canyon than any of the other canyons. After an average thickness for colluvium was calculated for Steed Canyon, it was then compared to the thicknesses of

Lightning and Centerville Canyons. Soil depths for Lightning and Centerville were decreased and increased respectively, based on how their average colluvium thicknesses, from Figure 9, compared to Steed Canyon's average thickness. For example the average thickness in Lightning Canyon measurements was only 53 percent of the average thickness in Steed Canyon. Therefore, the soil depths for Lightning Canyon, in Table 13, are only 53 percent of the soil depths chosen for Steed Canyon.

Figure 14 is a continuous plot of soil depth versus ground-water ratio, at a factor of safety equal to one, for slopes of 25, 32.5 , and 40 degrees. Appendix D contains the data, generated using dLISA, for Figure 14. For these three slopes, Figure 14 can be used to determine what ground-water ratio is necessary to cause failure at a particular soil depth. For example, at a soil depth of 10 ft. on a 25 degree slope the ground-water ratio necessary to cause failure is 1.30. Any soil that is less than 10 ft. thick will be stable if the ground-water ratio remains at 1.30.

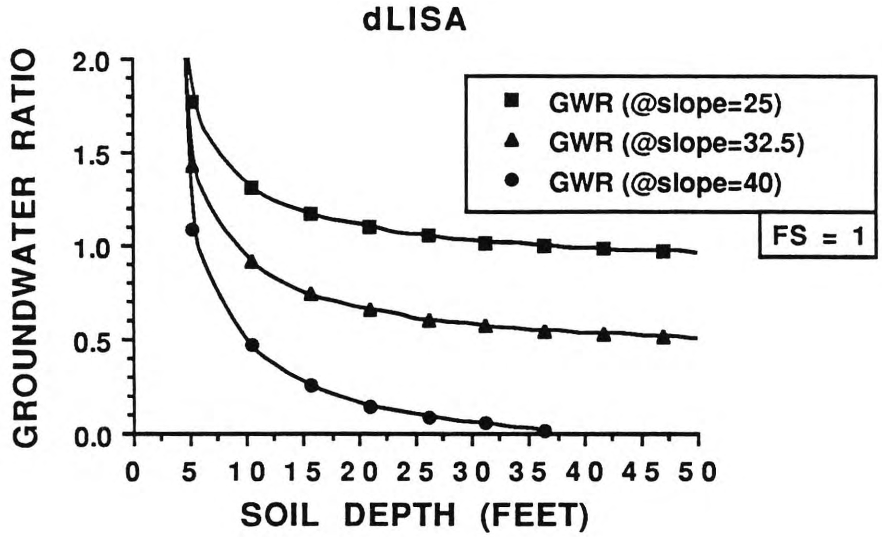


Figure 14. Threshold relationship between the ground-water ratio and soil depth for three different slopes.

DISCUSSION OF RESULTS

Observed colluvium thicknesses of the three drainage basins were thinnest in Lightning Canyon and thickest in Centerville Canyon. There is also a corresponding increase in drainage basin area from Lightning to Centerville Canyon. If we assume that drainage basin area increases with increasing age and degree of weathering in the drainage basin, then Centerville is the oldest and the most weathered drainage basin. Since the weathering has been greater in Centerville Canyon, then the colluvium accumulations are thicker. Possibly, the degree of weathering differences are lithologically controlled. Within the study area, the lithology that was most susceptible to weathering was schist. At the site of a stabilized detached block in Centerville Canyon, schist was exposed at the head scarp. Seismic refraction data of Table 4 also indicate the thickest colluvium is in Centerville Canyon.

Even though there was a difference in the percent fines between the Lightning and Centerville Canyon samples, on a 95 percent confidence level, the shear strength of three random samples from Lightning, Steed, and Centerville Canyons was almost identical. Because of the similar shear strength values, the colluvium in the three drainage basin's would be expected to fail under similar stress conditions. But, since Centerville Canyon contains less fines, the colluvium of this canyon would be more free draining than the colluvium of Lightning and Steed Canyons. Therefore, Centerville Canyon can be considered to be more stable than Lightning Canyon. Centerville Canyon has never failed catastrophically in historic time. Only a few surficial failures occurred in 1983 and 1984.

From the sensitivity analysis, it was determined that the infinite slope equation was most sensitive to changes in soil depth and surface slope. Changes in the ground-water ratio did not cause as great a change in the factor of safety but, the ground-water ratio is not static over time for any given site. Based on the sensitivity analysis, future data collection in the study area should concentrate on soil depth and surface slope. The ground-water ratio should be understood at each site, as well as any artesian pressures. Monitoring could be carried out in the future, throughout the study area. This piezometer monitoring should be carried out during natural failures, in order to define the relationship between the site specific ground-water conditions and failure mechanics. More study should be conducted in the upper mountain regions, where the debris flows were initiated. Data for colluvium thickness at these locations could be collected by using seismic refraction. After understanding which upper slopes are more unstable than others, these particularly unstable slopes could be identified and incorporated into a hazard evaluation.

LISA was used to determine five factors of safety (Table 14). Soil depths, surface slopes, and ground-water ratios are shown in Table 8 because, the infinite slope equation is most sensitive to these parameters. As discussed previously, conditions 1 and 2 are reasonable ranges of parameters for an entire drainage basin but, there are particular possible combinations of parameters that are geologically and geomorphically unrealistic. The combination of parameters, from the Monte Carlo iterations that were unlikely, caused the probability of failure not to be zero. So, if only the reasonable combinations of parameters are considered,

Table 14. Five soil depth and surface slope conditions modelled using LISA.

Condition	Soil Depth (ft)	Surface Slope (deg.)	Ground-water Ratio (D_w/D)	Factor of Safety (mean)	Probability of Failure
1	0 -40	10 - 40	0 - 0.5	1.85	0.019
2	10 - 40	5 - 25	0 - 0.5	3.47	0.000
3	0 - 10	25 - 40	0.9 - 1.0	1.37	0.145
4	5 - 10	25 - 32.5	0.9 - 1.0	1.21	0.018
5	0 - 5	32.5 - 40	0.9 - 1.0	2.09	0.000

then LISA predicts the stability of slopes (Condition 2). Slope stability was observed to exist in the field. Therefore, LISA seems to predict stability of the slope in Davis County, Utah, under the conditions that existed in the field during July and August of 1989. Conditions 3, 4, and 5 are representative of conditions in which a rainfall event or snowmelt causes the ground-water ratio to be between 0.9 to 1.0. Conditions 4 and 5 are most representative of what was observed in Steed Canyon, at the site of a debris flow that failed in 1983. Using the seismic refraction thickness from Steed Canyon (Table 3), the depth to the first refracting layer ranges from approximately 5 to 10 ft. This seismic line was run along a cross-slope bearing, perpendicular to the debris flow channel in Steed Canyon, that was formed in 1983. The corresponding ground slope at this location was measured to be 32 degrees, which was the upper limit of the slope for condition 4 of Table 8. For condition 4, there were 18 failures out of 1000 iterations (0.018 probability of failure). For condition 5, no failures occurred from the 1000 iterations, which implies stability during a thunderstorm or snowmelt that would cause a ground-water ratio of 0.9 to 1.0 to form. But, during 1983 and 1984 numerous debris flows did occur. If conditions 4 and 5 are representative of the failure mechanism, a rising phreatic surface on a infinite slope as described by Campbell (1975), then the probability of failure would be expected to be much greater. This is an indication that some other mechanism is causing the debris flow initiation. After modelling the slope stability using LISA, dLISA was used to define the conditions that caused failure in the past, and attempt to understand the initiation mechanics.

Three possible failure situations were created using dLISA. Three phreatic surfaces were chosen for the three situations. Each of the three situations were created by setting the factor of safety at one (failure) and, then solving for each parameter at a specified ground-water ratio. The parametric values of Table 12 were the values for the parameters, when their threshold value was not being determined. A moisture content of 20 percent was assumed, which is an approximate maximum value of the liquid limit based on Keaton (1988) and Santi (1988). In all three situations, the slope angle and soil depth combinations are larger than what exists in the field. Why is dLISA defining failure conditions that are unrealistic ? A possible reason is that the ground-water flow at initiation sites is not always parallel to the slope surface, especially if the initiation site is at the toe of a swale and the ground-water is being defined below a low permeability zone. As discussed previously, LISA and dLISA assumes a ground-water system with no artesian pressures.

Even though a ground-water ratio greater than one could not be input, dLISA could be forced into creating a ground-water ratio greater than one, by back calculation. Table 13 contains soil depth and slope situations that were assumed to solve for each of the ground-water ratio values. The values of the other parameters were presented in Table 12. For all nine situations of soil depth and surface slope, the ground-water ratio is greater than one. If a ground-water ratio from dLISA greater than one implies artesian pressures, then the initiation mechanism described by Mathewson and Santi (1987) may be applicable. Artesian pressures can develop where there are ground-water flow concentrations in swales, gullies, and other topographic depressions, if the surface materials have high infiltration capacities (Pierson, 1977; O'Loughlin, 1973; Johnson, 1987).

Another relationship that was developed, using dLISA, was between soil depth and ground-water ratio (Figure 14). At a soil depth of 10 ft and a slope of 32.5 degrees, the ground-water ratio necessary to induce failure, according to dLISA, is 0.95. It is unlikely that the above set of conditions has ever existed in the study area. If the ground-water ratio was 0.95 and the soil depth, on a 32.5 degree slope was increased above 10 ft the slope would become unstable. Figure 14 is another indication that debris flow initiation is due to more than a simple raising of a phreatic surface, because failures have occurred with ground-water ratios less than 0.95, when the soil depth was approximately 10 ft. So, it can be seen that if representative soil depths from the field are applied to Figure 14, on the appropriate slope, then the phreatic surface dLISA defines is higher than what existed at failure.

INITIATION MECHANISMS

Campbell (1975) stated that the initiation of a debris flow is dependent on the rate of infiltration into and through the upper layers of colluvium. If the rate of infiltration is equal to or less than the rate removed by the underlying bedrock, then the slope is stable (Campbell, 1975). However, if the infiltration rate is greater than the underlying rate of removal, then a temporary perched water table occurs above the less permeable bedrock (Figure 15).

Mathewson and Santi (1987) proposed that the 1983 and 1984 Utah debris flows on the metamorphic bedrock of the Wasatch Range (Figure 16) were initiated by artesian pore pressures that were built up where fractures in the bedrock intersect the base of colluvium. Pack (1984) determined the presence of artesian pressures in the ground-water system, associated with some of the 1983 debris flow sites in Davis County, Utah.

What determines the type of debris flow mechanism operating in an area? I would argue it is largely the geomorphology and geology. If the slope angles are uniform over great distances, with ground-water flow parallel to the slope, then the Campbell (1975) model dominates. But, if the geomorphology is dominated by swales, gullies, and other topographic depressions, then artesian pressures are more likely to develop if the colluvium contains heterogeneous hydraulic conductivities.

Soil drainage and moisture redistribution are two processes that are often ignored when debris flow initiation is studied. Moisture redistribution can take the form of water moving through the soil to a

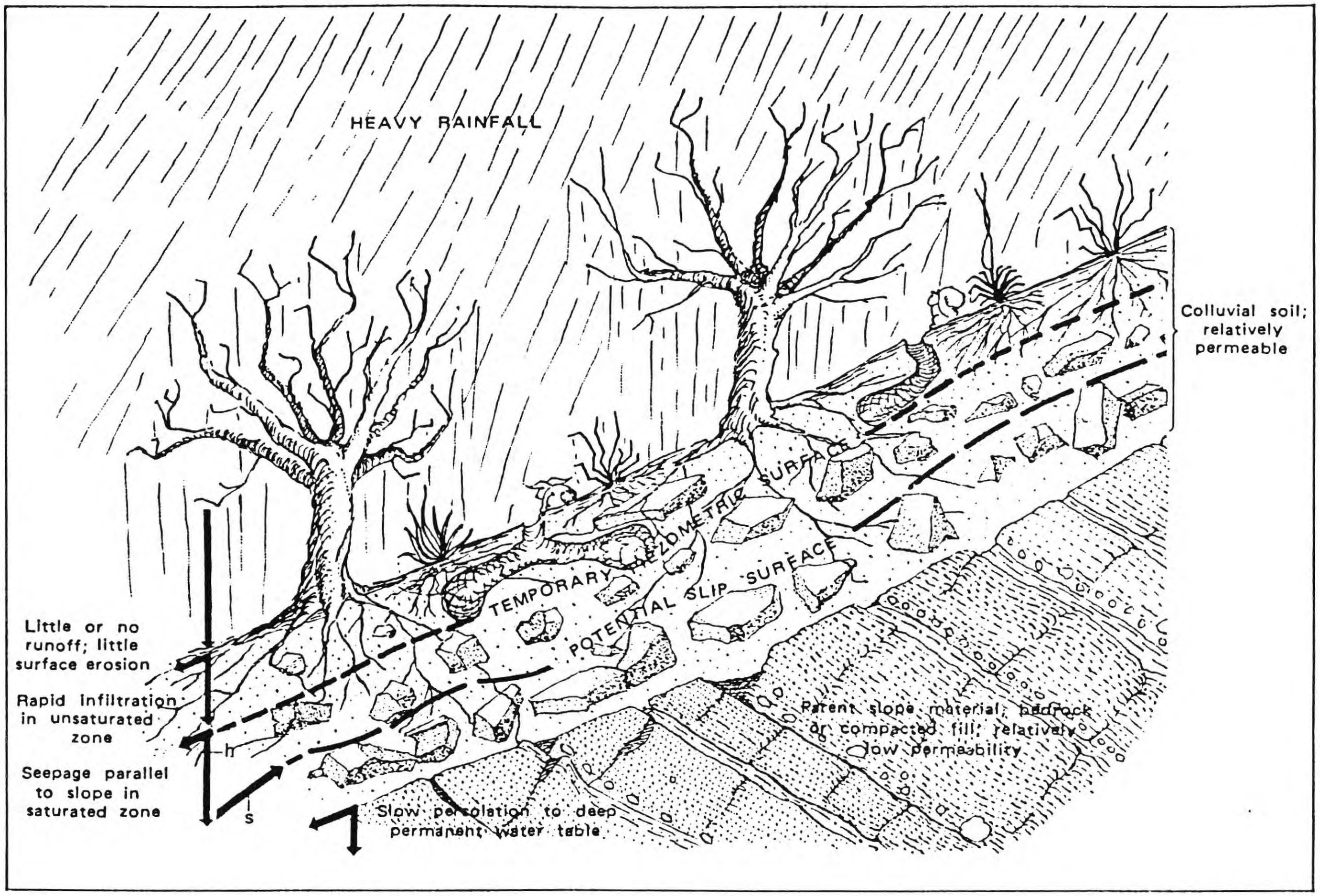


Figure 15. Diagram showing a temporary perched water table over a less permeable bedrock during heavy rainfall (from Campbell, 1975).

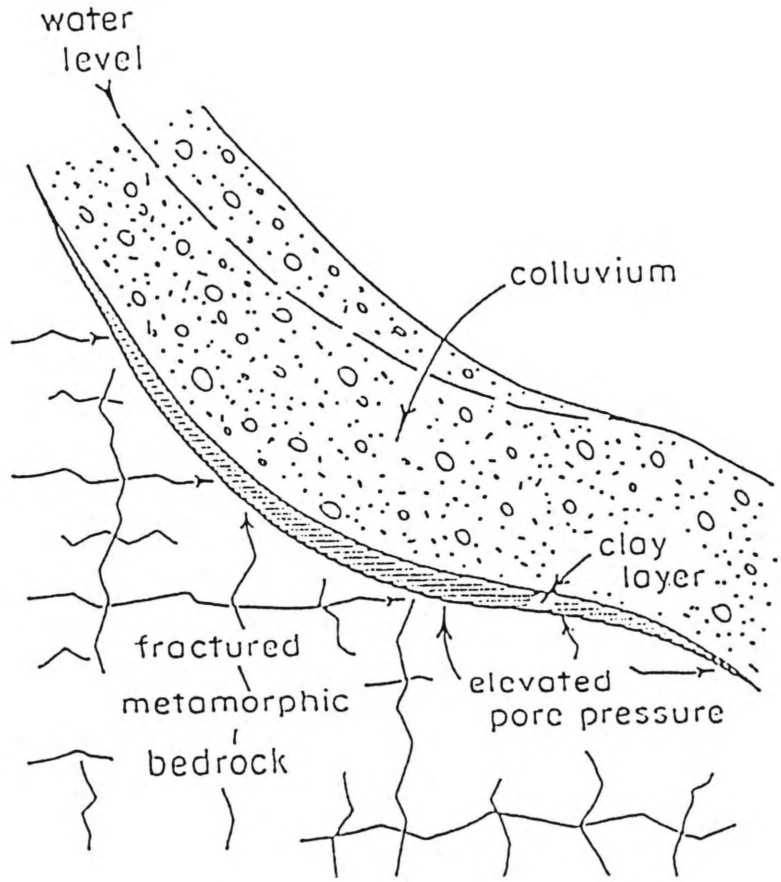


Figure 16. Diagram illustrating elevated pore pressures at the base of colluvium as a result of water flowing through fractured metamorphic bedrock (from Mathewson and Santi, 1987).

ground-water reservoir or evaporation (Johnson, 1987). This moisture, along with rainfall or snowmelt, generates the pore pressures that initiate debris flows. The most frequently discussed process of pore pressure generation involves the Campbell (1975) model, but Johnson (1987) describes three other mechanisms of pore pressure generation. These are: 1) the pinching out of more hydraulically conductive soil layers; 2) a flow constraint provided by areas of thinner soil associated with undulations of the soil-bedrock interface; and 3) flow through the bedrock mass into a relatively less permeable soil (Johnson, 1987). Harp and others (1990) studied soil pore-water pressures during initiation of induced slope failures in the Wasatch Range, Utah, and in the San Dimas Experimental Forest of southern California. In each of the experiments, trenches were dug to introduce water into the system. All three sites were instrumented with electronic piezometers and displacement meters to record pore pressures and slope movements during failures. Large variations, both spatially and temporally, occurred within the slopes at one of the sites in the Wasatch Range. This particular site was located at the head of a debris flow complex that was initiated during 1983. The cause for the continual changing pore pressures is probably due to the heterogeneous character of the colluvial slopes (Harp, et al., 1990).

Obviously, in order to fully evaluate debris flow occurrence in a region the geomorphology, the antecedent moisture conditions, the soil type, and the geology all need to be evaluated. To state that the increase in pore pressure at failure, is due only to a "temporary perched water table over low permeability bedrock" is grossly erroneous for areas where an infinite slope is not representative of the slope and heterogeneities exist in the hydraulic conductivity of the colluvium. An investigation should involve a program of both pore pressure and slope movement monitoring during failure, to understand this interrelationship at each site. Sites that are composed of heterogeneous soils will provide greater opportunities for pore pressure concentrations than sites that are composed of homogeneous soils where pore pressures are more predictable.

Haefeli (1948) proposed that for slopes with seepage parallel to the slope surface

$$FS = (1 - r_u)\frac{\tan ø'}{\tan \alpha}$$

where FS = factor of safety

$r_u = \frac{u}{\gamma H}$ (pore pressure ratio)

ø' = effective angle of internal friction

α = slope angle, deg.

u = pore pressure, psf

γ = total unit weight, pcf

H = depth to pore pressure u, ft

For the condition modelled in Steed Canyon, at the head of the debris flow scar which mobilized from the toe of a detached landslide block in 1983, $\alpha = 32^0$, ø' = 38^0, H = 5 ft, and γ = 135 pcf. The value of γ was calculated from the formula

$$\gamma_d = \frac{\gamma}{1 + w} \text{ (Das, 1985)}$$

where γ_d = dry unit weight of soil, 118 pcf

γ = total unit weight of soil, pcf w = moisture content.

A moisture content between 14 and 15 percent was assumed at failure, based on liquid limits of approximately 20 percent from Keaton (1988) and Santi (1988). For a colluvial deposit with large clasts and boulders, the moisture content at failure would likely be less than liquid limit, because of the inverse relationship between the liquid limit and grain size.

By substituting appropriate values into the equation proposed by Haefeli (1948) and back calculating the pore pressure at failure (FS = 1), u = 139 psf. If 139 psf is converted to a corresponding ground-water height, this corresponds to 2.23 ft of water. If 2.23 ft of water is expressed in terms of a ground-water ratio for 5 ft of colluvium, then the ground-water ratio = 0.45. Recall from dLISA, the ground-water ratio at failure for the same model conditions was 1.5. Which ground-water ratio is correct?

From dLISA, if the conditions at failure of the Steed Canyon debris flow are modelled (soil depth = 5 ft, slope = 32^0), assuming total saturation of colluvium (ground-water ratio = 1), then the FS = 1.30. This cannot be correct, because the conditions modelled at Steed did fail in 1983, which implies a factor of safety less than one.

When the equation proposed by Haefeli (1948) is used to calculate a factor of safety, for 5 ft of colluvium that is totally saturated on a 32^0 slope, the factor of safety is 0.68. This is a reasonable value for the situation modelled. Therefore, it appears that the Haefeli (1948) equation coincides more with empirical observations than dLISA.

If the back calculated pore pressure of 139 psf is used in the dLISA factor of safety equation (page 18), expressed as a ground-water ratio, then at a factor of safety equal to unity, a tree root cohesions can be back calculated. A saturated unit weight was calculated by assuming a porosity between 25 and 30 percent. Brady (1984) reports a range of porosities for sandy surface soils between 35 and 50 percent. Colluvium porosities of the field area are probably lower than this range, because of the gravel and boulder content which will lower the porosities.

If tree root cohesion is represented by C_r, then $C_r = 172.3$ psf or 1.2 psi. This value for tree root cohesion could be used to represent the amount of stability, expressed in psf or psi, due to tree root cohesion, failure plane roughness, and bedrock impedance at the toe of the Steed Canyon swale.

All of the above factors tend to stabilize the colluvium, at slope angles contrary to what is predicted by Haefeli (1948). According to Haefeli, when the ground-water level is at ground level and flowing parallel to the slope, a cohesionless sand will fail on a slope angle approximately 1/2 the angle of internal friction. Field observations contradict what is predicted by Haefeli's (1948) factor of safety equation, because many slopes greater than 1/2 the internal angle of friction have not failed. With every snowmelt in the spring, the colluvium of the upper mountain regions of Davis County, Utah, is likely saturated. Recall, that the model angle of internal friction was 38^0 and based on field observations the model slope angle was 32^0. Haefeli's (1948) equation does not consider soil cohesion, which also tends to stabilize the colluvium on a slope and contributes to the discrepancy between field observations and Haefeli's (1948) predictions.

In the study area, at the site of the debris flows that failed in 1983 and 1984 seepage not always parallel to the slope. Some amount probably drains into the fractured metamorphic bedrock and during recharge of the bedrock reservoir there is drainage into the colluvium form the bedrock. The bedrock reservoir is a reservoir of low storativity (Ala, 1990), which implies during snowmelt or heavy rainfall the peak discharge from the bedrock fractures occurs quickly. The seepage into the colluvium from the bedrock fractures could cause initiation of a debris flow, if internal drainage of the colluvium occurs at a slower rate than bedrock seepage into the colluvium. If seepage into the colluvium occurs, then the potential for the development of artesian pressures in the colluvium exists. A cause of the artesian pressures could be relatively lower hydraulic conductivity zones in the colluvium, acting to confine ground-water flow as it moves down the slope.

In Steed Canyon surface slopes were measured in the area of the detached landslide (slump block) studied by Montieth (1988) and Brooks (1986). Up-slope of the slump block the slope angle was approximately 35^0, within the block approximately 19^0, and downslope 32^0. Figure 17 is a sketch showing the surface slopes, underlying bedrock, colluvial cover, and piezometric surface. The changes in surface slopes were assumed to be a reflection of a changing bedrock dip. No piezometric data were collected for this study, therefore the sketch is not to scale. Since no springs were observed at this location in Steed Canyon during the field study, the piezometric surface was much shallower or did not exist in July and August of 1989. During a snowmelt or heavy rainfall event the

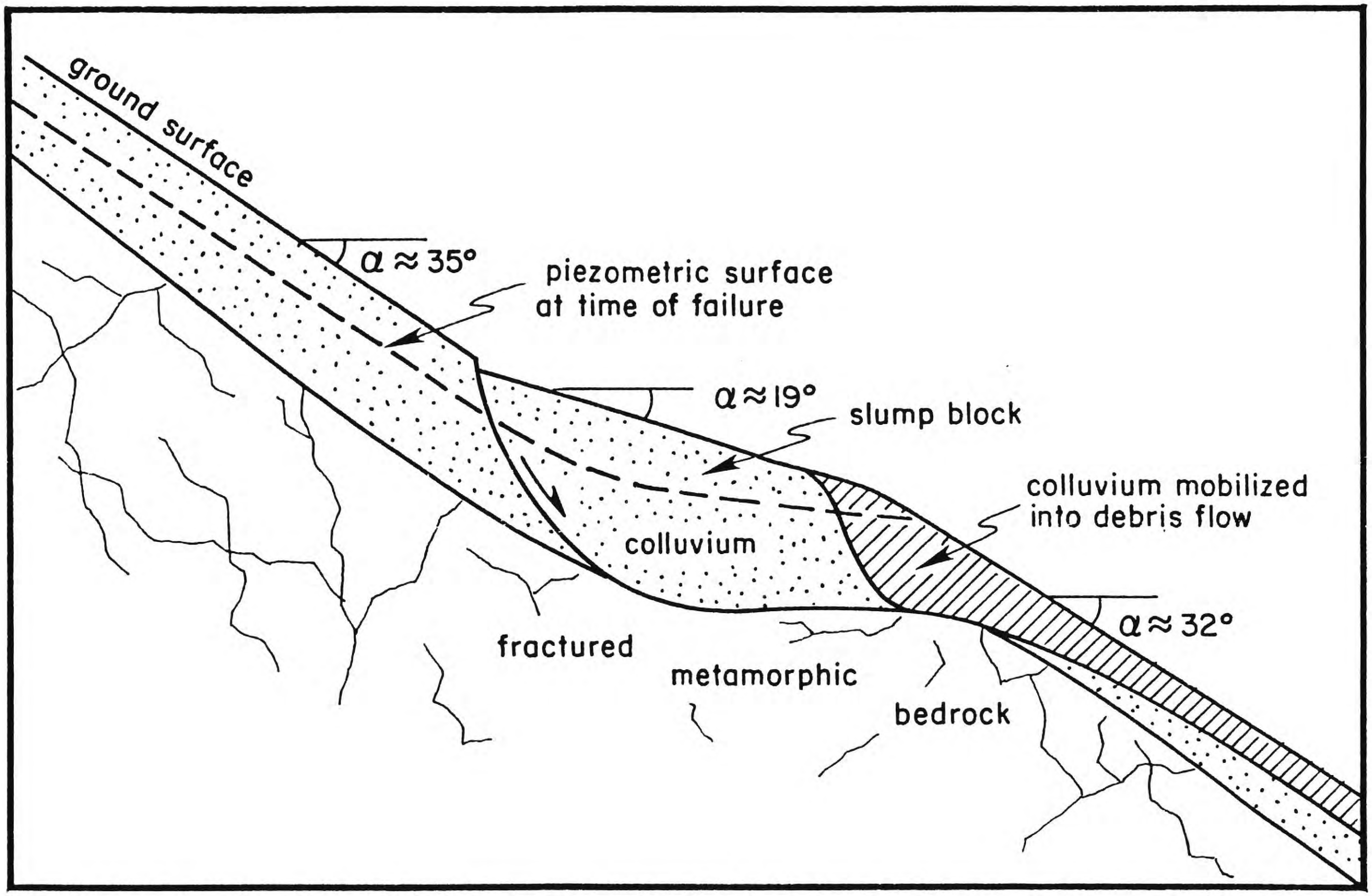


Figure 17. Cross-section showing the relationship between the surface slope, bedrock dip, piezometric surface, detached landslide block (slump block), and debris flow in Steed Canyon.

piezometric surface rises and can cause debris flow initiation where it intersects the ground surface. This initiation is caused by an increase in colluvium pore pressure as a result of the head of water up-gradient of where the piezometric surface intersects the ground surface. There is a ground water concentration within the swale because of a flattening of the bedrock surface which causes the hydraulic gradient to decrease.

CONCLUSIONS

By using a probabilistic model to examine the slope stability in Davis County, Utah, it was determined that the Campbell (1975) model of debris flow initiation was not the dominant mechanism. Debris flows initiated from upper mountain swales in 1983 and 1984 were due to ground-water flow concentrations in colluvium, which contains heterogeneous hydraulic conductivities. The relatively lower hydraulic conductivity zones act as a confining layer and can cause artesian pressures to develop as seepage from the bedrock fractures flows down slope and becomes confined between low hydraulic conductivity zone and the bedrock.

Based on this study and data collected in the field, laboratory, and estimates from the literature, the following conclusions can be made.

(1) The input parameters for LISA and dLISA are listed below:

soil depth (ft.)	5
slope (deg.)	32
tree surcharge (psf)	0
root cohesion (psf)	0
friction angle (deg.)	38
soil cohesion (psf)	190
dry density (pcf)	118
moisture content (%)	5 - 20 (liquid limit)
specific gravity	2.66

(2) The infinite slope equation was most sensitive to changes in soil depth and surface slope. Therefore, the geologic and geomorphic variations of a particular site affect the slope stability more than variations in material properties. At a given site the saturated thickness changes temporally. At any one site, for a reconnaissance level investigation, the

greatest effort should be spent collecting geologic, geomorphic, and hydrogeologic data.

(3) Slope stability for the study area, based on factors of safety and probabilities of failure from LISA indicate the slopes of the study area were stable when the field data were collected. This coincides with the empirical observations. LISA's low probabilities of failure for the 1983 and 1984 failure conditions implies some other mechanism caused failure than the Campbell (1975) style mechanism. The factor of safety equation LISA uses and the Campbell (1975) model are both based on the infinite slope equation with no artesian pressures present. Artesian pressures cannot be ruled out, based on the piezometers installed by Pack (1984).

(4) When dLISA was used to back calculate ground-water ratios at failure, values greater than one were calculated. For a phreatic surface, a ground-water ratio greater than one implies that the surface would be located above ground level indicating artesian pressures.

(5) Haefeli's (1948) factor of safety equation more accurately predicts the slope stability in Steed Canyon than dLISA.

(6) Slope stabilization occurs due to failure plane roughness, root cohesion, soil cohesion, and bedrock impedance at the toe of swales. This stabilization causes stability of colluvium on slopes that cannot be modelled as cohesionless sand sliding on a smooth failure plane of infinite length (infinite slope).

RECOMMENDATIONS

For this study, it was important to understand the initiation mechanism (s) of debris flows in the Wasatch Range of Davis County, Utah. LISA and dLISA were used to help understand the interrelationships of the different parameters involved in the debris flow process. From this study the following recommendations can be made.

(1) More studies should be conducted involving piezometer monitoring of slopes during failures. These data should then be analyzed for site specific applications. If numerous sites yield the same type of response during failure, then the different sites can be studied as a group. For any given area, grouping of data should done based on the amount of detail required for each project.

(2) It is critical that a geologist who understands the geologic and geomorphic processes operating at a particular site be involved when LISA is used. LISA is a very powerful tool if used and viewed in the proper perspective. From a range of parameter values, LISA can and will create unrealistic geologic and geomorphic conditions. It is imperative that the user understand this before LISA is used. An important statistical concept to understand is that a Monte Carlo simulation is a totally random simulation, that only uses parameter values that the user allows it to use. For example, if a slope of 60^0 and a soil depth of 50 ft are chosen in a Monte Carlo simulation this will be one iteration that will have a factor of safety less than 1. But, this situation would not exist in the field, so it is useless to model it.

(3) LISA and dLISA should be modified so that pore pressures can be input.

(4) Future models of the slope stability in Davis County, Utah, should consider the contributions failure plane roughness, root cohesion, soil cohesion, and bedrock impedance have on slope stabilization.

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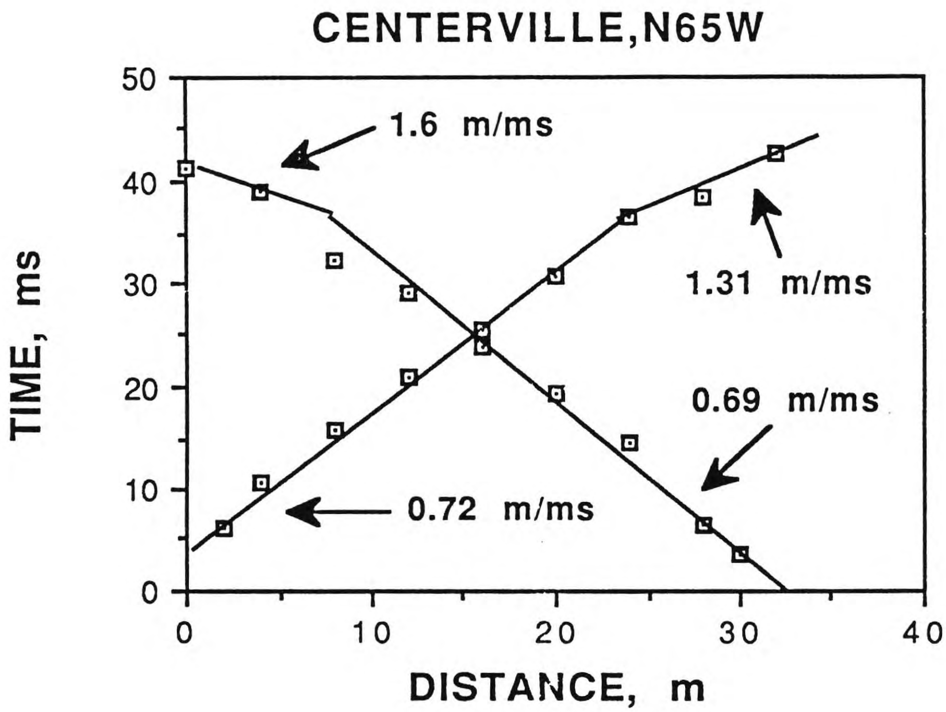
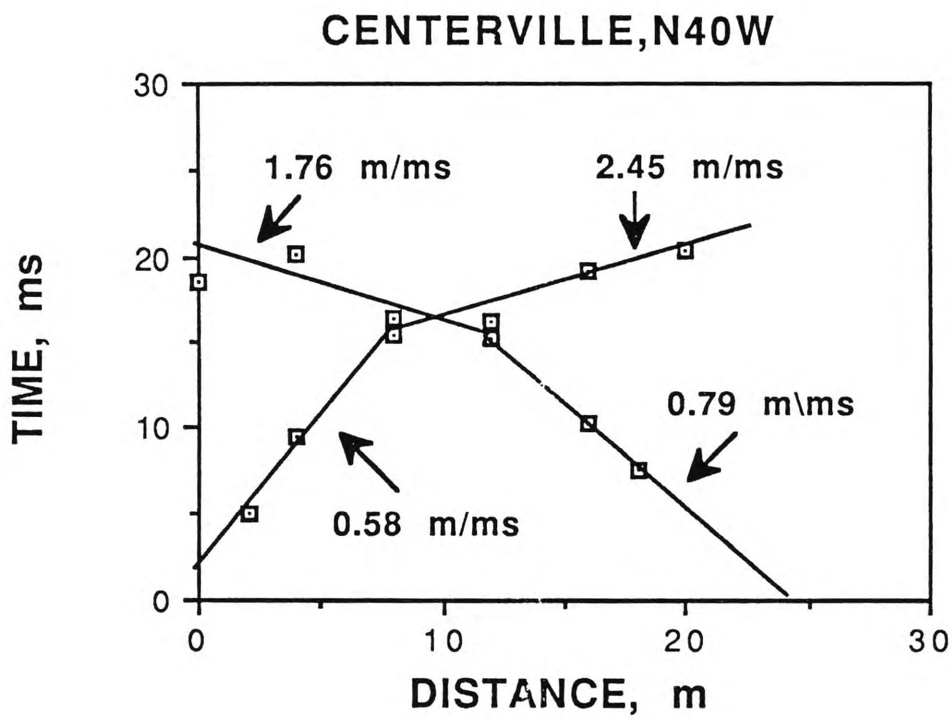
APPENDIX A

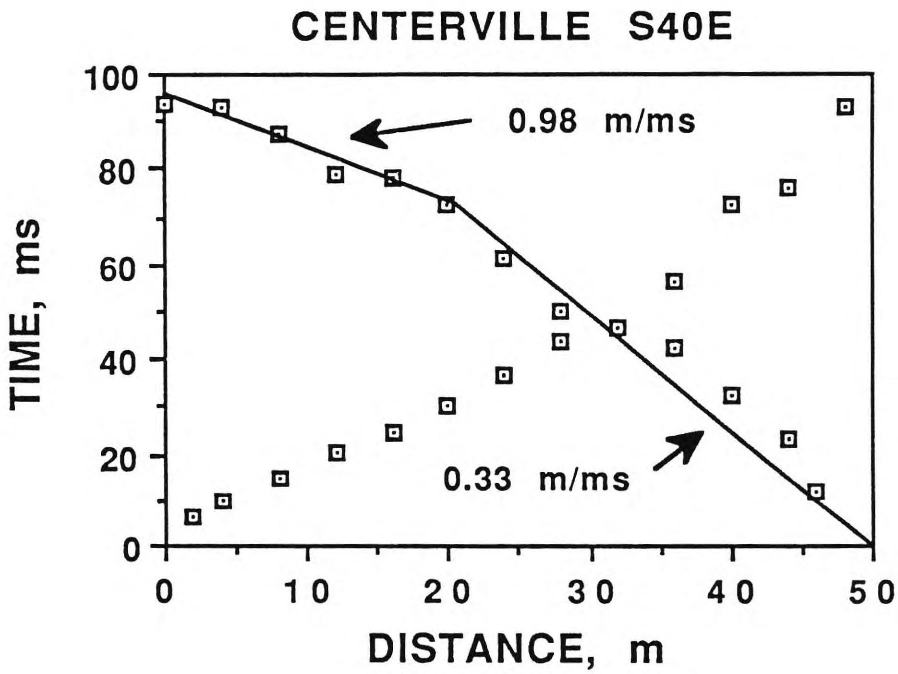
TRAVEL TIME VS. DISTANCE PLOTS



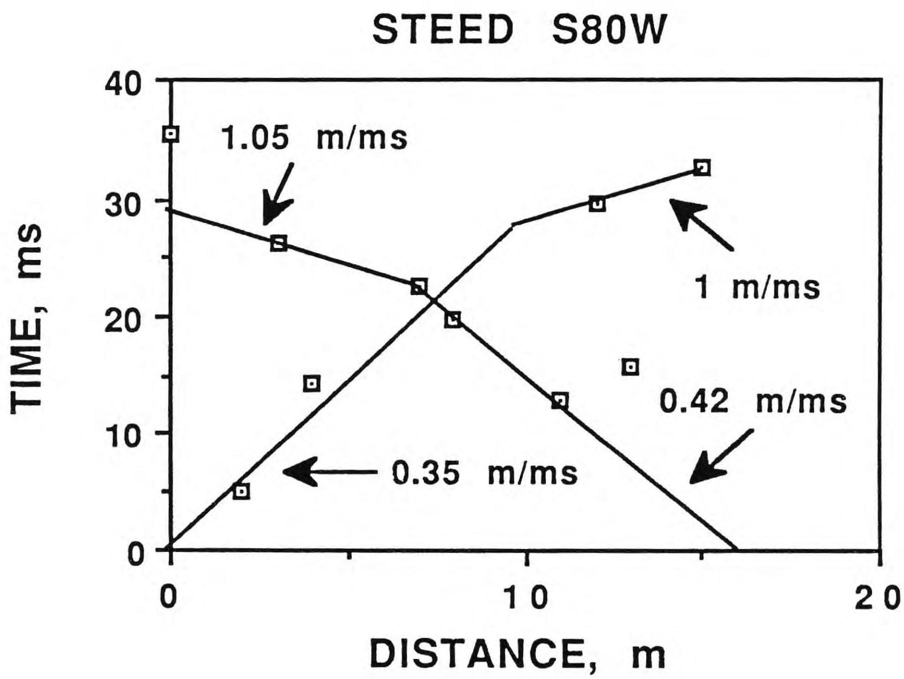
Appendix A1. Travel Time vs. Distance plots for a single channel seismic line in Centerville Canyon.



Appendix A1. Continued.



Appendix A1. Continued.

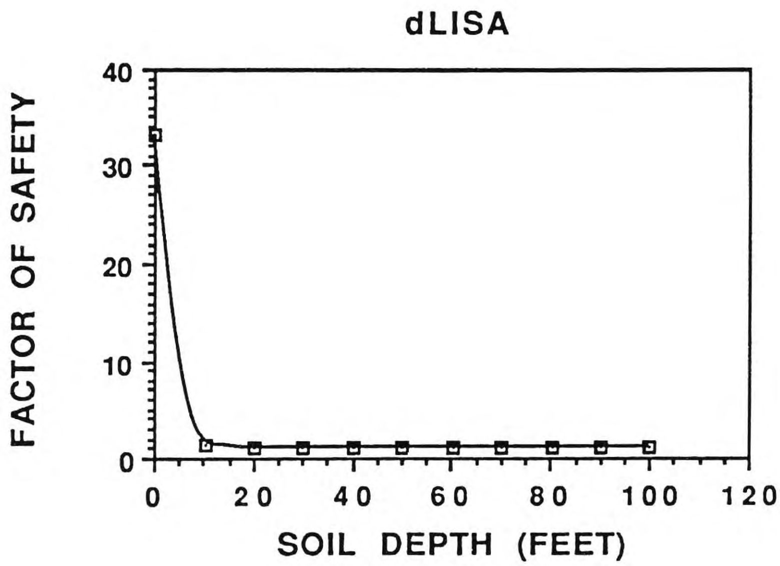


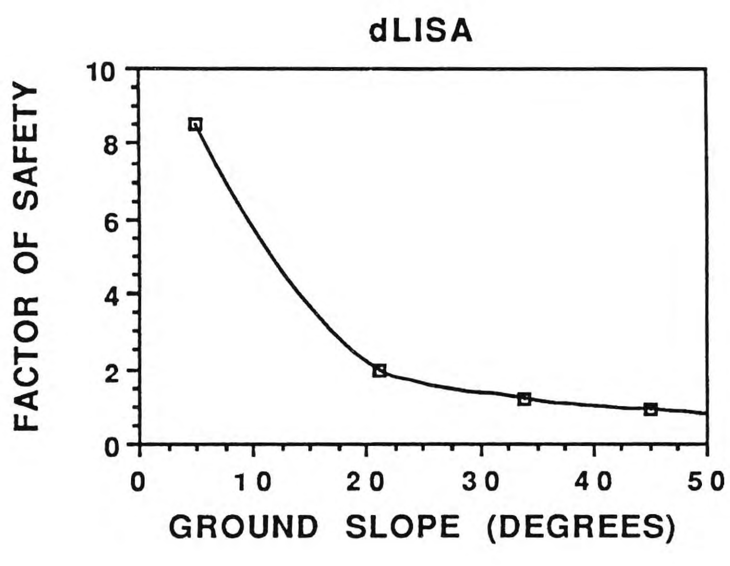
Appendix A2. Travel Time vs. Distance plot for a single channel seismic line in Steed Canyon.

APPENDIX B

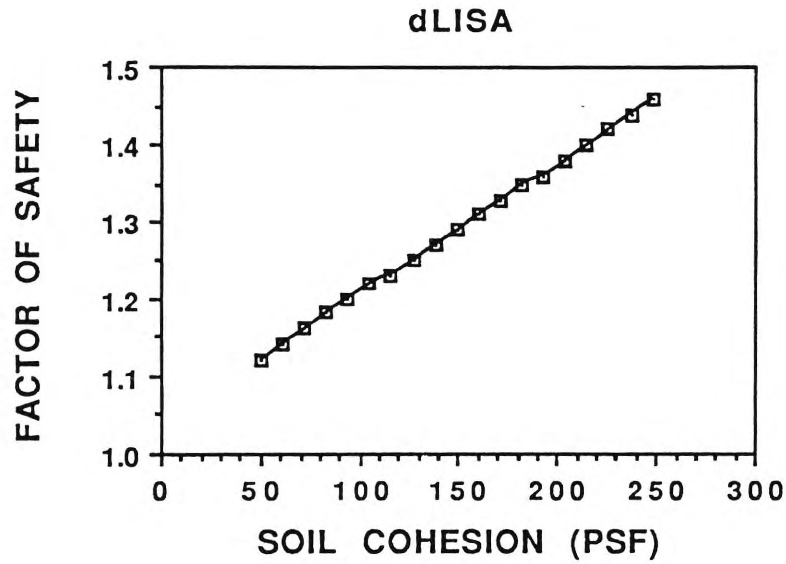
SENSITIVITY ANALYSIS

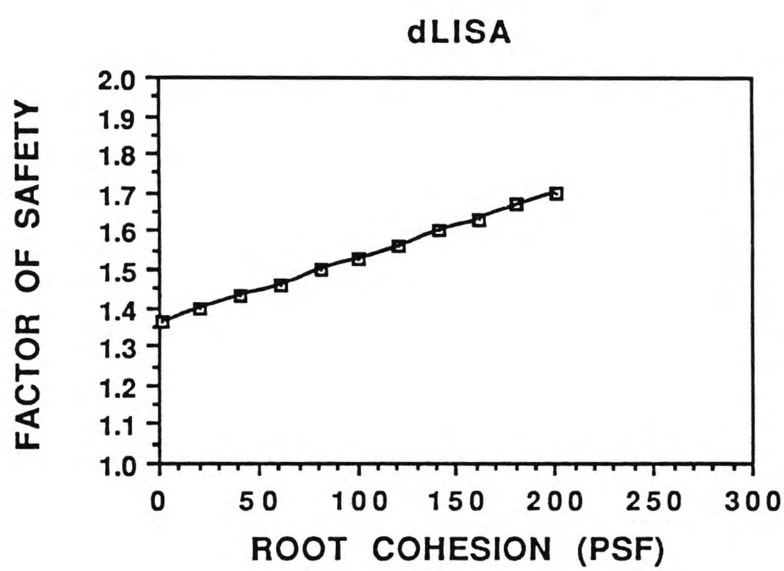
The Factor of Safety vs. Parameter plots were used to create the Sensitivity Analysis data (pages 80-83, Appendix B). Data for the plots were generated using dLISA, by varying the parameter and solving for the factor of safety.



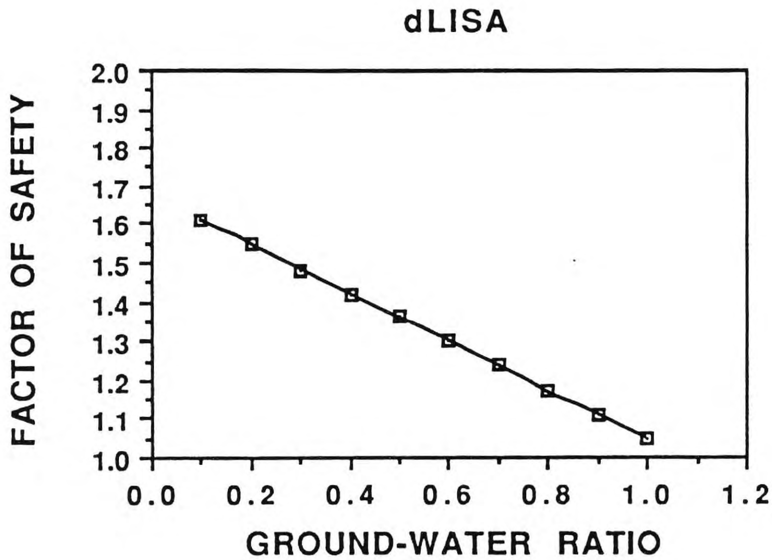


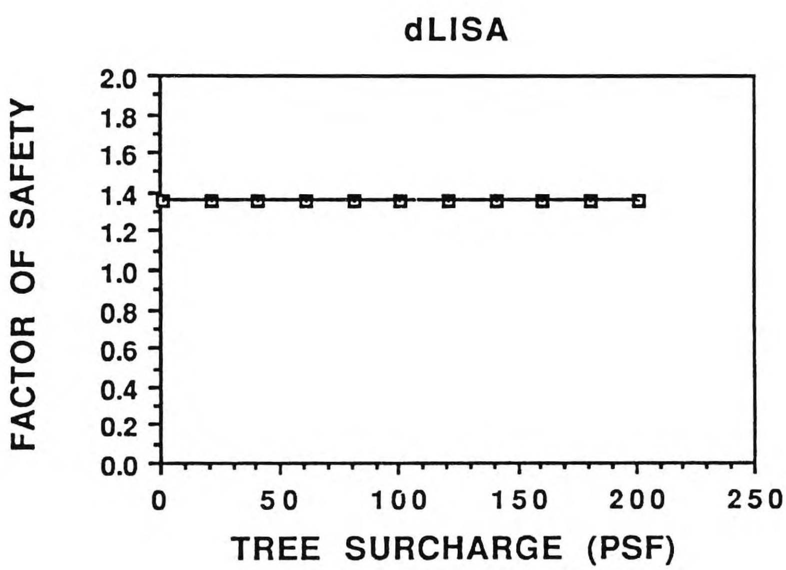
Appendix B1. Factor of Safety vs. Parameter plot.



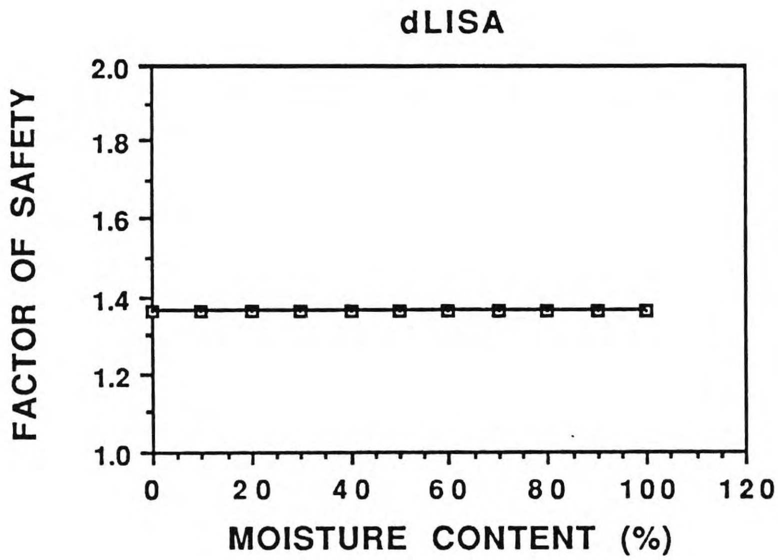


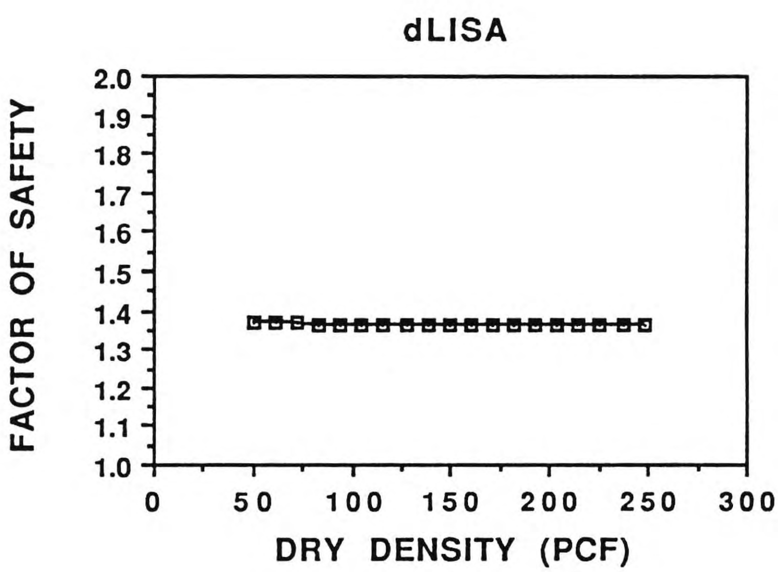
Appendix B1. Continued.



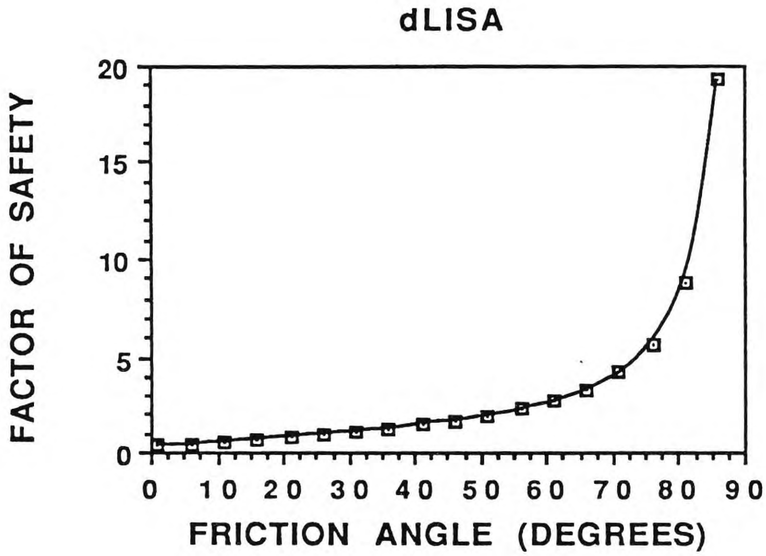


Appendix B1. Continued.





Appendix B1. Continued.



Appendix B1. Continued.

SOIL DEPTH (% change)	FS (% change)
0.000	0.000
10.000	99.100
20.000	99.600
30.000	99.750
40.000	99.840
50.000	99.910
60.000	99.940
70.000	99.970
80.000	99.970
90.000	100.000
100.000	100.000

SLOPE (% change)	FS (% change)
0.000	0.000
5.560	83.130
11.110	93.160
16.670	96.840
22.220	98.480
27.780	99.370
33.330	99.750
38.890	100.000

GROUNDWATER RATIO (% change)	FS (% change)
0.000	0.000
11.220	10.710
22.220	23.210
33.330	33.930
44.440	44.640
55.560	55.360
66.670	66.070
77.780	78.570
88.990	89.290
100.000	100.000

Appendix B2. Sensitivity Analysis data.

SOIL COHESION (% change)	FS (% change)
0.000	0.000
5.560	5.880
11.110	11.760
16.670	17.650
22.220	23.530
27.780	29.410
33.330	32.350
38.890	38.240
44.440	44.120
50.000	50.000
55.560	55.880
61.110	61.760
66.670	67.650
72.220	70.590
77.780	76.470
83.330	83.250
88.890	88.240
94.440	94.120
100.000	100.000

DRY DENSITY (% change)	FS (% change)
0.000	0.000
5.560	0.000
11.110	0.000
16.670	100.000
22.220	100.000
27.780	100.000
33.330	100.000
38.890	100.000
44.440	100.000
50.000	100.000
55.560	100.000
61.110	100.000
66.670	100.000
72.220	100.000
77.780	100.000
83.330	100.000
88.890	100.000
94.440	100.000
100.000	100.000

Appendix B2. Continued.

TREE SURCHARGE (% change)	FS (% change)
0.000	0.000
10.000	0.000
20.000	0.000
30.000	0.000
40.000	0.000
50.000	0.000
60.000	0.000
70.000	0.000
80.000	0.000
90.000	0.000
100.000	0.000

ROOT COHESION (% change)	FS (% change)
0.000	0.000
10.000	11.760
20.000	20.590
30.000	29.410
40.000	41.180
50.000	50.000
60.000	58.820
70.000	70.590
80.000	79.410
90.000	91.180
100.000	100.000

FRICTION ANGLE (% change)	FS (% change)
0.000	0.000
5.880	0.580
11.760	1.210
17.650	1.850
23.530	2.530
29.410	3.270
35.290	4.060
41.180	4.960
47.060	5.960
52.940	7.120
58.820	8.490
64.710	10.230
70.590	12.490
76.470	15.600
82.350	20.190
89.410	27.940
94.120	44.070
100.000	100.000

Appendix B2. Continued.

MOISTURE CONTENT (% change)	FS (% change)
0.000	0.000
10.000	0.000
20.000	0.000
30.000	0.000
40.000	0.000
50.000	0.000
60.000	0.000
70.000	0.000
80.000	0.000
90.000	0.000
100.000	0.000

Appendix B2. Continued.

APPENDIX C

FACTOR OF SAFETY VS. GROUND-WATER RATIO

These data were used to create Figure 10 for Lightning, Steed, and Centerville Canyons.

LEVEL I STABILITY ANALYSIS -- FACTOR OF SAFETY

Infinite Slope Equation

dLISA 1.02

	Soil depth (ft)	8.10
	Surface slope (deg)	20.00
	Tree surcharge (psf)	0.00
	Root Cohesion (psf)	0.00
SOLVE FOR	Groundwater ratio	
	Friction angle (deg)	38.00
	Soil cohesion (psf)	190.00
	Dry density (pcf)	118.00
	Moisture content (%)	20.00
	Specific gravity	2.66
VARIABLE	Factor of safety	

Factor of safety	Groundwater ratio
0.10	2.62
0.20	2.52
0.30	2.42
0.40	2.32
0.50	2.22
0.60	2.12
0.70	2.01
0.80	1.91
0.90	1.81
1.00	1.71
1.10	1.61
1.20	1.51
1.30	1.40
1.40	1.30
1.50	1.20
1.60	1.10
1.70	1.00
1.80	0.90
1.90	0.80
2.00	0.69

Dry density (pcf)	Moist density (pcf)	Saturated density (pcf)	Saturated moist cont (%)	moist cont (%)
118.00	136.04	136.04	15.29	15.29

Appendix C1. Factor of Safety vs. Ground-water Ratio for Lightning Canyon, on a 20^0 slope.

LEVEL I STABILITY ANALYSIS -- FACTOR OF SAFETY

Infinite Slope Equation

dLISA 1.02

	Soil depth (ft)	8.10
	Surface slope (deg)	20.00
	Tree surcharge (psf)	0.00
	Root Cohesion (psf)	0.00
SOLVE FOR	Groundwater ratio	
	Friction angle (deg)	38.00
	Soil cohesion (psf)	190.00
	Dry density (pcf)	118.00
	Moisture content (%)	20.00
	Specific gravity	2.66
VARIABLE	Factor of safety	

Factor of safety	Groundwater ratio
0.10	2.62
0.20	2.52
0.30	2.42
0.40	2.32
0.50	2.22
0.60	2.12
0.70	2.01
0.80	1.91
0.90	1.81
1.00	1.71
1.10	1.61
1.20	1.51
1.30	1.40
1.40	1.30
1.50	1.20
1.60	1.10
1.70	1.00
1.80	0.90
1.90	0.80
2.00	0.69

Dry density (pcf)	Moist density (pcf)	Saturated density (pcf)	Saturated moist cont (%)	moist cont (%)
118.00	136.04	136.04	15.29	15.29

Appendix C1. Factor of Safety vs. Ground-water Ratio for Lightning Canyon, on a 20^0 slope.

LEVEL 1 STABILITY ANALYSIS -- FACTOR OF SAFETY

Infinite Slope Equation

dLISA 1.02

	Soil depth (ft)	5.40
	Surface slope (deg)	30.00
	Tree surcharge (psf)	0.00
	Root Cohesion (psf)	0.00
SOLVE FOR	Groundwater ratio	
	Friction angle (deg)	38.00
	Soil cohesion (psf)	190.00
	Dry density (pcf)	118.00
	Moisture content (%)	20.00
	Specific gravity	2.66
VARIABLE	Factor of safety	

Factor of safety	Groundwater ratio
0.10	2.98
0.20	2.82
0.30	2.66
0.40	2.50
0.50	2.34
0.60	2.18
0.70	2.01
0.80	1.85
0.90	1.69
1.00	1.53
1.10	1.37
1.20	1.21
1.30	1.05
1.40	0.89
1.50	0.73
1.60	0.56
1.70	0.40
1.80	0.24
1.90	0.08

One or more values are impossible to solve for

Dry density (pcf)	Moist density (pcf)	Saturated density (pcf)	Saturated moist cont (%)	moist cont (%)
118.00	136.04	136.04	15.29	15.29

Appendix C2. Continued. Factor of Safety vs. Ground-water Ratio for Lightning Canyon, on a 30° slope.

LEVEL I STABILITY ANALYSIS -- FACTOR OF SAFETY

Infinite Slope Equation

dLISA 1.02

	Soil depth (ft)	2.70
	Surface slope (deg)	40.00
	Tree surcharge (psf)	0.00
	Root Cohesion (psf)	0.00
SOLVE FOR	Groundwater ratio	
	Friction angle (deg)	38.00
	Soil cohesion (psf)	190.00
	Dry density (pcf)	118.00
	Moisture content (%)	20.00
	Specific gravity	2.66
VARIABLE	Factor of safety	

Factor of safety	Groundwater ratio
0.10	4.41
0.20	4.17
0.30	3.94
0.40	3.70
0.50	3.47
0.60	3.23
0.70	3.00
0.80	2.77
0.90	2.53
1.00	2.30
1.10	2.06
1.20	1.83
1.30	1.60
1.40	1.36
1.50	1.13
1.60	0.89
1.70	0.66
1.80	0.43
1.90	0.19

One or more values are impossible to solve for

Dry density (pcf)	Moist density (pcf)	Saturated density (pcf)	Saturated moist cont (%)	moist cont (%)
118.00	136.04	136.04	15.29	15.29

Appendix C3. Continued. Factor of Safety vs. Ground-water Ratio for Lightning Canyon, on a 40^0 slope.

LEVEL I STABILITY ANALYSIS -- FACTOR OF SAFETY

Infinite Slope Equation

dLISA 1.02

	Soil depth (ft)	15.00
	Surface slope (deg)	20.00
	Tree surcharge (psf)	0.00
	Root Cohesion (psf)	0.00
SOLVE FOR	Groundwater ratio	.
	Friction angle (deg)	38.00
	Soil cohesion (psf)	190.00
	Dry density (pcf)	118.00
	Moisture content (%)	20.00
	Specific gravity	2.66
VARIABLE	Factor of safety	.

Factor of safety	Groundwater ratio
0.10	2.37
0.20	2.27
0.30	2.17
0.40	2.07
0.50	1.97
0.60	1.86
0.70	1.76
0.80	1.66
0.90	1.56
1.00	1.46
1.10	1.36
1.20	1.26
1.30	1.15
1.40	1.05
1.50	0.95
1.60	0.85
1.70	0.75
1.80	0.65
1.90	0.54
2.00	0.44

Dry density (pcf)	Moist density (pcf)	Saturated density (pcf)	Saturated moist cont (%)	moist cont (%)
118.00	136.04	136.04	15.29	15.29

Appendix C4. Continued. Factor of Safety vs. Ground-water Ratio for Steed Canyon, on a 20^0 slope.

LEVEL I STABILITY ANALYSIS -- FACTOR OF SAFETY

Infinite Slope Equation

dLISA 1.02

	Soil depth (ft)	10.00
	Surface slope (deg)	30.00
	Tree surcharge (psf)	0.00
	Root Cohesion (psf)	0.00
SOLVE FOR	Groundwater ratio	
	Friction angle (deg)	38.00
	Soil cohesion (psf)	190.00
	Dry density (pcf)	118.00
	Moisture content (%)	20.00
	Specific gravity	2.66
VARIABLE	Factor of safety	

Factor of safety	Groundwater ratio
0.10	2.54
0.20	2.38
0.30	2.22
0.40	2.06
0.50	1.89
0.60	1.73
0.70	1.57
0.80	1.41
0.90	1.25
1.00	1.09
1.10	0.93
1.20	0.77
1.30	0.61
1.40	0.44
1.50	0.28
1.60	0.12

One or more values are impossible to solve for

Dry density (pcf)	Moist density (pcf)	Saturated density (pcf)	Saturated moist cont (%)	moist cont (%)
118.00	136.04	136.04	15.29	15.29

Appendix C5. Continued. Factor of Safety vs. Ground-water Ratio for Steed Canyon, on a 30^{0} slope.

LEVEL I STABILITY ANALYSIS -- FACTOR OF SAFETY

Infinite Slope Equation

dLISA 1.02

	Soil depth (ft)	5.00
	Surface slope (deg)	40.00
	Tree surcharge (psf)	0.00
	Root Cohesion (psf)	0.00
SOLVE FOR	Groundwater ratio	
	Friction angle (deg)	38.00
	Soil cohesion (psf)	190.00
	Dry density (pcf)	118.00
	Moisture content (%)	20.00
	Specific gravity	2.66
VARIABLE	Factor of safety	

Factor of safety	Groundwater ratio
0.10	3.27
0.20	3.04
0.30	2.81
0.40	2.57
0.50	2.34
0.60	2.10
0.70	1.87
0.80	1.64
0.90	1.40
1.00	1.17
1.10	0.93
1.20	0.70
1.30	0.46
1.40	0.23

One or more values are impossible to solve for

Dry density (pcf)	Moist density (pcf)	Saturated density (pcf)	Saturated moist cont (%)	moist cont (%)
118.00	136.04	136.04	15.29	15.29

Appendix C6. Continued. Factor of Safety vs. Ground-water Ratio for Steed Canyon, on a 40^0 slope.

LEVEL I STABILITY ANALYSIS --- FACTOR OF SAFETY

Infinite Slope Equation

dLISA 1.02

	Soil depth (ft)	15.54
	Surface slope (deg)	20.00
	Tree surcharge (psf)	0.00
	Root Cohesion (psf)	0.00
SOLVE FOR	Groundwater ratio	
	Friction angle (deg)	38.00
	Soil cohesion (psf)	190.00
	Dry density (pcf)	118.00
	Moisture content (%)	20.00
	Specific gravity	2.66
VARIABLE	Factor of safety	

Factor of safety	Groundwater ratio
0.10	2.36
0.20	2.26
0.30	2.16
0.40	2.06
0.50	1.96
0.60	1.85
0.70	1.75
0.80	1.65
0.90	1.55
1.00	1.45
1.10	1.35
1.20	1.25
1.30	1.14
1.40	1.04
1.50	0.94
1.60	0.84
1.70	0.74
1.80	0.64
1.90	0.53
2.00	0.43

Dry density (pcf)	Moist density (pcf)	Saturated density (pcf)	Saturated moist cont (%)	moist cont (%)
118.00	136.04	136.04	15.29	15.29

Appendix C7. Continued. Factor of Safety vs. Ground-water Ratio for Centerville Canyon, on a 20^0 slope.

LEVEL 1 STABILITY ANALYSIS -- FACTOR OF SAFETY

Infinite Slope Equation

dLISA 1.02

	Soil depth (ft)	10.36
	Surface slope (deg)	30.00
	Tree surcharge (psf)	0.00
	Root Cohesion (psf)	0.00
SOLVE FOR	Groundwater ratio	
	Friction angle (deg)	38.00
	Soil cohesion (psf)	190.00
	Dry density (pcf)	118.00
	Moisture content (%)	20.00
	Specific gravity	2.66
VARIABLE	Factor of safety	

Factor of safety	Groundwater ratio
0.10	2.52
0.20	2.36
0.30	2.20
0.40	2.04
0.50	1.88
0.60	1.72
0.70	1.55
0.80	1.39
0.90	1.23
1.00	1.07
1.10	0.91
1.20	0.75
1.30	0.59
1.40	0.43.
1.50	0.27
1.60	0.10

One or more values are impossible to solve for

Dry density (pcf)	Moist density (pcf)	Saturated density (pcf)	Saturated moist cont (%)	moist cont (%)
118.00	136.04	136.04	15.29	15.29

Appendix C8. Continued. Factor of Safety vs. Ground-water Ratio for Centerville Canyon, on a 30^0 slope.

LEVEL I STABILITY ANALYSIS -- FACTOR OF SAFETY

Infinite Slope Equation

dLISA 1.02

	Soil depth (ft)	5.18
	Surface slope (deg)	40.00
	Tree surcharge (psf)	0.00
	Root Cohesion (psf)	0.00
SOLVE FOR	Groundwater ratio	
	Friction angle (deg)	38.00
	Soil cohesion (psf)	190.00
	Dry density (pcf)	118.00
	Moisture content (%)	20.00
	Specific gravity	2.66
VARIABLE	Factor of safety	

Factor of safety	Groundwater ratio
0.10	3.23
0.20	2.99
0.30	2.76
0.40	2.53
0.50	2.29
0.60	2.06
0.70	1.82
0.80	1.59
0.90	1.35
1.00	1.12
1.10	0.89
1.20	0.65
1.30	0.42
1.40	0.18

One or more values are impossible to solve for

Dry density (pcf)	Moist density (pcf)	Saturated density (pcf)	Saturated moist cont (%)	moist cont (%)
118.00	136.04	136.04	15.29	15.29

Appendix C9. Continued. Factor of Safety vs. Ground-water Ratio for Centerville Canyon, on a 40^0 slope.

APPENDIX D

GROUND-WATER RATIO VS. SOIL DEPTH

These data were used to create Figure 15.

LEVEL I STABILITY ANALYSIS -- FACTOR OF SAFETY

Infinite Slope Equation

dLISA 1.02

VARIABLE	Soil depth (ft)	
	Surface slope (%)	47.00
	Tree surcharge (psf)	0.00
	Root Cohesion (psf)	0.00
SOLVE FOR	Groundwater ratio	
	Friction angle (deg)	38.00
	Soil cohesion (psf)	190.00
	Dry density (pcf)	118.00
	Moisture content (%)	20.00
	Specific gravity	2.66
	Factor of safety	1.00

Soil depth (ft)	Groundwater ratio
0.10	48.45
5.30	1.77
10.50	1.32
15.70	1.17
20.90	1.10
26.10	1.05
31.30	1.02
36.50	1.00
41.70	0.98
46.90	0.97
52.10	0.96
57.30	0.95
62.50	0.94
67.70	0.94
72.90	0.93
78.10	0.93
83.30	0.93
88.50	0.92
93.70	0.92
98.90	0.92

Dry density (pcf)	Moist density (pcf)	Saturated density (pcf)	Saturated moist cont (%)	moist cont (%)
118.00	136.04	136.04	15.29	15.29

Appendix D1. Back calculated ground-water ratio for a 25^0 slope.

LEVEL I STABILITY ANALYSIS --- FACTOR OF SAFETY

Infinite Slope Equation

dLISA 1.02

VARIABLE	Soil depth (ft)	
	Surface slope (%)	64.00
	Tree surcharge (psf)	0.00
	Root Cohesion (psf)	0.00
SOLVE FOR	Groundwater ratio	
	Friction angle (deg)	38.00
	Soil cohesion (psf)	190.00
	Dry density (pcf)	118.00
	Moisture content (%)	20.00
	Specific gravity	2.66
	Factor of safety	1.00

Soil depth (ft)	Groundwater ratio
0.10	55.33
5.30	1.43
10.50	0.92
15.70	0.74
20.90	0.66
26.10	0.60
31.30	0.57
36.50	0.54
41.70	0.53
46.90	0.51
52.10	0.50
57.30	0.49
62.50	0.48
67.70	0.48
72.90	0.47
78.10	0.46
83.30	0.46
88.50	0.46
93.70	0.45
98.90	0.45

Dry density (pcf)	Moist density (pcf)	Saturated density (pcf)	Saturated moist cont (%)	moist cont (%)
118.00	136.04	136.04	15.29	15.29

Appendix D2. Continued. Back calculated ground-water ratio for a 32.5^0 slope.

LEVEL I STABILITY ANALYSIS --- FACTOR OF SAFETY

Infinite Slope Equation

dLISA 1.02

VARIABLE	Soil depth (ft)	
	Surface slope (%)	84.00
	Tree surcharge (psf)	0.00
	Root Cohesion (psf)	0.00
SOLVE FOR	Groundwater ratio	
	Friction angle (deg)	38.00
	Soil cohesion (psf)	190.00
	Dry density (pcf)	118.00
	Moisture content (%)	20.00
	Specific gravity	2.66
	Factor of safety	1.00

Soil depth (ft)	Groundwater ratio
0.10	66.31
5.30	1.09
10.50	0.47
15.70	0.26
20.90	0.15
26.10	0.09
31.30	0.05
36.50	0.02

One or more values are impossible to solve for

Dry density (pcf)	Moist density (pcf)	Saturated density (pcf)	Saturated moist cont (%)	moist cont (%)
118.00	136.04	136.04	15.29	15.29

Appendix D3. Continued. Back calculated ground-water ratio for a 40^0 slope.

VITA

James Storey Eblen was born in Wichita Falls, Texas, on November 8, 1963. His parents are the late W. E. (Bill) Eblen Jr., and Mary Ann (Eblen) Marshall. He has a sister, Nanette E. Moody and brother, W. E. (Bill) Eblen III. Nanette was born on September 5, 1959, and Bill was born on October 7, 1962.

He graduated from Canton High School in Canton, Texas, in May of 1983. He received his Bachelor of Science degree in Geology (Engineering Geology Option) in May of 1988, from Texas A&M University in College Station, Texas. He entered the Master of Science program in Geology (Center for Engineering Geosciences) at Texas A&M University in August of 1988.

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