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A Sensitivity Analysis of Factors that Determine Rock Slope Stability at the Lariat Clay Pit, South-Central Jefferson County, Colorado

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ABSTRACT

The Lariat Pit of the Robinson Brick Company is located on the eastern flank of the Dakota hogback, in south-central Jefferson County, Colorado. The mine produces high-quality clay, used in bricks. Shales of the Benton Group are stripped from the underlying rocks of the Dakota Formation, which are steeply-dipping. The resulting dip-slope pit wall may be unstable.

A study was undertaken to determine the potential for slope failure in the dip-slope west wall of the pit. The objectives of the study were threefold:

1. To assess the stability of the west wall of the pit in the present configuration.

2. To assess the stability of the pit wall in the proposed final configuration.

3. To determine the relative degree of influence of various slope parameters on the Factor of Safety.

Field data were collected and analyzed using conventional procedures. Stereographic analysis of fracture data revealed two potential failure modes: a plane failure subparallel to the pit wall and a wedge formed by the intersection of a joint set with a fault. A stability analysis, using the modified Limiting-Equilibrium method, was conducted on the potential failure geometries; the slope in the (idealized) present configuration was found to be stable.

A sensitivity analysis was conducted using the technique of parameter variation. The results of this analysis indicated that the factor of safety was highly sensitive to the effective slope angle of the pit wall. This determination allowed the investigation to focus on the probable cause of small-scale slope failures in the pit - the undercutting of the slope toe during mining operations. Undercutting increases the effective angle of the slope and can render a slope unstable. As a result of this observation, a recommendation was made that employees at the pit be advised of the potential hazards.

Use of sensitivity analysis identified the likely cause of slope failure so that a simple, cost-effective means of hazard mitigation could be recommended. This technique provides engineers a tool for conducting detailed hazard assessments or cost-benefit studies.

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ACKNOWLEDGEMENTS

If it is true that we all are composite products of our environment, then I have doubtless been significantly affected by numerous individuals since I began my academic career at the Colorado School of Mines. However, it is not difficult for me to identify those persons most responsible for the successes I have experienced in the last few years; this paper is tribute to their support.

Special thanks must be directed to the members of my thesis committee; as individuals, they have each had a profound influence on my development as a professional and as an person. Dr. Greg Holden has followed my progress through undergraduate and graduate years, and graciously consented to serve on the defense comittee. Dr. Keith Turner has consistently stressed the importance of practical solutions to engineering problems. Dr. John Abel has made available to me the full range of his profound knowledge of rock mechanics; the analytical approaches adopted here are a result of his guidance. Dr. Kenneth Kolm has been ever enthusiastic in describing the possibilities present in life and in the world. His editorial expertise contributed greatly to the clarity of this report. Most importantly, his ready friendship and consistently good advice were invaluable in advancing my academic career.

Certain individuals made significant contributions to the development of this analysis. First, I wish to thank Mr. Mike Leydich of the Robinson Brick Company for making the Lariat Pit facility available for the investigation. Mr. Earnesto Villaescusa provided capable assistance in the field and during laboratory testing of rock samples. Computer programs used in the reduction of data taken in the field were kindly made available to the author by Dr. John Abel. The assistance of Greg Naugle was invaluable in constructing plots of analyses.

Finally, I acknowledge with gratitude the unqualified support given to me by my dear wife, Donna Hallewell. Graduate students can be impossible folk, from time to time; while I occasionally lost sight of my goals, she never did.

INTRODUCTION

The Lariat Pit of the Robinson Brick Company is located in south-central Jefferson County, Colorado (Figure 1). The pit, located on the eastern flank of the east-dipping Dakota Hogback at the edge of the Colorado Front Range, has been operated since the 1960's and supplies high-quality clay for brick manufacturing. A series of small rock-slope failures in the pit wall (all involving less than a few hundred cubic yards of material) prompted a request for an investigation by managers of the Robinson Brick Company; there was concern that a larger, possibly catastrophic failure, might occur. As a result, in the Spring of 1986, a study was begun to determine the potential for slope failure in the west wall of the pit.

Purpose and Scope

The objectives of the study were threefold:

1. To assess the stability of the west wall of the pit in the present configuration;
2. To assess the stability of the pit wall in the proposed final configuration; and

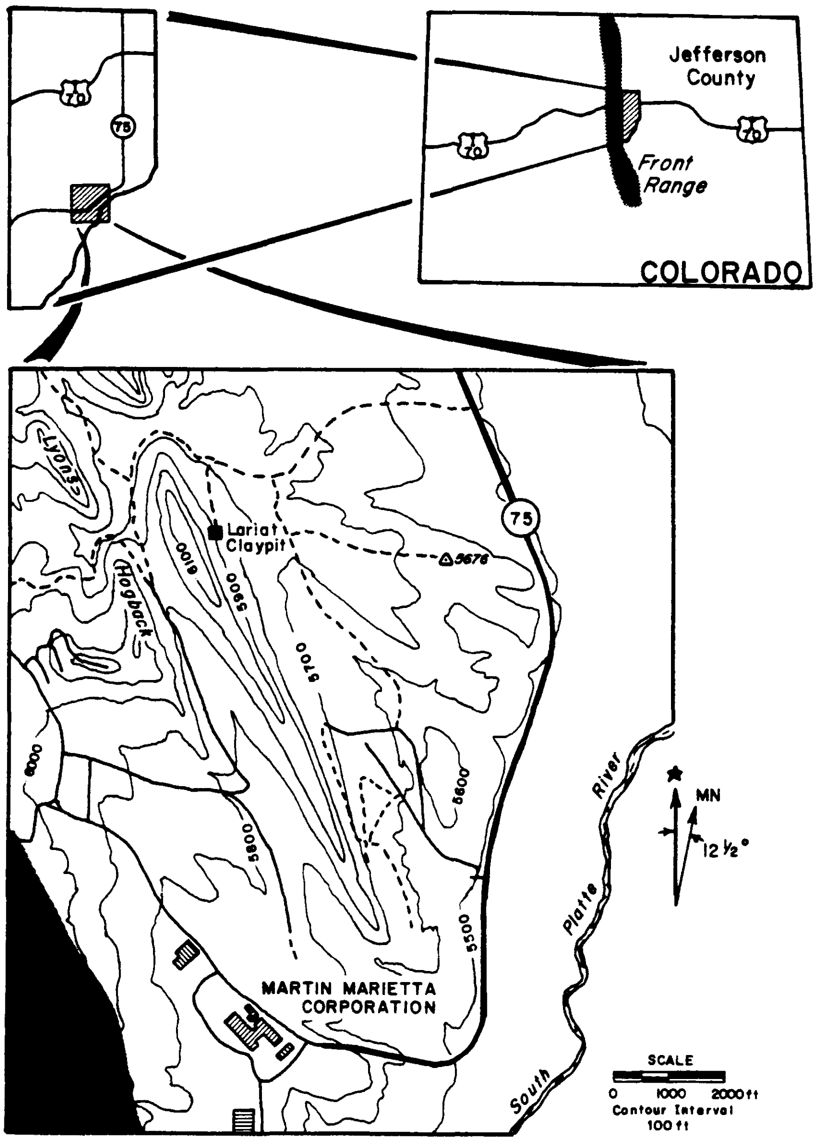

Figure 1. Map showing the location of the Lariat Pit in south-central Jefferson County, Colorado. Significant topographic and cultural features near the site are shown, including the Dakota and Lyons hogbacks, the South Platte River, and the main plant of the Martin Marietta Corporation.

3. To determine the relative degree of influence of various slope parameters on the Factor of Safety.

The results of the study were to include recommendations for simple, cost-effective hazard mitigation.

Field work consisted of the collection of data at the site, including information about fracture orientations, dimensions, and spacing, that were used to determine the likely geometries of potential slope failures, and to estimate properties of the rock mass. The dimensions and geometry of the rock slope were also observed in the field for use in subsequent stability calculations. Samples of the Dakota sandstone were taken in the field, and were analyzed for rock material properties in the Rock Mechanics laboratory of the Colorado School of Mines.

A preliminary slope stability and sensitivity analysis was conducted in the Spring of 1986; stability was assessed using the modified limiting-equilibrium method (Abel, 1983). Further field data were collected in June of 1987, and were used to refine the stability analysis. Computer programs to be used in the final sensitivity analysis were developed in the Fall of 1987; these programs were designed for the technique of parameter variation, and were used to confirm the preliminary results obtained in 1986.

Analytical Methods in the Determination of Slope Stability

Stability analysis of the slope was conducted following the limiting-equilibrium method described by Hoek and Bray (Hoek and Bray, 1981), and incorporating modifications suggested by Abel (1983), which will be discussed in a subsequent section. The analysis essentially consists of a static force balance. The stability of a rock slope is expressed in terms of a factor of safety, which is given as the ratio between those forces tending to retain a given failure block in place, and those forces tending to move the block outward and downward (Figure 2). In the case of a wedge failure, for example, the factor of safety (F.S.) is given as:

$$\text{F.S.} = \frac{\text{(Shear resistance to failure along structure \#1} + \text{Shear resistance to failure along structure \#2)}}{\text{(Component of weight of failure block directed down-plunge of line of intersection)}}$$

In the case of plane failure, only the shear resistance along the single planar failure structure is considered, while the driving force is the component of weight of the block directed down-dip along the failure structure. The factor of safety is determined for a unit width of slope face (one foot), for the assumed worst-case failure conditions, incorporating known or estimated rock properties and slope geometries. The worst-case failure conditions include the following assumptions:

1. The potential failure structure is daylighted just at the toe of the slope; therefore, the greatest possible volume of material can be involved.
2. The strike of the potential failure plane, in the case of plane failure, is exactly parallel to the slope face. The bearing of the line of intersection is exactly normal to the strike of the slope face in the case of wedge failure. These geometries provide the least resistance to the downward and outward movement of a potential failure block.
3. Finally, for the most conservative assumptions (which were not used in this analysis), all joints in the slope are assumed to be continuous; i.e., no intact rock is present in the slope.

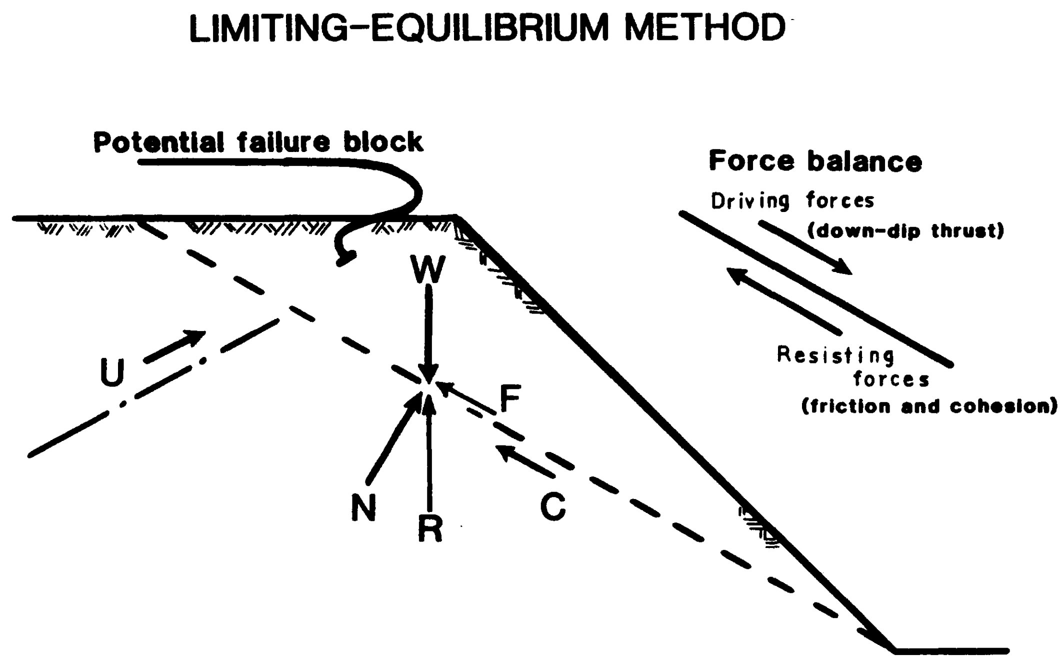

Figure 2. Static force-balance diagram in the limiting-equilibrium method. The principal driving force is the component of weight of the potential failure block directed parallel to the potential failure plane; the principal resisting forces are friction and cohesion. Friction depends on the component of weight of the block directed normal to the potential failure plane; this is reduced by some uplift force, which depends on the water pressure acting across the failure plane.

Mathematically, the determination of factor of safety for a given slope in the case of plane failure can be expressed as:

$$\frac{[(N-U)\tan(\phi_{ror}+i)] \times P_b + C_{ror} \times P_b \times S + [(N-U)\tan(\phi_{int})] \times P_i + C_{int} \times P_i \times S}{T}$$

where

θ = the modal dip of the potential failure plane
i = the irregularity angle at the selected level of confidence
S = length of the potential failure surface

W = the weight of rock above the potential shear plane (per foot of face)
W_u = the weight of any water contained within a failure block (per foot of face)
T = $(W + W_u) \sin \theta$, the total down-dip driving force (per foot of face)

N = $(W + W_u) \cos \theta$, the normal force acting across the potential failure plane (per foot of face)
U = the hydraulic uplift force acting across the potential failure plane (per foot of face)

Φ_{ror} = the rock-on-rock angle of surface friction
C_{ror} = the rock-on-rock cohesion
Φ_{int} = the intact-rock angle of internal friction
C_{int} = the intact-rock cohesion (reduced by some rock-mass strength reduction factor)

P_b = the proportion of broken rock along the potential failure plane
P_i = the proportion of intact rock along the potential failure plane

In the case of wedge failure, the relation is:

$$\frac{\begin{array}{l}[R_1 \times \tan(\phi_{ror} + i_1)] \times P_{b1} + C_{ror} \times P_{b1} \times A_1 \\ + [R_1 \times \tan(\phi_{int})] \times P_{i1} + C_{int} \times P_{i1} \times A_1 \\ + [R_2 \times \tan(\phi_{ror} + i_2)] \times P_{b2} + C_{ror} \times P_{b2} \times A_2 \\ + [R_2 \times \tan(\phi_{int})] \times P_{i2} + C_{int} \times P_{i2} \times A_2\end{array}}{T}$$

where

θ = the plunge angle of the potential failure wedge
i_1 = the irregularity angle along Structure 1 at the selected level of confidence
i_2 = the irregularity angle along Structure 2 at the selected level of confidence
A_1 = area of the potential failure surface along Structure 1 (per foot of face)
A_2 = area of the potential failure surface along Structure 2 (per foot of face)

W = the weight of rock above the potential failure wedge (per foot of face)
W_u = the weight of any water contained within the failure wedge (per foot of face)
T = $(W + W_u) \sin \theta$, the total down-plunge driving force (per foot of face)

R_1 = the normal reaction force acting across Structure 1 (per foot of face)
R_2 = the normal reaction force acting across Structure 2 (per foot of face)

Φ_{ror}= the rock-on-rock angle of surface friction
C_{ror}= the rock-on-rock cohesion
Φ_{int}= the intact-rock angle of internal friction
C_{int}= the intact-rock cohesion (reduced by some rock-mass strength-reduction factor)

P_{b1} = the proportion of broken rock along Structure 1
P_{i1} = the proportion of intact rock along Structure 1
P_{b2} = the proportion of broken rock along Structure 2
P_{i2} = the proportion of intact rock along Structure 2

The following should be noted:

1. Shear resistance to failure along a potential failure surface, or design joint, is increased by the irregularity angle at the selected level of confidence (discussed in a subsequent section).
2. The strength parameters of the rock material - cohesion and friction angles - are weighted by the relative proportions of intact and broken rock estimated to be present along a failure structure (discussed in a subsequent section).

Thus, the following data are required for performing a stability analysis using the Modified Limiting-Equilibrium method:

1. The geometry and characteristics of the rock slope, including slope angles, slope height, types of rock present in the slope, and local hydrology.
2. The statistical distribution of joint orientations present within the rock slope, and the orientations of any faults present in the slope. These data allow estimates to be made of the orientations of design joints, so that potential failure geometries

can be determined.

3. The engineering properties of the rock. These include properties of the rock <u>material</u> (density, cohesion, and friction angles), as well as properties of the rock <u>mass</u> (rock-mass strength-reduction factors, relative proportions of broken and intact rock present within the slope). Rock material properties are determined by laboratory testing of rock samples, while information concerning the spacings and trace lengths of joints within sets allows rock mass properties to be estimated.

Using these data, the limiting-equilibrium equations can be solved analytically; this is conveniently done using a microcomputer (Abel, 1982). Certain of the slope parameters can be changed sequentially to determine the relative effect that each parameter has on the resultant factor of safety; this examination of relative effects constitutes a sensitivity analysis. The technique of parameter variation was simplified by the use of two computer programs (PLANESHR and WEDGESHR) that allow some slope parameters to be varied within specified limits. These routines are based on Abel's (1982) original programs; compiled versions are included as an Appendix to this report.

Previous Work

Lithologic characteristics and distribution of the Dakota Group sandstones are summarized by Leroy and Weimer (1971), MacMillan and Weimer (1976), Waage (1961), and Weimer (1970). Numerous methods for collecting joint data are outlined by Baecher and others (1977), Call (1972), Cruden (1977), Fecker and Rengers (1971), Goodman (1980), Hoek and Pentz (1968), LaPointe and Hudson (1985), Major and others (1977), Piteau (1972), Priest (1985), and Priest and Hudson (1981). A slightly modified detail-line survey was used to collect joint data for this project; the method is standardized by the International Society for Rock Mechanics (International Society for Rock Mechanics, Commission on Standardization of Laboratory and Field Tests, 1978), and is presented clearly by Hoek and Bray (1981).

Methods of conducting joint data analysis are presented by Call (1972), Call and others (1976), Fecker and Rengers (1971), Goodman (1980), Hoek and Bray (1981), Major and others (1977), and Priest (1985). Joint data are often analyzed probabilistically (Baecher and others, 1977; Call, 1972; Cruden, 1977; Goodman, 1980; Hoek and Bray, 1981; Hudson and Priest, 1983; Piteau, 1972; Priest and Hudson, 1981; Warburton, 1980). Several investigators (Baecher and others, 1977; Warburton, 1980) have established that the Poisson distribution is the appropriate model for describing the variation of joint orientations about a central value. A stereoplot of poles to joints can be contoured at selected levels of significance of the Poisson distribution; these contours are then taken as a representation of the irregularity angles associated with the various joint sets present in a slope (Fecker and Rengers, 1971). The irregularity angle, in turn, is a measure of roughness of a discontinuity (Figure 3), and is related to the shear strength of the discontinuity (Patton, 1966).

Numerous qualitative and semi-quantitative methods for determining the relative degree of stability of a rock slope have been proposed (Goodman, 1980; Hoek and Pentz, 1968; Major and others, 1977; Piteau, 1972; Priest, 1985; Warburton, 1980). Most of these methods require only an analysis of the relationships between slope and fracture geometries; some probabilistic models (Major and others, 1977; Serrano and Castillo, 1974; Warburton, 1980) have also been used. The approach adopted in this study, originally proposed by J.F. Abel (1983) of the Colorado School of Mines, is analytical, and incorporates probabilistic analysis as a part of the stereological interpretation of joint data. Abel's (1983) approach is similar to the Limiting-Equilibrium

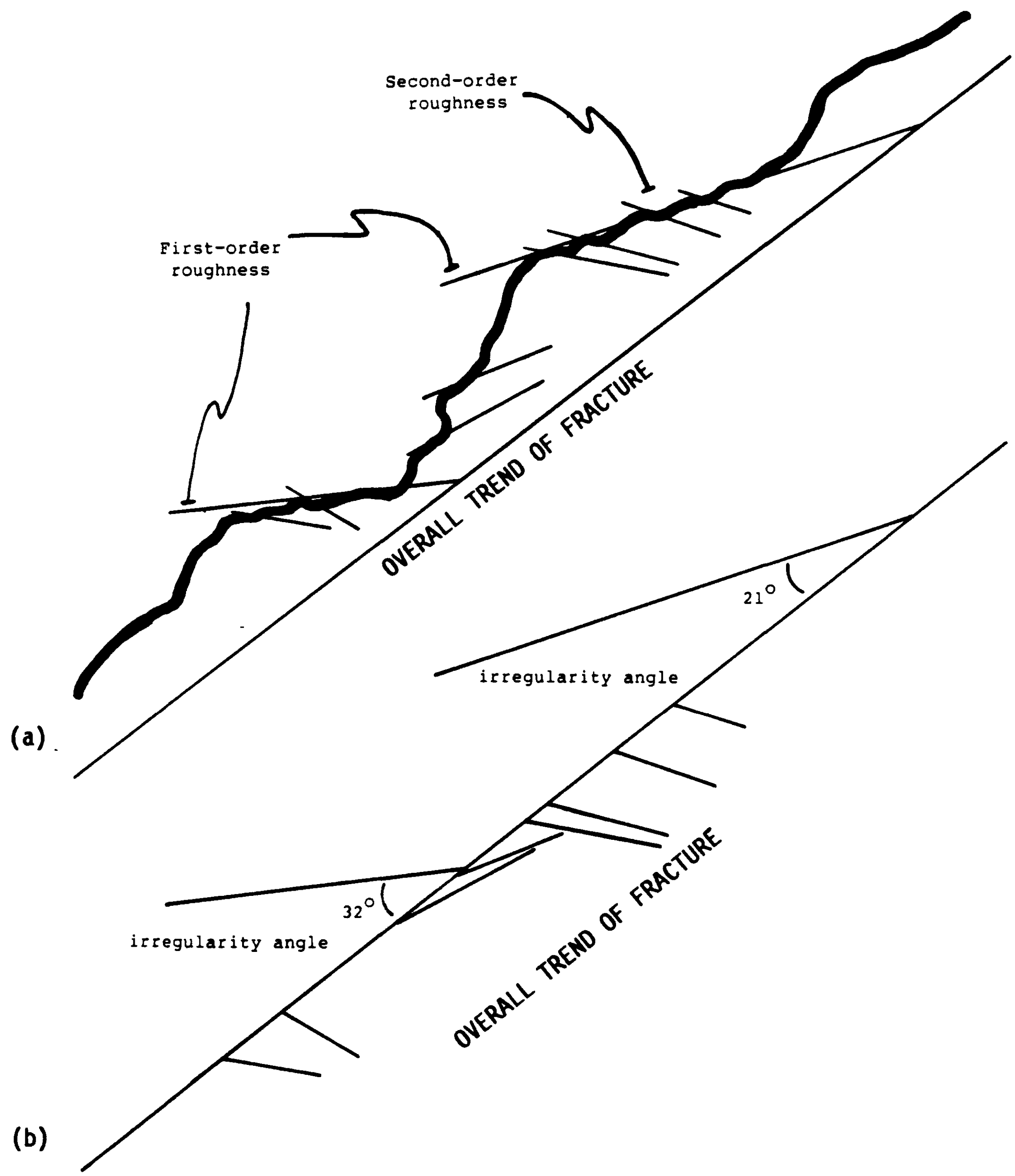

Figure 3. (a) Idealized rough joint. Both first- and second-order irregularities are shown; the accompanying irregularity angles (i-angles) are shown in (b). (After Patton, 1966)

method of Hoek and Bray (Hoek and Bray, 1981); however, the analysis differs significantly in its treatment of rock material strength parameters, rock mass strength parameters, and the possible presence of intact rock in the slope. Therefore, the analytical method used in this study will be referred to as the Modified Limiting-Equilibrium method of Abel.

Sensitivity Analysis

General procedures used to conduct a sensitivity analysis are well established (Dief, 1986, p. 173 ff.). Suppose there exists an arbitrary function F such that

$$F = f(p,q,r)$$

where (p,q,r) are some independent variables. If one of these parameters, p for example, is varied by a small amount, Δp , the solution becomes

$$F^{\circ} = f^{\circ}(p + \Delta p,q,r)$$

For values of Δp that are sufficiently small, this new expression may be approximated as

$$F^{o} = f^{o}(p + \Delta p,q,r) = f^{o}(p,q,r) + \Delta p \times \partial F^{o}/\partial p$$

Note that the new, "perturbed" value of the function depends on the so-called 'sensitivity coefficient' - the first derivative of the function with respect to the varied parameter. This observation suggests the kernel of an analytical approach to sensitivity analysis: parameters can be varied within a narrow range, and the resulting effects on the function noted. The given function is most sensitive to the parameter that induces the steepest slope as the function is varied.

However, the technique of sensitivity analysis as applied to engineered structures, particularly in the fields of earth mechanics, is not well documented in the literature. Hoek and Londe (1974) note that the information that is most useful to an engineer is information that indicates the response of an excavation to changes in significant parameters. This is in fact the purpose of a sensitivity analysis: to determine which parameters are most significant to a design, and to predict the effects of changes in those parameters on the factor of safety of the engineered structure (in this case, a rock slope). The basic principles of parameter variation are outlined by Hoek and Bray (1981, p. 196 ff.). However, these authors examine the effects of varying only one parameter: height of water in a headwall fracture. Such an exposition of the technique of sensitivity analysis must be regarded as incomplete.

BACKGROUND INFORMATION

Location and Access

The Lariat Pit is located in south-central Jefferson County, Colorado, in NW 1/4 6S-69W-22, 6th P.M. (Figure 1). The pit is 9 miles southwest of the center of Littleton, Colorado; access is via Colorado Highway 75. The site is due north of the main plant of the Martin Marietta Company, at Waterton. Martin Marietta property abuts the pit site to the west and south; land to the northeast and east is the property of the State of Colorado (the Chatfield State Recreation Area). The site is bounded to the north by property owned by the Denver Tech Center.

The pit site is located on the boundary between the High Plains and the Southern Rocky Mountains physiographic provinces (Figures 4 and 5). In addition to the nearby Colorado Front Range, the most distinctive topographic feature near the site is the Dakota hogback, a sharp-crested, north-south-trending linear ridge that extends nearly unbroken for approximately 17 miles through Jefferson County. Local relief of about 450 feet is present between the crest and base of the hogback near the Lariat Pit (Figure 4).

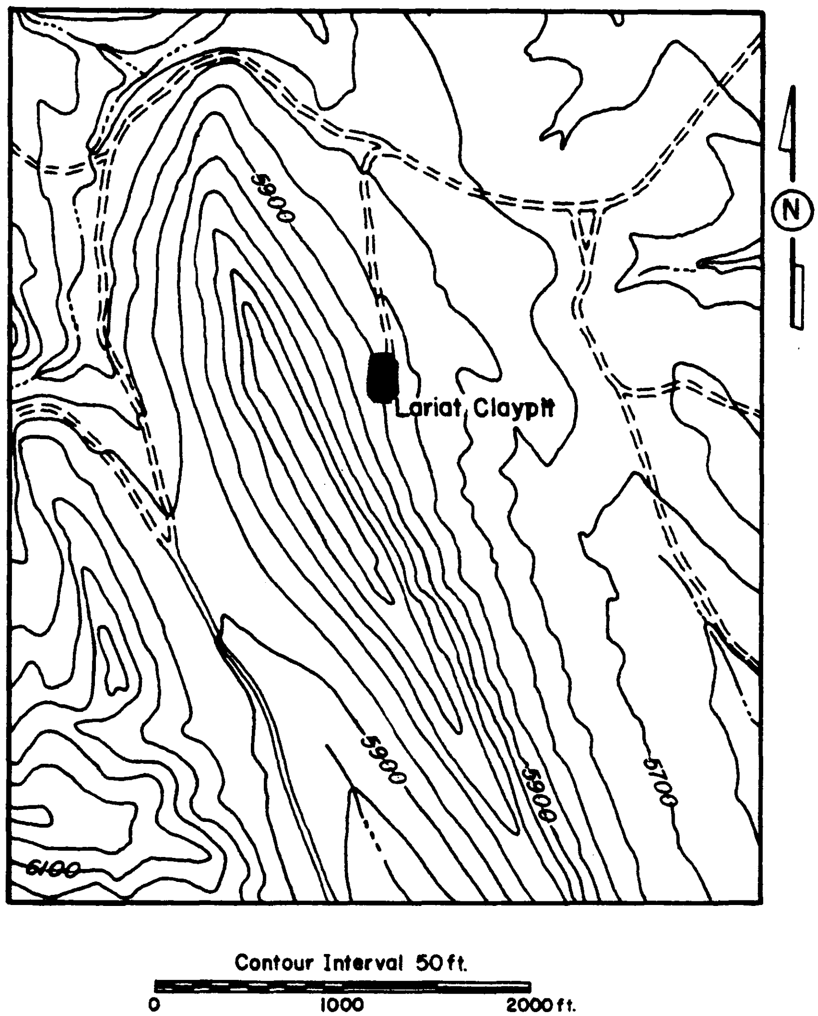

Figure 4. Topography of the Dakota hogback in the vicinity of the Lariat Pit (after U.S. Geological Survey 7.5' Quadrangle map, 1971).

Figure 5. A view of the Dakota hogback, looking northwest. The hogback is the linear ridge in the middle ground; it extends as a nearly unbroken topographic eminence for about 15 miles north of this point. The main plant of the Martin Marietta Corporation is located in the water gap at left center (Waterton gap); the South Platte River flows onto the Great Plains from the Rocky Mountains through this gap.

The Lariat Pit is beyond the horizon to the north (center of the photograph).

General Geology

The Dakota hogback is composed of east-dipping strata of Mesozoic age (Figure 6), which are predominantly marine or marine shoreface in origin (Weimer, 1970; MacMillan and Weimer, 1976; Waage, 1961). A sandstone of the Dakota Group forms the crest of the hogback. Rocks of the Dakota Group are widely distributed throughout the western United States; in central Colorado, Dakota Group rocks are commonly sandstones of Lower Cretaceous age. The unit of interest at the study site is the "J" interval of the South Platte Formation, Dakota Group (Weimer, 1970; Leroy and Weimer, 1971), but will be referred to for simplicity as 'Dakota sandstone' (Figure 7). The total thickness of Dakota Group rocks near the site (taken from a measured section, Section 14 of Waage; Waage, 1961) is about 355 feet (South Platte Formation, 283 feet; Lytle Formation, 72 feet).

The Dakota sandstone at the study site consists of gray to white, well-sorted, rounded to subrounded, medium- to fine-grained quartz sand in an extensively bioturbated matrix of silty clay. Little small-scale stratification or segregation of grains is apparent in petrographic analysis; this apparent homogeneity is confirmed through mechanical testing of rock samples (in a later section). The clays,

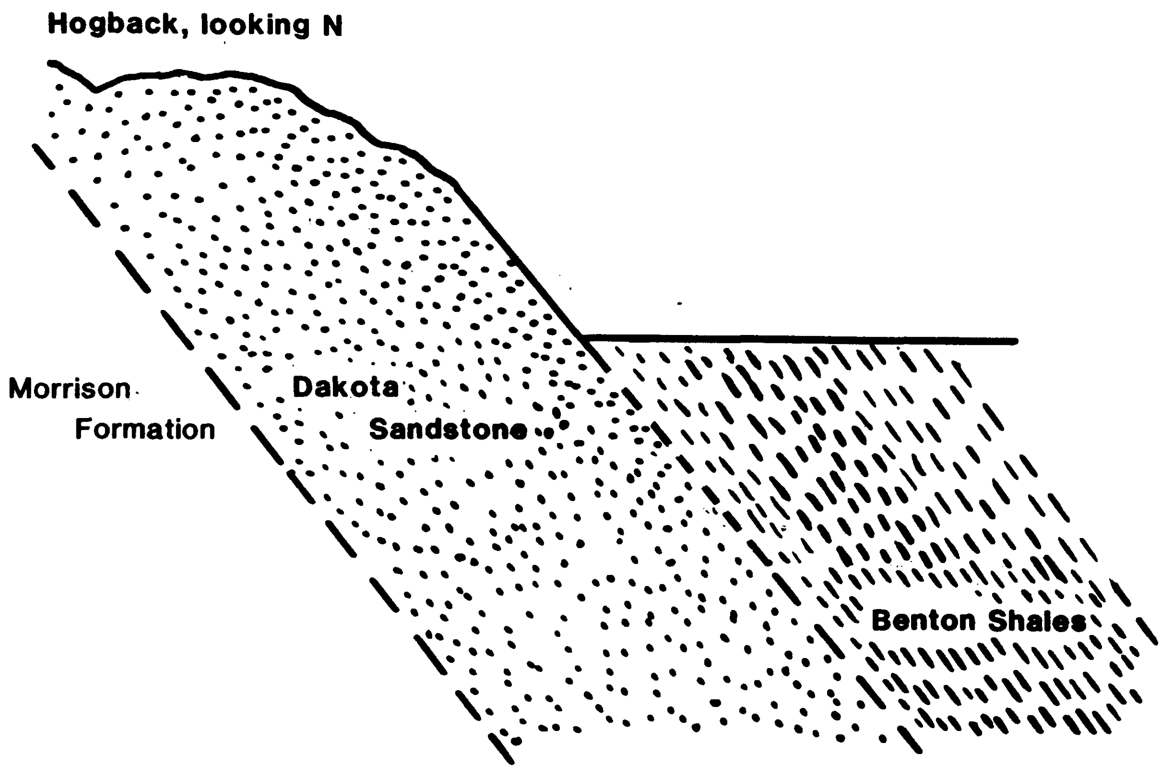

Figure 6. Generalized cross section through the Dakota hogback, looking northwest along the ridge. The general stratigraphic and structural relationships present at the site are shown. Dip of bedding in the Dakota sandstone at the site varies between 52° and 59° NE. The contact with Benton Group shales occurs near the slope break, at the base of the eastern flank of the hogback.

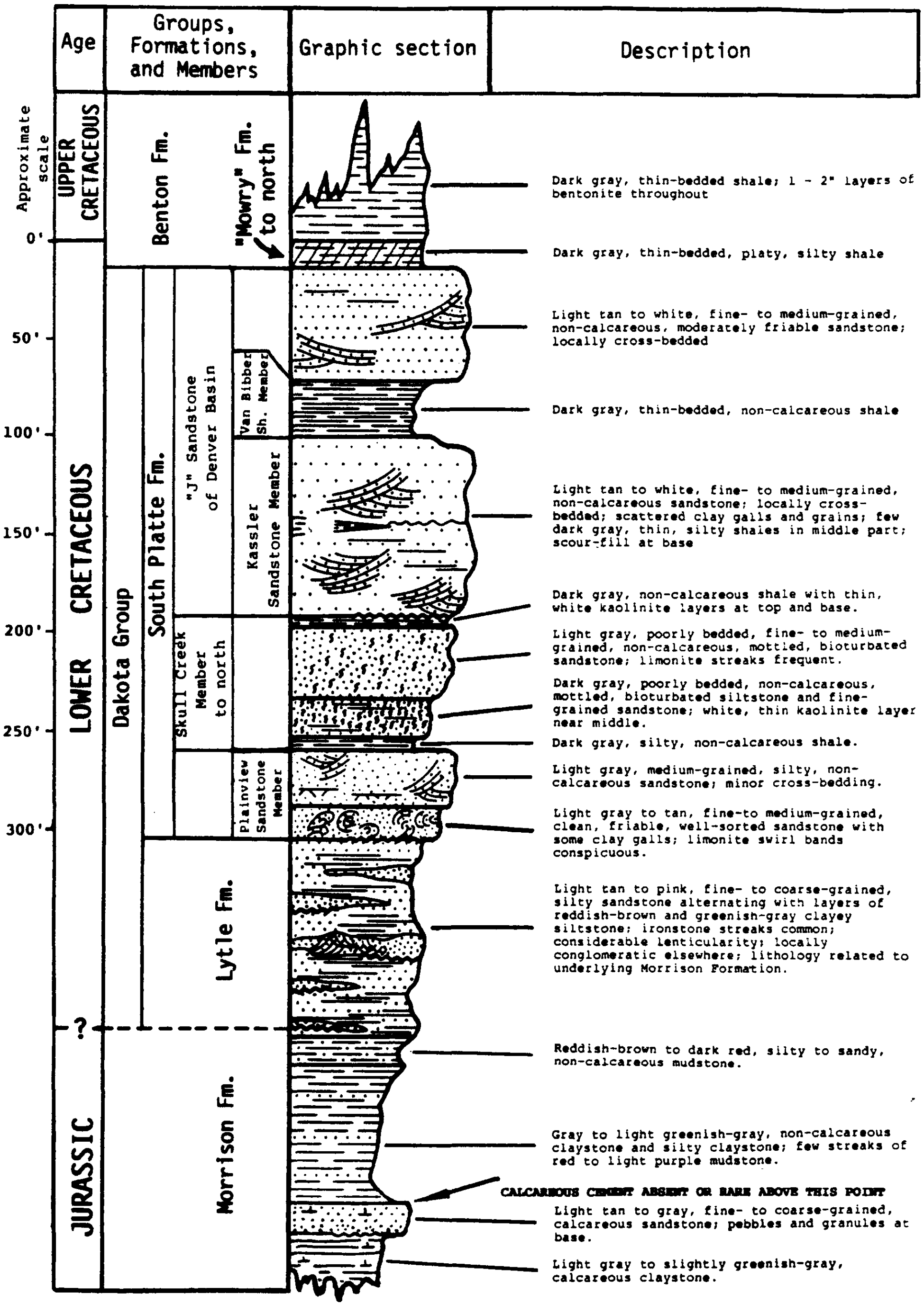

Figure 7. Part of the stratigraphic column of the Front Range of Colorado, showing Mesozoic units composing the Dakota hogback (after Leroy and Weimer, 1971, Fig. 3).

viewed in thin section, are primarily illite, and compose approximately 20 - 25% of the rock by volume; therefore, the Dakota sandstone at the site is characterized as a wacke. Minor siliceous cement is also present. The Dakota sandstone at the site tends to be friable, due to the high volume of clay.

The contact of the Dakota sandstone with the overlying shales of the Benton Group is near the base of the hogback, and along the slope break northwest along strike. The Benton Group shales consist of poorly indurated, gray to black silty claystones. The clay material consists of illite with some bentonite; the high quality of the clays makes this shale an ideal raw material in brick manufacture.

In the central part of the Colorado Front Range, the Dakota hogback is an erosional remnant of the faulted eastern limb of a large monoclinal drape fold that was formed during Laramide time (Matthews, 1976). The steep easterly dip of the sedimentary units which form the hogback (Figures 5, 6) is an expression of this locally antiformal structure. The strike of bedding in the pit wall is generally N24W, while bedding dip varies from 52°NE to 59°NE. Strike of the beds is parallel to subparallel to the trend of the hogback, while the natural slope angle is generally 5° - 15° less than the dip of the beds.

The hogback is dissected by numerous, small en echelon faults that are subparallel to the trend of a major north-south trending fault complex. Rocks in the hogback tend to be extensively jointed as a consequence of intensive regional deformation during Laramide and later times.

Mining Conditions

A thin veneer of alluvial-colluvial material (less than 5 feet thick) covers the Benton Shale east of the hogback; therefore, shallow pit extraction methods have been utilized exclusively during mining. Conditions at the study site are nearly ideal for pit mining: a pass with a single-tooth ripper is sufficient to break up the shale bedrock to a depth of about 2 feet; the broken shale is then scraped from the steeply dipping bedding surface of the underlying Dakota sandstone, which is left standing as a dip-slope pit wall (Figure 8). Broken shale remains in spoil piles at the base of the slope for later loading into trucks (Figure 9). In extending or deepening the mine, a bulldozer will make a pass northwest or southeast along strike at the contact between the Dakota sandstone and Benton shale. Frequently, the bulldozer will miss the contact by several feet during a stripping pass. As noted, Dakota sandstones tend to be fri-

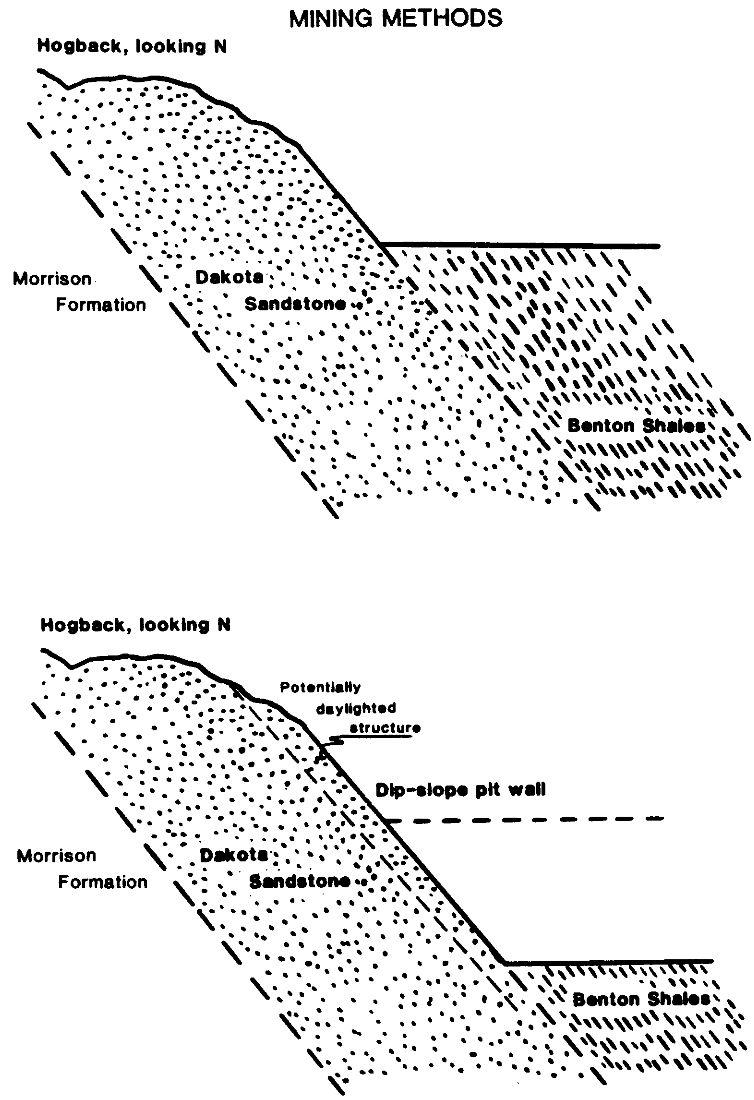

Figure 8. Mining conditions at the Lariat Pit. The present face height is about 100 feet (upper diagram); as mining proceeds, a bulldozer removes material from the contact between the Dakota sandstone and the overlying Benton shales. The pit is deepened and the Dakota sandstone remains as a dip-slope pit wall (lower diagram).

Figure 9. Panoramic view of the Lariat Pit, looking west toward the Dakota hogback. The white pit-wall face is Dakota sandstone. Broken Benton shale has been placed in a spoil pile below the slope. The pit is about 1000 feet in length, and is approximately parallel to the strike of the Dakota beds.

able in outcrop, and can be easily removed by a dozer blade; thus the dip-slope pit wall is often undercut, and the effective slope angle increased (Figure 10). The presence of several discontinuous benches in the west wall (Figure 9) indicates the frequency of this occurrence.

Currently, the Lariat Pit is about 1000 feet in length, and is approximately parallel to the strike of the Dakota sandstone (Figure 9). The vertical face height (toe to crown of the slope) varies with location in the pit, and is greatest (about 100 feet) near the central part of the slope (Figure 11). Recent excavation (summer of 1987) has increased the total face height in one location to about 130 feet.

The northwest end of the pit wall is penetrated by an adit, which was driven perpendicularly to strike (Figure 12). This adit, which extends several hundred feet into the hogback, was not driven during the present mining operation, but was probably used to extract refractory fireclay from a lower stratigraphic interval of the Dakota sandstone (Waage, 1961).

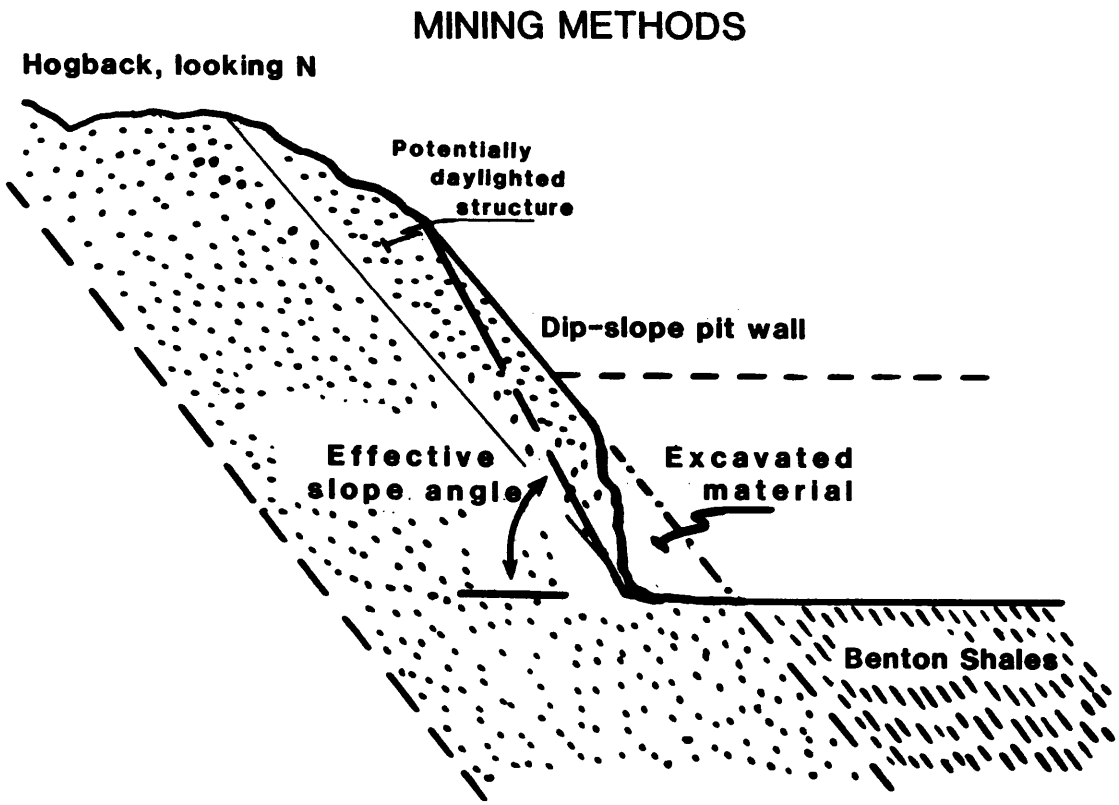

Figure 10. Diagram showing the increase in effective slope angle due to undercutting of the slope toe. The effective slope angle is the angle subtended by a ray constructed from the slope crest to the slope toe, ignoring intervening masses of material. The material lying above this ray doesn't add to the strength of the rock mass in the pit wall, and is not considered in stability calculations.

Figure 11. View along the wall of the Lariat Pit, looking southeast. Note the steep angle of the dip-slope wall (here about 52^{o}); the total face height of the wall at this point is about 130 feet. Recent mining activity has left depressions at the slope toe. Standing water is present in the excavations several days after a heavy local rainfall; this gives some indication of the impermeable nature of the Benton shales, located at the base of the slope.

Figure 12. Adit at the northwest end of the pit wall. The adit was used to investigate the hydrology and three-dimensional geometry of the rock slope.

FIELD INVESTIGATIONS

Fracturing in Rock Masses

The stability of rock slopes depends largely upon the presence and nature of discontinuities in the rock mass (Call, 1972; Goodman, 1980; Hoek and Bray, 1981; Major and others, 1977; Terzaghi, 1962). A principle objective of field investigations is to define the characteristics of these discontinuities, including: (1) modal orientations of joint sets; (2) joint spacing; (3) joint trace length; (4) presence or absence of faults; and (5) fault orientations (Hoek and Bray, 1981; ISRM, 1978). Depending on the method of analysis to be used, additional data to be collected can include joint roughness, joint aperture, joint continuity and waviness, and the presence and character of infilling material (Goodman, 1980; ISRM, 1978). The dimensions of the slope, and the presence or absence of water in the slope should also be noted (Hoek and Bray, 1981). If water is present, the potentiometric surface should be determined or estimated, as the stability of a given slope is dependent on the degree of saturation (Goodman, 1980; Hoek and Bray, 1981; Major and others, 1977; Piteau, 1972).

Data Collection Methods

Detail line surveys (ISRM, 1978) were used to collect fracture data. To conduct a detail line survey, the spacing, trace lengths, and orientations of all joints located along a randomly-located line that is located parallel to the slope face are measured (Figures 13 & 14). The modal orientations of joint sets can then be determined (Hoek and Bray, 1981; Hoek and Brown, 1980; Piteau, 1972; Priest, 1985). Separate structural regions which may be present in the rock slope are also identified (Piteau, 1972).

Several criteria must be observed if detail-line data are to be a statistically valid representation of the population of discontinuities in the rock mass:

1. The detail lines should be randomly located (Hoek and Bray, 1981; ISRM, 1978).
2. 60 to 80 joints should be sampled per 'locality' (Müller, 1963; Pincus, 1951).
3. The orientation of the slope face (a two-dimensional surface) may conceal the presence of joint sets that are subparallel to the slope (Terzaghi, 1965). To obtain a three-dimensional representation of rock-mass discontinuities, data should be collected from locations other

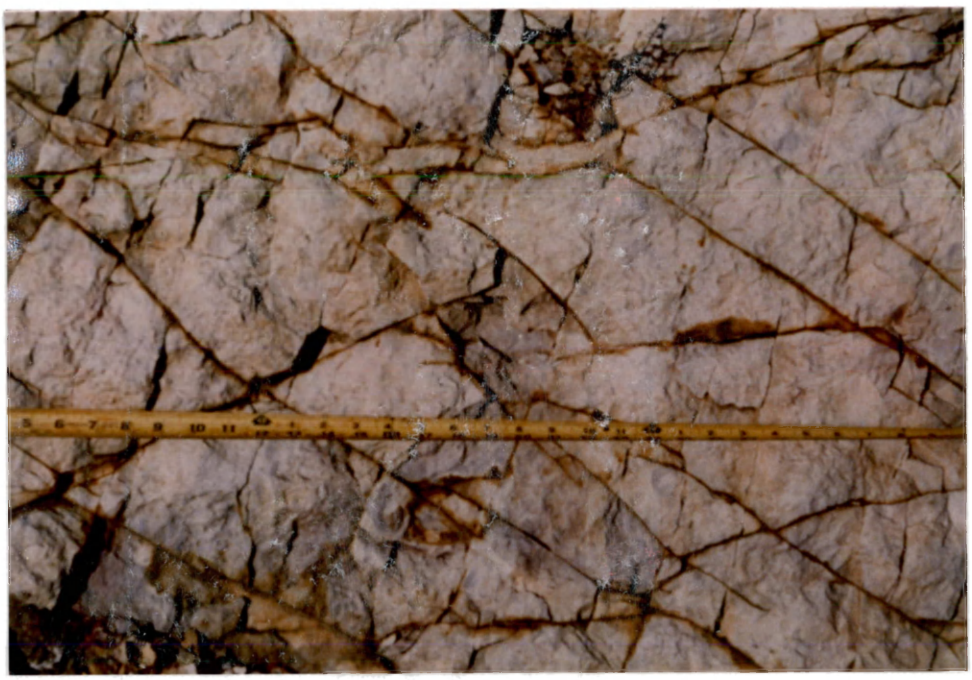

Figure 13. Close-up of a detail line. The tape is stretched along a marked section of the slope face, and the point at which each fracture intersects the tape is noted. The orientation, trace length, fracture aperture, and fracture filling (if present) of each joint are also noted.

Figure 14. Fracture orientation is measured at the point where it intersects the detail line.

than exposed slope faces. These data can be collected around noses in the slope, from oriented core, or from **tunnels or adits.**)

In addition, Cruden (1977) recommends that trace lengths of joints be estimated using that part of the joint trace that extends to one side of a given detail line.

Fracture and Slope Data

Eight detail lines were constructed at various locations on and within the slope; the attitude, orientation, and height of the slope were noted at each location. Overall slope height is about 100 feet, on average; the slope angle is generally 51°, and a back-slope angle of 15° is generally present. These values were used in subsequent calculations. Joint data, consisting of 290 data points were measured (Appendix 1). Five detail lines (Lines 1 and 2; Lines A and B; and Line B1) were constructed on the slope face. Two detail lines (Lines C and B2) were constructed inside the adit at the northwest end of the pit. One detail line (Line 3) was constructed on the flank of a rock nose, located in the south-central part of the pit. The three-dimensional construction of detail lines in the slope ensured that the total joint population was sampled adequately.

The Modified Limiting-Equilibrium method of analysis (Abel, 1983), used to assess the stability of the rock slope, requires information about the following joint properties:

1. Modal orientations of joint sets
2. Irregularity angles of joint sets
3. Spacing of joints within sets
4. Joint trace lengths
5. Presence and orientations of faults

These data were noted for all detail lines. Most joints in the slope were open (unfilled), and aperture spacing was approximately .04 inches (Appendix I).

Inspection of the pit wall revealed some small en echelon faults, spaced at intervals of about 60 to 80 feet. These faults strike in a northeasterly direction, and are all steeply-dipping (Table 1). The slope hydrology was assessed by examining the adit in the northwest end of the pit. The inspection revealed that no water was present within the slope, and no visible hydrologic effects, such as seeps, significant alteration of minerals, or discoloration of Dakota wall rock, were observed. This indicated that the slope usually is dry; accordingly, the effects of water were not considered in subsequent stability analyses.

Finally, samples of the Dakota sandstone were collected for later laboratory analysis.

Table 1. Fault data.

FAULT	ORIENTATION
F1	N41E/82NW
F2	N29E/73NW
F3	N29E/77SE

Note: The orientations of three small en echelon faults visible in the slope face of the Lariat Pit are listed. Displacement for each fault is less than two feet. Faults are numbered consecutively from south to north, with fault F1 being the southernmost. Fault traces are defined by slightly displaced bedding, and by fracture zones several inches in width, filled with hematite-stained gouge. Apparent dip of fault planes, as seen in the pit wall, shallows with depth.

STEREOGRAPHIC ANALYSIS

Once field data have been collected, a stability analysis proceeds in three general stages: (1) identification of potential failure geometries; (2) estimation of physical characteristics of the slope and slope material; and (3) construction of a static force balance, based on (1) and (2). Joint orientations for each detail line are first plotted on a Schmidt equal-area stereonet, as poles to joint planes; data are then filtered by plotting poles as poles per 1% of stereonet area (Call and others, 1976; Priest, 1985). This is facilitated by a computer program by Call and others (1970); a printer-definition routine allowed the stereonets to be printed on a standard-width line printer (Appendix 1).

Joint Orientations and Identification of Structural Regions

A rock mass is by nature both heterogeneous and anisotropic; however, for analytical purposes it is necessary to divide the rock mass or rock slope into parts within which jointing characteristics are statistically similar. The stability conditions of each part of the rock slope can then be assessed separately. The smaller parts within which joint characteristics are similar are called structural regions; the systematic sets of joints occurring within these structural regions are called design joints (Piteau, 1972). Designation of a structural region is made exclusively on the basis of similarities of design joint orientations (Piteau, 1972, p. 309 ff.).

The orientation data for all detail lines in this study were plotted and compared. The fractures represented in all eight detail lines were assigned to a single structural region, because all design joints were represented in every detail line, and all showed similar orientations. Accordingly, data from all the detail lines were combined, and fracture orientations were plotted as a single stereonet to represent the fracture geometries of a single structural region (Figure 15).

Contouring of Plotted Data

Plotted joint data are contoured to show the distribution of joint orientations about a mean (or modal) value (Call, 1972; Call and others, 1976; Goodman, 1980; Hoek and Bray, 1981; Priest, 1985). Contouring intervals based on the critical values of some population distribution at selected levels of significance are used for statistical

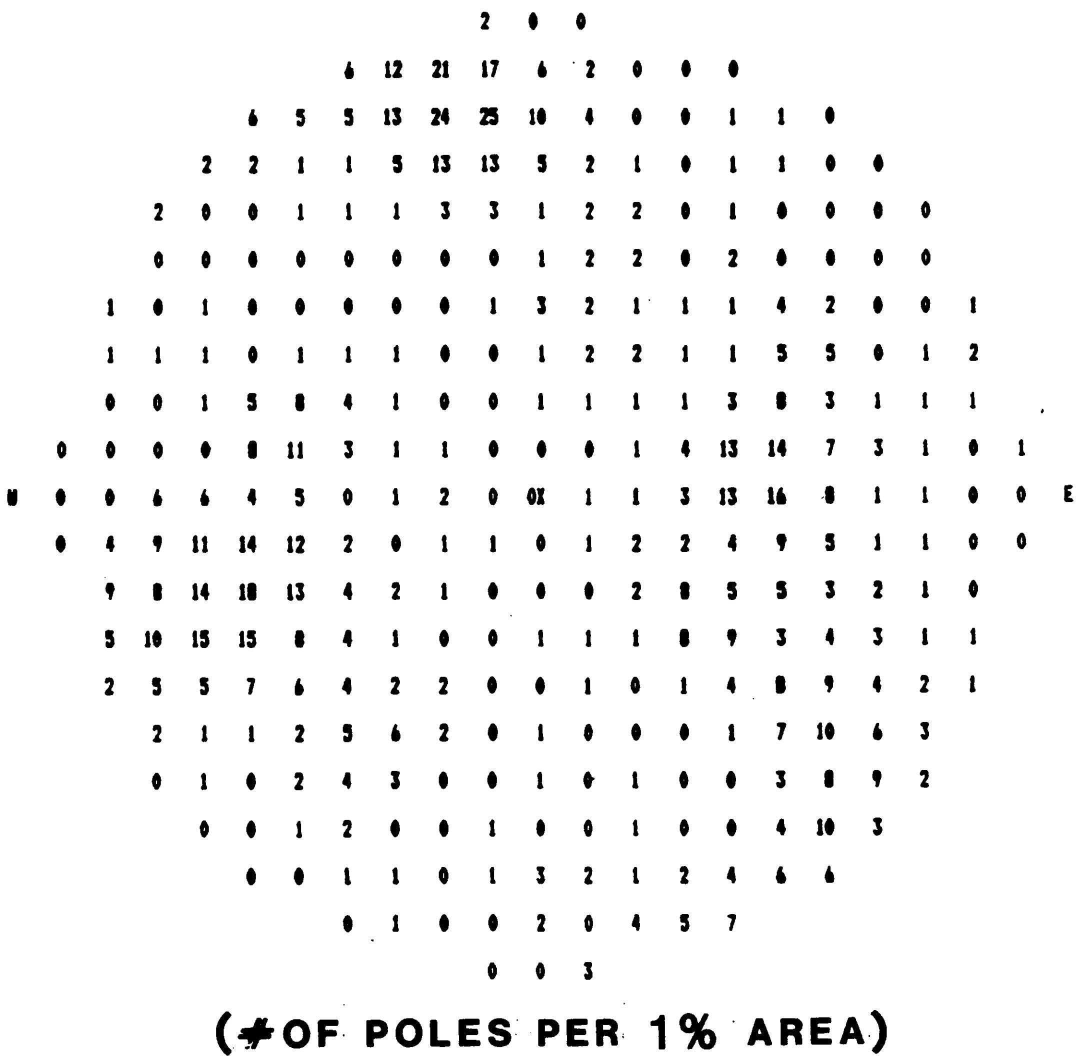

Figure 15. Stereoplot of poles to joints for the combined joint orientation data of all detail lines. The stereonet used by the program is a Schmidt equal-area net (lower hemisphere); numbers represent raw poles per 1% of the stereonet area. Poles were plotted using the SCHMOD program (Call and others, 1970); output was produced on a line printer using a printer redefiniton routine.

accuracy. The joint sets of the single structural region were contoured manually at the 80%, 98%, and 99.9% levels of significance, based on the critical values of the Poisson distribution (Figure 16). The countoured data indicate the level of confidence that an individual joint, having a given orientation, is a member of the indicated joint set. The probable orientations of the principal systems of fractures within a slope can now be determined from the contoured data (Call, 1972; Hoek and Bray, 1981; Priest, 1985).

The relative degree of roughness of the joints can also be estimated from a contoured polar plot (Figure 17). The irregularity angle for a joint set, at some level of confidence, is the up-dip angular separation between the modal orientation of the design joint, and the contour at that confidence level (Fecker and Rengers, 1971; Call, 1972).

Five principal sets of joints were identified at a level of significance of 99.9%. The orientations of these joint sets (strike/dip) are (1) N01W/38SW; (2) N40E/63NW; (3) N77E/66SE; (4) N18W/51NE; and (5) N11E/41SE. In addition, other joint-set characteristics (irregularity angles, minimum and average fracture spacing, maximum fracture trace length) were determined (Table 2).

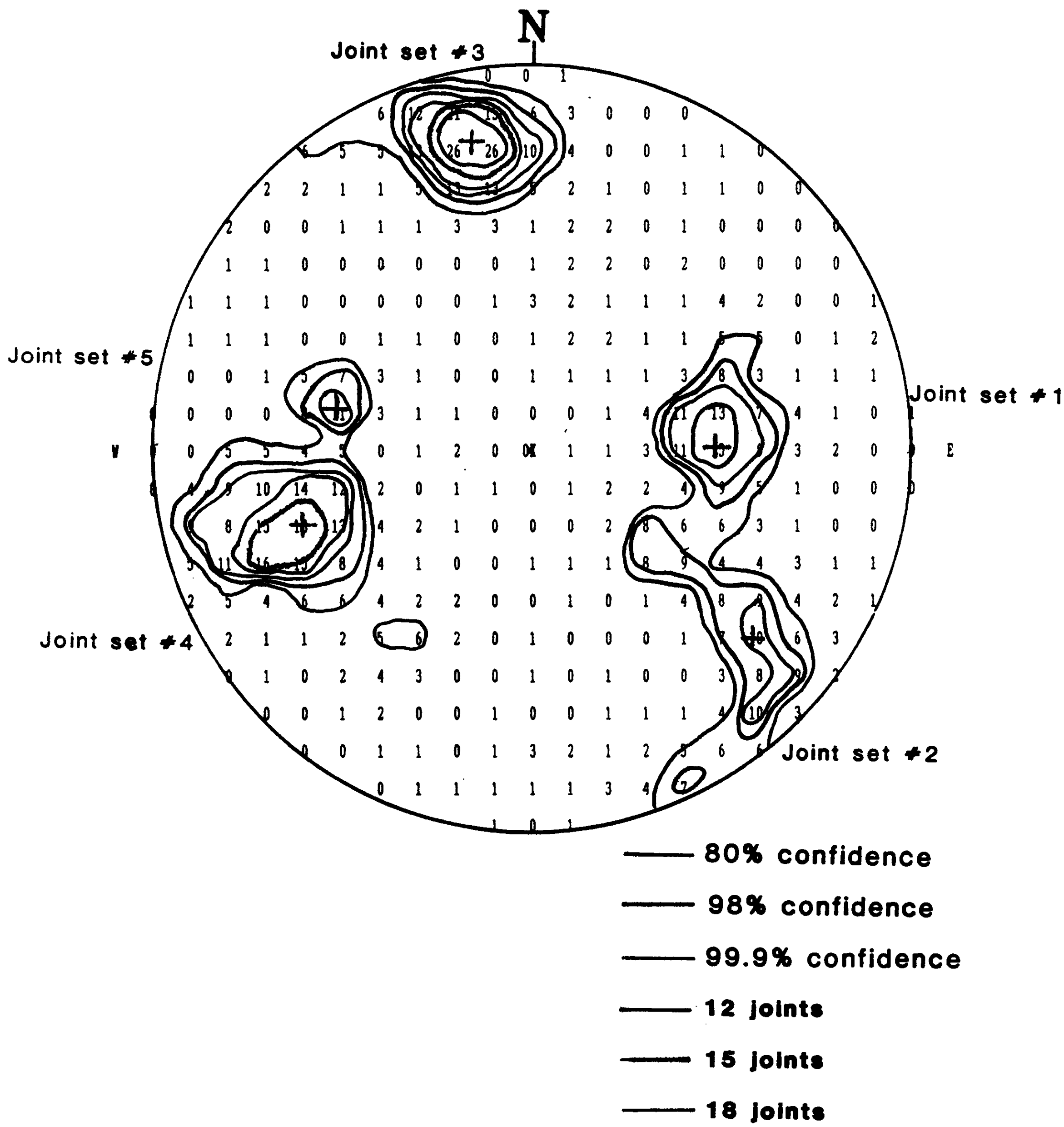

Figure 16. Contoured stereoplot of poles to joints for all detail line data. Contour intervals are based on the critical values of the Poisson distribution at the 80%, 98%, and 99.9% levels of significance. Five joint sets can be identified at the 99.9% level of significance.

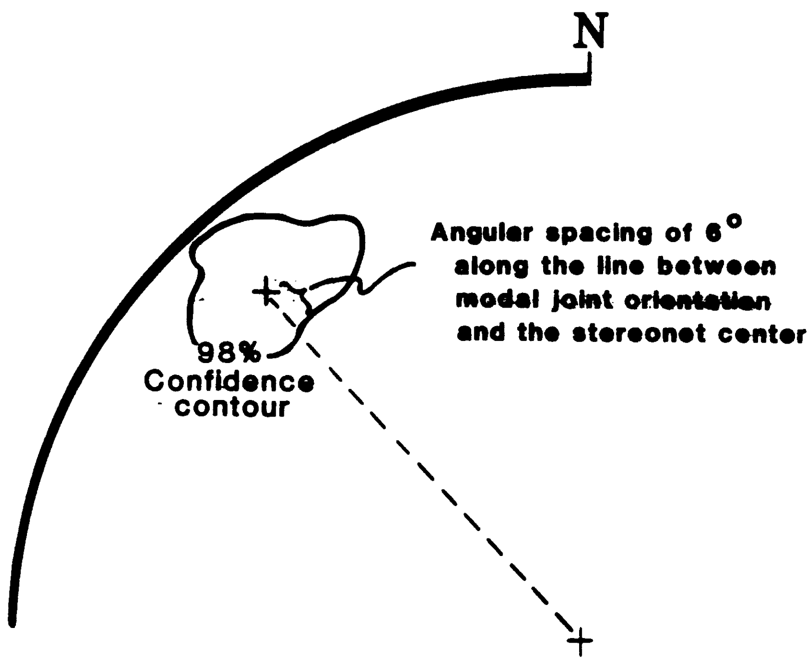

Figure 17. Enlarged part of a contoured stereoplot, showing an irregularity angle of 6° at the 98% level of confidence. The value of the irregularity angle is determined by measuring the angular separation between the modal value of orientation of the joint set (the contour peak) and the 98% confidence contour, along a line between the modal orientation value and the origin of the stereonet.

Table 2. Characteristics of the five joint sets present in the west wall of the Lariat Pit.

	Modal orientation (degrees)	Irregularity angles (degrees)		
		80% conf	98% conf	99.9% conf
Joint Set 1	N01W/38SW	14	12	11
Joint Set 2	N40E/63NW	18	13	4
Joint Set 3	N77E/66SE	17	15	14
Joint Set 4	N18W/52NE	17	14	12
Joint Set 5	N11E/41SE	7	5	3

	Average joint spacing (feet)	Minimum joint spacing (feet)	Maximum trace length (feet)
Joint Set 1	1.1	0.1	40
Joint Set 2	0.9	0.1	40
Joint Set 3	1.0	0.1	30
Joint Set 4	0.5	0.24	10
Joint Set 5	0.5	0.1	40

Note: Average and minimum joint spacings are the adjusted (true) joint spacings, rather than the apparent joint spacings recorded in a detail-line survey.

Potential Failure Geometries

A kinematic analysis was used to determine which joint sets, or combinations of fracture sets, were capable of producing slope failure. Three general failure modes are possible in slopes of hard intact rock: plane failure, involving a single discontinuity set; wedge failure, involving two discontinuitiy sets; and toppling failure, usually involving detached blocks. Methods commonly used to conduct a kinematic analysis are discussed by Call (1972), Goodman (1980), Hoek and Bray (1981), Major and others (1977), and Priest (1985).

Joint Set 4 is capable of causing a potential plane failure in the slope face. The strike of Joint Set 5 is 35° out of parallel with the slope orientation; this difference is sufficient to remove this joint set from consideration as a potential plane-failure geometry. The remaining joint sets do not daylight out of the slope, and are not a problem.

A non-parametric analysis was conducted, after Abel (1983), to determine the least stable potential failure wedge. This analysis showed that the wedge formed by Joint Set 4 and Fault 3 has the greatest likelihood of failure. The bearing and plunge of the line of intersection of the two planes is N42E/46NE, and the wedge dihedral angle is 132 degrees. A conceptual stereoplot (Figure 18) shows the pertinent characteristics of the slope and the two potential failure geometries.

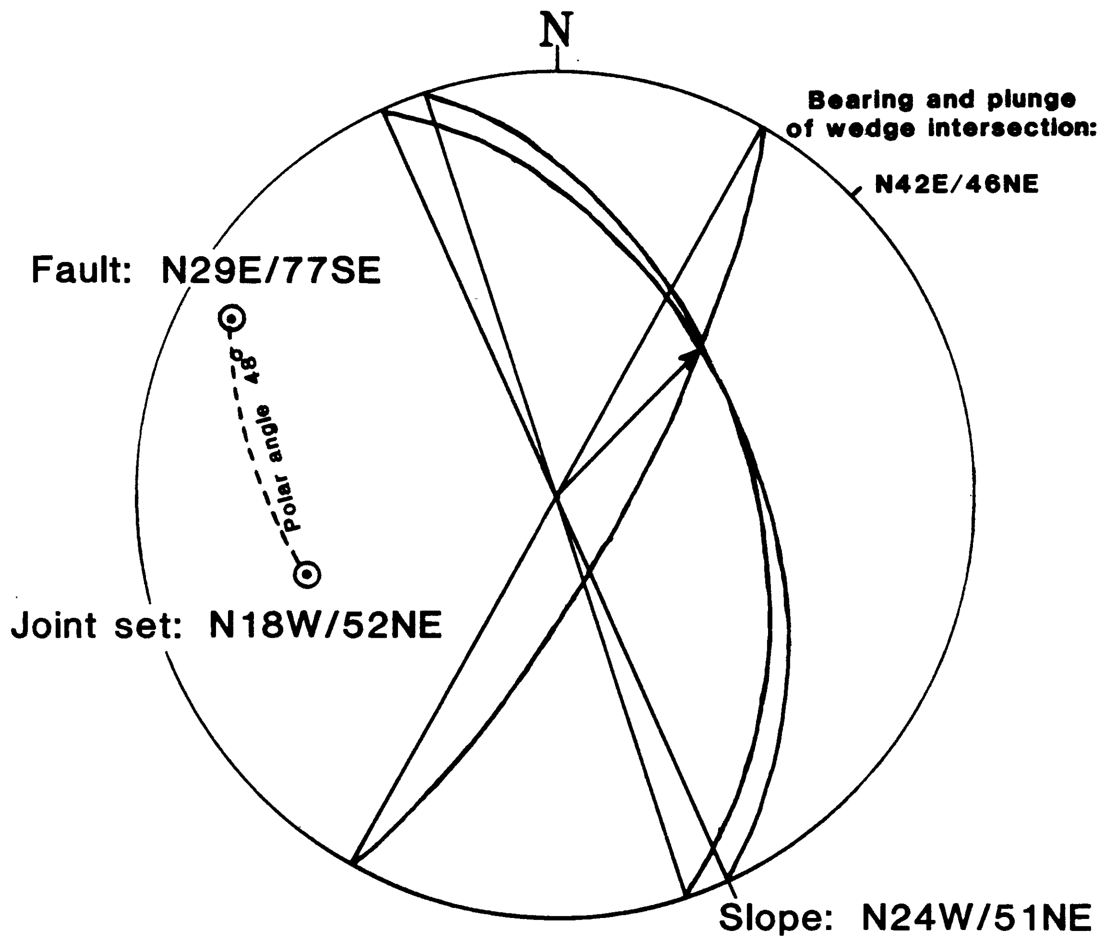

Figure 18. Conceptual stereoplot showing the geometric relationships between the rock slope and the two potential failure geometries. The potential plane failure involves Joint Set 4, while the potential wedge failure involves Joint Set 4 and Fault 3. The line of intersection of the failure wedge plunges out of the wall at an angle of 46° and bears N42E. The wedge dihedral angle is 132°.

ENGINEERING PROPERTIES OF THE DAKOTA SANDSTONE

Rock as an Engineering Material

It was necessary to determine the engineering properties of the design material, in this case the Dakota sandstone, prior to commencing a stability analysis. Two general classes of engineering properties were considered: rock material properties and rock mass properties. Rock material properties are those physical characteristics of the rock that can adequately describe the response of the rock to an applied stress, if it is considered to be an ideal engineering material. These properties include material density, cohesion, and friction angles. Rock mass properties are those physical characteristics which must be considered in describing a rock mass as a non-ideal, inhomogeneous geologic material. Such properties include an estimate of the relative proportions of broken and intact material in the rock mass, and a factor for reducing the apparent strength of the material; these are used to modify the idealized description of the engineering properties of rock.

Rock Material Properties

Rock material properties were determined by laboratory testing of samples of the Dakota sandstone. Two-inch (NX) cores were taken from sandstone blocks that had been collected in the field. The density of the rock was determined from the cores, and a series of uniaxial, triaxial and direct-shear tests were then performed.

Testing the rock in uniaxial and triaxial compression will yield an estimate of the characteristic response of intact rock material to an applied stress (ASTM,1986; Goodman, 1980; Hendron, 1968; Hoek, 1968; Hoek and Bray, 1981; ISRM, 1981). Testing was conducted in the Rock Mechanics Laboratory under the direction of Dr. John F. Abel; tests were performed using standardized methods (ASTM 1986; ISRM 1981); test results are shown in Appendix 2. Results of all tests were corrected to the 'idealized' 2:1 length-to-diameter ratio to adjust for end effects, as recommended by ASTM (ASTM, 1986).

Test results were plotted as failure strength vs. confining stress (Figure 19), and gave a typical linear Mohr-Coulomb failure envelope. Note that this linear approximation of intact-rock strength is one of the significant departures of Abel's Modified Limiting-Equilibrium

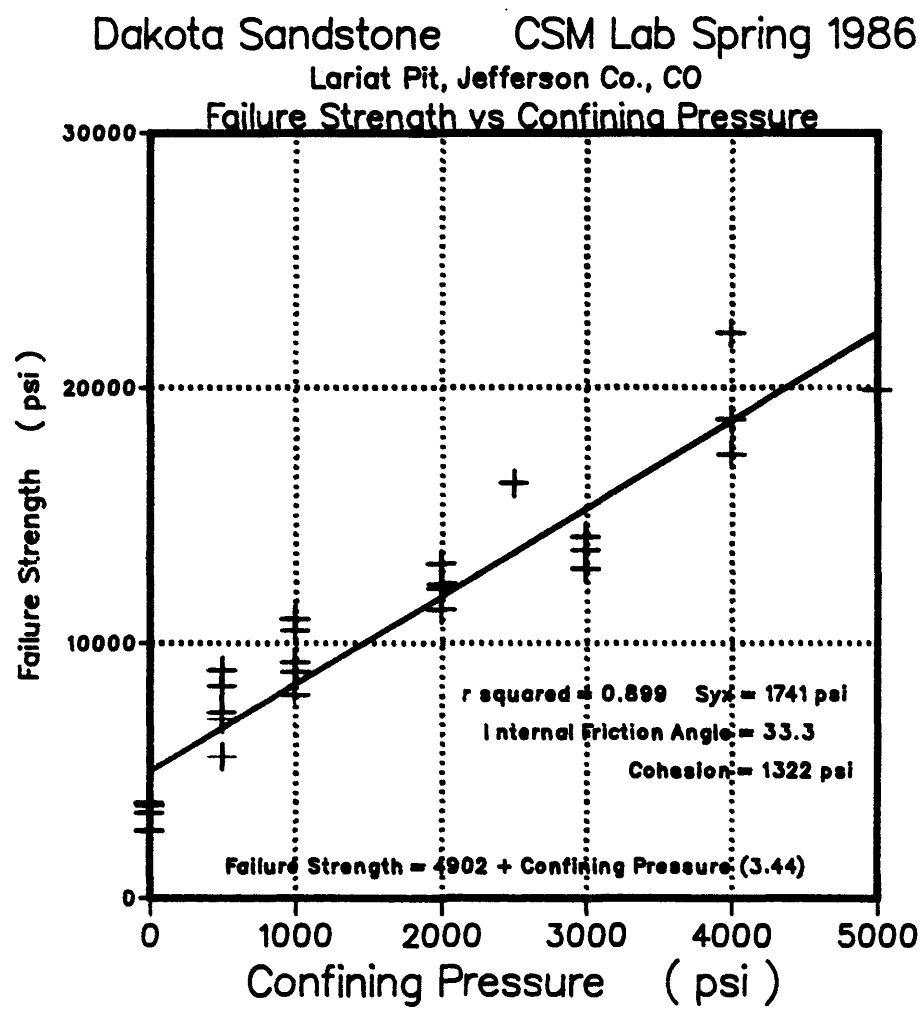

Figure 19. Linear Mohr-Coulomb failure envelope, showing the material properties of intact Dakota sandstone. Testing was conducted under uniaxial and triaxial compression; test results are given in Appendix 2.

method from the standard method of Hoek and Bray (1981). Hoek prefers the use of a curved failure envelope to describe rock strength under all conditions of failure (Hoek, 1968; Hoek and Bray, 1981; Hoek and Brown, 1980, pp. 135 ff.). However, other authors (Goodman, 1980; Hendron, 1968) assert that the linear Mohr-Coulomb approximation of rock material strength is sufficiently accurate for most analytical purposes. Goodman, in particular, states that the linear approximation is quite acceptable as long as

$$\sigma_3 < \sigma_{unconfined}$$

Since surface workings in rock are always at relatively shallow crustal depths, and confining stress is generally correspondingly low, Goodman's restriction seems reasonable.

An estimate of the shear strength of <u>broken</u> rock material along idealized smooth discontinuity planes (joints) is derived by testing the rock in direct shear (ISRM, 1981). Testing was conducted in the Rock Mechanics Laboratory under the direction of Dr. John F. Abel; tests were performed using standardized methods (ASTM,1986; ISRM 1981); results are shown in Appendix 2. Note that the design of the series of direct-shear tests required that each pair of core samples be tested twice. In the first series of tests, one member of a sample pair was sheared past the second, stationary sample. During the second series of tests, the order was reversed: the sample that had been stationary was placed atop the previously mobile sample, and the pair was sheared again. This procedure was adopted to examine the uniformity of the testing process. As expected, values of shearing resistance for the second series of tests were consistently lower than values resulting from the first test series. This occurred because surface asperities present on the rock samples were sheared off during the first series of tests. The resulting, fewer asperities presented less shear resistance during the second series of tests.

Test results were plotted as shear strength vs. applied normal stress (Figure 20); this again gave a linear Mohr-Coulomb failure envelope. Consideration of the failure envelope in the case of direct shear gives an estimate of rock-on-rock cohesion and the friction angle for broken rock, while the failure envelope in the case of uniaxial and triaxial compression gives an estimate of the intact-rock cohesion and the friction angle for intact rock. These properties are sufficient to describe the engineering behavior of the rock material for conditions of applied stress; rock material properties are listed in Table 3.

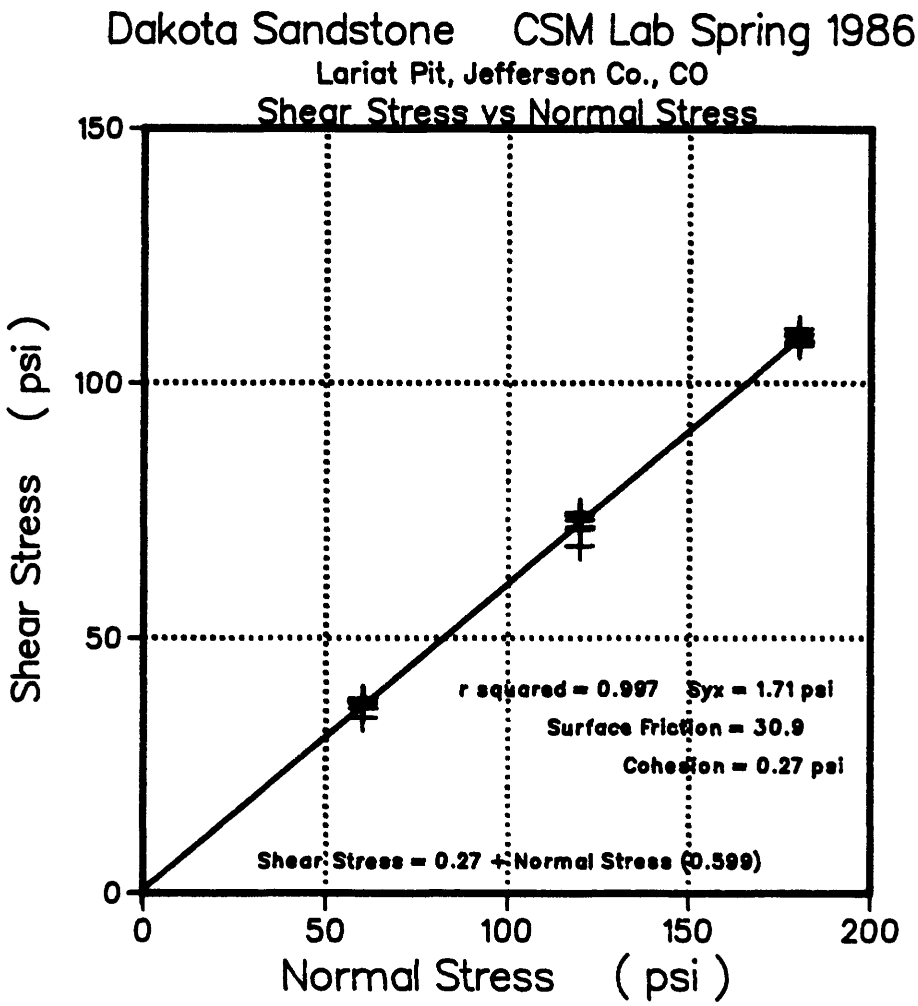

Figure 20. Linear Mohr-Coulomb failure envelope, showing the material properties of broken Dakota sandstone. Testing was conducted in direct shear; test results are given in Appendix 2.

Rock Mass Properties

Rather than using the unqualified estimates of rock material properties derived in the laboratory, it is better to approximate the behavior of a non-ideal geologic material, under conditions of applied stress, using estimates of rock mass properties. It has been recognized (Call, 1972; Hoek and Bray, 1981; Terzaghi, 1962) that jointing in a rock mass is often discontinuous in nature. Terzaghi (1962, p. 252 ff.), states:

> A body of rock - for instance the body of rock underlying a slope within a distance approximately equal to the height of the slope - is said to contain continuous joints if it is possible to construct sections across the body which nowhere cut across the intact rock. These sections may be approximately plane, irregularly warped, or stepped-up like similar sections across a body of dry brick masonry.
>
> ...In addition to continuous joints, every body of rock contains more or less discontinuous joints with very different degrees of discontinuity. A section following a discontinuous joint cannot enter an adjacent joint without cutting across intact rock. Those portions of a section which are located in intact rock will be referred to as <u>gaps</u>. The ratio between the area of the joints located in a section and the total area of the section represents the <u>effective joint area</u> of the rock formation along the section. A shear failure along the section is resisted by both pressure-conditioned shearing resistance and the cohesion of the rock located in the gaps between joints.

Terzaghi continues by stating that it is impractical to attempt to determine the relative proportions of intact and broken rock in the slope; and that in any case, the cohesion of the intact rock contributes little to the overall strength of a rock slope. An apparent contradiction occurs in this argument, however, because further in his discussion, Terzaghi states that the cohesion of the intact rock is the reason natural rock slopes stand at high slope angles (Terzaghi, 1962, p. 254).

According to Call (Call, 1972, pp. 33 ff.), it is possible to derive an estimate of the relative proportions of broken and intact rock located along a given design joint, based on the degree of continuity of the trace of a fracture. Under the assumed worst-case conditions, the total length of a given joint is the sum of both the continuous fracture length and that part of the joint which is discontinuous, the joint 'gap'. An empirical worst-case approximation of intact-rock gap in a design joint can be taken as the true minimum spacing between joints of the same joint set (Figure 21). The proportions of intact and broken rock along a given design joint can then be estimated as:

$$\% \text{ Broken rock(1D)} = \text{MAX LENGTH}/(\text{MAX LENGTH} + \text{MIN SPACE}) \times 100,$$

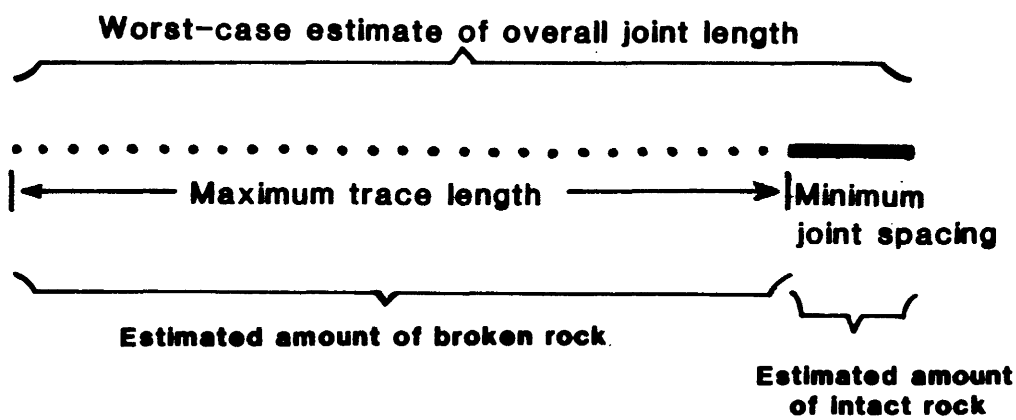

Figure 21. One-dimensional estimate of the proportion of broken rock along a design joint. The minimum joint spacing (MIN SPACE) is taken as a conservative estimate of the discontinuous gaps along the joint, while the maximum trace length (MAX LENGTH) is assumed to be a conservative estimate of the continuity of the design joint. The one-dimensional geometric proportion of broken rock is then

% Broken rock(1D) = MAX LENGTH/(MAX LENGTH + MIN SPACE) X 100.

where

MIN SPACE = minimum joint spacing of all joints in the set
MAX LENGTH = maximum trace length of all joints in the set

and

$$\% \text{ Intact rock(1D)} = 100 - \% \text{ Broken rock(1D)}$$

for the one-dimensional case. Note that the minimum joint spacing is the adjusted (true) joint spacing, rather than the apparent joint spacing recorded in a detail-line survey. Spacings are adjusted according to the recommendations of Call and others (1976), Call (1972), Cruden (1977), Hudson and Priest (1983), ISRM (1978), Major and others (1977), and Priest and Hudson (1981). While the joint trace and the joint gap (intact rock) do not necessarily lie in the same plane, they are assumed to be colinear in deriving this empirical estimate of intact and broken rock proportions.

Joints are best approximated as two-dimensional planar features; for the two-dimensional case (Figure 22), the corresponding expression is:

$$\% \text{ Broken rock(2D)} = (\text{MAX LENGTH})^2/(\text{MAX LENGTH} + \text{MIN SPACE})^2 \times 100,$$

where

MIN SPACE = minimum joint spacing of all joints in the set

MAX LENGTH = maximum trace length of all joints in the set

and

$$\% \text{ Intact rock(2D)} = 100 - \% \text{ Broken rock(2D)}$$

These estimates of the proportions of broken and intact rock in the design joint along the assumed failure plane were used throughout the subsequent stability analyses. Since offset is assumed to have occurred along <u>fault</u> planes, but not along joint planes, the proportion of intact rock along faults is conservatively estimated as 0%.

An important consideration in the determination of the mechanical properties of unjointed rock is the effect of specimen size on the strength and deformation characteristics of the rock material (Bandis and others, 1981; Bieniawski, 1968; Pratt and others, 1972). Because rock is an inherently inhomogeneous, anisotropic material containing microscopic discontinuities (grain boundaries) as well as

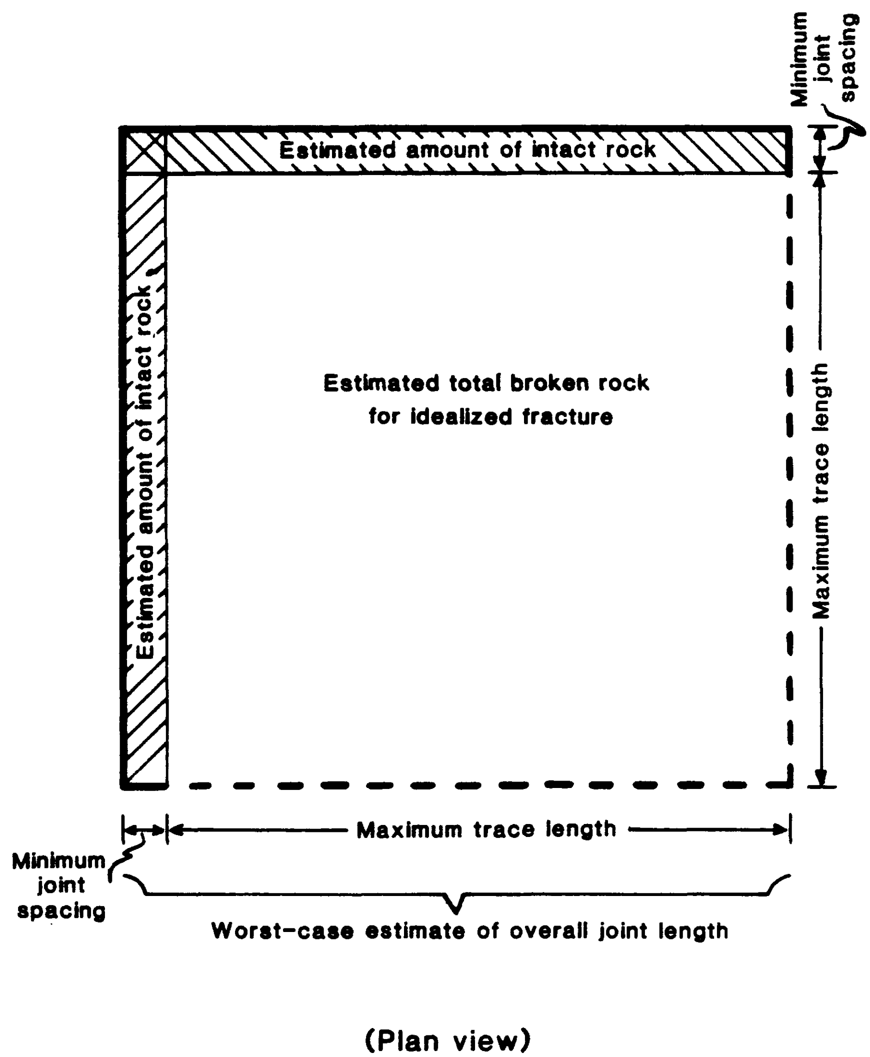

Figure 22. Two-dimensional estimate of the proportion of broken rock along a design joint. The minimum joint spacing (MIN SPACE) is taken as a conservative estimate of the discontinuous gaps along the joint, while the maximum trace length (MAX LENGTH) is assumed to be a conservative estimate of the continuity of the design joint. The two-dimensional geometric proportion of broken rock is then

$$\% \text{ Broken rock(2D)} = (\text{MAX LENGTH})^2/(\text{MAX LENGTH} + \text{MIN SPACE})^2 \times 100.$$

regions of compositional variation, it can be expected that material properties will change as the volume of material under consideration increases. According to Bieniawski (1968, p. 325):

> It may seem that the compressive strength of a material is a property easy enough to determine and that this can be done during a simple laboratory test. While this may be so, to a certain extent, in the case of such materials as steel, the compressive strength of coal is one of the most difficult properties to establish experimentally. It has been shown that there are many factors that influence properties of coal, such as anisotropy, behavior of cracks and fissures, pore pressure, environment, rate of loading, time effects, and specimen size and shape.
>
> ...Since coal is not a continuous solid material but contains various discontinuities such as cracks, cleats, and bedding planes, the strength of coal is of necessity a statistical value depending on how many and what types of discontinuities are present.

The larger the volume of sample tested, the greater is the influence of material anisotropy.

To account for the scale effects inherent in the testing process, various investigators (Bieniawski, 1968; Pratt and others, 1972) recommend that the specimen-derived value of intact-rock cohesion be reduced by some factor. This strength-reduction factor is dependent on the ratio of specimen size (2 inches) to the minimum size of intact-rock joint gaps found in the slope (3 inches). The Bieniawski relationship (Bieniawski, 1968, p. 331, Fig. 10) was chosen, because the Dakota sandstone was formed under conditions of sedimentation and burial, rather than by igneous processes (Pratt and others, 1972). This relation was used to determine the rock-mass strength-reduction divisor (1.2).

The estimation of the relative proportions of broken and intact rock in the slope, and the corollary of rock-mass strength reduction are the second major departure of the Modified Limiting-Equilibrium method from the standard method of Hoek and Bray. These rock mass properties for the Dakota sandstone are listed in Table 3.

Table 3. Engineering properties of the Dakota sandstone.

Rock Material Properties		
Density of the rock material:	124.6 lbs. per cu. ft.	
Properties of intact rock		
Intact-rock cohesion:	1322 psi	
Angle of internal friction:	33.3°	
Properties of broken rock		
Rock-on-rock cohesion:	0.27 psi	
Angle of surface friction:	30.9°	

Rock Mass Properties		
Proportion of intact rock		
	1-dimensional	2-dimensional
Joint Set 4	2.3%	4.6%
Fault 3	0%	0%
Rock mass strength-reduction divisor:		1.2

RESULTS AND SENSITIVITY ANALYSIS

Initial Analytical Solution

The Modified Limiting-Equilibrium method was used to estimate the slope factor of safety, incorporating slope and rock parameter values that had been determined. Initial trials yielded values for the Factor of Safety ranging from 4.5 - 8.0. Because failure had already been observed in the slope, in conditions similar to those described in the analytical solution, this was an unsatisfactory result. The high factor of safety indicated that some slope parameter, or parameters, had not been described as it (or they) actually occurred at the site. Accordingly, a method of sensitivity analysis was devised, in an attempt to determine which parameters might critically affect slope stability, within the narrow ranges of parameter values possible at the site. The analysis might also assist in identifying the causes of previous slope failures.

Sensitivity Analysis

The analytical solution of the Modified Limiting-Equilibrium method is based upon estimates of the following physical parameters:

1. Geometry of the rock slope, including height of the slope and effective slope angle;
2. Orientations of the design joints and faults that define the failure block;
3. Properties of the rock material, including material density, and cohesion and friction angles of both intact and broken rock;
4. Properties of the rock mass, including estimates of the relative proportions of intact and broken rock in the slope, and the rock- mass strength reduction divisor; and
5. The hydrology of the rock slope.

For the purpose of this analysis, it was assumed that the rock material properties and rock mass properties were constant, and not subject to significant variation. This assumption is justified, as laboratory testing of the rock properties is intended to closely define the physical properties of the material, and the Dakota sandstone approximates an ideal, homogeneous engineering material. Similarly, estimates of rock mass properties represent a 'closest approximation' to description of actual conditions.

Orientations of design joints show a significant degree of natural variation, that may locally affect the stability of the rock slope. During the initial analysis of plane failure, for example, the modal dip angle of the design joint was determined to be 52°; this angle was used in subsequent calculations. However, the design joint does not strike parallel to the orientation of the rock slope; inspection of the conceptual stereonet (Figure 18) shows that a plane-failure block moving out of the slope may actually slide down an apparent modal dip angle as low as 47°. The angle of plunge of the wedge in the wedge-failure case is subject to similar variation.

Mining conditions at the site, including slope undercutting, preclude a constant attitude of the pit wall. Thus the effective slope angle is subject to variation, given the actual conditions. Because the company mining plan calls for deepening of the pit, the slope height will also be changed during future operations. Slope height is often the parameter of most interest to pit operators - a determination of the depth to which mining can safely proceed is a matter of economic importance. Finally, it was assumed that the orientation of the pit wall was constant along its length, and that the rock slope remains dry.

Parameter variation. The parameters that were varied during the sensitivity analysis, and the ranges of variation, are listed in Table 4. Parameter variation was accomplished conveniently through use of microcomputer programs that allow the sequential variation of a single parameter over a specified range. These programs are included in Appendix 3. The effects of parameter variation for specified ranges on resultant factors of safety were noted. The calculation results are reported for one-dimensional factors of safety at the 98% level of confidence; this seemed to be a reasonably conservative value.

<u>Dependence of Factor of Safety on Dip of Failure Surface</u>. The variation of factor of safety with dip of failure structure, in the case of plane failure, is shown for two different slope heights: 100 ft and 500 ft (Figure 23). Both curves are nearly linear, and indicate that for this slope, the dip of a potential plane-failure structure has little effect on the factor of safety. A similar result is obtained in a plot of the variation of factor of safety with angle of plunge of the line of intersection in the case of wedge failure (Figure 24). Two inferences can be drawn from this observation: 1) Possible slight variations in the field determination of joint orientations is not critical; and 2) the idealized true planar structures considered

Table 4. Ranges of parameter variation during sensitivity analysis.

PARAMETER VARIED	VARIATION RANGE
Plane failure:	
Dip of failure surface	47^{o} - 52^{o}
Slope height	100 ft - 500 ft
Effective slope angle	47^{o} - 60^{o}
Wedge failure:	
Plunge angle of wedge	47^{o} - 52^{o}
Slope height	100 ft - 500 ft
Effective slope angle	47^{o} - 60^{o}

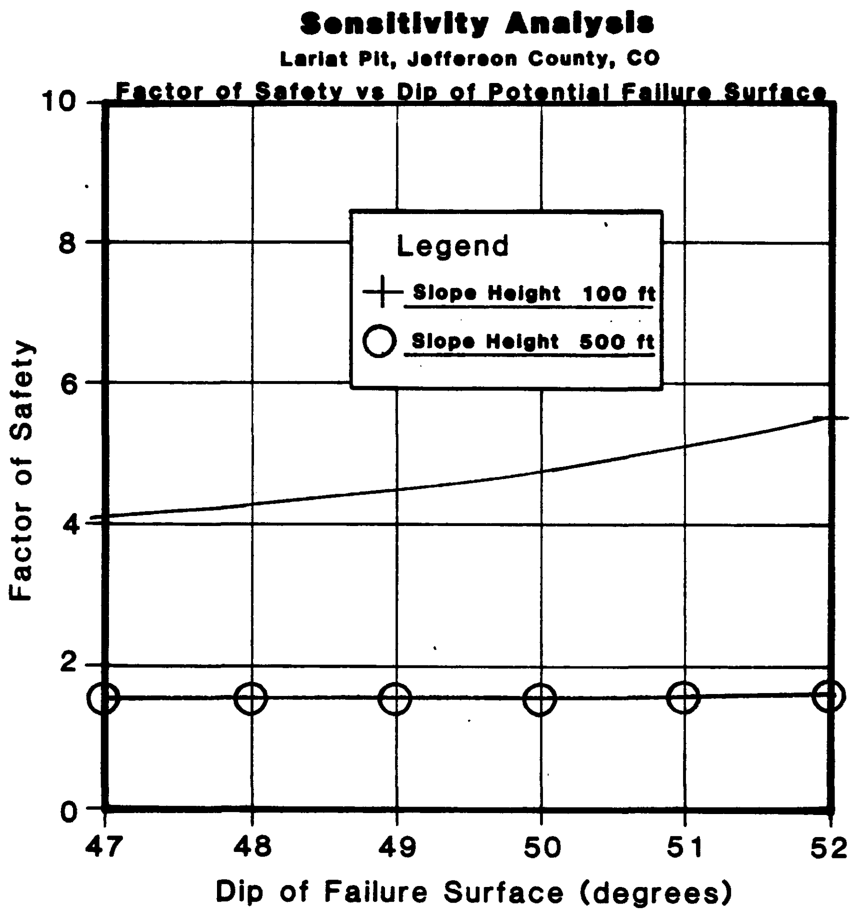

Figure 23. Variation of slope factor of safety versus dip of the potential failure surface in the case of plane failure. The effective slope angle is taken to be 60°. Two slope heights (100 and 500 ft) are shown. The curves are nearly linear, indicating that the factor of safety is relatively insensitive to small variations in the dip of the failure plane.

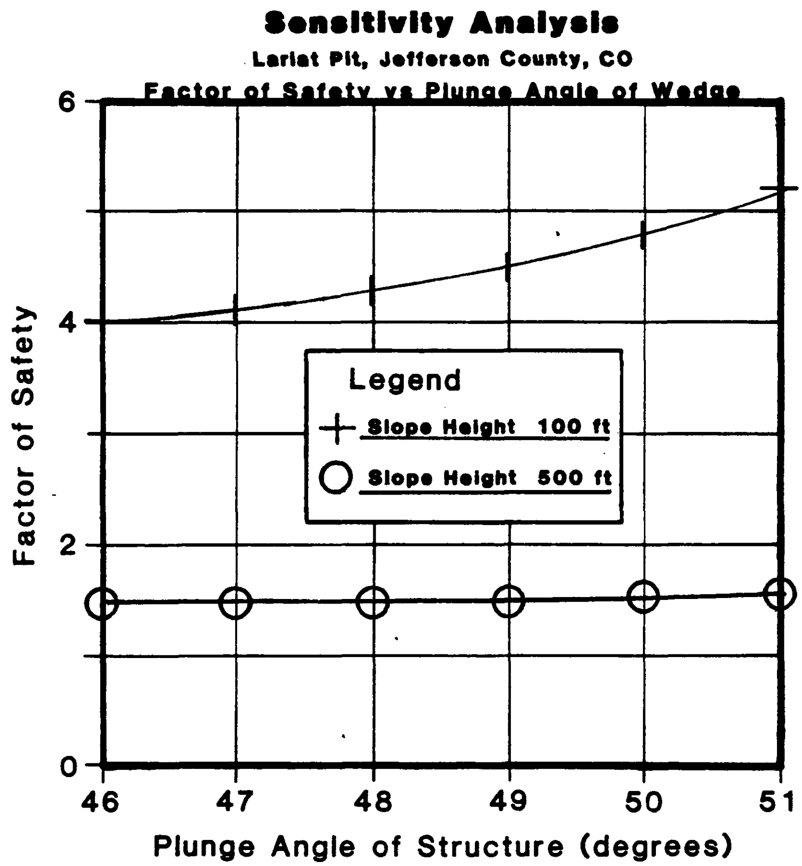

Figure 24. Variation of slope factor of safety versus angle of plunge for the potential failure wedge. The effective slope angle is held constant at 60°. Two slope heights (100 and 500 ft) are shown. The shallow slopes of the curves indicate that the factor of safety is only moderately sensitive to small variations in the plunge angle of the wedge.

during a stability analysis, with fracture roughness accommodated by irregularity angles, are probably a reasonable approximation of actual joint conditions, as minor variations in structural attitude apparently have little effect on stability.

Dependence of Factor of Safety on Slope Height. The variation of factor of safety with slope height was considered for slopes ranging from 100 ft (the actual slope) to 500 ft in height (an exaggerated slope). The analysis revealed only minor dependence of stability on slope height for the Lariat Pit (Figures 23 and 24), and indicated that the pit could be extended to a depth of 500 ft while maintaining an adequate factor of safety. In this analysis, both plane failure and wedge failure conditions were considered; effective slope angles of 60 degrees were considered in both cases. In the plane failure condition, dip of the potential failure structure was varied between 47 degrees and 52 degrees, while the plunge of the line of intersection of the structures in the wedge-failure case was varied between 46 degrees and 50 degrees.

Dependence of Factor of Safety on Effective Slope Angle. The variation of factor of safety with effective slope angle, in the case of plane failure, was considered for angles ranging from 47 degrees - the apparent dip of the potential failure plane projected onto the normal to the actual slope - to 60 degrees (a steep slope). The dip of the potential failure surface was held constant at 47 degrees, while slope heights ranging from 100 to 500 ft were considered (Figure 25).

This analysis revealed that stability of the pit wall is highly sensitive to changes in effective slope angle. If an acceptable factor of safety for an open-pit slope is taken to be 1.3 (Hoek and Bray, 1981), then the west wall of the Lariat Pit may become unsafe at an effective slope angle of 60 degrees, if the pit is deepened to 500 feet. The factor of safety is similarly dependent on effective slope angle in the wedge-failure case (Figure 26).

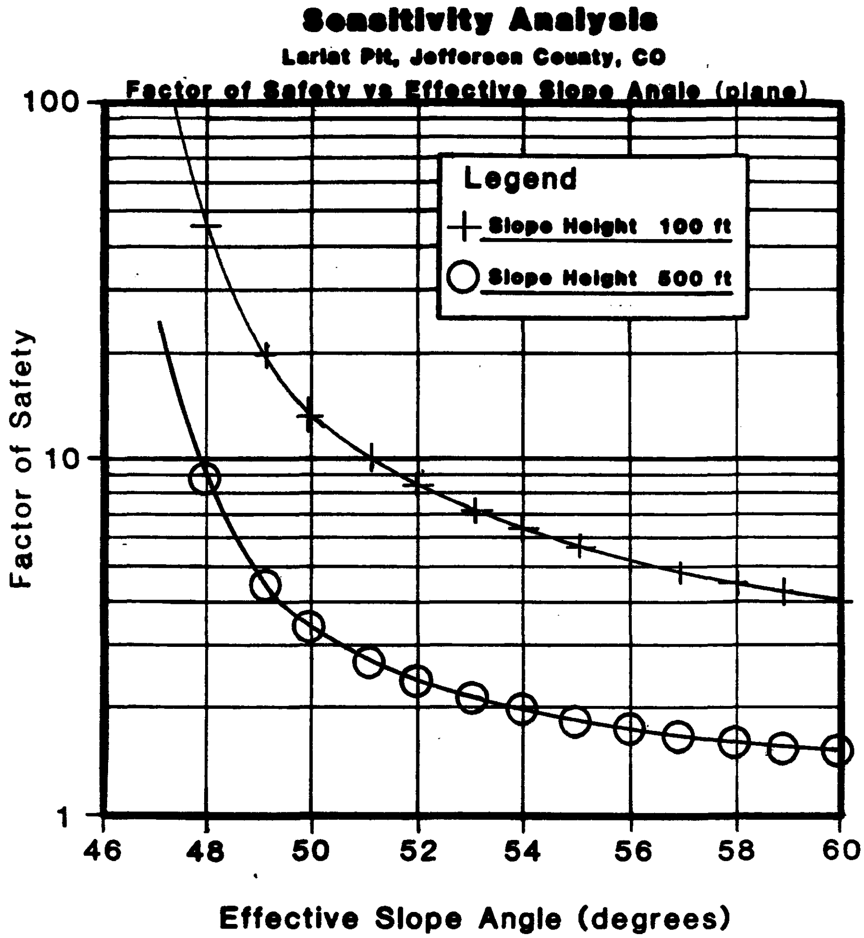

Figure 25. Variation of slope factor of safety versus effective slope angle, in the case of plane failure. The dip of the design joint is constant at 47°. Two slope heights, 100 and 500 ft, are shown. The steep slopes of the curves (note semi-log scale) indicate that the factor of safety is highly sensitive to small variations in the effective slope angle.

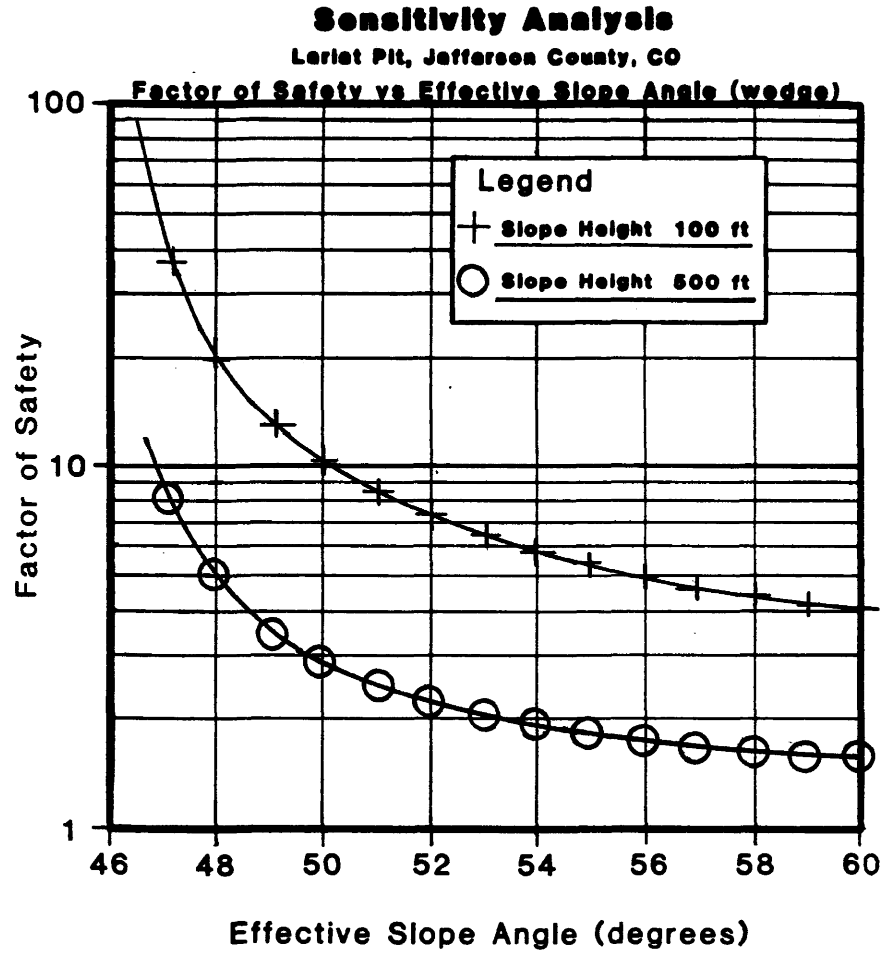

Figure 26. Variation of slope factor of safety versus effective slope angle, in the case of wedge failure. The plunge angle of the wedge intersection is constant at 46°. Two slope heights, 100 and 500 ft, are shown. The steep slopes of the curves (note semi-log scale) indicate that the factor of safety is highly sensitive to small variations in the effective slope angle.

Results of the Sensitivity Analysis

The sensitivity analysis showed that the stability of the rock slope at the Lariat Pit was strongly dependent on the effective slope angle of the pit wall. The effective slope angle of the pit wall is steepened as undercutting of the the toe occurs during mining (Figure 10). Undercutting that proceeds only 3 feet into a 52^{o} dip slope is sufficient to increase the effective slope angle by one degree in a 100 foot slope. Undercutting is common during dip-slope stripping, and it is probable that continued mining, under the present conditions, would eventually result in some significant slope failures.

CONCLUSIONS AND RECOMMENDATIONS

A study was undertaken to determine the potential for slope failure in the dip-slope west wall of the Lariat Pit. The objectives of the study were threefold:

1. To assess the stability of the west wall of the pit in the present configuration.
2. To assess the stability of the pit in the proposed final configuration.
3. To determine the relative degree of influence of various slope parameters on the Factor of Safety.

Preliminary stability analyses were unsatisfactory, so a sensitivity analysis, using the technique of parameter variation, was devised.

When a sensitivity analysis had been performed on various factors which can influence the stability of a slope, it was apparent that the stability of the west wall of the Lariat Pit was highly sensitive to slight changes in the effective slope angle. The method of mining employed at the Lariat Pit (dip-slope stripping) commonly results in undercutting of the toe of the slope, with a concomitant increase in the effective slope angle. The owners of the pit are encouraged to inform working personnel, in particular the heavy equipment operators, regarding the hazards associated with careless stripping practices. Employee education, coupled with strict supervision of mining activities at the pit should result in a lower incidence of undercutting. The improved factors of safety should allow future operations along the dip-slope west wall of the pit to extend the face to a height of 500 feet.

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Appendix 1

DETAIL-LINE DATA

LARIAT PIT DATA: DETAIL LINE #1

Data collected by J.W. Anthony and E.A. Villaescusa, 1 February 1986

Line located at base of rock slope, about 4 ft above slope toe. Line is near the center of the pit wall.
Line is horizontal, and is laid directly along the rock slope. Attitude of the slope at this point is N26W/58NE. Bedding attitude of the Dakota sandstone is N19W/59NE.

FRACTURE ORIENTATION	TAPE DISTANCE (feet)	TRACE LENGTH (feet)	COMMENTS
N13W/39SW	0	20	sub-parallel to detail line
N22W/53SW	0	4	
N69W/77NE	2.5	.5	
N01E/48NW	3.25	.25	
N16W/77SW	5	2	
N03E/21SE	9.5	1	
N40W/44NE	9.5	.25	
N24W/53NE	10	.25	
N21W/67NE	10.5	.25	
N56W/51NE	10.5	.75	
N34W/55NE	11	.25	
N37W/19NE	11.75	1	
N32E/75NW	12	.5	
N05W/68SW	13	.5	
N01W/55SW	13	2	
N37E/79NW	13.5	2	
N15W/37SW	14	2	sub-parallel to detail line
N16W/50NE	14.5	4	
N20E/43SE	14.5	2	
N20E/89NW	14.7	.25	
N29W/50NE	14.75	.75	
N31E/82SE	15	.5	
N60W/78SW	15	.5	
N85W/72SW	16	.5	
N22W/66NE	16	1	
N14E/56NW	18.5	2	
N86E/58SE	18.5	1	
N09W/38SW	20	.5	
N63E/70NW	20	1	

DETAIL LINE #1 (continued)

FRACTURE ORIENTATION	TAPE DISTANCE (feet)	TRACE LENGTH (feet)	COMMENTS
N03E/40NW	20.5	.25	
N10W/50NE	20.5	.5	
N09W/59SW	21	2	
N04W/46NE	23.5	.25	
N02E/41NW	23.5	.25	
N84W/66SW	24	.25	
N19W/52NE	24	.5	
N06W/29SW	26.5	.25	
N02W/60NE	26.5	4	
N46E/52NW	26.5	1	
N54W/57SW	27	.25	
N75E/76NW	27.5	.5	
N85E/63SE	28	.25	
N14W/84NE	29	1	

```
SCHMIDT EQUAL-AREA NET        0  0  0
 LOWER HEMISPHERE     0  0  0  0  0  1  0  0  0
               0  0  0  0  0  1  3  2  0  0  1  1  0
            0  0  0  0  0  0  2  2  1  0  0  1  1  0  0
         0  0  0  0  0  0  0  1  1  0  0  0  1  0  0  0  0
         1  1  0  0  0  0  0  0  0  0  0  0  1  0  0  0  0
      1  1  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
      1  0  0  0  0  0  0  0  0  0  0  0  0  0  0  1  0  1  1
      0  0  0  0  1  1  0  0  0  0  0  0  0  1  1  1  1  1  1
   0  0  0  0  0  1  0  1  0  0  0  0  0  1  4  3  1  2  1  0  0
W  0  0  1  1  1  1  0  1  1  0  0X 0  0  1  4  4  2  1  1  0  0  E
   0  0  0  1  3  3  0  0  1  1  0  0  0  0  0  2  2  1  0  0  0
      1  0  2  4  4  0  0  1  0  0  0  0  0  0  0  1  1  0  0
      1  2  2  4  3  1  0  0  0  0  0  0  0  0  0  0  0  0  1
      0  0  0  1  1  1  0  0  0  0  0  0  0  1  1  0  1  1  1
         0  0  0  0  1  1  0  0  0  0  0  0  1  1  0  2  2
         0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  1  0
            0  0  0  0  0  0  0  0  0  0  1  1  0  0  0
               0  0  1  1  0  0  0  0  1  1  1  0  0
   RAW POINT COUNT    0  1  0  0  0  0  1  1  0        LARIAT PIT DATA:  DETAIL LINE #1
   NUMBER OF POINTS 43        0  0  0                  ANTHONY-VILLAESCUSA  1 FEBRUARY 1986
```

Figure 27. Stereoplot of poles to joint planes for joint data of Detail Line #1. Number of poles per 1% area are shown.

LARIAT PIT DATA: DETAIL LINE #2

Data collected by J.W. Anthony and E.A. Villaescusa, 1 February 1986

Line located just north of an undercut nose in the south-central part of the pit.
Line is horizontal, and placed along the rock slope about 10 ft above the base of the slope. Slope attitude at this point is about N24W/73NE. The slope is 100 - 120 ft in height.

FRACTURE ORIENTATION	TAPE DISTANCE (feet)	TRACE LENGTH (feet)	COMMENTS
N19W/61NE	0	40	
N65E/88NW	0	1	
N16W/62NE	.25	20	
N21W/59NE	.5	15	
N31W/59NE	.75	2	
N17W/66NE	1	40	
N14W/72NE	1	8	
N01W/65NE	1.25	8	
N24W/37SW	1.25	1	
N28W/53NE	1.5	10	
N09W/80NE	1.5	2	
N13W/74NE	2.75	2	
N29W/49SW	2.75		
N36W/55SW	2.75		
N19W/65NE	3	1	
N67E/88SE	3		
N25W/71NE	3.25	.25	
N22W/74NE	3.5	.25	
N83W/34SW	3.5		
N17W/82NE	3.75		
N19W/87NE	3.75	3	
N87E/70NW	4	4	
N04W/67NE	4	30	
N66E/78NW	4.5	10	
N08W/78NE	4.5	5	
N07W/79NE	4.75	.5	
N26W/66NE	4.75		
N36W/78NE	5	3	
N11W/28SW	5.25	.5	
N10W/81NE	5.25	10	
N75E/89NW	5.5	8	
N14W/83NE	5.5		

DETAIL LINE #2 (continued)

FRACTURE ORIENTATION	TAPE DISTANCE (feet)	TRACE LENGTH (feet)	COMMENTS
N12W/65NE	6	25	
N06W/45SW	6	1	
N13W/64NE	6		
N18W/68NE	6.25	10	
N76E/83NW	6.25	12	
N11W/34SW	6.5	7	
N76E/82NW	6.75		
N21W/75NE	7.75		
N57E/81NW	7.75		
N27W/81NE	8		
N55E/84NW	8		
N06W/55SW	8		

```
SCHMIDT EQUAL-AREA NET                   0  0  0
 LOWER HEMISPHERE                  2  2  1  0  0  0  0  0  0
                             1  0  0  0  0  0  0  0  0  0  0  0  0
                          0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
                       0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
                       0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
                    0  0  0  0  0  0  0  0  0  1  1  0  0  0  1  1  0  0  1
                    0  0  0  0  0  0  0  0  0  0  0  0  0  0  2  1  0  0  1
                    0  0  0  0  0  0  0  0  0  0  0  0  0  1  2  0  0  0  0
                 0  0  0  0  0  0  0  0  0  0  0  0  0  1  2  1  2  1  0  0  1
              W  0  0  2  2  0  0  0  0  0  0  0X 0  0  1  1  1  2  1  0  0  0  E
                 0  4  7  3  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
                    8  8  8  2  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
                    4  8  9  4  1  0  0  0  0  0  0  0  0  0  0  0  0  0  0
                    2  4  2  1  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
                       2  1  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
                       0  1  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
                          0  0  0  0  0  0  0  0  0  0  0  0  2  1  0
                             0  0  0  0  0  0  1  1  0  1  2  2  1
   RAW POINT COUNT                 0  0  0  0  1  0  2  2  3        LARIAT PIT DATA:  DETAIL LINE #2
   NUMBER OF POINTS   42                    0  0  0                 ANTHONY-VILLAESCUSA  1 FEBRUARY 1986
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Figure 28. Stereoplot of poles to joint planes for joint data of Detail Line #2. Number of poles per 1% area are shown.

LARIAT PIT DATA: DETAIL LINE #3

Data collected by J.W. Anthony and E.A. Villaescusa, 1 February 1986

Line located on the south side of a short nose, in the southern part of the pit. The nose is steep-walled (nearly vertical) due to undercutting.

Line is horizontal, and is laid directly on the rock cut, about 5 feet above the base of the slope. The line bears about N52W (obliquely into the slope). In this perspective, fracture traces are easily lost in bedding-plane partings; hence trace length was not noted for fractures taken along this line.

FRACTURE ORIENTATION	TAPE DISTANCE (feet)	TRACE LENGTH (feet)	COMMENTS
N19W/50NE	0		
N01W/41SW	0		
N16E/46NW	0		
N25W/59NE	0.25		
N11W/30SW	0.5		
N09E/40NW	0.75		
N78E/70SE	0.75		
N21W/57NE	0.75		
N06E/51NW	1.25		
N85E/71NW	1.5		
N28W/61NE	2		
N83E/61SE	2		
N48E/69NW	2.5		
N35W/56NE	2.75		
N49E/83NW	3		
N58W/54NE	3		
N05E/09NW	3		

```
SCHMIDT EQUAL-AREA NET                   0   0   0
 LOWER HEMISPHERE                0   0   0   0   0   0   0   0   0
                         1   0   0   0   1   2   0   0   0   0   0   0   0
                     0   0   0   0   0   1   1   1   0   0   0   0   0   0   0
                 0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0
                 0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0
             0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0
             0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0
             0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0
         0   0   0   0   0   0   0   0   0   0   0   0   0   1   1   1   0   0   0   0   0
     W   0   0   0   0   0   0   0   0   0   0   0X  1   1   1   2   2   1   0   0   0   0   E
         0   0   0   0   0   0   0   0   0   0   0   1   0   0   1   2   2   0   0   0   0
             0   0   1   3   1   0   0   0   0   0   0   0   0   0   1   1   0   0   0
             0   0   3   4   1   0   0   0   0   0   0   0   0   0   0   0   0   0   0
             0   0   1   2   1   0   0   0   0   0   0   0   0   0   0   0   0   0   0
                 0   0   0   0   1   1   0   0   0   0   0   0   0   0   0   0   0
                 0   0   0   0   1   1   0   0   0   0   0   0   0   1   1   0   0
                     0   0   0   0   0   0   0   0   0   0   0   0   0   1   1
                         0   0   0   0   0   0   1   1   0   0   0   0   1
   RAW POINT COUNT               0   0   0   0   0   1   0   0   0          LARIAT PIT DATA:  DETAIL LINE #3
   NUMBER OF POINTS   17                 0   0   0                          ANTHONY-VILLAESCUSA  1 FEBRUARY 1986
```

Figure 29. Stereoplot of poles to joint planes for joint data of Detail Line #3. Number of poles per 1% area are shown.

LARIAT PIT DATA: DETAIL LINE A

Data collected by J.W. Anthony and J.M. Anthony, 29 March 1986

Line located near northern end of pit, along a fresh cut. Slope attitude N24W/51NE; line is horizontal along cut. Slope at this point is approximately 100 feet in height.

FRACTURE ORIENTATION	TAPE DISTANCE (feet)	TRACE LENGTH (feet)	COMMENTS
N73E/56NW	1	1	
N51E/72NW	1.2	1	
N73E/62SE	1.5	1	
N79E/60SE	2	20	
N26E/47NW	2.2	3	discont
N67E/61SE	2.75	2	
N73E/89NW	3.5	1	
N81E/72SE	3.6	14	
N41E/35NW	3.75	3	
N83E/72SE	4.7	6	
N83E/79SE	4.75	4	
N81E/80SE	5.75	1	
N79W/57SW	7.2	25	
N34E/62NW	7.3	4.5	
N79E/55SE	7.75	5	
N41E/86NW	8	1	
N40E/66NW	8.75	.75	
N29E/38NW	9	.5	
N77E/72SE	9.25	20	
N28E/59NW	9.75	.5	
N78E/67SE	10.25	10	
N16E/56SE	10.75	40	hematite-filled
N81E/69SE	11.75	1	
N16W/40SW	12	4	
N46E/68NW	12.1	1	
N81E/62SE	12.5	6	
N77E/70SE	12.6	2	
N65E/82NW	12.75	1	
N42E/75NW	12.75	1	
N39E/59NW	14	10	
N23W/26SW	14	12	
N80E/71SE	14	1.5	
N86E/76SE	14.25	.5	
N39E/40NW	14.5	2	

DETAIL LINE A (continued)

FRACTURE ORIENTATION	TAPE DISTANCE (feet)	TRACE LENGTH (feet)	COMMENTS
N79E/78SE	14.75	10	
N85E/75SE	15.75	30	
N41E/35NW	16.5	40	
N81E/73SE	16.5	1	
N38E/75NW	16.75	1	
N45E/77NW	17.5	1	
N37E/84NW	17.75	.75	
N52E/68SE	18.5	8	
N24E/44NW	19.25	40	

```
SCHMIDT EQUAL-AREA NET                     0   0   0
  LOWER HEMISPHERE             1   1   5   8   2   0   0   0   0
                       0   0   0   0   8  12   2   0   0   0   0   0   0
                   1   0   1   0   2   7   5   0   1   1   0   0   0   0   0
               2   0   0   1   1   1   2   1   0   1   1   0   0   0   0   0   0
               0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0
           0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0
           0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0
           0   0   1   1   0   0   0   0   0   0   0   0   1   1   1   0   0   0   0
      .0   0   0   0   1   0   0   0   0   0   0   0   1   1   1   1   0   0   0   0   0
   W   0   0   0   0   0   0   0   0   0   0  0X   0   0   0   0   0   0   0   0   0   0   E
       0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0
           0   0   0   0   0   0   0   0   0   0   0   0   2   2   2   0   0   0   0
           0   0   0   0   0   0   0   0   0   0   0   0   3   4   2   1   1   0   0
           0   0   0   0   0   0   0   0   0   0   0   0   0   1   1   4   1   0   0
               0   0   0   0   0   0   0   0   0   0   0   0   0   3   6   3   1
               0   0   0   0   0   0   0   0   0   0   1   0   0   2   4   5   2
                   0   0   0   0   0   0   0   0   0   1   0   0   1   2   1
                       0   0   0   0   0   0   0   0   0   0   1   0   0
  RAW POINT COUNT              0   0   0   0   0   0   0   1   1          LARIAT PIT DATA:  DETAIL LINE A
  NUMBER OF POINTS   43                    0   0   1                      ANTHONY-ANTHONY 29 MARCH 1986
```

Figure 30. Stereoplot of poles to joint planes for joint data of Detail Line A. Number of poles per 1% area are shown.

LARIAT PIT DATA: DETAIL LINE B

Data collected by J.W. Anthony and J.M. Anthony, 29 March 1986

Line located on a narrow bench, about 40 feet above toe of the pit wall, near north-central part of the pit.
Line is horizontal, and along rock slope; attitude of the wall at this point is about N21W/50NE.

FRACTURE ORIENTATION	TAPE DISTANCE (feet)	TRACE LENGTH (feet)	COMMENTS
N77E/74SE	0	3	
N74E/71SE	.25	4	
N11W/36SW	.25	1	
N81E/77SE	.75	1	
N76E/80SE	1	1.5	
N74E/75SE	1.25	4	
N01W/33SW	1.75	3	
N73E/82SE	2.5	4	
N71E/80SE	2.75	1	
N75E/76SE	3.5	8	
N80E/26NW	3.5	1	
N85E/64SE	3.6	1	
N44E/45NW	4	1	
N52E/82NW	4.75	1.5	
N11W/37SW	5	6	
N61E/88NW	5.5	10	
N65E/89NW	5.5	1	
N88W/67SW	6.3	3	
N41E/61NW	7	1	
N59E/89NW	7.5	4	
N14E/45SE	8	1	
N45E/75NW	8.25	1	
N17W/43SW	8.5	.5	
N30E/54NW	8.75	2	
N35E/60NW	9	3	
N27E/72NW	9.25	1	
N77E/57SE	9.5	2	
N24E/66SE	10	1.5	
N09E/44SE	10.1	1	
N51E/89NW	10.5	1	
N69E/71SE	10.75	1	
N45E/75NW	10.75	1	
N65E/70SE	11	1	
N34E/65NW	11.2	.5	

DETAIL LINE B (continued)

FRACTURE ORIENTATION	TAPE DISTANCE (feet)	TRACE LENGTH (feet)	COMMENTS
N42E/56NW	11.25	.5	
N35E/56NW	11.5	4	
N05W/46SW	12	1	
N54E/89NW	12.2	8	hematite-filled
N50E/88NW	12.25	2.5	
N47E/78NW	12.5	.5	
N79W/34SW	13	3.5	
N36E/75NW	13	40	
N19E/46SE	13.5	20	
N68E/73SE	14.25	12	
N81E/79SE	15.25	2	
N11E/40SE	15.25	30	
N26E/57NW	15.75	1	
N75E/71SE	16	5	
N01W/45SW	17	6	

A fault zone is present near the north end of Detail Line B; fault traces are defined clearly in the Dakota sandstone, and are defined in the overlying shales by slickensides, and by hematite filling. The cross-sectional view of faulting allowed by the slope cut indicates that fault attitude shallows to the northeast with depth.

Fault orientations are as follows:

FRACTURE ORIENTATION	COMMENTS
N41E/82NW	southernmost fault
N29E/73NW	next fault north
N29E/77SE	northernmost fault

```
SCHMIDT EQUAL-AREA NET                 0   0   0
 LOWER HEMISPHERE
                            2   5   7   2   0   0   0   0   0
                    4   3   2   9   8   4   2   1   0   0   0   0   0
                1   2   0   1   3   3   2   2   0   0   0   0   0   0   0
            0   0   0   0   0   0   1   1   0   0   0   0   0   0   0   0   0
            0   0   0   0   0   0   0   0   0   1   0   0   0   0   0   0   0
        0   0   1   0   0   0   0   0   0   1   1   0   0   0   0   0   0   0   0
        0   1   1   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0
        0   0   0   1   2   0   0   0   0   0   0   0   0   0   1   0   0   0   0
    0   0   0   0   2   4   1   0   0   0   0   0   0   0   3   5   1   0   0   0   0
W   0   0   0   0   0   2   0   0   0   0   0X  0   0   0   3   3   2   0   0   0   0   E
    0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0
        0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   1   0   0   0
        0   0   0   0   0   0   0   0   0   1   1   0   0   1   2   3   2   1   0
        0   0   0   0   0   0   0   0   0   0   1   0   1   1   5   5   2   1   0
            0   0   0   0   0   0   0   0   0   0   0   0   0   2   4   1   0
            0   0   0   0   0   0   0   0   0   0   0   0   0   0   3   3   0
                0   0   0   0   0   0   0   0   0   0   0   0   1   6   1
                    0   0   0   0   0   0   0   0   0   0   1   4   4
    RAW POINT COUNT         0   0   0   0   0   0   0   0   2          LARIAT PIT DATA:  DETAIL LINE B
    NUMBER OF POINTS   49               0   0   0                      ANTHONY-ANTHONY  29 MARCH 1986
```

Figure 31. Stereoplot of poles to joint planes for joint data of Detail Line B. Number of poles per 1% area are shown.

LARIAT PIT DATA: DETAIL LINE C

Data collected by J.W. Anthony and J.M. Anthony, 29 March 1986

Line located within a mine adit, which is near the northern end of the pit. Tape zero is against the north post of the adit.
Line is run along the north wall of the adit, at a height of about 3 ft above the adit floor. Axis of the adit is nearly perpendicular to strike of the slope face, and runs about N57E.

The adit is about 5 ft in breadth, and 6 ft high. As apparent trace lengths are limited by the dimensions of the adit, and can be shortened deceptively, trace length data were not taken in the adit, and should not be included in final joint data summary.

FRACTURE ORIENTATION	TAPE DISTANCE (inches)	TRACE LENGTH (feet)	COMMENTS
N41W/61NE	10		bedding-plane
N35W/50SW	13		
N09E/44SE	14		
N31W/57NE	14		
N62W/54NE	18		
N16E/49SE	19		
N19W/46NE	21		bedding-plane
N13E/40SE	23		
N13E/43SE	23		
N56W/65NE	32		bedding-plane
N46W/50SW	32		bedding-plane
N09W/43SW	34		
N38W/50NE	41		
N15W/44SW	44		
N55W/55NE	49		
N16E/52SE	51		
N14E/46SE	56		
N64W/39NE	56		
N12W/44NE	59		
N62W/41NE	62		
N01W/47SW	67		
N31W/53SW	68		

DETAIL LINE C (continued)

FRACTURE ORIENTATION	TAPE DISTANCE (inches)	TRACE LENGTH (feet)	COMMENTS
N14E/17SE	68		
N21W/51SW	69		
N46W/54NE	69		bedding-plane
N51W/52NE	73		
N85W/68NE	75		
N23W/49SW	79		
N89W/46NE	83		

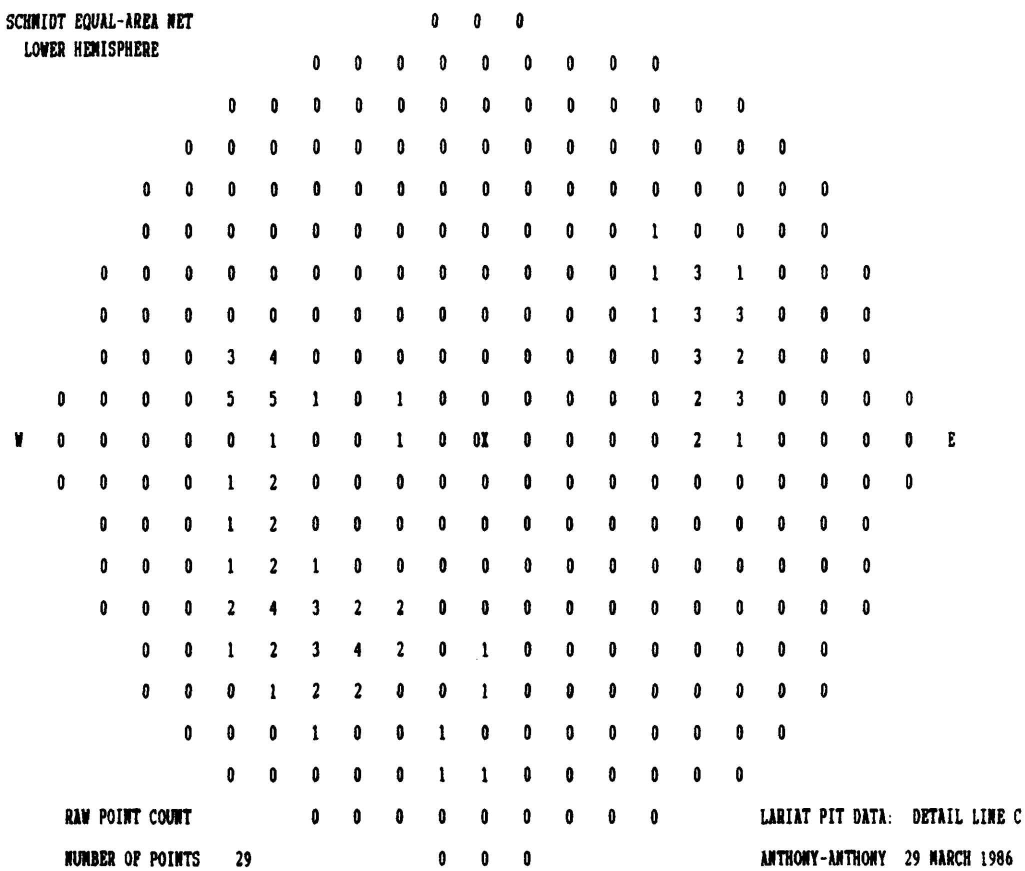

Figure 32. Stereoplot of poles to joint planes for joint data of Detail Line C. Number of poles per 1% area are shown.

LARIAT PIT DATA: DETAIL LINE B1

Data collected by J.W. Anthony and G.D. Naugle, 8 September 1987

Line located in north-central part of pit, and varies in height above pit floor from 1 ft (north end) to 8 ft (south end).
Line is laid horizontal, along the slope face; attitude of the face at this point is N26W/51NE. The slope is between 110 - 130 ft in height.

FRACTURE ORIENTATION	TAPE DISTANCE (inches)	TRACE LENGTH (feet)	COMMENTS
N78E/66SE	0	4	
N82E/68SE	2.5	3	
N40E/41NW	3	2	
N76E/67SE	3.5	4	
N34E/40NW	6	6	
N89E/78SE	6	1	
N72E/76SE	8	6	
N89E/35SE	10	3	
N89W/78SW	12	10	
N40E/34NW	13	20	
N80E/68SE	16	2	
N40E/55NW	20	5	
N68E/75SE	20	4	
N20E/40NW	23	5	
N60E/82SE	23.5	6	
N70E/77SE	25	2	
N30W/36NE	26	1	
N64E/82SE	27	2	
N85E/75SE	32.5	3	
N42E/28NW	33.5	2	
N35E/30NW	36	10	
N85W/76SW	36	1	
N54W/26SW	37	1	
N04W/62NE	41	2	
N36E/30NW	41	5	
N75W/50SW	41	2	
N50W/33SW	44	1	
N73E/82SE	46	2	
N75E/86SE	49	1	
N23E/26NW	52	2	
N80E/78SE	52	1	
N76E/76SE	57	3	

DETAIL LINE B1 (continued)

FRACTURE ORIENTATION (inches) (feet)	TAPE DISTANCE	TRACE LENGTH	COMMENTS
N80E/80SE	60.5	1	
N30E/39NW	61	5	
N22E/25NW	64	1	

```
SCHMIDT EQUAL-AREA NET                    0  0  0
 LOWER HEMISPHERE                   1  4  8  5  4  2  0  0  0
                              0  2  3  4  9  7  3  1  0  0  0  0  0
                           0  0  0  0  0  2  3  0  0  0  0  0  0  0  0
                        0  0  0  0  0  0  0  0  0  1  1  0  0  0  0  0  0
                        0  0  0  0  0  0  0  0  1  1  1  0  0  0  0  0  0
                     0  0  0  0  0  0  0  0  1  1  0  0  0  0  0  0  0  0  0
                     0  0  0  0  0  0  0  0  0  0  1  2  1  0  0  0  0  0  0
                     0  0  0  0  0  0  0  0  0  0  0  1  0  0  0  0  0  0  0
                  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
               W  0  0  1  1  0  0  0  0  0  0  0X 0  0  0  0  0  0  0  0  0  0  E
                  0  0  1  1  0  0  0  0  0  0  0  0  2  2  1  1  0  0  0  0  0
                     0  0  0  0  0  1  1  0  0  0  0  2  6  3  2  0  0  0  0
                     0  0  0  0  0  1  0  0  0  0  0  1  5  4  0  0  0  0  0
                     0  0  0  0  0  0  0  0  0  0  0  0  0  1  1  0  0  0  0
                        0  0  0  0  0  0  0  0  0  0  0  0  0  1  0  0  0
                        0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
                           0  0  0  0  0  0  0  0  0  0  0  0  0  0  0
                              0  0  0  0  0  0  0  0  0  0  0  0  0
   RAW POINT COUNT                  0  0  0  0  0  0  0  0  1        LARIAT PIT DATA:  DETAIL LINE #B1
   NUMBER OF POINTS   37                     0  0  0                 ANTHONY-NAUGLE  8 SEPTEMBER 1987
```

Figure 33. Stereoplot of poles to joint planes for joint data of Detail Line B1. Number of poles per 1% area are shown.

LARIAT PIT DATA: DETAIL LINE B2

Data collected by J.W. Anthony and G.D. Naugle, 8 September 1987

Line located within a mine adit, which is near the northern end of the pit. Tape zero is against the south post of the adit.
Line is run along the north wall of the adit, at a height of about 4 ft above the adit floor. Axis of the adit is nearly perpendicular to strike of the slope face, and runs about N57E.

The adit is about 5 ft in breadth, and 6 ft high. As apparent trace lengths are limited by the dimensions of the adit, and can be shortened deceptively, trace length data, while taken in the adit, should not be included in final joint data summary.

FRACTURE ORIENTATION	TAPE DISTANCE (inches)	TRACE LENGTH (feet)	COMMENTS
N04W/64NE	0	6	
N01W/60SW	1.5	3	
N24W/72NE	4	2	
N35W/36NE	5	8	
N55W/68NE	8	6	
N15W/44NE	11	8	
N06W/50NE	15	4	
N22W/54NE	16	2	
N23W/50NE	17	8	
N60W/39SW	18	3	
N14W/58NE	20	8	
N84W/20SW	21	3	
N12W/57NE	21.5	6	
N07W/50NE	23	5	
N80W/84NE	24.5	1	
N18W/48NE	28	5	
N14W/40NE	30	1	
N20W/43NE	32	1	
N12W/54NE	35	5	
N14W/50NE	35.5	2	
N20W/64NE	39	3	
N20E/40NW	45	1	
N20W/36NE	46	2	
N04E/47NW	52	1	
N12W/52NE	55.5	8	

DETAIL LINE B2 (continued)

FRACTURE ORIENTATION	TAPE DISTANCE (inches)	TRACE LENGTH (feet)	COMMENTS
N04E/38NW	59	5	
N06E/42NW	62	2	
N36E/33SE	63	1	
N16E/35SE	65	1	
N05W/47NE	73	1	

```
SCHMIDT EQUAL-AREA NET                0   0   1
  LOWER HEMISPHERE          0   0   0   0   0   0   0   0   0
                    0   0   0   0   0   0   0   0   0   0   0   0   0
                0   0   0   0   0   0   0   0   0   0   0   0   0   0   0
            0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0
            0   0   0   0   0   0   0   0   0   0   1   0   0   0   0   0   0
        0   0   0   0   0   0   0   0   0   0   0   1   1   0   0   0   0   0   0
        0   0   0   0   0   1   1   0   0   1   1   0   0   0   0   0   0   0   0
        0   0   0   0   0   2   1   0   0   1   1   0   0   0   0   0   0   0   0
    0   0   0   0   0   1   1   0   0   0   0   0   0   0   0   0   0   1   0   0   0
W   0   0   1   1   3   1   0   0   0   0   0X  0   0   0   1   3   1   1   1   0   0   E
    0   0   1   5   10  7   2   0   0   0   0   0   0   0   2   4   1   0   0   0   0
        0   0   4   8   6   3   1   0   0   0   0   0   0   1   1   0   0   0   0
        0   1   2   2   1   1   1   0   0   0   0   0   0   0   0   0   0   0   0
        0   1   1   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0
            0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0   0
            0   0   0   1   1   0   0   0   0   0   0   0   0   0   0   0   0
                0   0   1   1   0   0   0   0   0   0   0   0   0   0   0
                    0   0   0   0   0   0   0   0   0   0   0   0   0
    RAW POINT COUNT         0   0   1   1   0   0   0   0   0         LARIAT PIT DATA:  DETAIL LINE #B2
    NUMBER OF POINTS   30               1   0   0                     ANTHONY-NAUGLE  8 SEPTEMBER 1987
```

Figure 34. Stereoplot of poles to joint planes for joint data of Detail Line B2. Number of poles per 1% area are shown.

Appendix 2

TEST RESULTS FOR DETERMINATION OF ROCK MATERIAL PROPERTIES

UNIAXIAL AND TRIAXIAL TEST RESULTS

Tests performed in the Rock Mechanics Testing Laboratory of the Colorado School of Mines, Golden, CO.

Uniaxial and triaxial testing performed by J.W. Anthony and E.A. Villaescusa, 5 March, 7 March, and 12 March 1986.

Samples are from the "J" interval, South Platte Formation, Dakota Group sandstones, and were obtained from the Lariat Pit of the Robinson Brick Company.

Test Results

Average sample density:	124.6 lbs/ft^3
Estimated uniaxial strength:	4902 psi
Angle of internal friction:	33.3 degrees
Intact-rock cohesion:	1322 psi

Statistics:

Number of samples:	27
Multiple correlation coefficient squared (R^2):	0.899
Standard error of estimate of y based on x:	1741 psi
t-calculated goodness-of-fit:	14.895

Test Data

Sample number	Length (inches)	Diameter (inches)	Weight (g)	Density (lbs/ft^3)	Confining stress (psi)	Failure load (lbs)	Corrected failure strength* (psi)
RB 1/1	3.980	2.140	461.3	122.7	500	32340	8898
RB 1/2	3.819	2.148	441.7	121.5	500	25740	6989
RB 1/3	3.731	2.139	433.5	123.1	500	30360	8287
RB 1/4	3.632	2.141	438.6	127.7	1000	29260	7932

*The corrected failure strength is the actual failure strength of the specimen, corrected to an idealized specimen length-to-diameter ratio of 2:1, as suggested for uniaxial testing by ASTM.

Test Data (continued)

Sample number	Length (inches)	Diameter (inches)	Weight (g)	Density (lbs/ft^3)	Confining stress (psi)	Failure load (lbs)	Corrected failure strength (psi)
RB 1/5	4.057	2.144	466.7	121.3	0	13024	3582
RB 1/6	3.993	2.139	461.6	122.4	1000	39600	10903
RB 1/7	4.020	2.148	464.7	121.4	1000	38280	10463
RB 1/8	3.583	2.144	431.2	126.9	2000	41800	11271
RB 1/9	4.030	2.140	462.5	121.4	2000	47520	13073
RB 1/10	3.328	2.146	378.7	119.7	2500	60940	16229
RB 1/11	4.044	2.146	465.7	121.2	0	13486	3700
RB 1/12	3.706	2.143	424.2	120.8	0	12122	3296
RB 1/13	3.930	2.141	474.9	127.7	3000	49720	13605
RB 1/14	3.965	2.139	471.9	126.1	3000	51480	14128
RB 1/15	3.595	2.146	415.5	121.6	4000	82060	22100
RB 1/16	3.979	2.142	466.6	123.9	4000	63360	17349
RB 1/17	4.002	2.143	453.5	119.6	5000	72600	19872
RB 2/1	3.972	2.153	478.5	128.1	0	9372	2594
RB 2/2	3.940	2.136	473.5	127.6	0	9548	2637
RB 2/3	3.859	2.137	464.1	127.6	500	26400	7252
RB 2/4	3.988	2.135	472.0	125.8	500	19976	5519
RB 2/5	3.952	2.138	473.7	127.1	1000	32120	8836
RB 2/6	3.918	2.135	469.1	127.3	1000	33440	9214
RB 2/7	3.964	2.137	475.8	127.4	2000	44660	12288

Test Data (continued)

Sample number	Length (inches)	Diameter (inches)	Weight (g)	Density (lbs/ft^3)	Confining stress (psi)	Failure load (lbs)	Corrected failure strength (psi)
RB 2/8	3.949	2.136	472.2	127.0	2000	44000	12110
RB 2/9	3.954	2.137	473.9	127.2	3000	46860	12870
RB 2/10	3.912	2.136	468.5	127.2	4000	68200	18727

DIRECT SHEAR TEST RESULTS

Tests performed in the Rock Mechanics Testing Laboratory of the Colorado School of Mines, Golden, CO.

Direct shear testing performed by J.W. Anthony and E.A. Villaescusa, 26 February 1986. Samples are from the "J" interval, South Platte Formation, Dakota Group sandstones, and were obtained from the Lariat Pit of the Robinson Brick Company.

Four rock core samples were used, in two separate sets of tests. In the first set of tests (RB 1/*/S), one core sample was placed atop a second, stationary sample, and the two were sheared. In the second set of tests (RB 2/*/S), the position of the samples was reversed: the sample which had been stationary was placed atop the second sample, and the two were sheared again. This convolving of samples allows an estimate to be made of the accuracy of the test procedure.

Test Results

Average sample density:	124.6 lbs/ft^3
Angle of surface friction:	30.9 degrees
Peak rock-on-rock cohesion:	0.27 psi

Statistics:

Number of tests:	18
Multiple correlation coefficient squared (R^2):	0.997
Standard error of estimate of y based on x:	1.71 psi
t-calculated goodness-of-fit:	72.819

Test Data

Sample number	Diameter (inches)	Density (lbs/ft^3)	Normal load (lbs)	Normal stress (psi)	Shear load (lbs)	Shear stress at failure (psi)
RB 1/1/S	2.140	124.6	217.25	60.40	134.65	37.44
RB 2/1/S	2.140	124.6	217.25	60.40	134.65	37.44
RB 1/1/S	2.140	124.6	432.50	120.25	257.52	71.60

Test Data (continued)

Sample number	Diameter (inches)	Density (lbs/ft^3)	Normal load (lbs)	Normal stress (psi)	Shear load (lbs)	Shear stress at failure (psi)
RB 2/1/S	2.140	124.6	432.50	120.25	265.93	73.93
RB 1/1/S	2.140	124.6	649.25	180.51	393.85	109.50
RB 2/1/S	2.140	124.6	649.25	180.51	392.16	109.03
RB 1/2/S	2.140	124.6	217.25	60.40	136.33	37.90
RB 2/2/S	2.140	124.6	217.25	60.40	132.97	36.97
RB 1/2/S	2.140	124.6	432.50	120.25	267.61	74.40
RB 2/2/S	2.140	124.6	432.50	120.25	255.83	71.13
RB 1/2/S	2.140	124.6	649.25	180.51	388.79	108.09
RB 2/2/S	2.140	124.6	649.25	180.51	385.43	107.16
RB 1/3/S	2.140	124.6	217.25	60.40	129.59	36.03
RB 2/3/S	2.140	124.6	217.25	60.40	122.87	34.16
RB 1/3/S	2.140	124.6	432.50	120.25	262.56	73.00
RB 2/3/S	2.140	124.6	432.50	120.25	244.05	67.85
RB 1/3/S	2.140	124.6	649.25	180.51	397.20	110.43
RB 2/3/S	2.140	124.6	649.25	180.51	387.11	107.63

Appendix 3

PROGRAM DOCUMENTATION: PLANESHR and WEDGESHR

PLANESHR Documentation

Program History: 'PLANESHEAR' is based on an original program developed by J.F. Abel of the Colorado School of Mines, in the Fall of 1983.

This modified and extended version was prepared by J.W. Anthony, Department of Geology and Geological Engineering, Colorado School of Mines, Golden, Colorado 80401, USA. It was developed to implement a sensitivity analysis, conducted at the Lariat Clay Pit of the Robinson Brick Company, Lakewood, Colorado.

Purpose: The program is intended to estimate the stability in plane shear of the worst-case orientation of a daylighted joint set in a rock slope, and is structured so that the Factor of Safety may be determined for a given slope over a wide range of slope parameters. The factor(s) of safety is (are) determined using the limiting-equilibrium method of Hoek and Bray (1977), and incorporating modifications suggested by Abel (1983). Principal modifications include the assumption of intact rock, and a linear Mohr-Coulomb criterion to describe the failure conditions of rock.

All the assumptions inherent in the method will apply.

Implementation: The program is written in an advanced form of the BASIC programming language, and is compiled under the Microsoft QuickBASIC compiler, ver. 2.0. The program is menu-driven, and offers the user a number of options for examining various rock-slope failure conditions.

Interactive input is received directly from the keyboard; no provision has been made for retrieving data from a file, or for storing data in a file. Output can be directed to the screen or to a line printer connected to a parallel port.

System Requirements:	'PLANESHEAR' is compiled for use on an IBM PC or compatible system. Minimum system requirements include: 320K internal RAM; color monitor; color graphics adapter. A version of PLANESHEAR is also available for monbochrome monitors; this version does not require a graphics adapter.
Disclaimer:	The programmer assumes that the program user is familiar with the modified limiting-equilibrium method of Abel (1983). The programmer extends no warranty for program results to any user, and assumes no liability for damages resulting from inaccurate or erroneous program results.

WEDGESHR Documentation

Program History:

'WEDGESHEAR' is based on an original program developed by J.F. Abel of the Colorado School of Mines, in the Fall of 1983.

This modified and extended version was prepared by J.W. Anthony, Department of Geology and Geological Engineering, Colorado School of Mines, Golden, Colorado 80401, USA. It was developed to implement a sensitivity analysis, conducted at the Lariat Clay Pit of the Robinson Brick Company, Lakewood, Colorado.

Purpose:

The program is intended to estimate the stability in wedge failure of the worst-case orientation of two daylighted fractures in a rock slope, and is structured so that the Factor of Safety may be determined for a given slope over a wide range of slope parameters. The factor(s) of safety is (are) determined using the limiting-equilibrium method of Hoek and Bray (1977), and incorporating modifications suggested by Abel (1983). Principal modifications include the assumption of intact rock, and a linear Mohr-Coulomb criterion to describe the failure conditions of rock.

All the assumptions inherent in the method will apply.

Implementation:

The program is written in an advanced form of the BASIC programming language, and is compiled under the Microsoft QuickBASIC compiler, ver. 2.0. The program is menu-driven, and offers the user a number of options for examining various rock-slope failure conditions.

Interactive input is received directly from the keyboard; no provision has been made for retrieving data from a file, or for storing data in a file. Output can be directed to the screen or to a line printer connected to a parallel port.

System Requirements: 'WEDGESHEAR' is compiled for use on an IBM PC or compatible system. Minimum system requirements include: 320K internal RAM; color monitor; color graphics adapter.

A version of WEDGESHEAR is also available for monbochrome monitors; this version does not require a graphics adapter.

Disclaimer: The programmer assumes that the program user is familiar with the modified limiting-equilibrium method of Abel (1983). The programmer extends no warranty for program results to any user, and assumes no liability for damages resulting from inaccurate or erroneous program results.