

DESIGN OF OPEN PIT SLOPES

by

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ABSTRACT

Determining the maximum angle to which a pit slope can be mined requires information on the geometry and strength of the geologic structures, the magnitude and distribution of stresses, and the ground water conditions. The primary objective of mineral exploration and development, however, is the evaluation of ore reserves. Thus such information is either not obtained or appears on geologic maps and drill logs as detail which is not readily useable to the engineer for whom the determination of slope angles is often only one of many problems to be solved. As a result, slope angles are usually arbitrarily chosen rather than designed. Investigations directed toward collecting data on the geomechanical properties of the rock mass can provide much of the information necessary to improve this choice of slope angle. By use of the stereographic projection the geometric relationship between a proposed slope and the geologic structures can be evaluated. Parameters such as planarity, continuity, and type of termination can be used to assign a relative importance to joint sets and to other geologic structures.

INTRODUCTION

The determination of the optimum slope angle and general configuration of the slope for an open pit is a critical part of pit design. With the increased sophistication of pit design, brought about largely by the use of computers, the demand for more precise slope information has become greater. However, the complexity of the rock mass associated with a typical open-pit ore body precludes an easily obtained slope design. Furthermore, unless specific studies are conducted, the data for slope design must come from a number of sources not particularly oriented toward slope design.

Since the primary objective of mineral exploration is the location of potentially minable ore bodies, preliminary investigations are oriented toward the genesis of ore mineralization and favorable structural environments rather than the geomechanical properties of the rock. After the location of a potential ore body, additional work is aimed toward proving the grade and dimensions of the ore and determining the metallurgical characteristics for mill design. As a result, the information necessary to determine the optimum slope angles, i.e., the geometry and character of geologic structures, the magnitude and distribution of stresses and the ground water conditions, is often not obtained or may be buried as incidental details on geologic maps and drill logs. Slope angles are therefore often chosen rather arbitrarily by the engineer, who usually doesn't have the time or facilities to ferret out the needed information in order to evaluate the potential slope conditions. This choice of slope angle can be improved by utilizing what data are available and conducting additional investigations to provide needed information.

DESIGN CONCEPTS

The ultimate objective of slope design, as with pit design in general, is to maximize profit. Since a flat slope requires more waste stripping than a steep slope, the steepest possible unsupported final pit slope

would yield the lowest cost per ton of ore mined and hence the greatest profit. Slope design, therefore, consists in large part of determining the maximum slope that can be mined without slope failure.

Defining slope failure is not as simple as would appear at first glance. From the theoretical standpoint, in which the rock of a slope is considered an elastic material, any displacement beyond recoverable strain constitutes failure. This, however, is not a satisfactory definition for a mine operator who often is successfully mining a slope which has theoretically "failed." Thus, it is necessary to distinguish between "theoretical failure" and what could be termed "operational failure." When the rate of ground movement is greater than the rate at which the slide material can be economically mined, or the movement is actively damaging a permanent facility--such as a skip--it is an operational failure.

Even after accepting criteria for failure, the maximum slope angle cannot be precisely determined. Two factors contribute to this: (1) the uncertainty in the design technique and (2) the influence of unpredictable natural phenomena, such as precipitation and seismic activity. Given sufficient data, a relationship possibly could be established between the probability of failure and the slope angle, such as the hypothetical one shown in Fig. 1. By utilizing such a relationship, in conjunction with figures on the savings that could be realized by increasing the slope, the choice of slope angle may be put on a profit versus economic risk basis--essentially a management decision.

The mining method adds a constraint on the maximum slope angle, particularly for operating slopes inside the ultimate pit limits. The necessity for having haul roads and access benches may limit the overall slope. Operating slopes in a rail pit, where track laying and maintenance are significant costs, are especially critical in this respect. A few inches displacement or a small bench slide can often be negotiated by trucks but may make a level impassable for rail haulage.

Safety considerations also enter into slope design, resulting in more conservative slope angles

than simple economics would dictate. By a system of displacement and microseismic monitoring to provide warning, personnel safety can be maintained even where the possibility of a slide is high. The greater the probability of a slide, the more expensive the safety precautions become; this cost must be considered in slope design.

Slope design should also include consideration of artificial means of increasing the stability of a slope. The effectiveness of drainage in preventing and containing slides in soil has been well established (Terzaghi, 1950, pg. 120) and should be applicable to rock slopes. Rock bolts, tendons and grouting offer possibilities for increasing the strength of a rock mass.

MECHANICS OF SLOPE FAILURE

For purposes of analysing the stability of a slope, the following basic assumptions are made:

1. The rock mass is heterogeneous. Chemical composition and physical properties will vary with position in the rock mass.
2. The rock mass is discontinuous. Geologic structures (faults, jointing, foliation) transect the rock mass so that physical properties vary abruptly.
3. The rock mass is anisotropic. Physical properties are a function of orientation as well as position, i.e., there are preferred orientations of jointing, faulting, foliation and other structures.
4. The strength of the discontinuities is so much less than that of the intact rock that the failure surface will be primarily along the discontinuities.
5. The strength, or resistance to shear, of a discontinuity is a function of the normal stress on the surface. This can

be expressed as some modification of the Coulomb equation:

$$\tau_f = C + \sigma_n \text{ TAN } \phi$$

C = cohesion

σ_n = normal stress

ϕ = friction angle

On the basis of these assumptions the stability of a proposed slope can be determined in principle if one can (1) measure the attitude and position of the discontinuities, (2) determine a cohesion and angle of internal friction for the discontinuities and (3) compute the stresses acting on the discontinuities. As an illustration, consider the simple situation of a planer discontinuity, such as a fault, dipping into a pit (Fig. 2). A unit segment of the discontinuity would be acted on by a stress which would be a combination of the weight of the overlying material, a stress induced by the configuration of the pit, and possibly a regional tectonic stress. The stress distribution can be resolved into a normal stress and a shear stress factor along the discontinuity. If a value for C and ϕ can be assigned to the discontinuity, the factor of safety can be computed.

Obtaining the necessary data for a rigorous solution is virtually impossible at present because of the complexity of any real rock mass. This is not a satisfactory conclusion for the mine engineer who must have some slope angle to use in his pit design. However, the situation is far from hopeless. By a common sense application of the principles that are known about rock behavior and utilization of available data, slope angles can be designed on something more than opinion.

GEOLOGIC STRUCTURE

As stated above, the attitude and position of the discontinuities in the rock must be known in order to determine potential failure surfaces. Usually some information can be obtained from existing geologic maps and sections, although additional mapping oriented toward

slope design is warranted. The amount of information available is highly variable. In an operating open pit, the geologist may have detailed bench maps as well as cross sections and level maps showing the subsurface. If in the past there had been underground mining in the district, there may be miles of geologically mapped workings. On the other hand, an exploration prospect may have only a few drill holes and a reconnaissance map.

The results of any slope design technique are of course only as good as the input data. A quick appraisal of potential slope conditions for a rough economic evaluation may be done with existing geologic information. Because geologic mapping is relatively inexpensive (a competent geologist with his Brunton at an outcrop or in a drift can produce more useful information per investigation dollar than any other technique), additional investigation is economically justified for any slope design beyond a quick appraisal.

A basic requisite for obtaining geologic structural information for slope stability studies is a conventional geologic map showing the major structures and lithologic units. Major structures would be faults and contacts having dimensions of the same order of magnitude as the pit. The attitude and position of these structures relative to the proposed slopes must be determined by surface and drill hole intercepts.

The area between major structures can be considered a structural unit, or structural domain, containing large numbers of secondary structures such as jointing, minor faults, bedding and foliation. These secondary structures constitute, for practical purposes, an infinite population of discontinuities with a range of attitudes but are rarely randomly oriented. The cases where jointing has been reported as random are commonly situations where the jointing patterns are complex giving an appearance of randomness.

The mapping and analysis of jointing and other geologic structures has been dealt with extensively in the geologic literature. The most comprehensive recent publication is by Friedman (1964). John (1962), McMahon (1967), Terzaghi (1964) and Weaver and Call (1965) have treated the application of geologic mapping to rock mechanics and slope stability.

An ideal joint or fracture set would consist of parallel plane surfaces separated by a distance referred to as the spacing. The attitude is described (and measured in the field) by the bearing of the intercept of the plane with a horizontal reference plane. Two alternate methods of describing the attitude of a plane can be used--the bearing and plunge of the normal to the plane, or the bearing and plunge of the dip. These are more useful for analytical purposes because they can be mathematically treated by vector analysis. Any real joint set will of course only approximate the ideal model. The strike and dip will vary as well as the spacing. For a quick graphic method of presenting joint set orientation, the equal area ~~stereographic~~ SCHMIDT projection can be used (Phillips, 1954). If a planar surface is placed at the center of a reference sphere, the normal to the plane is projected to the surface of the sphere. This intersection, called the pole, will be a point on the surface of the reference sphere. The normal can be projected to either the upper or the lower hemisphere. Conventionally in geologic literature the lower hemisphere is used but not consistently. In upper hemisphere projection the poles are in the dip direction. For this reason the upper hemisphere is used in this paper. Converting from one to the other merely requires rotating the diagram 180°.

Two basic ~~stereographic~~ projections are used in structural geology--the Wulff or equalangular net and the Schmidt or equalarea net. The Schmidt equalarea net is constructed such that the area on the projection plane is proportional to the corresponding area on the reference plane. In the case of the Wulff net, the distance from the center of the projection is proportional to the dip angle and the areas are not equivalent. Since we are dealing with a large number of joints and other structures which appear on the reference sphere as clusters of points, the equalarea net should be used rather than the equalangle net. If a Wulff net is used, the same concentration of fractures will appear less dispersed at a low angle than at a high angle.

By plotting the poles of all of the joints, fractures, and other structures on the Schmidt net, a point distribution plot is obtained. By utilizing a 1% counting circle, a contour plot showing the concentration of poles can be obtained. A concentration of poles

represents a preferred orientation and can be classified as a joint or fracture set.

When statistically treating joint attitudes it may be more convenient to use joint set intensity rather than spacing in order to describe the distance between joints in terms of numbers of joints. Intensity is the number of joints per unit distance along the normal to a plane oriented at the mean attitude of the joint set. When spacing is unequal, the usual case, intensity can be described as a simple mean:

$$I = \frac{h}{D}$$

I = Intensity

D = Distance

h = total no. of
of points
observed

By computing the intensity for a standard distance (100' is a good value) regardless of the actual distance sampled, the intensity of various joint sets can be compared directly (Weaver and Call, 1965).

Sampling Methods

To obtain the orientation pattern of discontinuities for a structural domain, statistical sampling methods should be used. The following techniques can be applied.

Joint Set Mapping

This method is not a true statistical technique but will yield good results if a competent geologist does the mapping. It has the virtue of being the fastest (and thereby the lowest cost) method. From fresh surface exposures, joint sets can be distinguished and a good estimate can be made of the mean attitude and spacing of the joint set.

Spot Sampling

Another method, utilized by McMahon (1967), is to measure the attitude of a number of arbitrarily

chosen joints. A minimum of 50 joints should be measured for regular joint patterns and up to several hundred for irregular joint patterns. The attitudes of the fractures are plotted on an equalarea net from which the preferred orientations are determined.

Detail Line

A less subjective method is to lay a tape along an exposure and measure the attitude of every fracture that intersects the tape. The line is made long enough for at least 100 observations to be made for each sample.

Stereoscopic Photography

If a joint or other discontinuity forms a surface on an outcrop or pit face, its attitude may be determined by the use of stereoscopic photographs taken with a phototheodolite. This method has been used in the Rio Tinto Pit slope stability study (Pentz, 1967).

The above methods require that the rock in place be examined directly or exposed to a camera viewpoint and are, therefore, limited to surface exposures and accessible underground workings. Where surface exposures and underground workings are limited or nonexistent, as in the case of new ore bodies covered with alluvium, a down hole technique is necessary.

Oriented Core

If drill core can be oriented by marking rods or by using drill hole survey equipment, the attitude of fractures in the core can be measured.

Bore Hole Camera

An alternate down hole method utilizes a Corps of Engineers type bore hole camera or a TV camera to observe the walls of a drill hole. Although the method has potential, at present it is a difficult and expensive technique yielding uncertain results.

Oriented Sample Corrections

All but the first, and under some circumstances, the second of the above sampling methods result in linear samples that have a low probability of measuring joints parallel with the sample orientation. Since any statistical analysis, including the stereographic plot, is based on the assumption that all structures have an equal chance of being sampled, the amount of this bias must be evaluated and if possible corrected. Ruth Terzaghi (1963) has proposed the "blind zone" approach where the locus of poles for joints parallel to the sample line is plotted on a stereonet. This constitutes the "blind zone" in which a joint would not be observed. This approach is particularly useful in designing a sampling program.

The observed intensity for joints intersecting the sample line at other than 90° can be converted to true intensity by applying a weighting factor.

For horizontal sample lines, the weighting factor would be as follows:

$$I = \frac{I_o}{\sin \delta \sin (|b-a|)}$$

I = intensity
 I_o = observed intensity
 δ = dip of joint
 a = strike of joint
 b = line bearing

Since the inverse sine goes to infinity as $(|b-a|)$ or go to 0, the weighting factor becomes extremely large for small values of $(|b-a|)$ and δ . It is therefore necessary to limit the weighting function. If all intercepts at less than 5° are set to 5° , the weighting factor will not exceed 100. Figure 5 shows the weighting factors for a horizontal line sample. This graph can also be used to obtain the true spacing of a joint set when the spacing is measured at an angle to the set by dividing the observed spacing by the weighting factor.

Sample Density

To maximize the information obtained per dollar invested, a sequential sampling technique should be used.

The variability of rock joint patterns from pit to pit and from place to place in a pit precludes a fixed sample density being satisfactory in all situations. Within a structural domain, a small number of widely spaced samples should be taken. These data should be plotted on a stereonet. Subsequent sampling should continue until the last sample makes no significant change in the overall pattern.

DISCONTINUITY STRENGTH

The strength, or resistance to sliding, of a geologic discontinuity is determined by the geometry of the discontinuity and the composition of the surfaces and/or filling material, if any. By mapping these properties of joints and other structures, their relative strengths may be approximated and with sufficient data collected, correlations with testing and actual field performance can be established. Therefore, in conjunction with orientation mapping, the following properties of geologic structures should be considered: planarity, continuity, termination, and type of material.

Planarity

Patton (1966) has demonstrated that the surface irregularities of a discontinuity influence the resistance to shear. In his field studies he measured the geometry of irregularities in detail. He showed in laboratory tests that the effect of irregularities could be introduced into the Coulomb equation by adding the inclination of the irregularities to the shear angle

$$\tau_f = c - \sigma \tan(\phi - i)$$

This system may be appropriate for major structures, but is probably not practical for joint surveys. The broad classification of planar, wavey or irregular joint traces may be more satisfactory (Fig. 6).

Continuity

Considering a potential failure surface parallel to a discontinuity, the amount of intact rock would be a measure of the cohesion. Karl Terzaghi (1962) pointed out that it is impractical to measure continuity defined as

$$C = \frac{A_f}{A}$$

C = continuity

A_f = area of fracture surface

A = total area of section

because the total fracture surface cannot be observed. By measuring the length of the trace of a discontinuity on an exposed face, a one dimensional estimate of continuity can be obtained.

Fracture Termination

The manner in which a discontinuity terminates is a factor which should be considered. A joint which terminates against a cross structure is less likely to propagate than one which tails out into the rock. Three types of termination can be recognized: simple termination in rock, termination against a cross structure and horsetailing, Fig. 6.

Fracture Filling

As can be seen in Fig. 7, the coefficient of friction varies widely with the mineralogy of the surfaces. Thus, data on wall rock and filling composition are necessary.

The effect of water on shear strength is handled in soil mechanics by Terzaghi's Effective Stress Principle whereby the normal stress is reduced by the pore pressure.

$$\tau_f = C + (\sigma_n - uh) \text{TAN } \phi$$

u = weight of water

h = head of water

The low porosity and permeability of a rock mass make it difficult to obtain good data on water pressure in rock slopes. Water is unquestionably a factor, however. Hammel (1967) has shown a significant statistical correlation between rainfall and movement in his analysis of an open pit slide.

Laboratory direct shear tests of soil and of rock have shown that the shear strength of a sample under a constant normal load varies with the shear displacement (Bjerrum, 1954) (Patton, 1966). Typically, the shear resistance increases rapidly with displacement, reaching a peak then dropping to a residual value. This makes displacement monitoring of pit slopes important from the safety standpoint because the drop in strength will result in acceleration of the slide unless the displacement also changes the driving forces.

The friction angle ϕ has also been found to vary with the normal stress. For a limited range of normal stress the Coulomb equation is satisfactory, however.

STRESS

The stresses at a point in an open pit slope are the result of the weight of the overlying material, regional tectonic stresses and stresses induced by the configuration of the pit. Determination of the stress distribution of a slope with irregular geometry and varying elastic properties is an extremely difficult problem. Including the effects of the nonelastic behavior of a discontinuous rock mass makes the problem even more formidable. However, analytical techniques such as the finite element analysis and physical model studies show considerable promise in providing reasonable estimates (Goodman and Taylor, 1966). Also the recent proliferation of in situ stress measurement instrumentation (Griswald, 1963) provides a means of actual measurement.

CLASSIFICATION OF FAILURES

In order to differentiate between various slide mechanisms it is useful to classify slope failures.

Although there is a large variety of slides, the following classification used by Lacy (1963) is adequate to cover most open pit failures.

1. Ravelling--individual rock fragments falling or rolling down a slope steeper than the angle of repose. "Running ground" where many fragments are involved would fall in this category.
2. Translational or Plane Shear--sliding of a more or less intact rock mass on one or more geologic discontinuities.
3. Rotational Shear--rotational failure along an arcuate surface. This occurs in soils and incompetent rock where the shear strength of the rock is close to that of the discontinuities.

APPLICATION TO PIT SLOPES

The following is a discussion of how some potential failure conditions can be recognized on the basis of geologic information, and slope angles can be chosen to avoid obvious slope failures.

The first step is to divide the pit area into structural units. All too often when adverse conditions result in a failure in one portion of a pit, all the slope angles are reduced even though other parts of the pit are in an entirely different rock and could stand at a much steeper slope.

Next the probable failure type must be determined for each structural unit, including the major discontinuities.

The importance of major structures cannot be overemphasized. The majority of pit slope failures are associated with a fault or other discontinuity that has been (or should have been) known from conventional geologic mapping. No amount of detailed statistical analysis of minor discontinuities is a substitute for knowing the position and physical characteristics of major geologic structures.

Plane Shear Failures

If the rock of a slope is competent but is transected by a number of discontinuities, plane shear is the most probable failure. In this case rigid body mechanics often can be applied to obtain a reasonable estimate of stability. In this technique, which has been described by Wittke (1963) and by Goodman (1966), the forces acting on the block (primarily the weight of the block) are resolved into normal and shear forces on the potential failure surfaces. The shearing resistance is computed using the Coulomb equation and the stability is determined by comparing the shearing forces with the shearing resistance.

Defining the failure surfaces for this type of analysis requires examination of the geology. The following are some common types of potential failure conditions.

Type 1a----Major Discontinuities Dipping Into Pit

This is the simplest and most obvious potential failure condition. If the slope is reasonably straight or convex in plan and the structure more or less parallel to the slope, a wedge or rock will be "daylighted" if the slope is steeper than the discontinuities. As can be seen in Fig. 10, when the discontinuity is at a high angle the driving forces are much greater than the normal forces on the discontinuity. Hence the discontinuities must have considerable cohesion to keep the wedge from sliding. Since major discontinuities have little cohesion, it is apparent that to be stable the slope must be flatter than the discontinuities. For low angle discontinuities the normal forces are high compared to the driving forces; therefore, a flat discontinuity would be stable regardless of the slope angle, unless water pressure greatly reduces the effective normal force or the discontinuity is part of a compound failure where lateral thrust is developed.

A "daylighted" wedge should be stable if the discontinuity is dipping at less than the angle of internal friction and should slide if above the angle of internal friction, provided cohesion and water pressure are minimal. In the intermediate range of dip (25° to 40°) some knowledge of the shear strength of the structure is necessary to determine if failure will occur.

Where the slope is straight or convex in plan and the "daylighted" wedge is thin compared to its length, the end conditions do not contribute significantly to the resistance to sliding. Usually there are high angle structures at right angles to the pit face, which will define the ends of the wedge.

Type 1b----Minor Structure Dipping Into Pit

When there is a minor structure, such as a joint set, dipping into the pit, the situation is similar to the case of the major structure, with these exceptions:

1. The shear strength will tend to be higher, particularly if the jointing is discontinuous; thus "daylighted" wedges will be stable at higher angles.
2. The jointing will be repetitive and the exact position of the failure cannot be predicted although the probability of failure may be the same.
3. The jointing will tend to vary in attitude so that picking an angle which will not "daylight" a wedge is more difficult. This is one reason why the probability of failure approach is necessary. If the scatter of pole points on the stereonet is converted to a frequency curve relative to dip, it can be seen that no single attitude will describe the orientation. As the slope angle is increased into the range of the joint set, there will be an increasing probability that a "daylighted" wedge will fail.

Type 2----Intersecting Discontinuities

Two nonparallel discontinuities striking at an angle to the pit face and dipping in opposite directions can form a tetrahedron that is free to slide into the pit. The direction and dip of the line of intersection will be the direction of motion. This intersection can be found by descriptive geometry, stereographic projection or calculation. The dip of the intersection will always be less than the dip of the flatter of the two structures.

The discontinuities can be either major structures such as faults, minor structures such as joints, or any combination.

As with single structures, steeply dipping intersections represent a maximum slope angle. For lower angle intersections the stability can be calculated by using the vector techniques described by Wittke (1963) and by Goodman (1966).

Type 3----Intersection With Toe Discontinuity

In a pit slope transected by discontinuities in a number of orientations, compound failure surfaces can develop.

Intersections dipping steeper than the pit slope should not constitute a failure condition unless a third discontinuity forms a surface along which the intersecting wedge can slide into the pit. This third discontinuity can be flat or even dipping into the slope, because the upper part of the failure block (unit No. 1 in Fig. 13) exerts a lateral force on the lower unit.

Type 4----Intersection With Headwall Discontinuity

Commonly the upper portion of an intersection failure will be defined by a high angle structure. Lateral thrust developed by the upper segment can result in failure of an intersection that would otherwise be stable.

Additional Types

Discontinuities dipping into the pit may also be combined with toe and headwall discontinuities in the same manner as the intersection failure. Additional, more complex configurations can occur with combinations of discontinuities but the principles used in defining the failure surface and establishing the stability are the same as the simpler types.

Ravelling and Rock Falls

If a slope is steeper than the angle of repose of the rock making up the slope, loose rock will fall or

roll down the slope. The angle of repose is the maximum slope of a pile of loose fragments and is approximately equal to the angle of internal friction of the rock. Angularity, grading and surface roughness effect the angle of repose. Lacy (1965) has measured the angle of repose of rocks in Arizona open pit porphyry copper operations. Values ranged from 33° to 42° .

Relatively continuous, closely spaced discontinuities in two or more orientations will produce loose rock. The prevalence of these conditions may be established by geologic mapping.

If it is determined that rockfall conditions could exist in a pit slope, catch benches, presplit blasting and other control measures should be planned. Ritchie (1963) has studied rockfall in highway cuts and has developed design criteria which can be applied to open pit slopes.

Rotational Failure

A material that can be considered approximately isotropic and homogeneous or with simple horizontal layering will fall along a circular arc. Soils and very soft rock fall in this category. A number of workers in soil mechanics, in particular Taylor and Bishop, have developed methods of computing the stability of a slope for this type of failure. Stability is based in the magnitude of the weight moment around the center of the circle compared to the cohesive and internal friction moments along the failure surface. These solutions can be obtained in standard soil mechanics references.

Structural units composed of soils (gravels, silts, clays) and soft rock, such as poorly consolidated shales or deeply weathered intrusives, should be analysed by the slip circle method. The basic criteria is that the material is sufficiently uniform and isotropic that conventional soil testing will give a satisfactory approximation of the strength.

CONCLUSION

The design of open pit slopes is an interdisciplinary problem involving mining economics, geology and rock mechanics. As with any interdisciplinary problem an important aspect is proper communication. The mining engineer and geologist working together can apply principles and techniques developed by rock mechanics research to determine potential slope-failure conditions. Although much more research is needed in the mechanics of slope failure and techniques for designing slopes, it is possible to choose slope angles that will minimize the occurrence of hazardous and expensive slope failures, but without unnecessarily flattening pit slopes. Also mine layout can be modified to reduce the effect of a failure on a mining operation.

Slope design should be a continuing process whereby design slopes are reevaluated throughout the life of the mine as new data are collected.

Case history studies are needed as a check on design techniques and to provide information for future design. (Back analysis of slides is an excellent way of obtaining discontinuity strength data.) Case histories need not be failures; a stable high angle slope is in many ways as significant as a failure. A large measure of the success of earth slope design is due to case history studies. Rock slope design can also profit from such studies.

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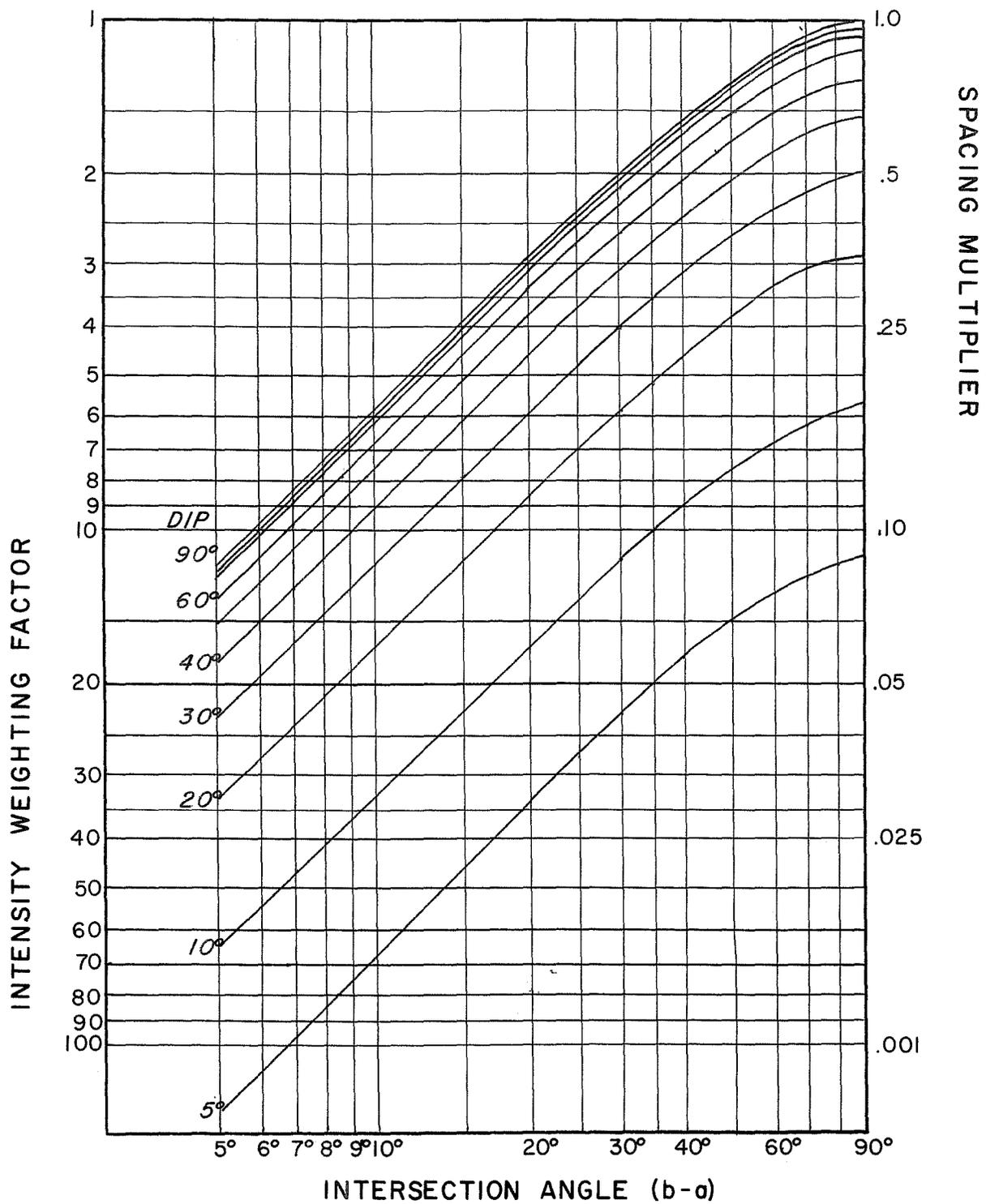
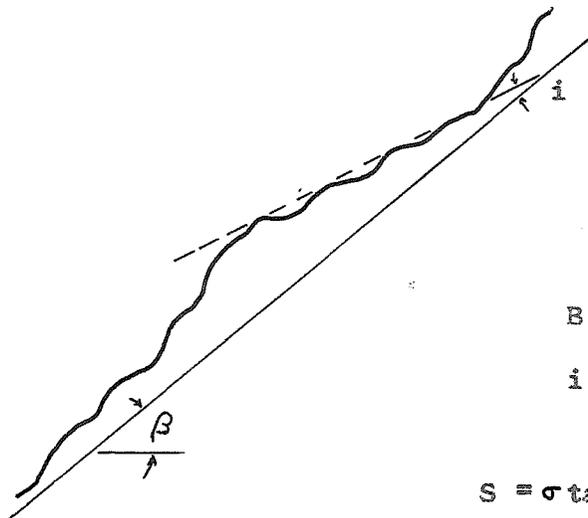


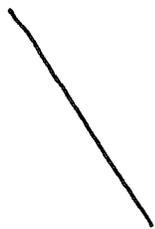
Fig. 5 Intensity and Spacing Corrections for Horizontal Linear Samples



B = average dip
 i = angular deviation

$$S = \sigma \tan(\phi + i)$$

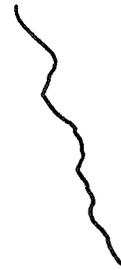
ANGULAR DEVIATION OF FIRST-ORDER IRREGULARITIES (after Patton 1966)



planar



wavy



irregular

PLANARITY



termination
 in rock



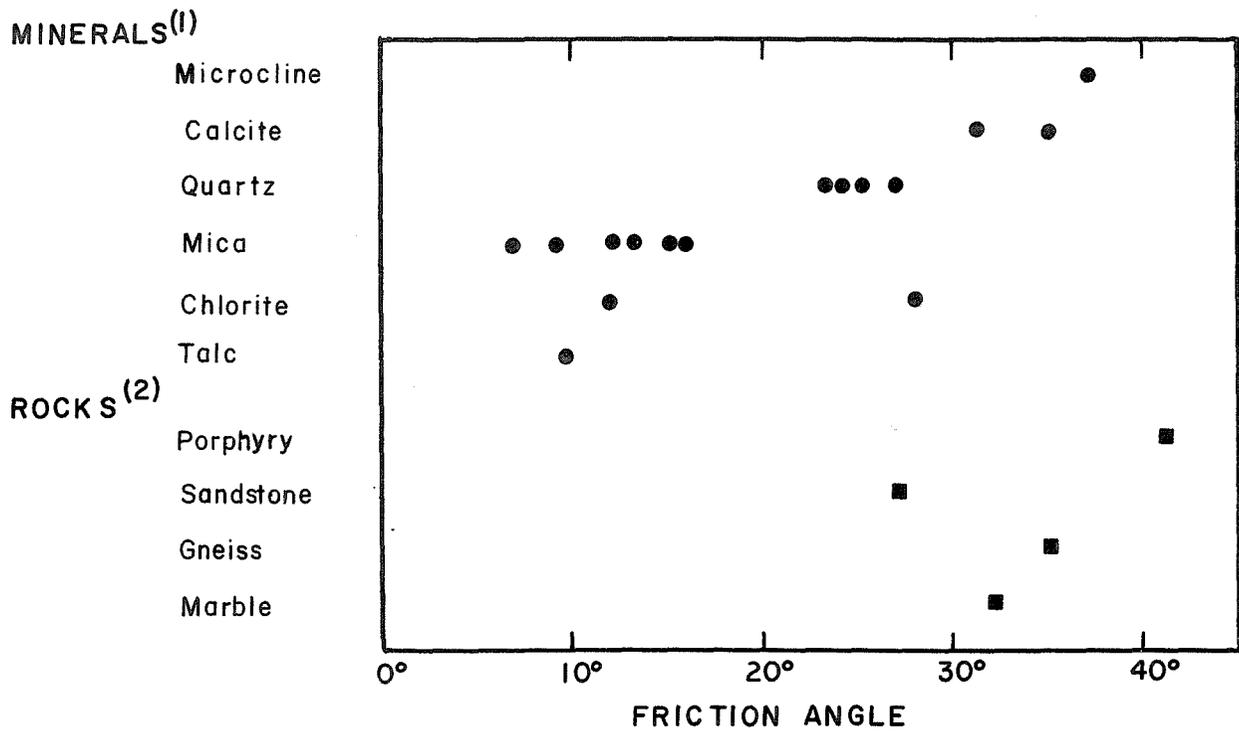
termination against
 cross structure



horsetailing

TERMINATION

Fig.6 Geologic discontinuity geometry



(1) Horn and Deere (1962) static friction from direct shear tests of saturated smooth surfaces.

(2) Jaeger (1959) triaxial tests of natural shear failure surfaces.

FIG. 7 FRICTION ANGLE FOR VARIOUS SURFACES

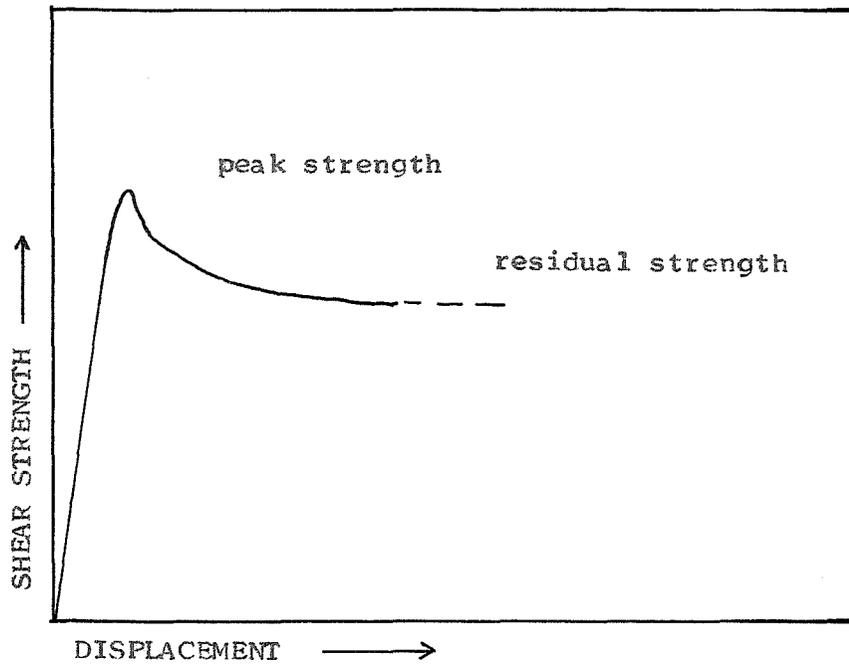
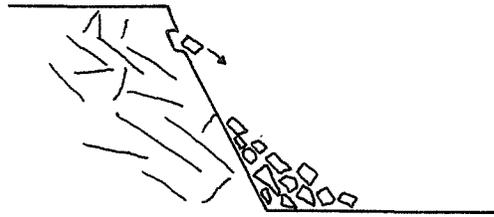
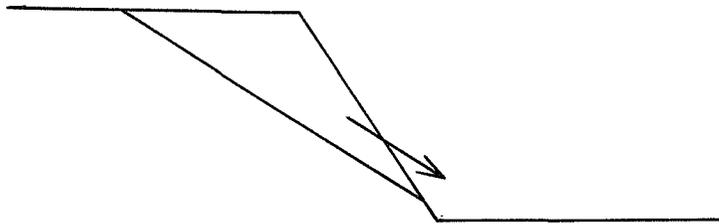


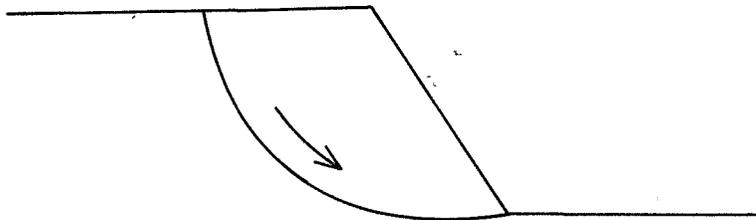
Fig. 8 Shear Strength Vis Displacement



RAVELLING



TRANSLATIONAL FAILURE



ROTATIONAL FAILURE

(after Lacy 1963)

Fig. 9 Classification of Slope Failure

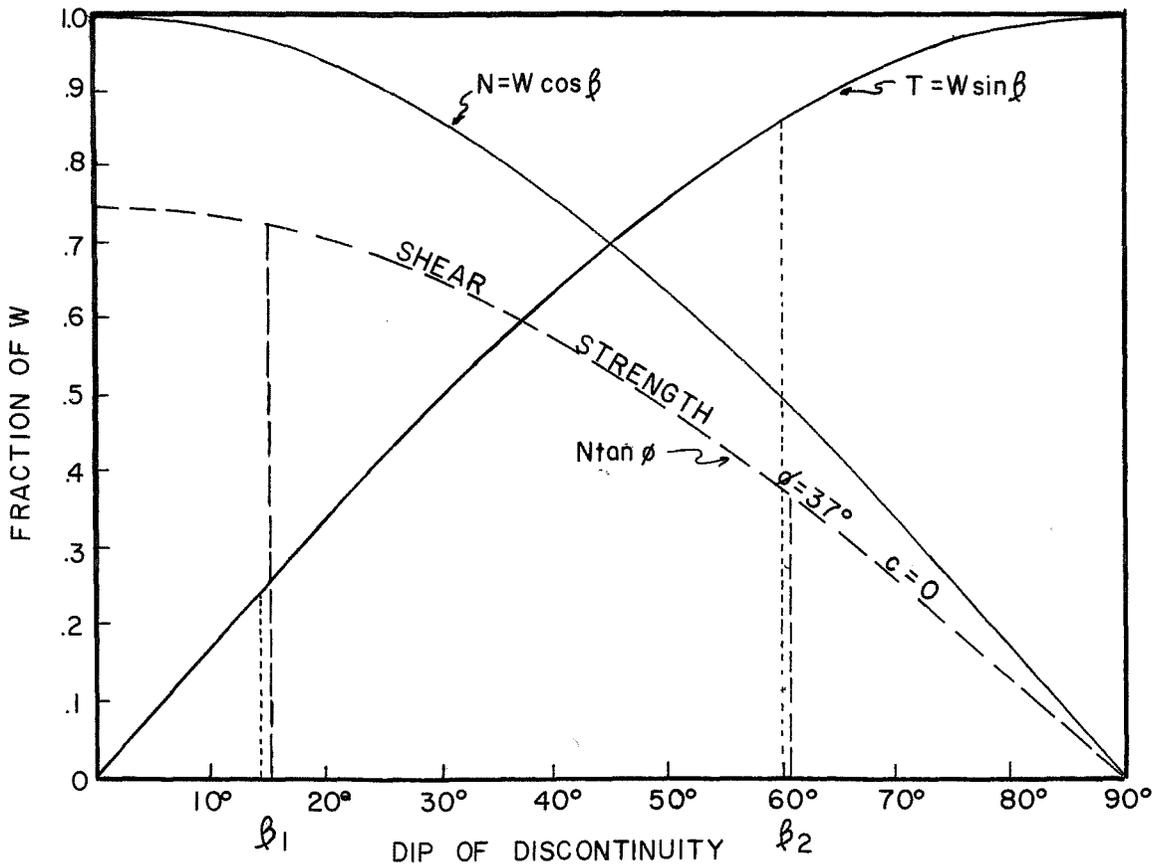
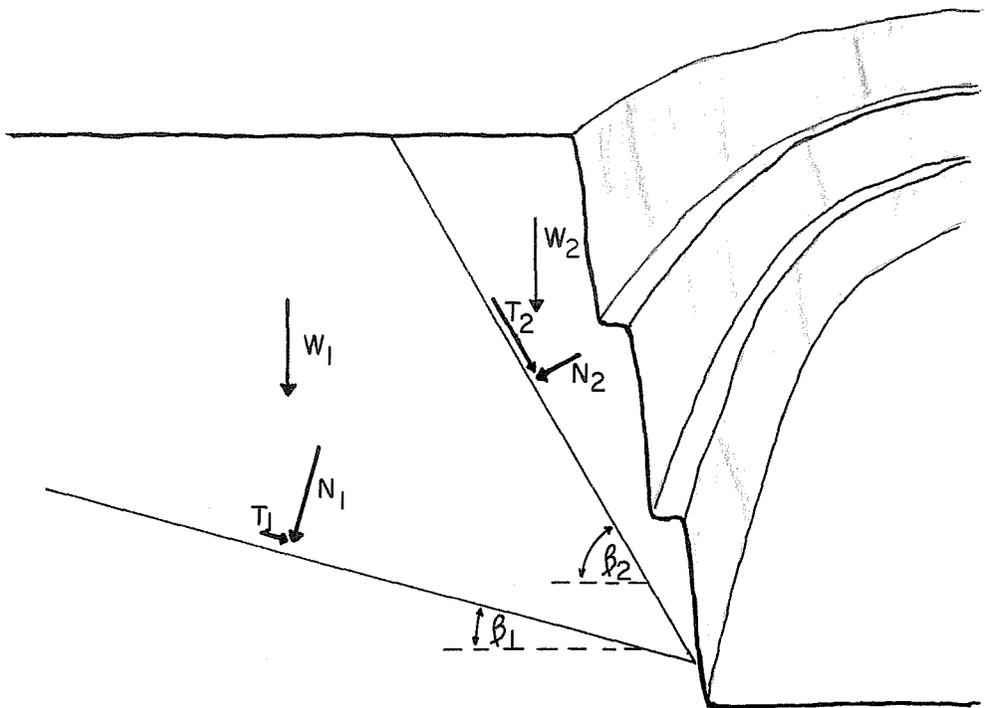
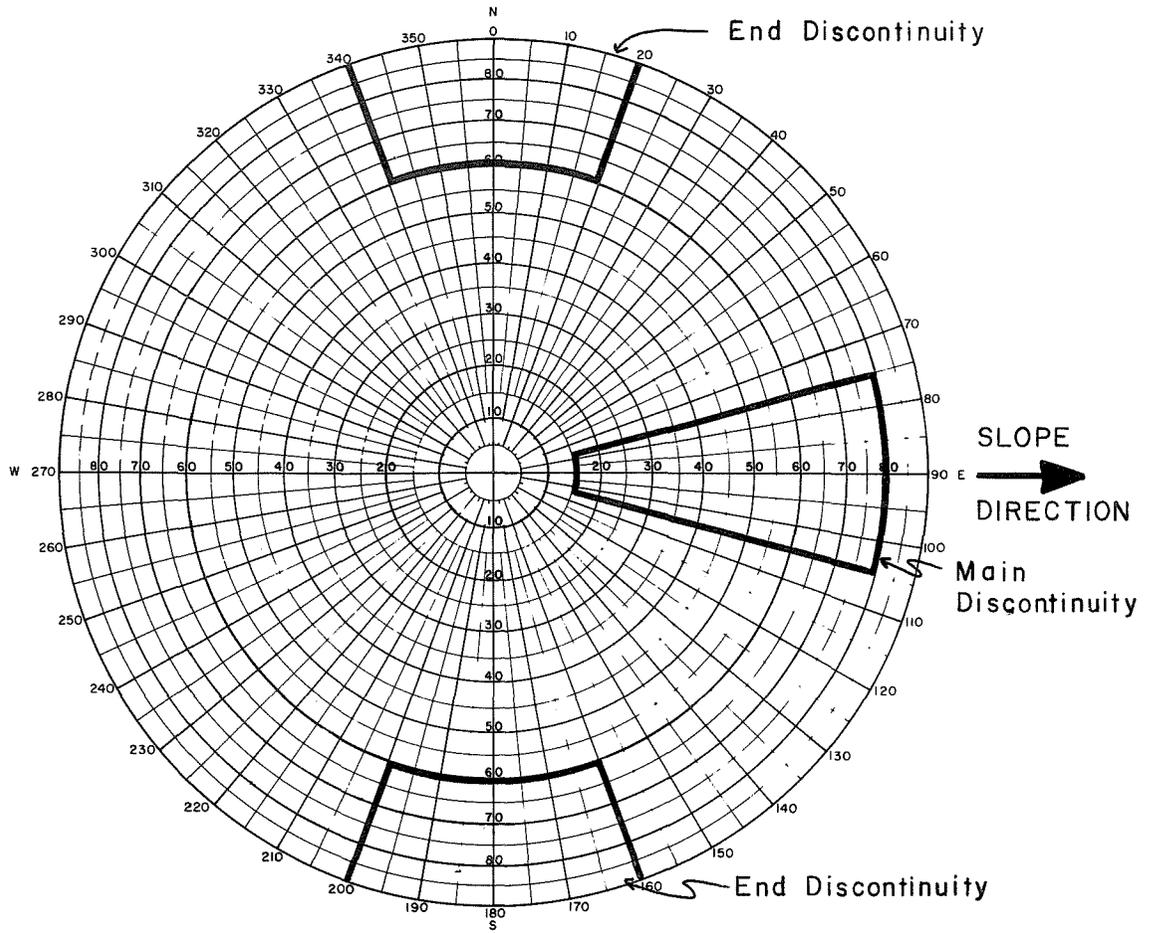


Fig. 10 Stability of Slope vis Dip of Discontinuity for simple Rigid Block Slides



UPPER HEMISPHERE PROJECTION OF DISCONTINUITY ATTITUDE RANGE

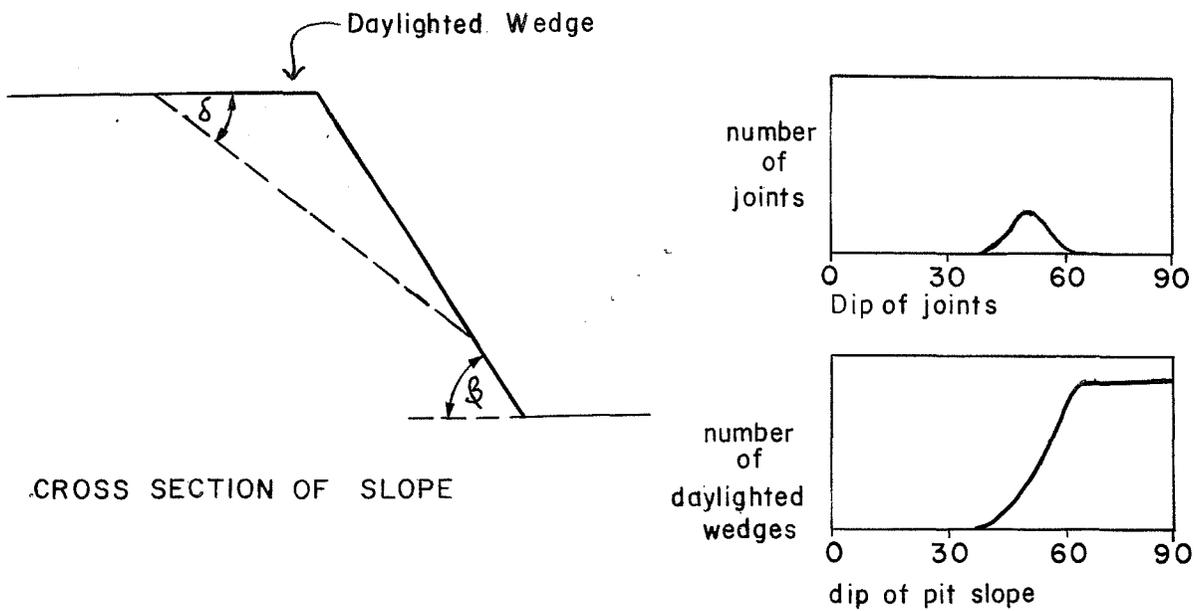
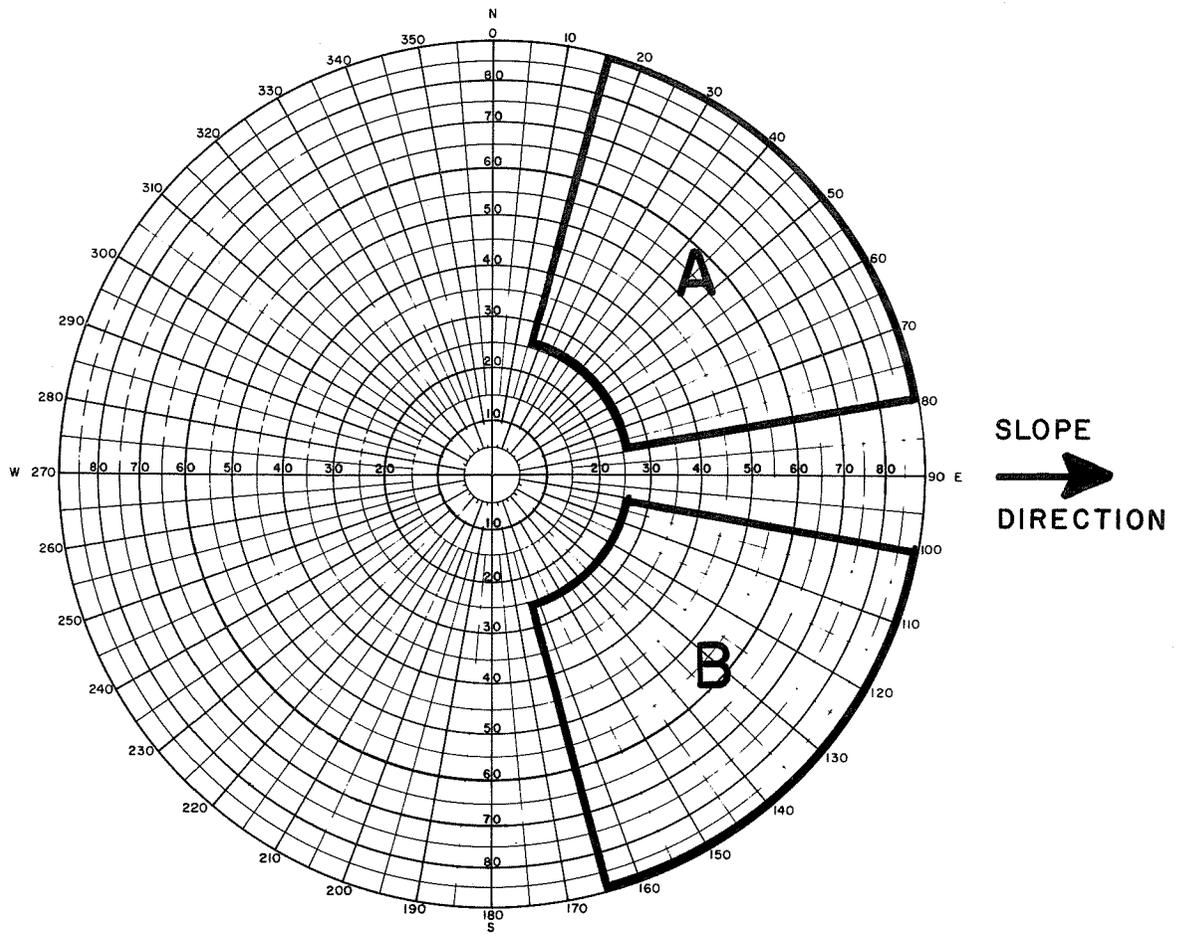
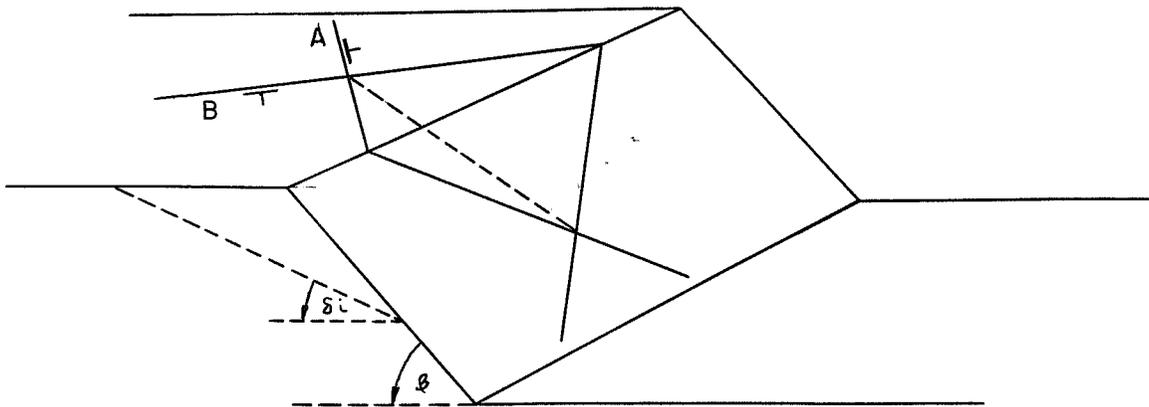


FIG. II DISCONTINUITY DIPPING INTO PIT

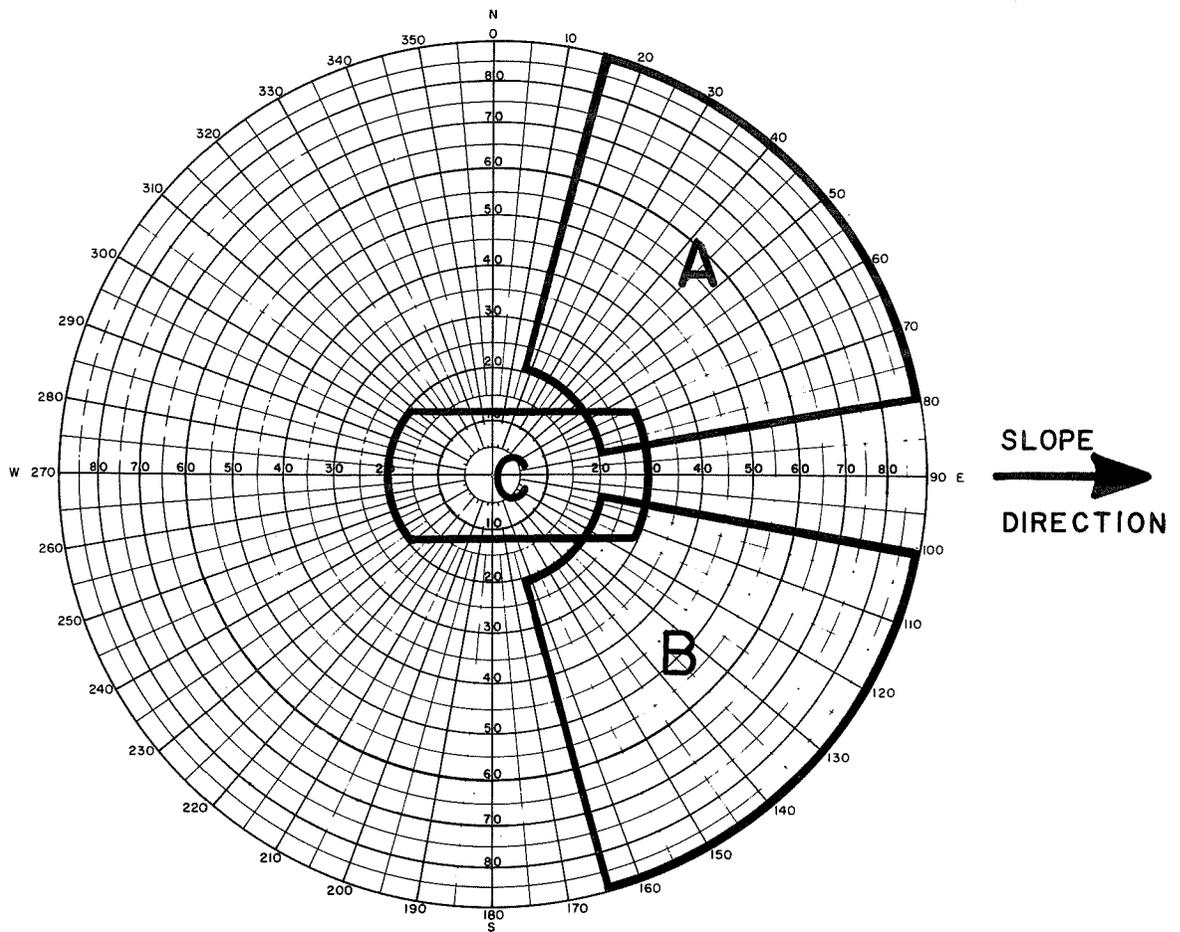


UPPER HEMISPHERE PROJECTION OF DISCONTINUITY ATTITUDE RANGE

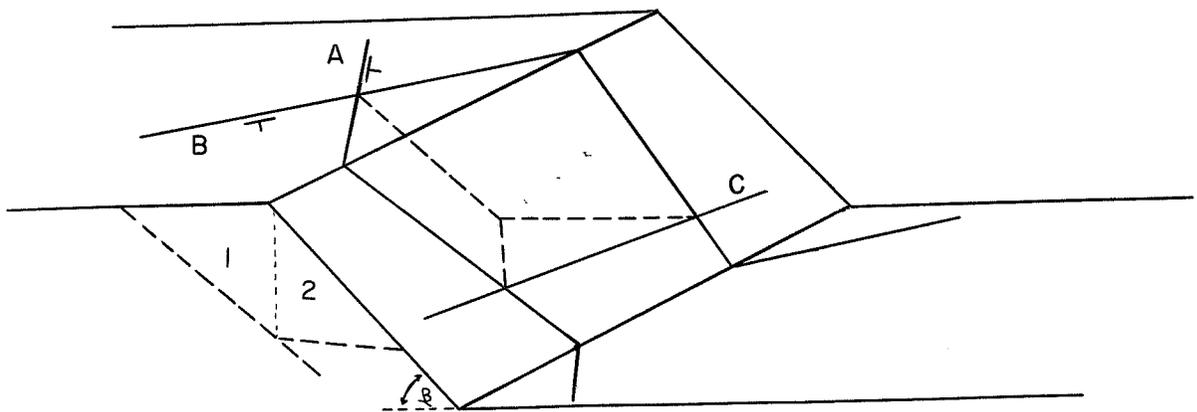


ISOMETRIC SKETCH OF FAILURE GEOMETRY

FIG. 12 SIMPLE INTERSECTION OF DISCONTINUITIES

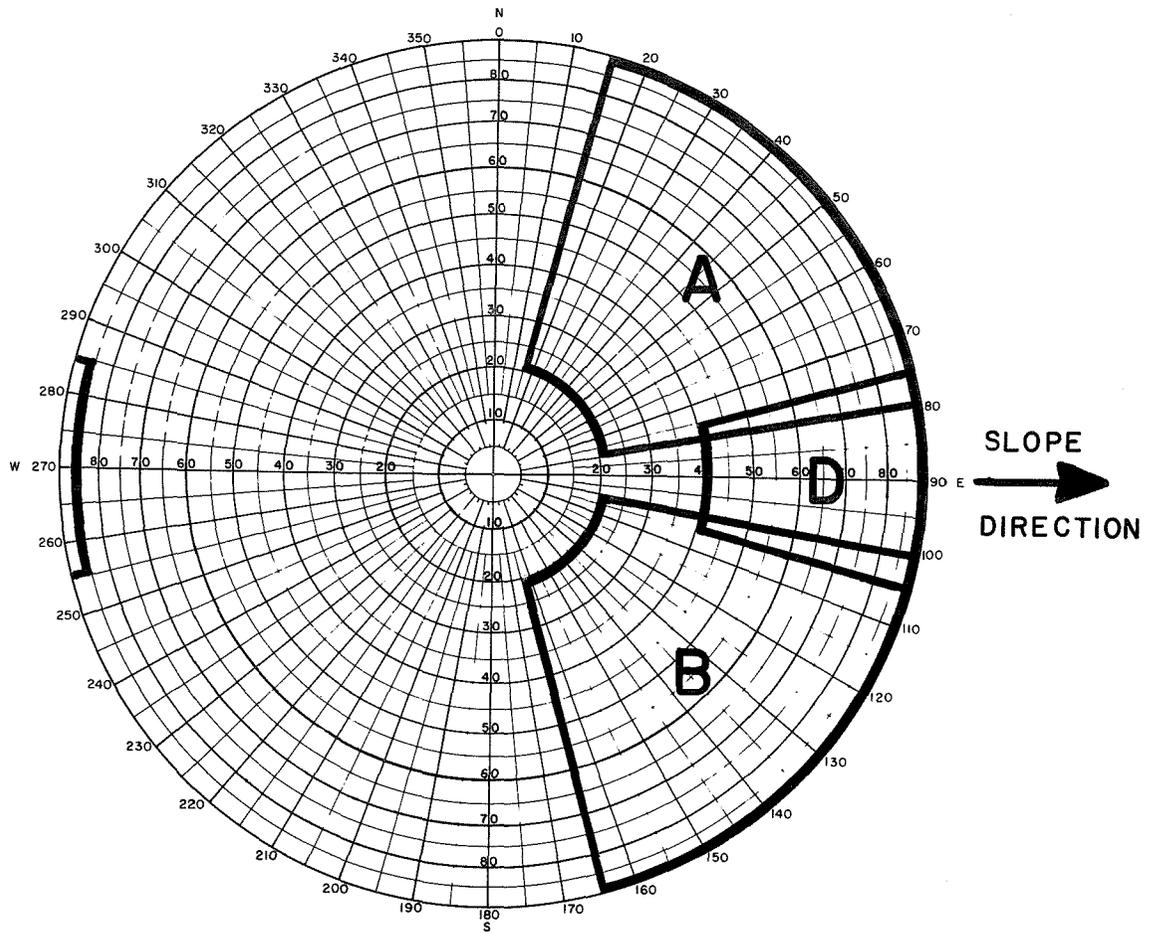


UPPER HEMISPHERE PROJECTION OF DISCONTINUITY ATTITUDE RANGE

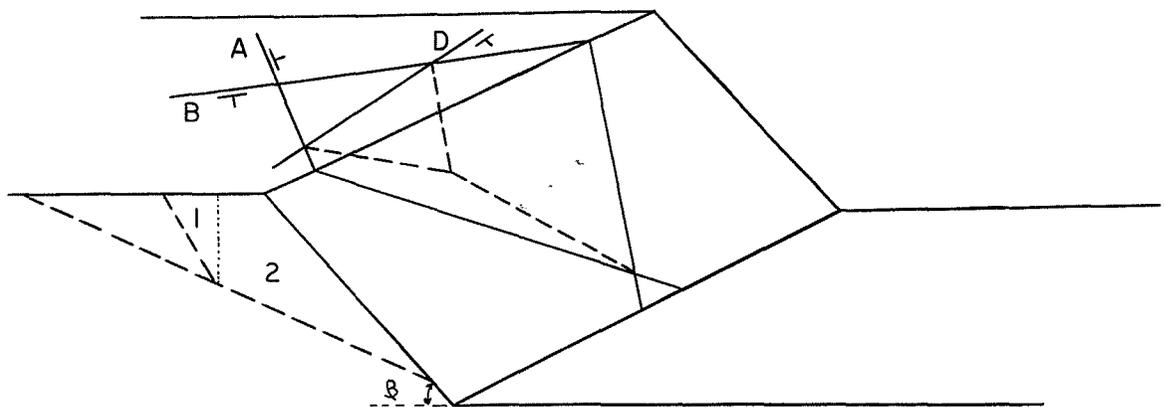


ISOMETRIC SKETCH OF FAILURE GEOMETRY

FIG. 13 INTERSECTION WITH TOE DISCONTINUITY

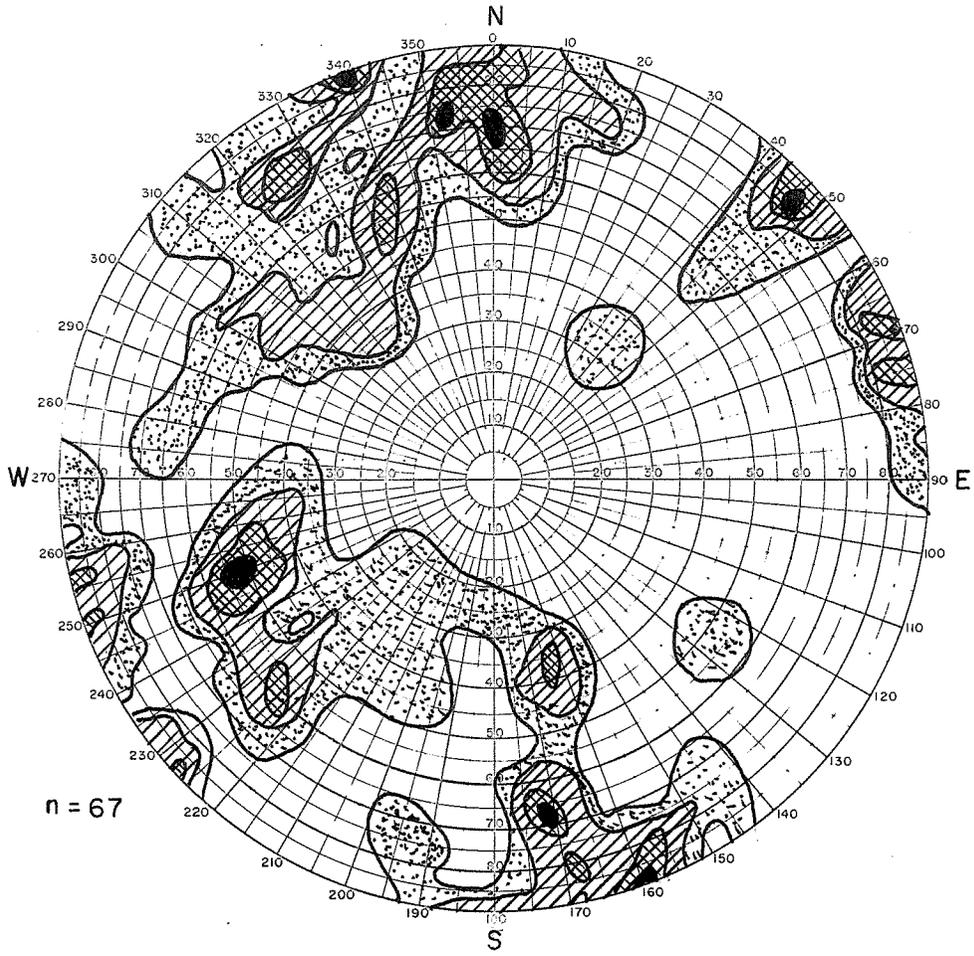


UPPER HEMISPHERE PROJECTION OF DISCONTINUITY ATTITUDE RANGE



ISOMETRIC SKETCH OF FAILURE GEOMETRY

FIG. 14. INTERSECTION WITH HEADWALL DISCONTINUITY



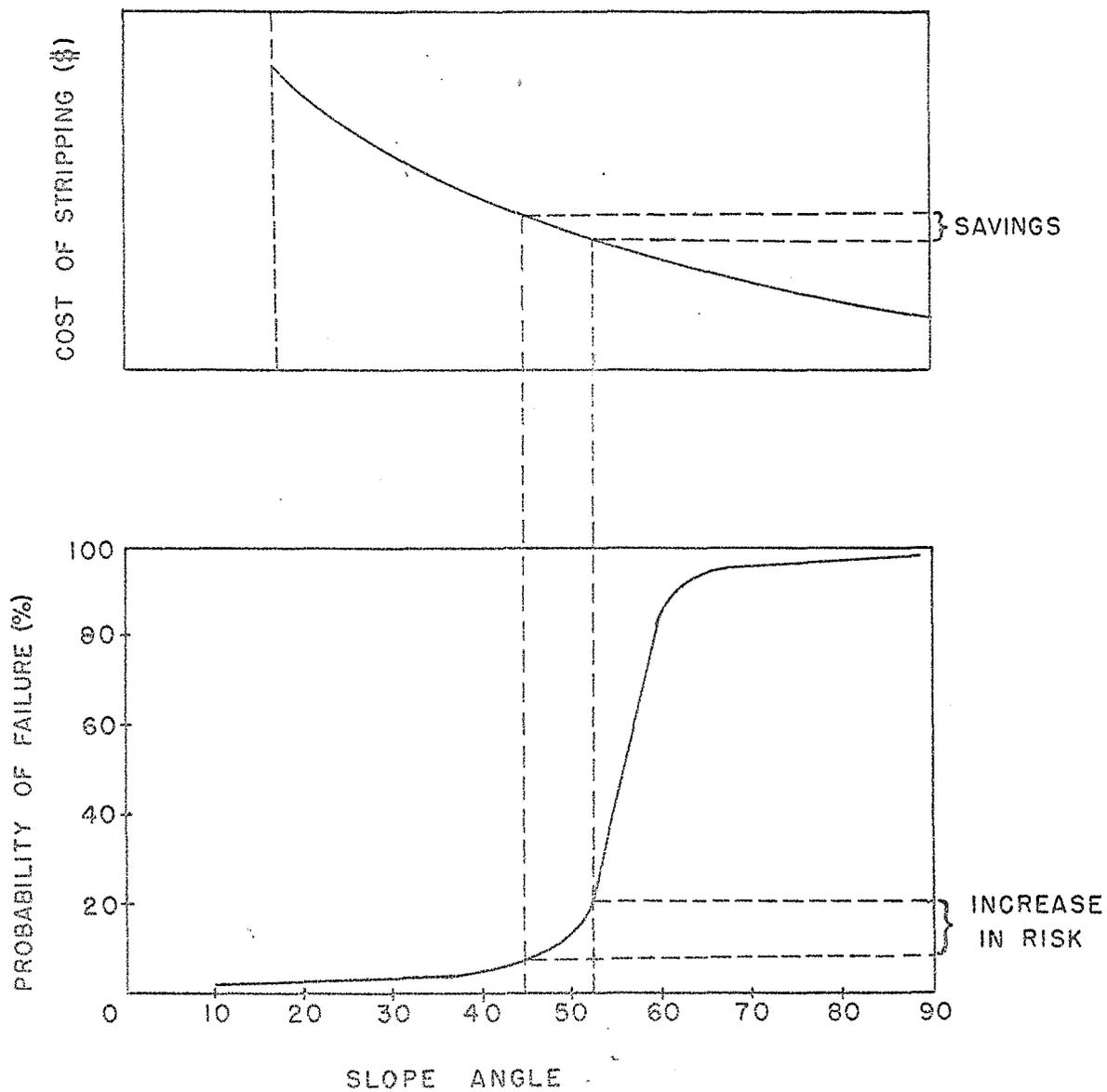
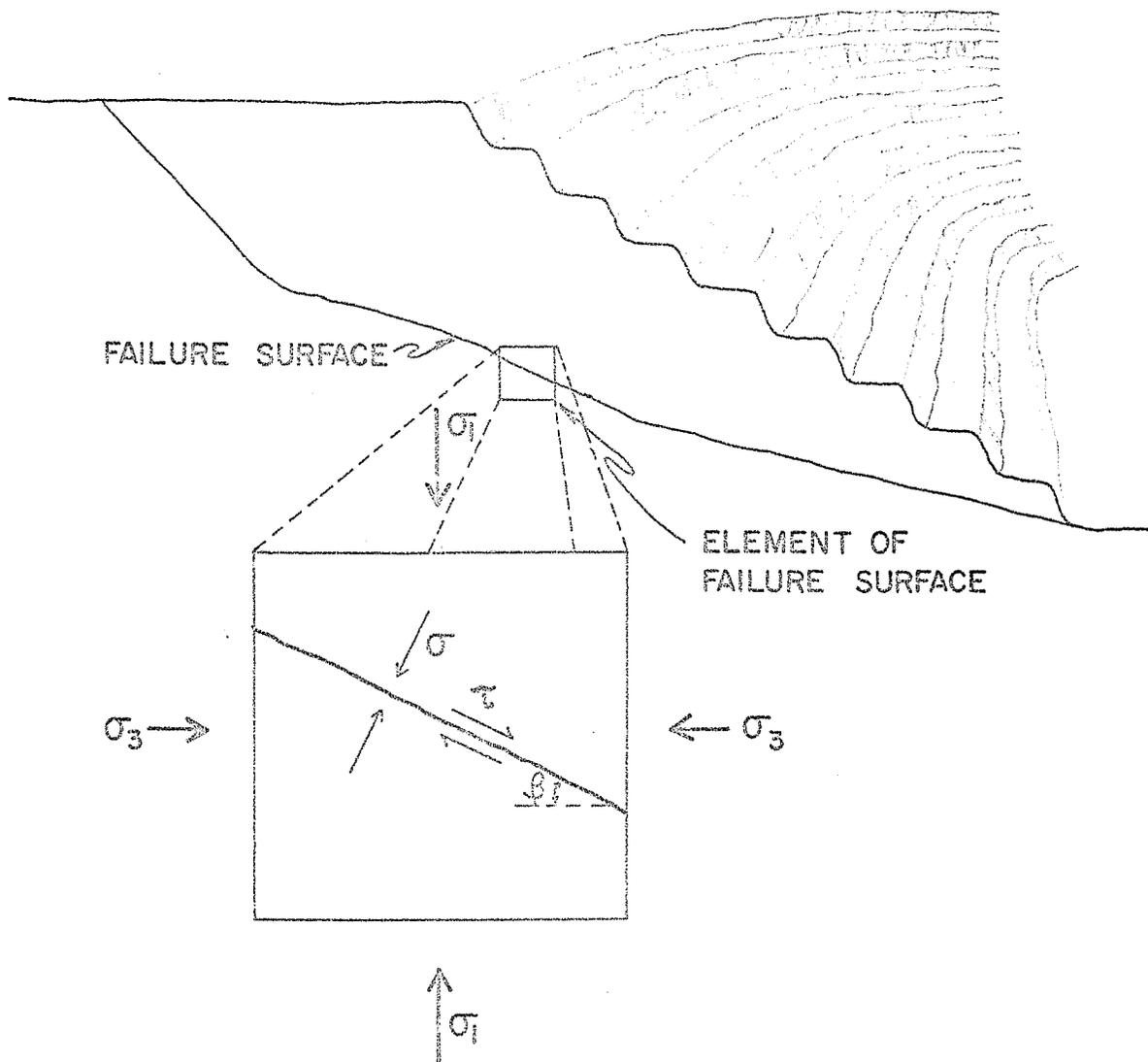


Fig. 1 Hypothetical Probability of Failure Curve



$$\text{FACTOR OF SAFETY} = \frac{\text{Shear Strength}}{\text{Shear Stress}} = \frac{f(\sigma, c, \phi)}{\tau}$$

Fig. 2 Theoretical Stability Conditions

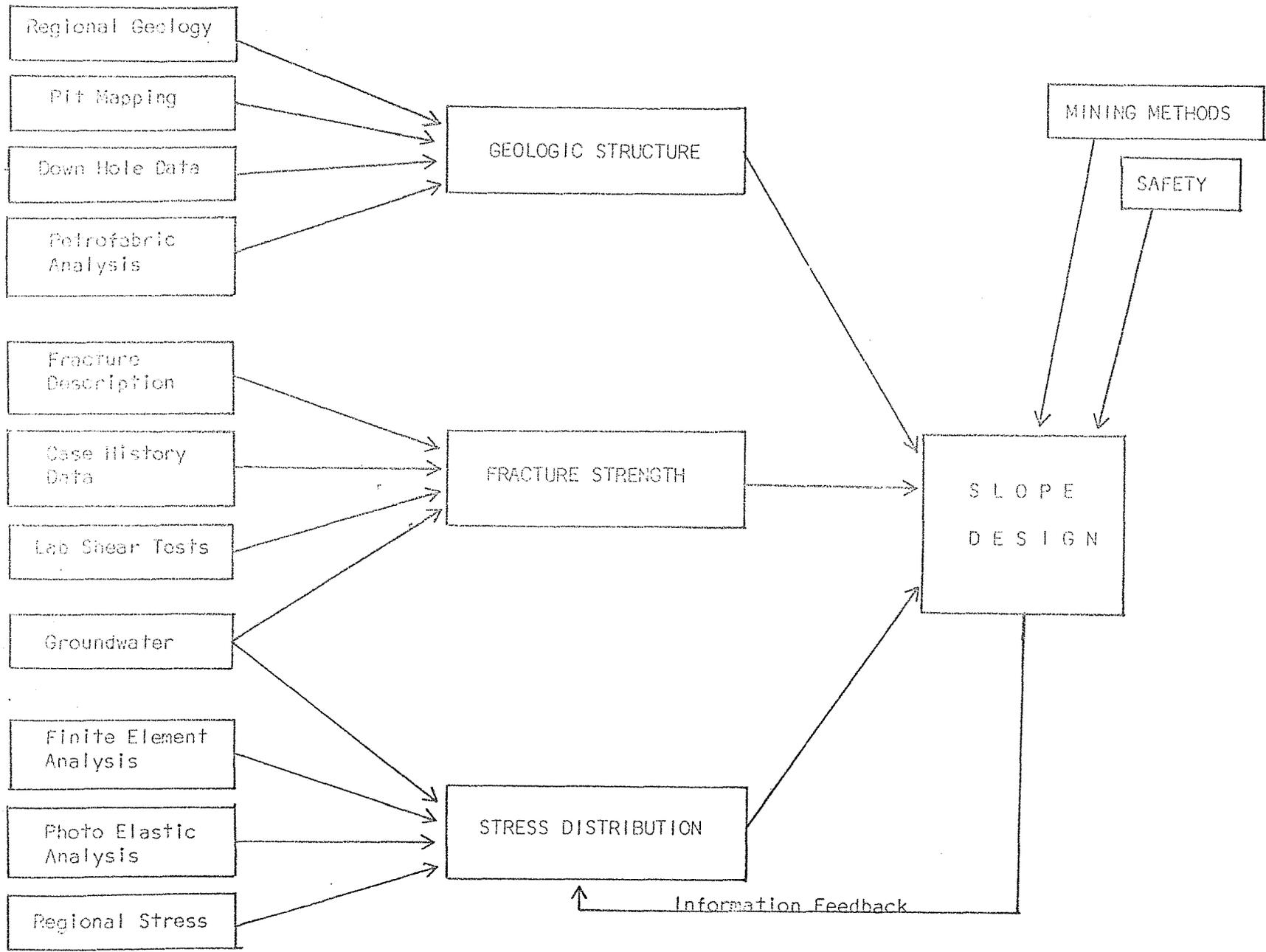


Fig. 3 Information Input for Slope Design

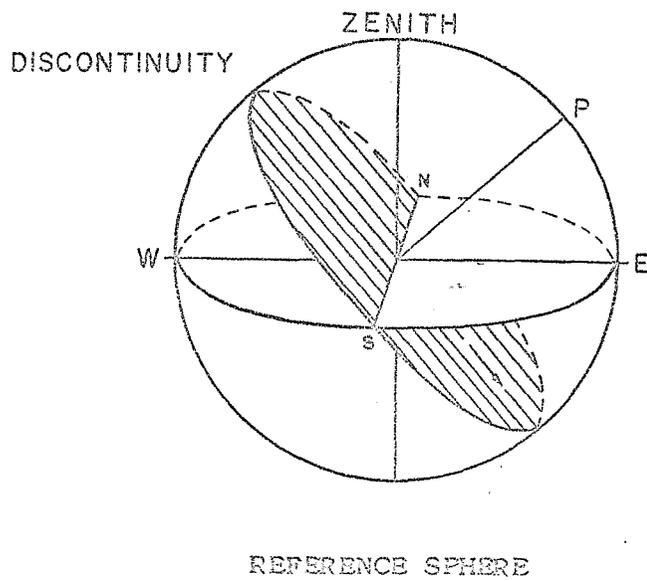
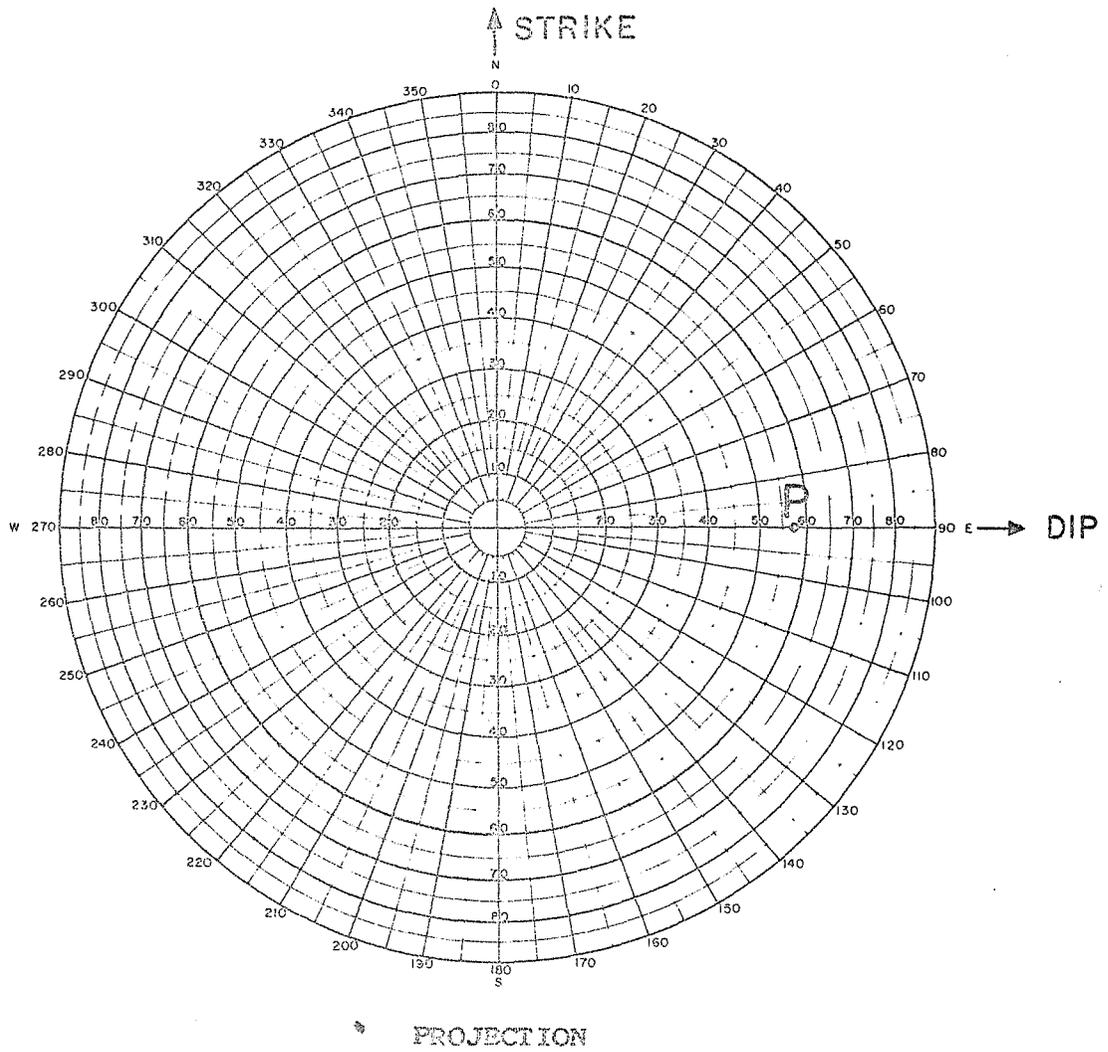


Fig. 4 Stereographic Projection

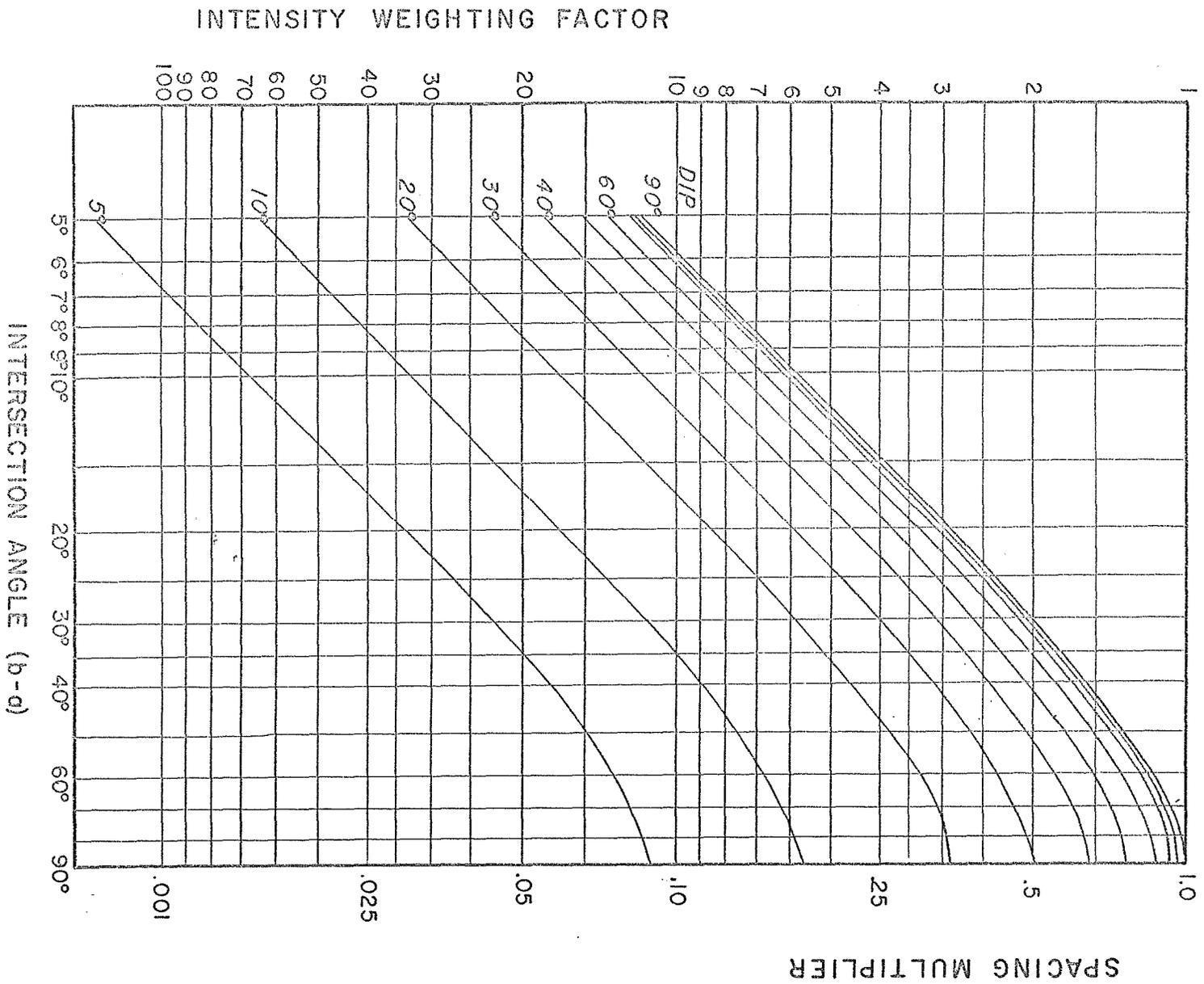
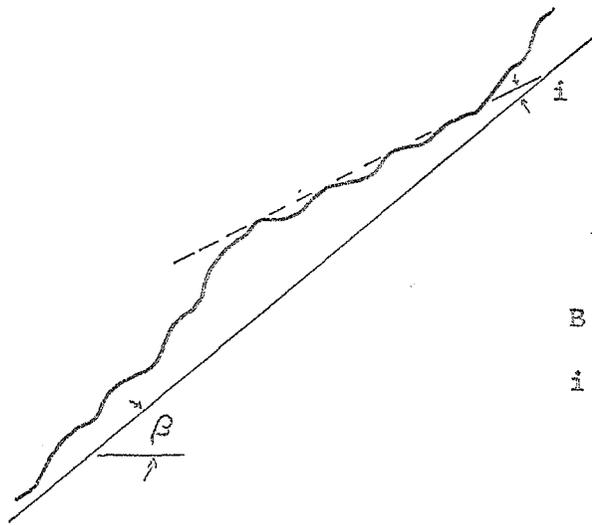


Fig. 5 Intensity and Spacing Corrections for Horizontal Linear Samples



B = average dip

i = angular deviation

ANGULAR DEVIATION OF FIRST-ORDER IRREGULARITIES (after Patton 1966)



planar



wavy



irregular

FLANARITY



termination
in rock



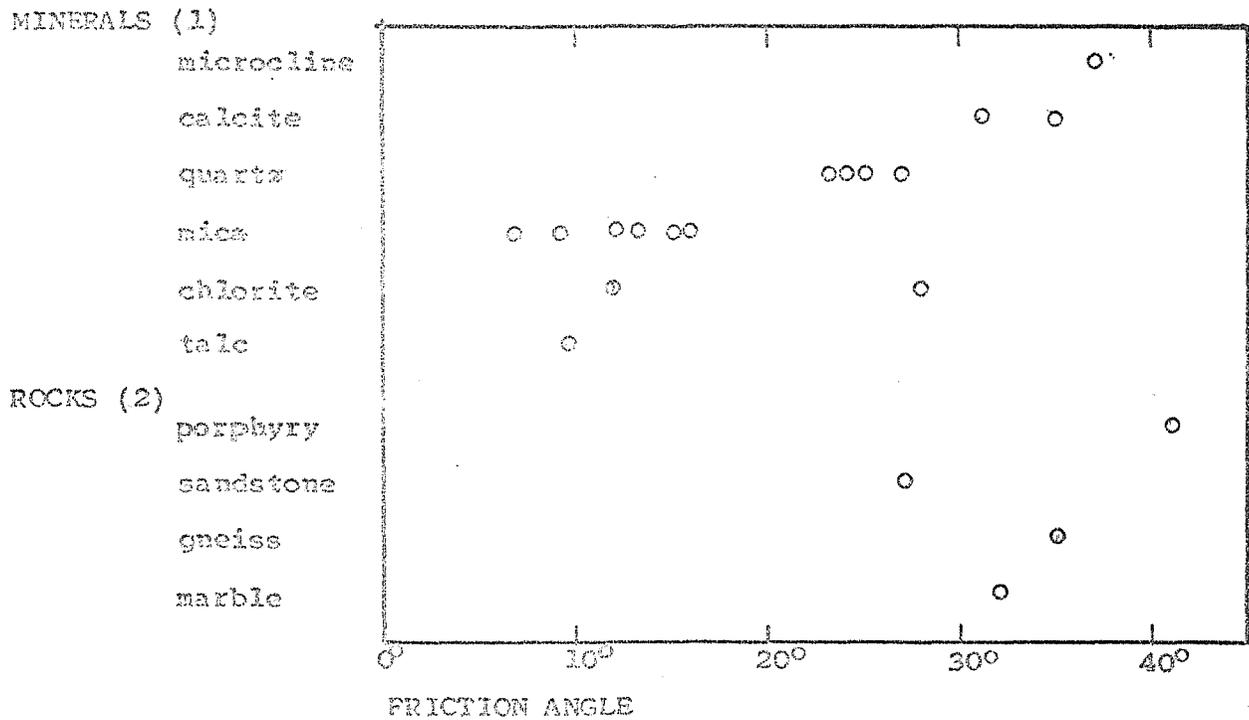
termination against
cross structure



horsetailing

TERMINATION

Fig.6 Geologic discontinuity geometry



1. Horn and Deere (1962) static friction from direct shear tests on saturated smooth surfaces.
2. Jaeger (1959) triaxial test of natural shear failure surfaces.

Fig. 7 Friction Angle for Various Surfaces

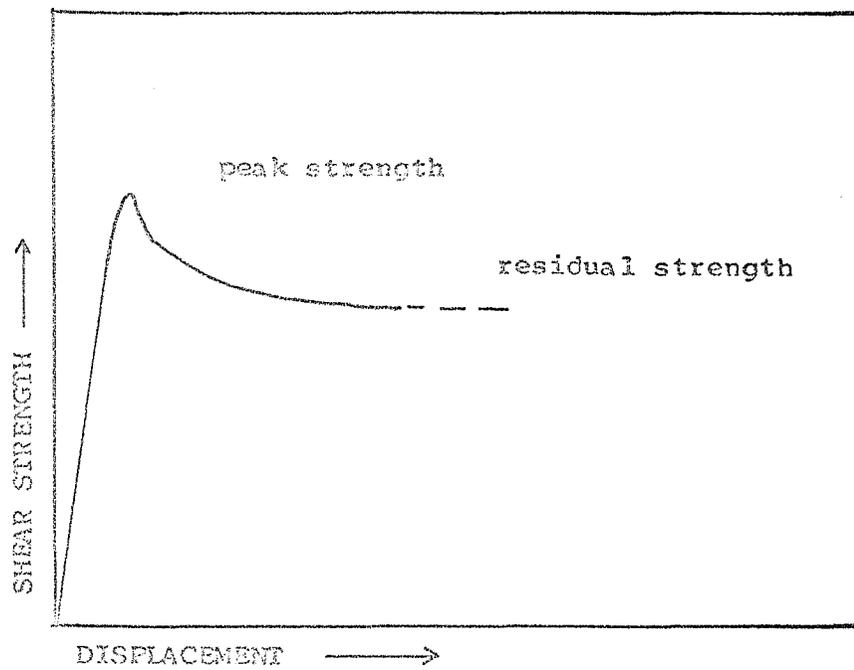
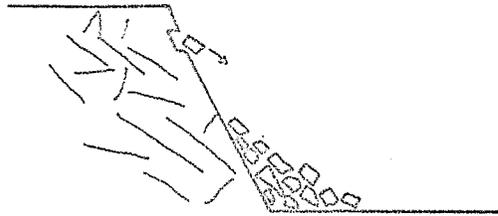
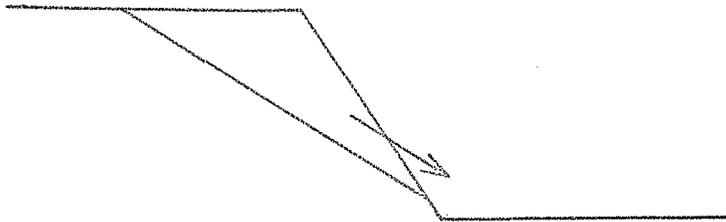


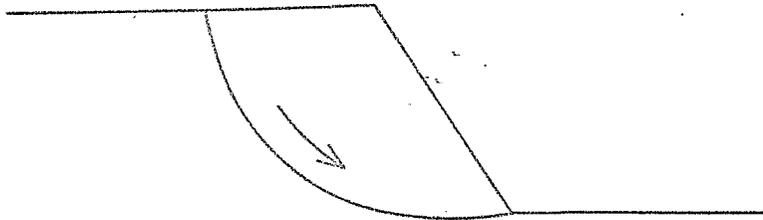
Fig. 8 Shear Strength Vis Displacement



RAVELLING



TRANSLATIONAL FAILURE



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(after Lacy 1963)

Fig. 9 Classification of Slope Failure

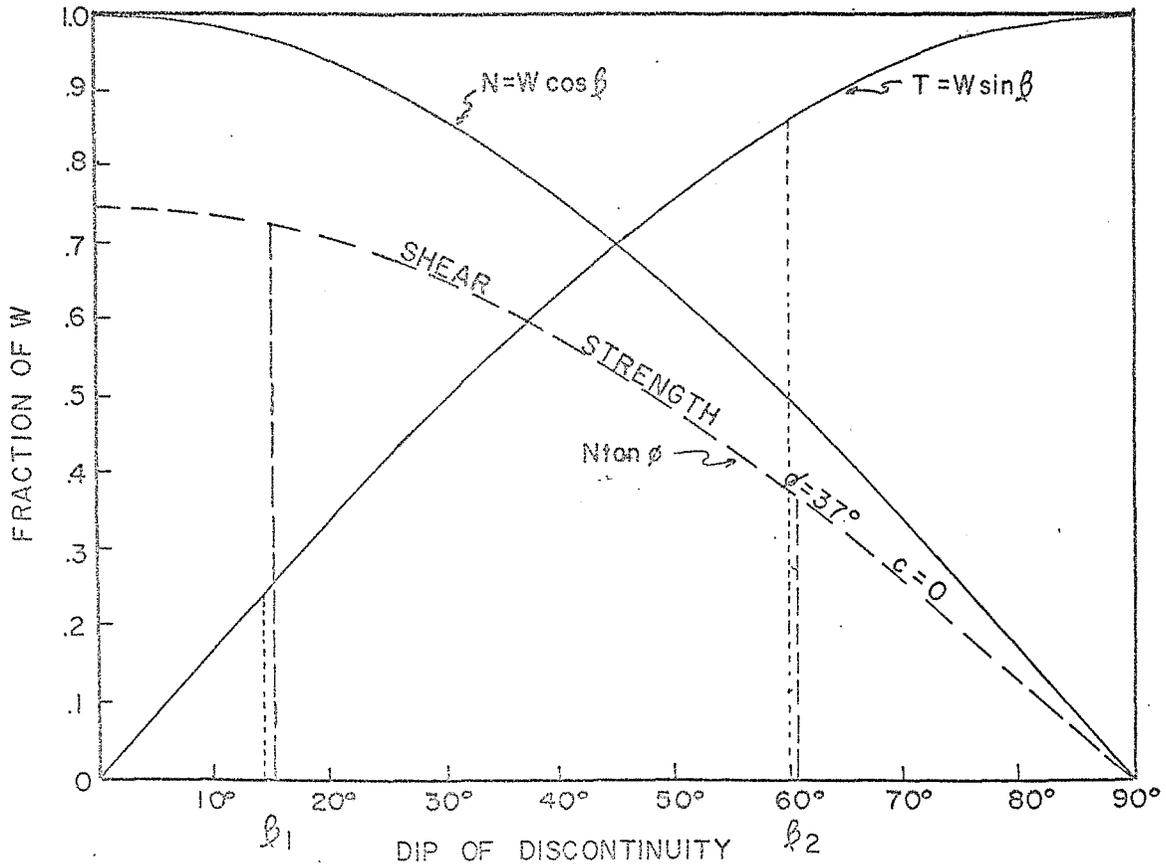
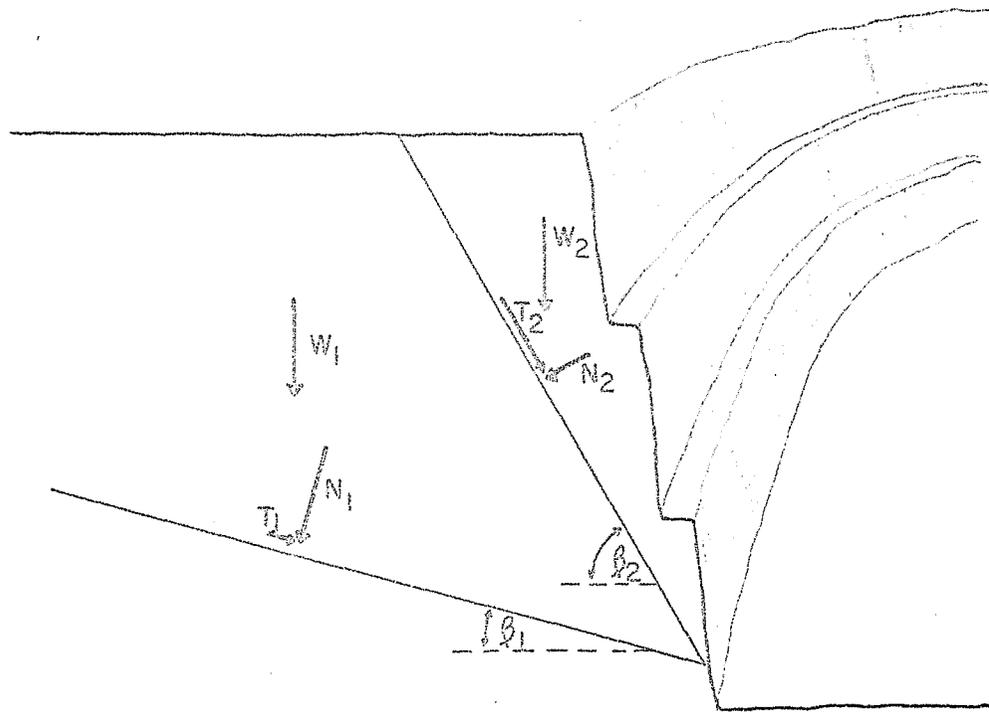
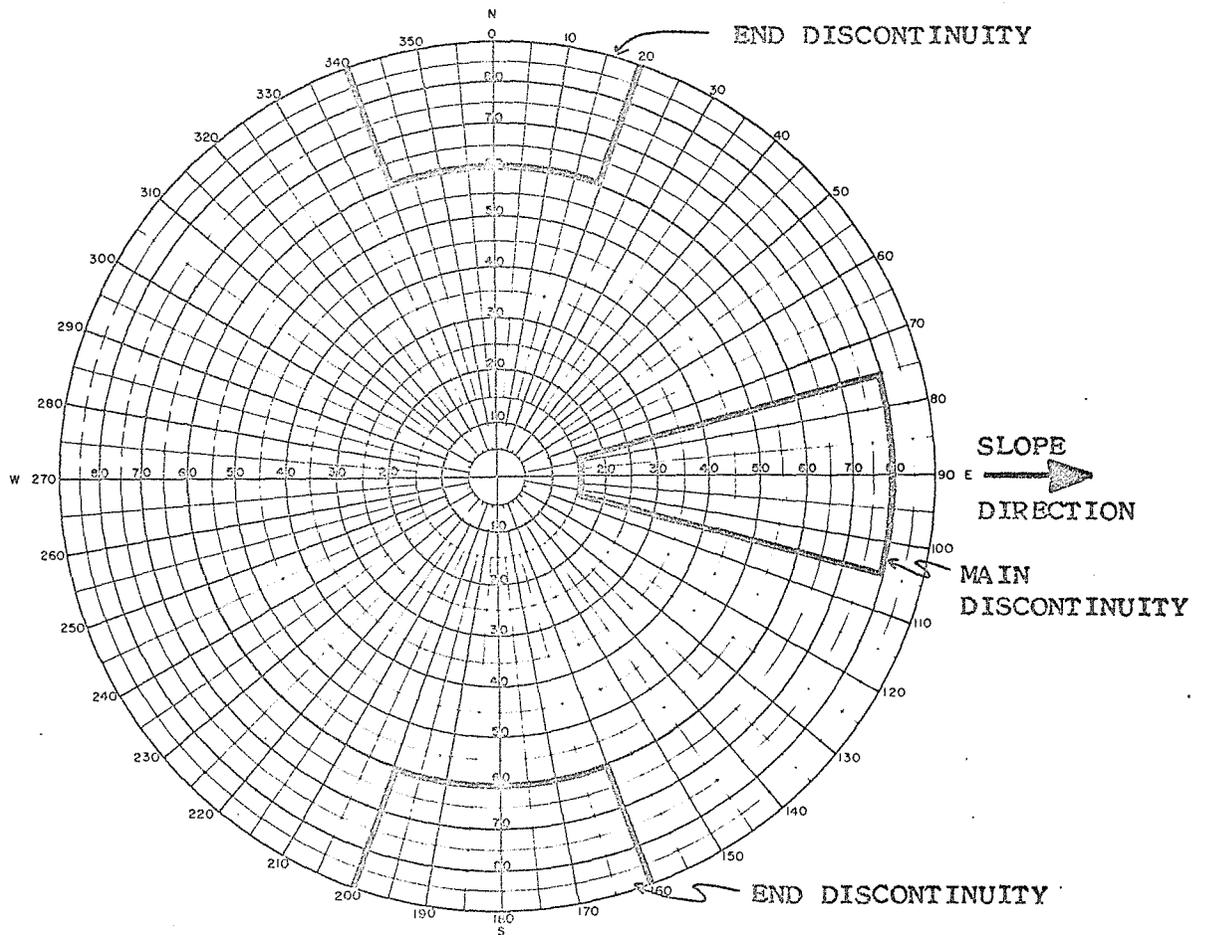
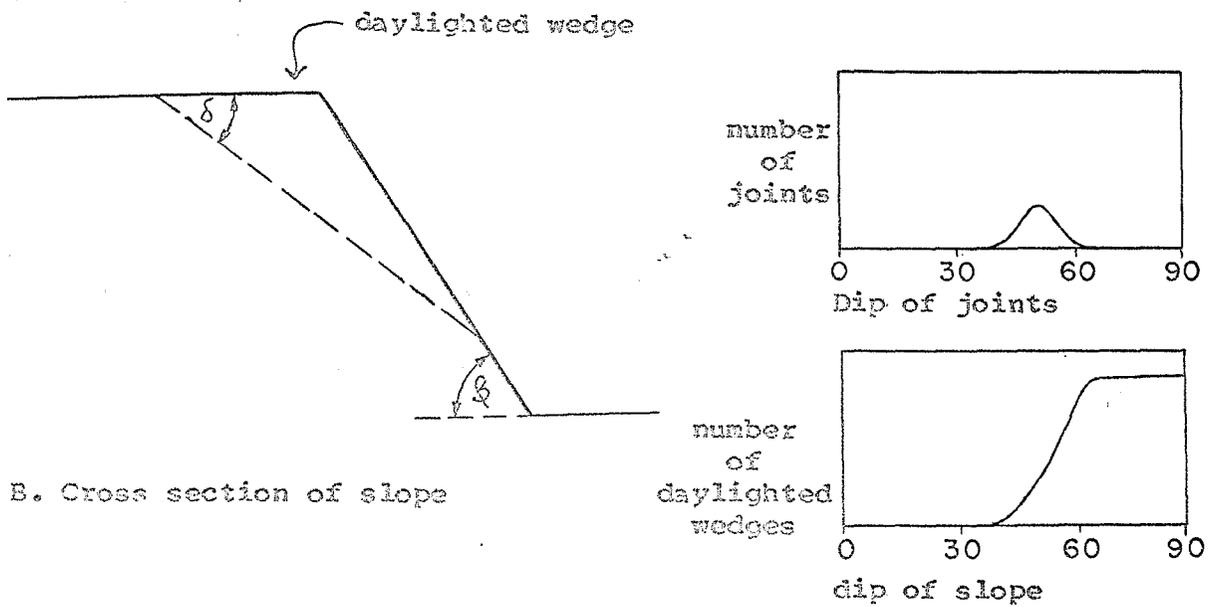


Fig. 10 Stability of Slope vis Dip of Discontinuity for simple Rigid Block Slides

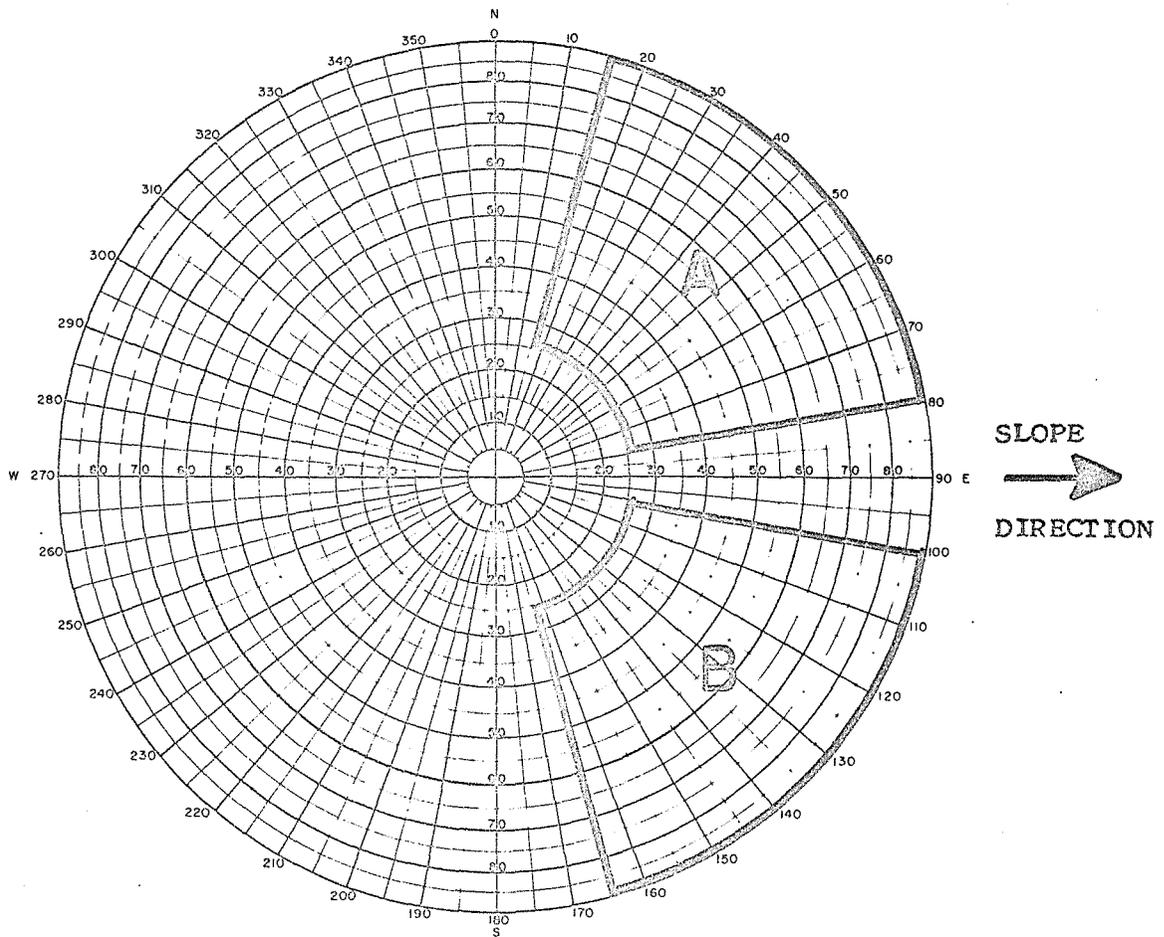


A. Upper Hemisphere Projection of the Attitude Range of Discontinuities

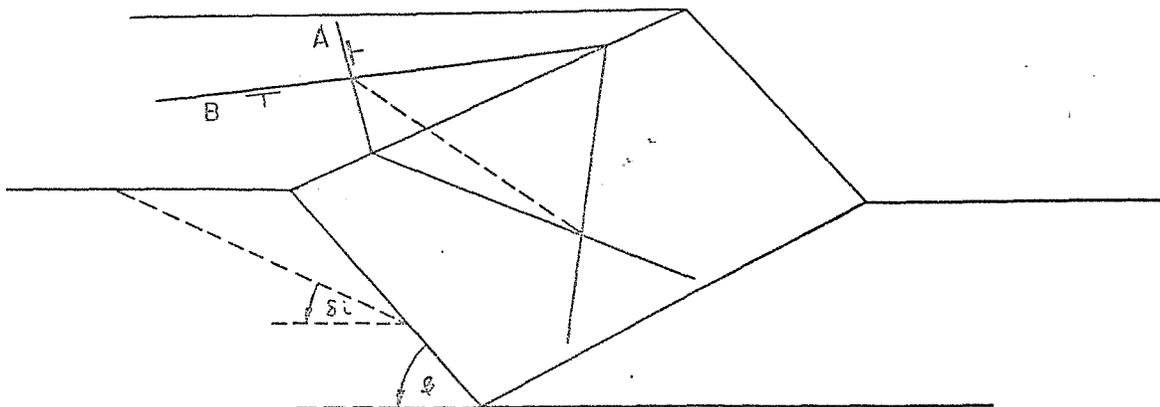


B. Cross section of slope

Fig. 11 Discontinuity Dipping into Pit

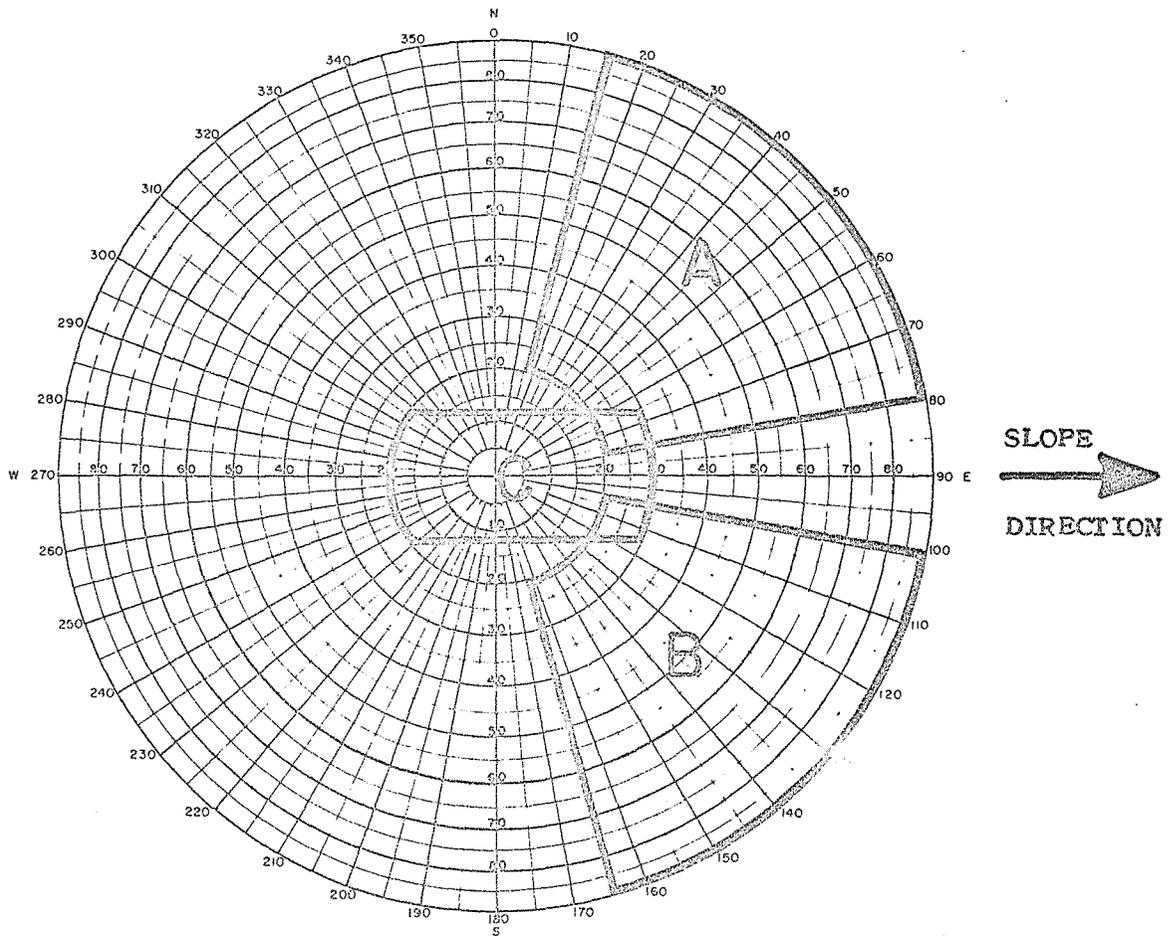


A. Upper Hemisphere Projection of the Attitude Range of Discontinuities

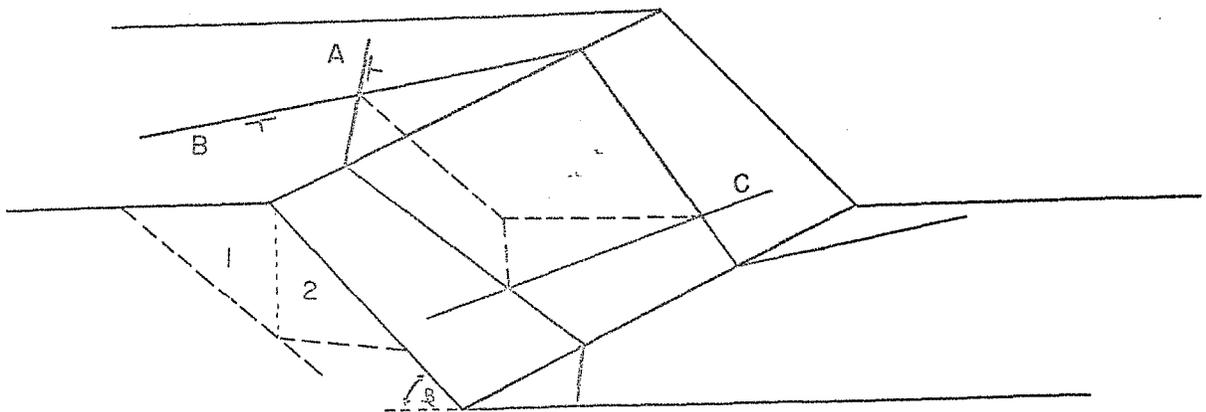


B. Isometric Sketch of Failure Geometry

Fig. 12 Simple Intersection of Discontinuities

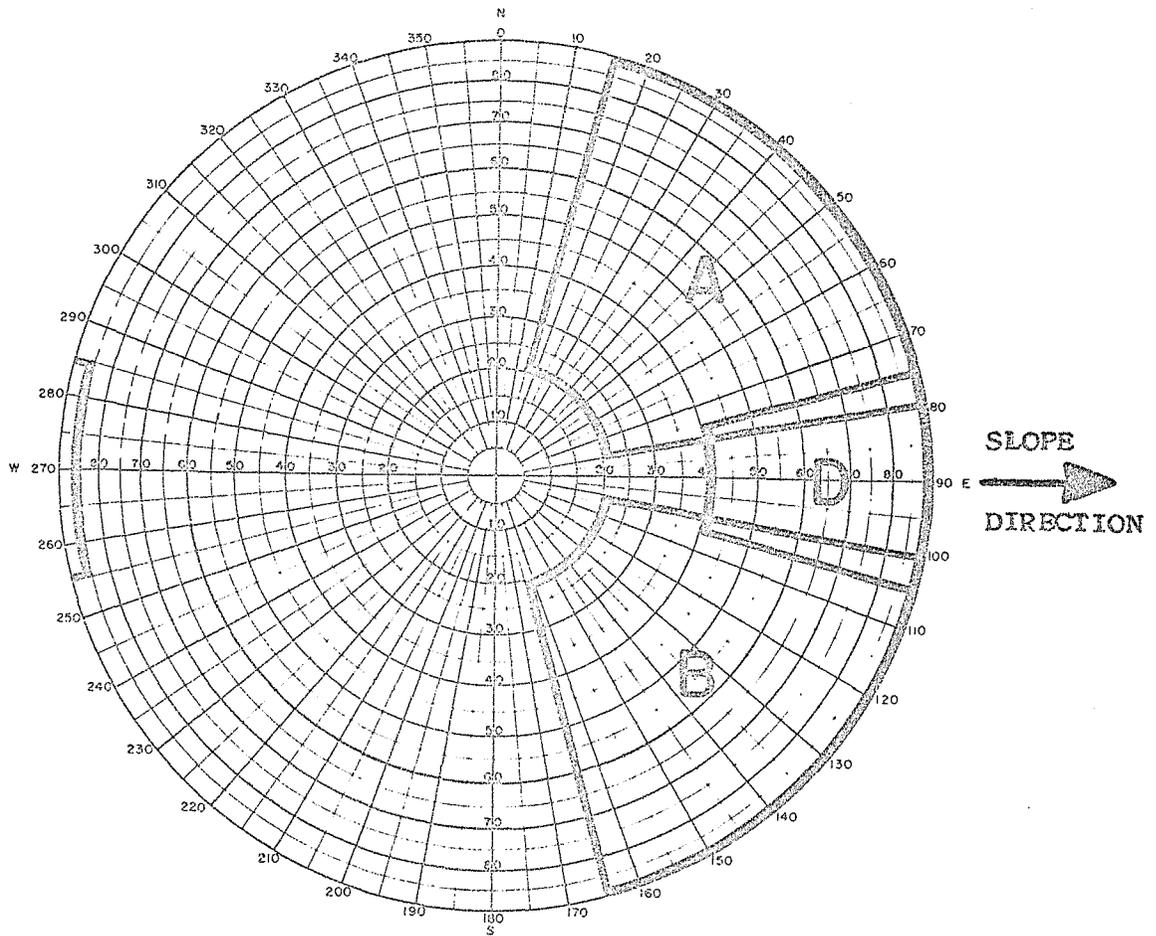


A. Upper Hemisphere Projection of the Attitude Range of Discontinuities

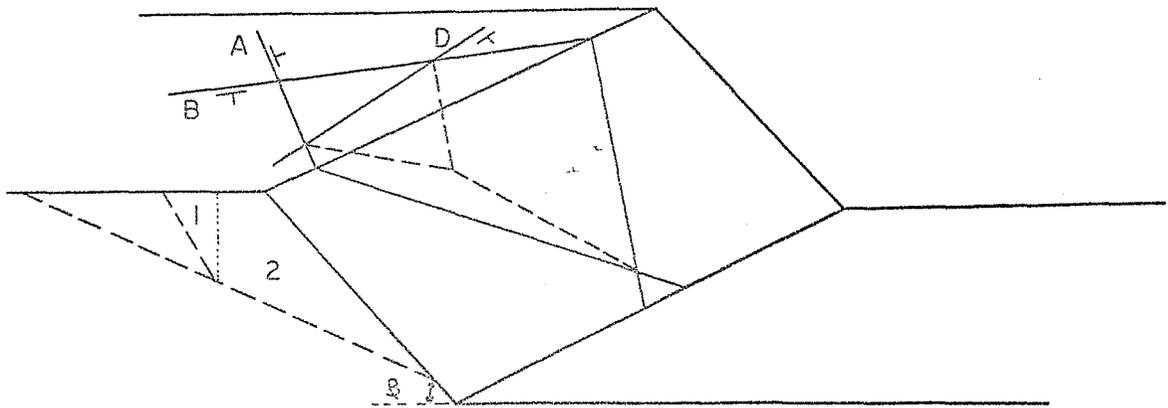


B. Isometric Sketch of Failure Geometry

Fig. 13 Intersection with Toe Discontinuity



A. Upper Hemisphere Projection of the Attitude Range of Discontinuities



B. Isometric Sketch of Failure Geometry

Fig. 14 Intersection with Headwall Discontinuity