PROBABILISTIC APPROACH TO SLOPE DESIGN

FOR THE AITIK MINE, SWEDEN

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ABSTRACT

A probabilistic analysis for optimum pit slope design is presented that is based on limiting equilibrium models. Modes of instability considered are plane shear, step path, three-dimensional wedge, slab, rotational shear, and ravelling. The open pit is first divided into design sectors. Modes of instability are determined for each sector based on fracture set orientations and their characteristics. Probability of instability for each sector is determined from the results of stability analyses on these models. Mine economics are then compared to failure costs to determine the economic optimum slope angles. Results from the slope stability study at Boliden's Aitik Mine are included.

INTRODUCTION

The purpose of this study was to determine the optimum slope angles for the final pit at the Aitik Mine. The term "optimum slope angles" implies some instability since completely stable slopes would give a conservative and costly design. Optimum slope angles are determined when the costs of flattening the slope equal the costs incurred from the instability.

Factors most critical to slope stability at Aitik are: (1) structural features (foliation, joints, faults, etc.) and their characteristics (orientation, length, spacing, waviness angle, and shear strength); (2) rock substance strengths; (3) hydrologic conditions; and (4) slope orientation. The basic premise is that in competent rock, as is the case at Aitik, the stability of a slope will mainly depend on the characteristics of the structural features. Given a potential failure geometry the hydrologic conditions also become critical to the slope stability.

The design procedure is presented first followed by the results from the study on the Aitik Mine.

DESIGN

Optimum slope angles for an open pit are a function of rock strength, rock fabric, orientation of the pit walls and cost of instability. Since these parameters vary from place to place within an open pit a single design analysis is not possible. The first step in the design is division of the mine into design sectors. Second, the conceptual models for stability analysis and probability of instability must be postulated. After the first 2 steps are completed for each design sector the results of the stability analyses and the conclusions with regard to slope angles can be presented.

Design procedures used to determine optimum slope angles are:

1. Define the design sectors based on location of facilities, pit wall orientation, structural domains, and rock type;

2. Define the distributions of the fracture set characteristics (dip, waviness angle, spacing, and length) and rock strengths for each design sector;

3. Define the pre-mine hydrologic regime and the drawdown curve expected during mining;

4. Define potential failure modes for each sector based on the pit wall orientations, structural domains, and rock mass strengths;

5. Develop probability of instability schedules for each failure mode within each design sector and calculate the overall probability of instability schedule for that sector; and

6. Weight the cost of failure with the probability of instability and compare the result with the cost of stripping to determine optimum slope angles.

MODES OF INSTABILITY

To make a quantitative analysis of slope stability, conceptual models of the modes of instability that are amenable to numerical analysis must be postulated (Figure 1). In any given slope one or more of the modes of instability may be possible, however, one mode is usually the least stable.

These modes of instability are models of reality, and the details of an actual slide are usually more complex. However, to model all the detail in any real rock slope would require a prohibitive amount of data collection and analysis. Analysis of each instability mode for each sector makes reasonable representation of the rock behavior possible. This rock behavior depends on the orientations and characteristics of joint sets, rock and joint strength, water conditions, and pit geometry.

Plane Shear

A fracture that dips in the same direction as the pit face and has a dip angle flatter than the slope angle presents a potential plane shear failure condition. For the given slope geometry, water conditions, shear strength, and dip for a continuous fracture the safety factor can be calculated using conventional stability equations. The shear strength can be expressed as a linear Mohr-Coulomb equation or a power failure law (Jaeger, 1971).

Shear strength of a specific single fracture cannot be measured directly, but instead must be estimated from direct shear test data. This data is variable and best represented by a distribution of values. Variability in the strength estimate means a probability of failure P_f must be calculated instead of a specific safety factor. The Monte Carlo method can be used to obtain a distribution of safety factors. This involves an iterative process of randomly sampling the shear strength distributions and calculating the safety factor. After a sufficient number of iterations ($\pm$ 200) a distribution of safety factors is obtained.

Since a safety factor less than 1 represents instability, an estimate of the probability of failure P_f is the ratio of the number of safety factors less than 1 to the total number calculated. For example, if 50 iterations out of 200 have a safety factor less than 1, P_f would be 25%. If the distribution of safety factors from a Monte Carlo run approximates a normal distribution the probability of failure P_f can be calculated from the area under the normal distribution curve that has safety factors less than 1 (Figure 2).

In addition to the probability of failure P_f for a continuous fracture at a specific dip the probability that such a failure plane will occur must be considered. This probability of expectancy for having a failure plane is the joint probability of a fracture having a specific dip and the probability of the length of the fracture being equal to or greater than the distance from the toe to the upper surface of the slope. This probability of expectancy P_E can be obtained from the dip and length distributions for the fracture set (Figures 3 and 4). Thus,

$$P_E = P_D \cdot P_L \tag{1}$$

Since P_f and P_E vary with dip, the probability of instability for a single fracture of a set is the summation of the joint probability of P_f and P_E over the range of dips.

$$P_F = \sum_{n=j}^{k} (P_f \cdot P_E) \tag{2}$$

where

P_F = Probability of instability of a single fracture for a given slope angle and slope height.

P_f = Probability of failure for a continuous fracture at a specific dip.

P_E = Probability of expectancy.

k = Maximum dip of the failure plane.

j = Minimum dip of the failure plane.

The above probability of instability is for a single fracture at the toe of the slope. However, in a vertical increment of the slope, for example one bench height, there will be a number of daylighted fractures of the same set with a comparable probability of instability. This number will depend on the fracture spacing. Thus, to assess the stability of a slope of a given height, an increment of slope height should be considered. This increment must be small relative to the total height as the probability of expectancy P_E is a function of slope height. A logical choice is one mining level as this is the instantaneous increment the slope height is increased during mining. The incremental probability of instability is then

$$IP_F = 1 - (1 - P_F)^N \tag{3}$$

where N is the number of joints daylighted in the increment.

Gumbel extreme length distributions can be used rather than measured lengths, but the IP_F would be calculated differently (Figure 5). The N number is the number of required extreme lengths within a design cell. The equation for probability then becomes

$$IP_F = \sum_{n=j}^{k} (P_F \cdot P_E) \qquad (4)$$

where

$$P_E = P_D \cdot [1 - (1 - P_L)^N]$$

P_D = Probability of dip expectancy.

P_L = Gumbel length probability.

N = Number of mapping cells per design cell.

This incremental probability of instability when combined for all failure modes can be used in the economic analysis computer programs developed by the University of Arizona for the Canada Centre for Mineral and Energy Technology Pit Slope Design Manual (CANMET, 1976). Alternatively, a net failure volume can be calculated as

$$NFV = IP_{F1}(V_1 + IP_{F2}(V_2 - V_1) + IPF_3(V_3 - V_2). \; . \; . \; . \; . \; . \; .IP_{FH}(V_H - V_{H-1})) \qquad (5)$$

where

V = Volume of failure.

$1, 2, \ldots H$ = Height increments.

Since the plane shear is a two-dimensional analysis the volumes would be for a unit slope length.

Step Path

Where mean lengths for daylighted joint sets are less than 1m, the probability of a continuous joint more than 30m long is very small. In these cases the failure path would be a combination of joints (Figure 6). The step path geometry is composed of 2 joints with similar dip direction. A reasonable range for dips is 20° to 70° for the master joint on which sliding occurs, and 50° to 90° for the cross joint that connects the master joints. When a complete step path cannot be developed for slopes rock bridges are formed (Figure 7). Tensile failure of the rock bridges usually occurs before shearing since in rock the shear strength is generally higher than the tensile strength. The probability of instability P_F is calculated similarly to the plane shear case except for the probability of expectancy P_E. The probability of expectancy P_E is determined from a computer model that samples the length, spacing, dip, and overlap of the joint sets involved. A distribution of step path angles and step path heights results (Figures 8 and 9). From these 2 distributions, assuming tensile rock bridges occur between step paths, the percent intact rock is calculated and a distribution of Beta angles is obtained. The distribution of Beta angles is equal to the probability of expectancy because the probability of length is always 1 when tensile failure of the rock bridges is assumed. In the stability analysis the distribution of percent intact rock is used to determine the rock mass tensile strength. The stability analysis proposed by Jaeger (1971) is used.

Simple Three-Dimensional Wedge Failure

Two fractures with dip directions oblique to the pit slope can intersect and form a three-dimensional wedge block. If the plunge angle of the intersection is less than the slope angle and the bearing of the intersection is similar to the dip direction of the slope a potential failure geometry is formed. With this geometry present the daylighting criteria of the single plane shear failure applies to the trace of the intersection. Intersections with plunge angles flatter than the slope angle have the potential for sliding if restraint in the direction of the intersection has been removed.

For each design sector the wedge analysis in general follows 4 steps:

1. Define the potential wedge failure geometry from the mean orientations of the design sets;

2. Determine the kinematic possibility of wedge failure from the orientation of the slope, the orientation of both fractures, and the orientation of the intersection;

3. Conduct stability analyses using the fracture shear strengths, fracture orientations, and fracture characteristics to determine critical wedges; and

4. Conduct probabilistic analysis on critical wedges.

Defining fracture sets in the sector of interest is the first step in determining the potential wedge geometries. All possible intersection combinations are calculated from the mean orientations of these design sets. Kinematic tests are then conducted to determine wedges with sliding potential in the sector.

Stability analyses follow the kinematic tests to determine which of the wedges with sliding potential also have low shear strength. If wedges are present in critical orientations with respect to the slope and the included fractures have low strengths, a probabilistic analysis is done.

Probabilistic analysis of the three-dimensional wedge geometry uses equation (3) to calculate the incremental probability of instability IP_F. The N value changes to the number of possible wedges in a cell. The cell height is equal to the height of the mining increment and the cell length along the slope is equal to the height of the slope being considered. The equation to calculate the probability of instability P_F is:

$$P_F = P_W \cdot P_T \tag{6}$$

where

P_F = Probability of instability of a wedge formed from the intersection of 2 specified fracture sets at a given slope geometry.

P_W = The probability of failure for the simple wedge geometry including the dispersion of the intersection.

P_T = The probability that the intersection trace length will be long enough to extend from the toe to the upper surface of the slope.

The probability of an intersection trace length being long enough P_T is the combined probability of the fracture lengths from the two included sets being as long as the required intersection trace length. This is expressed mathematically as:

$$P_T = P_{L1} \cdot P_{L2} \tag{7}$$

where

P_{L1}, P_{L2} = The probabilities of having fractures from Set 1 or Set 2 that are long enough to extend along the line of the intersection from the toe to the upper surface of the slope.

Length probabilities P_{L1} and P_{L2} can be calculated from measured lengths from detail line data using the exponential distribution equation (Figure 4).

The required length L is calculated along the line of intersection:

$$L = \frac{H}{\sin\alpha \, \cos(\theta - \psi)} \tag{8}$$

where

H = Slope height taken from the mid-point of the mining increment to the upper slope surface.

α = The plunge of the wedge intersection.

ψ = The bearing of the wedge intersection.

Θ = The dip direction of the slope.

An alternative length probability can be calculated from the Gumbel extreme values of lengths (Figure 5). The length data comes from cell interval mapping. The Gumbel probability equation was presented previously in the plane shear analysis description and gives higher length probabilities than the measured length data.

Slab Failure

When a structure system such as foliation or bedding parallels the slope, slab failure is possible. Although the geologic structure may be steeper than the slope making plane shear failure impossible, failure of a slab could occur if 1 or more of the following conditions are met:

1. The weight of the slab exceeds the shear strength at the toe. The toe could either consist of intact rock that shears or contain a joint that releases the toe.

2. When high water pressures uplift the slab and cause shear failure or a moment about the toe or crest.

The analysis is most critical to water height and the nature of the toe (joint or intact).

Where a frozen face condition could occur during the winter months normal seepage of water may be blocked. Under these frozen conditions hydrostatic pressure on the slab could exceed the weight of the slab and cause uplift or overturning.

The analysis has not been developed to the stage where an incremental probability of instability can be calculated. However, a series of sensitivity analyses can be made to determine the critical water height and slab thickness so that drainage can be set up to prevent this failure mode.

Rotational Shear

Rotational shear is a common failure mode for soil and "soil-like" (altered rock) materials where the rock substance strength is low and the failure surface is controlled by the maximum shear surface rather than by low strength geologic structures. The rotational shear analysis consists of finding the maximum shear surface determined from the strength characteristics of the material and the geometry of the slope. Water levels play a significant role in the stability analysis.

In high strength rock such as at Aitik, true rotational failure through the rock would not occur. However a combination of fractures can result in a potential failure path that approximates the maximum shear surface. For this failure path the strength properties of the fractures would apply.

Ravelling

Ravelling can occur under the following conditions where the slope angle exceeds the angle of repose:

1. Where excessive weathering or overbreak from blasting has reduced the cohesive strength of the pit wall rock;

2. Where fractures with short lengths are daylighted by the bench face angle.

At the present time there is no specific calculation for stability with regard to ravelling. Design consists of empirically evaluating the potential for ravelling and choosing an appropriate catch bench width. The potential for ravelling is largely determined by the blasting and scaling techniques for the final face. The catch bench width is a function of the trajectory of falling rocks which depends on the bench face angle, the bench face height, the elastic properties of the rock, and the shape of falling rocks. Where it is considered necessary to clean off the catch benches the width depends on equipment size.

OPTIMUM SLOPE ANGLES

Optimum slope angles can be determine by two methods: cost of instability versus cost of stripping curves or a benefit-cost economic model.

Cost of Instability versus Cost of Stripping

The optimum slope angle is determined when the incremental increase in slope angle produces a larger incremental increase in the cost of instability than the incremental decrease in the cost of stripping. The cost of stripping can be estimated from the stripping volume and the unit cost of mining. The failure cost is determined by the volume of the failure, the unit cost of mining failed material, and the costs associated with any disruption in mining caused by a failure.

Stability analysis defines the most probable failure mode. Failure volumes are calculated on a unit volume basis from this predicted failure mode and the slope geometry. Net failure volumes are calculated from equation (5) for each increment of slope angle. The failure volume difference produced by each increment is calculated to produce the incremental failure volume curves (Figure 10). Costs associated with the different failure situations are estimated on a sector by sector basis for evaluation of optimum slopes. If removing failed material is the only consequence of failure the cost of the failure may be no more than the cost of stripping. If the failed material covers ore, damages facilities, or closes a haul road, for example, higher costs may be assigned to a failure in the sector where this could occur. This NFV method does not account for the time value or sequencing of money.

Benefit-Cost Model

It is possible to account for the time value and sequencing of money with a cost-benefit analysis developed through the Canada Centre for Mineral and Energy Technology during the development of the Slope Design Manual (CANMET, 1976; Kim, Y.C. and others, 1976a, 1976b). This analysis models the mining sequence of the open pit. Each design sector is evaluated at each mining period for the cost of instability. The occurrence of these instabilities is predicted by the probability of instability schedule for each sector. The resulting costs and benefits incurred from each mining period are discounted back to the present. A model for different slope design conditions can be produced and the optimum slope design more precisely selected based on the net present value from each slope design option.

The benefit-cost analysis recommends comparison of 3 pit designs and requires at least 2 pit designs.

RESULTS FROM THE AITIK STUDY

The slope stability study at Aitik was a practical application of the design procedures. The study included field data collection, rock strength testing, rock fabric analysis, and stability analysis.

Design Sectors

To obtain a realistic estimate of the potential final pit geometry at Aitik, a trial pit was designed using a 57° overall slope for the hanging wall with catch benches every 30m vertically and a slope angle of 45° for the footwall with no catch benches.

The pit area is considered to be one structural domain, therefore the four major design sectors -- Footwall, Hanging Wall, North End, and South End -- were chosen based on the relationship between pit wall orientation and geological structure. The Footwall Sector was further subdivided because of the high cost of instability associated with the mill location (Figure 11).

Geologic Structure

Call and others (1976) discuss the process of defining design joint sets and determining the distribution of joint set characteristics. Detail line mapping, fracture set mapping, and oriented drill core were used at Aitik to obtain the joint set orientations and their characteristics.

Foliation dips to the west about 47° and is the predominant structure trend at the Aitik mine. Other joint sets defined at Aitik include a cross joint set that dips 28° to the east, a flat joint set, and 8 vertical joint sets (Figure 12).

Distributions of length, spacing, waviness angle, and dip were determined for each of the joint sets. Exponential distributions were found for the length, spacing, and waviness angle while a normal distribution represented the dip (Figure 13). Foliation was estimated to have a mean length of 2.38m. The mean joint lengths on the other design sets ranged from .32m to 1.02m. Spacings ranged from .05m to .81m. Mean waviness angles considering all of the design sets were from 7° to 10°.

Rock Units for Design

Strength testing on rock samples showed Aitik rock generally weakest parallel to foliation. Based on the testing, the rock units at Aitik were combined into 5 groups to use in analysis and design. Classification of these rock groups is from a combination of two classifications; one proposed by Coates (1970) and the other by Deere (1968).

Skarnbanded Group. A high strength (1125-2250 kg/cm^2), high modulus ratio, elastic, layered, broken to very broken gneiss. Strength parallel to foliation is reduced 17% on the average.

Gneiss Group. Medium to high strength (560-2250 kg/cm^2), medium to high modulus ratio, elastic, layered, broken to very broken gneiss. Strength parallel to foliation is reduced 34% on the average.

Schist Group. Low to medium strength (280-1125 kg/cm^2), medium modulus ratio, elastic, layered, broken to very broken mica schist. Strength parallel to foliation is reduced 38% on the average.

Amphibolite Group. Low to medium strength (280-1125 kg/cm^2), medium to high modulus ratio, elastic, layered, broken to very broken amphibolite. Strength parallel to foliation is reduced 33% on the average.

Pegmatite Group. Medium to high strength (560-2250 kg/cm^2), medium to high modulus ratio, elastic, massive pegmatite.

When a failure path occurs along a natural joint, large scale direct shear strengths were applied. When failure through intact rock was expected the intact rock shear strength was used. The following natural joint shear strength values were used for the stability analyses.

Sector	Rock Unit	ø Wet	c Wet (kg/cm^2)	k	m
Hanging Wall, North End, South End	Schist	28.9°	2.38	1.46	.76
	Gneiss	30.4°	1.48	1.22	.82
Footwall	Schist	26.0°	0.82	0.76	.90
	Gneiss	29.7°	1.62	1.11	.86

Rock substance shear strength values of $\phi = 58.6°$ and $c = 126.6$ kg/cm^2 and a mean rock substance tensile strength of 82.2 kg/cm^2 were found for footwall gneiss.

In the hanging wall both schist and gneiss were considered. A mean rock substance tensile strength of 126.0 kg/cm^2 was found for schist and 82.2 kg/cm^2 was found for gneiss.

Optimum Slope Angles at Aitik

Based on the stability analyses, economic, and operating considerations the following slope angles were recommended:

	Footwall	Hanging Wall	North End	South End
Interramp Slope Angle	47°	57°	57°	57°
Bench Face Angle	61°	72°	72°	72°
Bench Width	6m	10m	10m	10m
Bench Interval	15m	30m	30m	30m

Hanging Wall

The stability analysis for the Hanging Wall Sector indicated that the most probable failure mode would be a step path failure (Cross Joint - 192/012) with the frozen face condition. The incremental probability of failure IP_F for the Cross Joint - 192/012 step path based on 15m mining increments is listed in Table 1. Based on these IP_F values, the incremental net failure volumes were calculated and compared against the incremental stripping volume (Figure 10).

Assuming the cost of instability is removal of the failed material and this cost is equal to the cost of stripping, the optimum slope angle would be 62.5° (Figure 10). Because of the steep slope of the incremental failure volume curve, the optimum slope angle would be reduced to 61° when the cost of removing failed material is assumed to be twice the stripping cost (Figure 10). Below 60° the failure volume is insignificant compared to the stripping volume.

For the normal drawdown step path and for the plane shear, the probability of instability is so low that the incremental failure volume does not approach the stripping volume for slope angles up to 90°.

Since the cost of instability optimum slope angles are in the range of expected bench face angles, the criterion for slope angles for the hanging wall is catch bench design.

Footwall

The footwall of the ore generally follows the foliation dip so there is no advantage to having a footwall slope angle steeper than the 40° to 50° dip of the foliation. Plane shear, step path, slab, wedge, rotational shear, and ravelling were considered potential failure modes on the footwall.

Daylighting the foliation gives a high probability of instability for the footwall. Plane shear and wedge failures are unlikely as the slope follows the dip of the foliation. Rotational shear through rock substance is unlikely, but an approximation to a rotational shear failure path could be formed by the combination of the Flat Set, the Foliation Set, and the north-south vertical sets. If the Flat Set is a near surface feature this failure path would not occur for high slopes.

A slab failure mode is possible for slopes with an orientation parallel to the foliation. A 5m thick slab on a 100m high slope with the frozen face condition will fail if drainage is not achieved. For dry slopes the slab slide has a low probability of failure.

Present benches in the Aitik pit and the step path analysis suggest that average bench face angles of 61° could be achieved in the footwall, but considerable backbreak to foliation would occur.

North and South Ends

The ends of the pit where the pit face is at right angles to the foliation have very few critical structures and the slope angles will depend on catch bench requirements. In the corners where the slope makes a transition from hanging wall and footwall a potential wedge failure geometry is present.

In the hanging wall corners the wedge would involve the Cross Joint Set and vertical set with the displacement predominantly plane shear on the Cross Joint. Because of the short Cross Joint lengths there is a low probability of failure for slope heights greater than the bench height. Therefore, the hanging wall slope angles can be continued to both end sectors.

In the footwall corners the control geometry is wedge failure of the foliation and vertical set combination with the displacement primarily plane shear on the foliation. Because of the larger lengths of the foliation this failure mode has a significant probability. To minimize the failure volume these corners should be cut as square as possible and should follow the geologic structure.

CONCLUSIONS

The current trend in pit slope design is statistical analysis where the probability of instability is evaluated as a function of slope angle and slope height. Pit optimization is the ultimate objective. With proper mining procedures an unstable slope may be more profitable than a stable slope at a much flatter angle. Presenting the stability analyses in the form of a probability of instability, stripping costs can be compared with failure costs to arrive at an economic optimum slope angle. An alternative to this direct comparison of costs is a cost-benefit model that includes the time value and sequencing of money based on probability of instability and pit economics.

The open pit should be divided into design sectors based on the location of facilities, pit wall orientation, structural domains, and rock types. Mapping should be conducted to develop distributions of the fracture set characteristics. Failure modes should be determined for each design sector and a probabilistic analysis done for each failure mode. When this work is completed the probability of instability schedules can be developed for input to the economic analyses.

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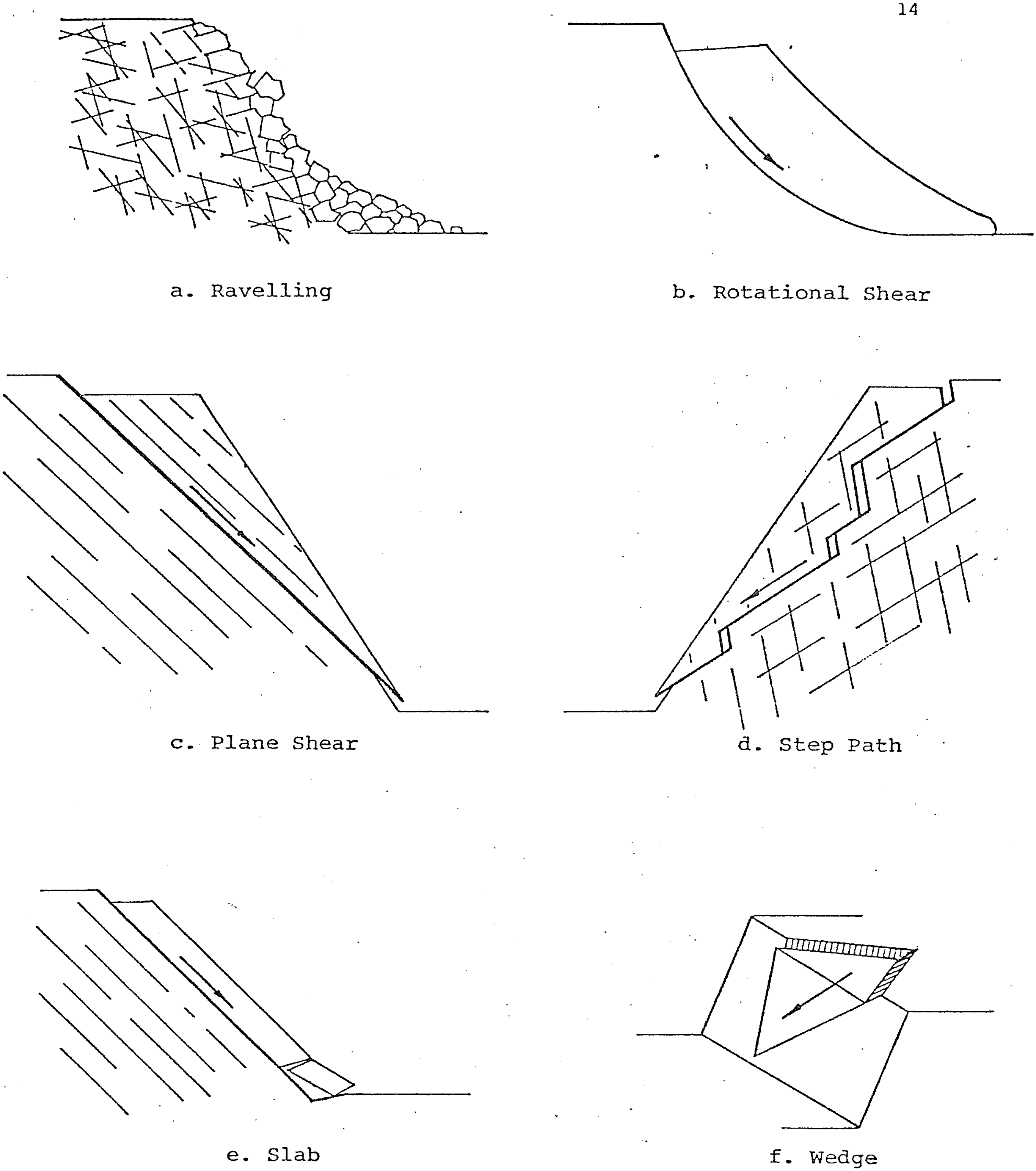

Figure 1. Potential Instability Modes

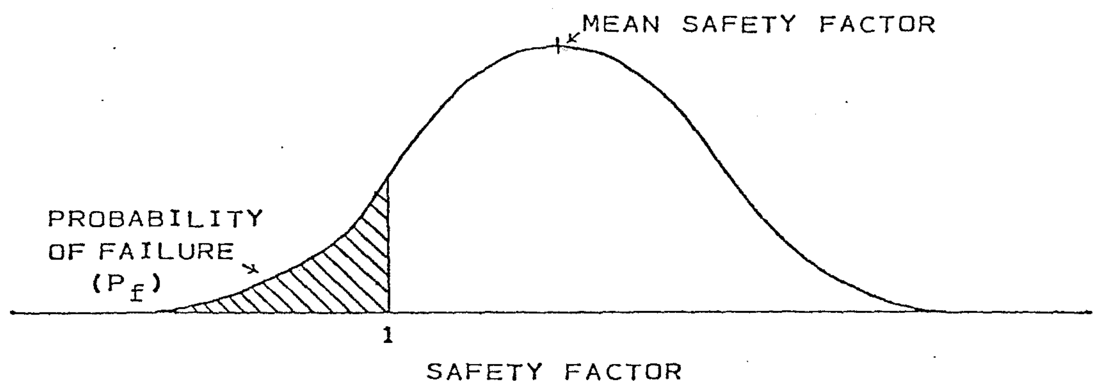

Figure 2. Example Distribution of Safety Factors to Calculate P_f

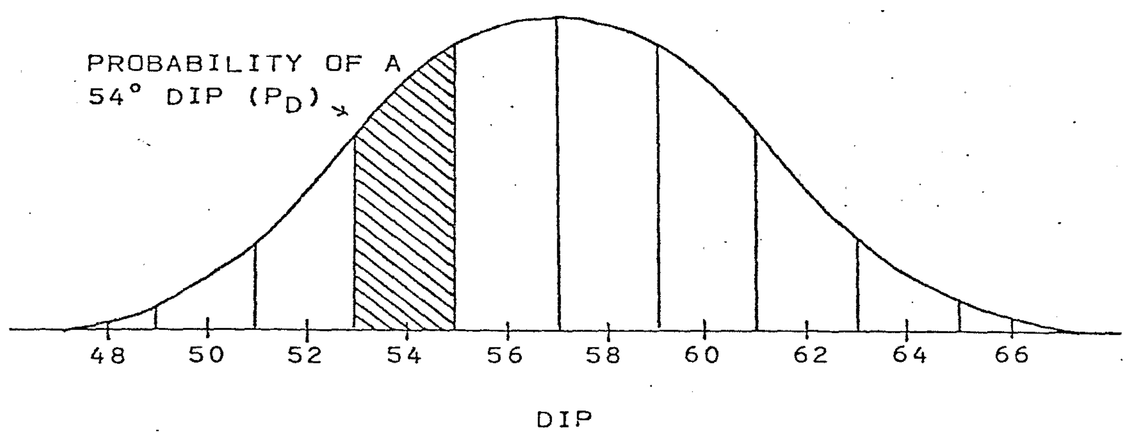

Figure 3. Example of Probability of a Dip Occurring to Calculate P_D

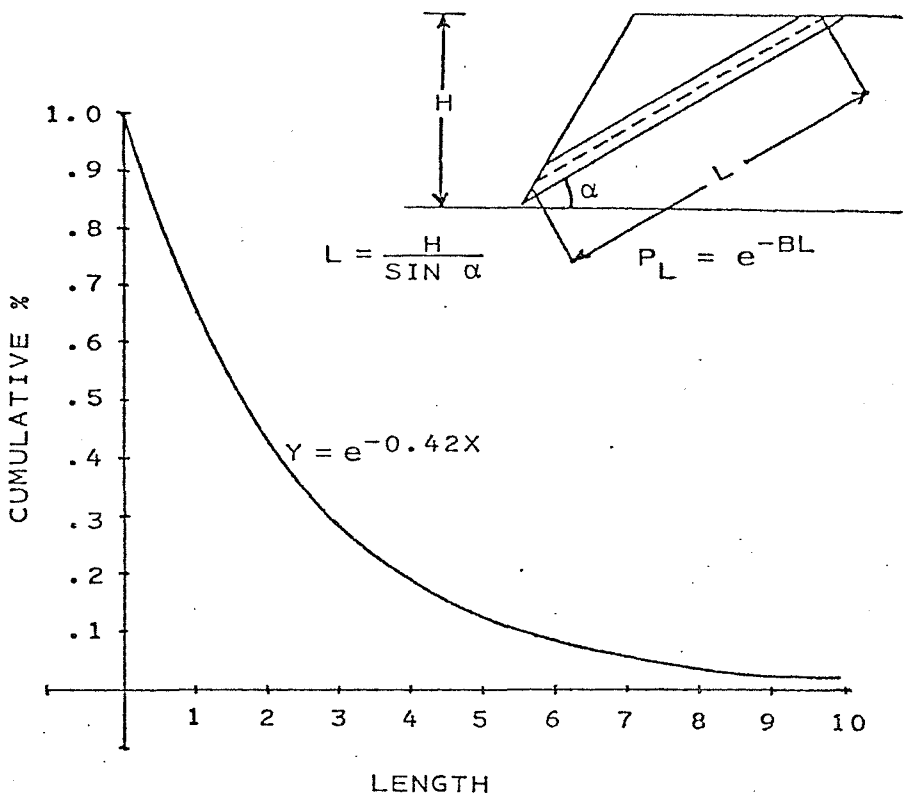

Figure 4. Example of a Cumulative Length Distribution to Calculate P_L.

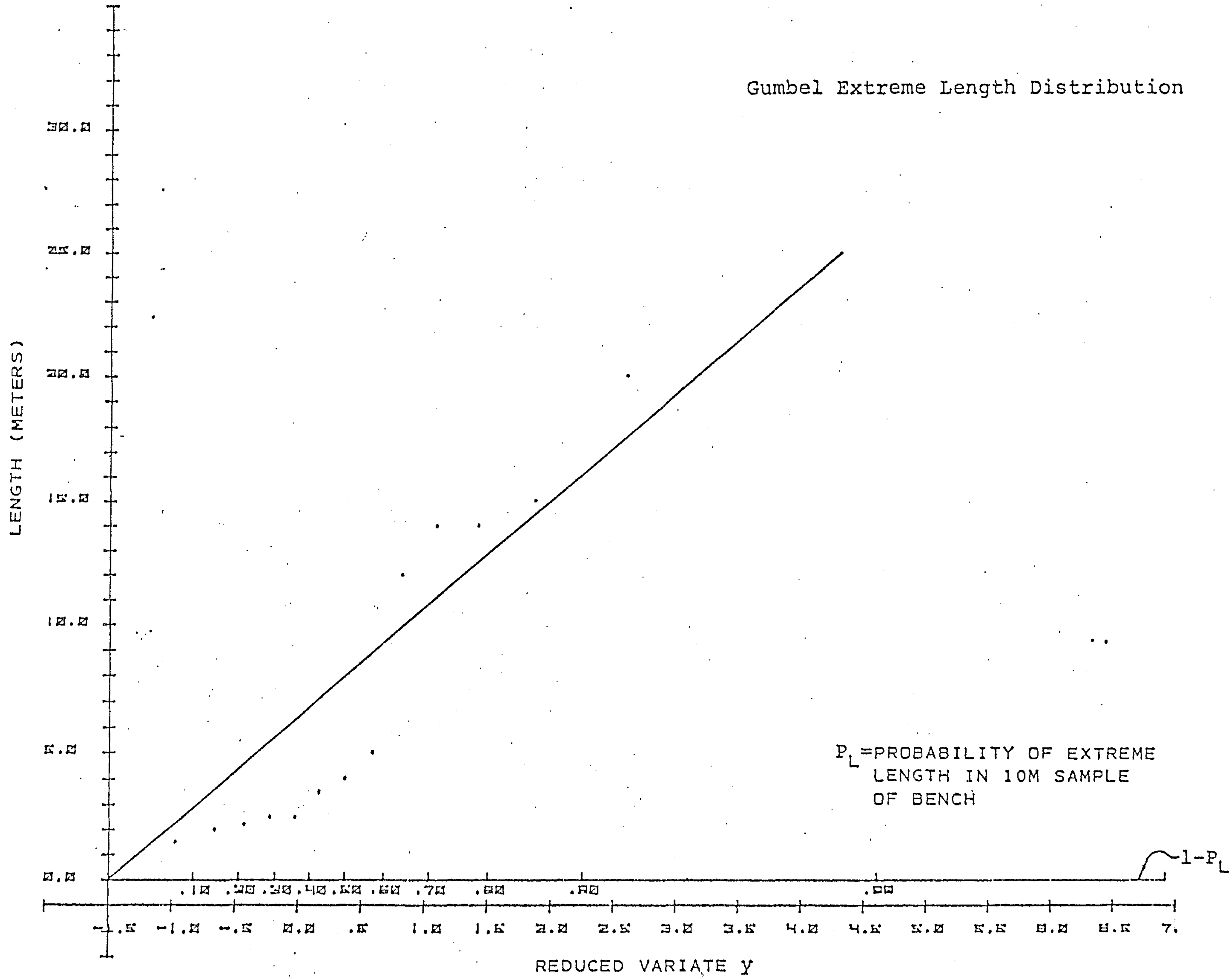

Figure 5.

$$1-P = e^{-e^{-y}}$$

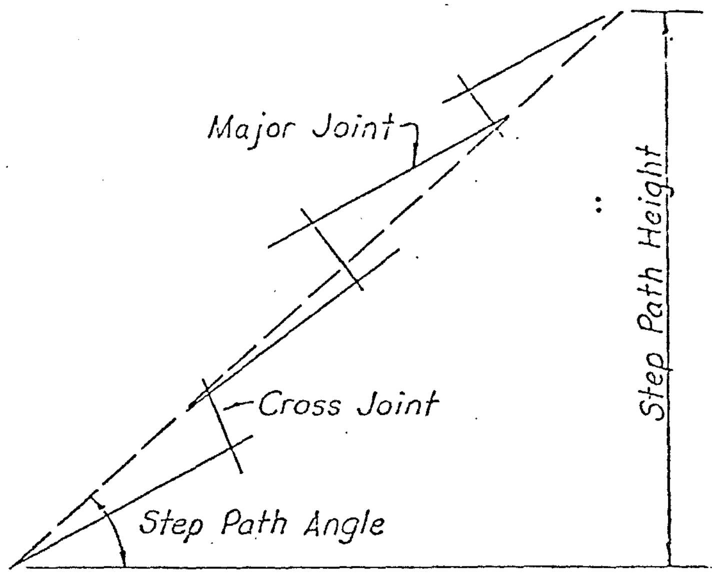

Figure 6. Geometry of Step Path

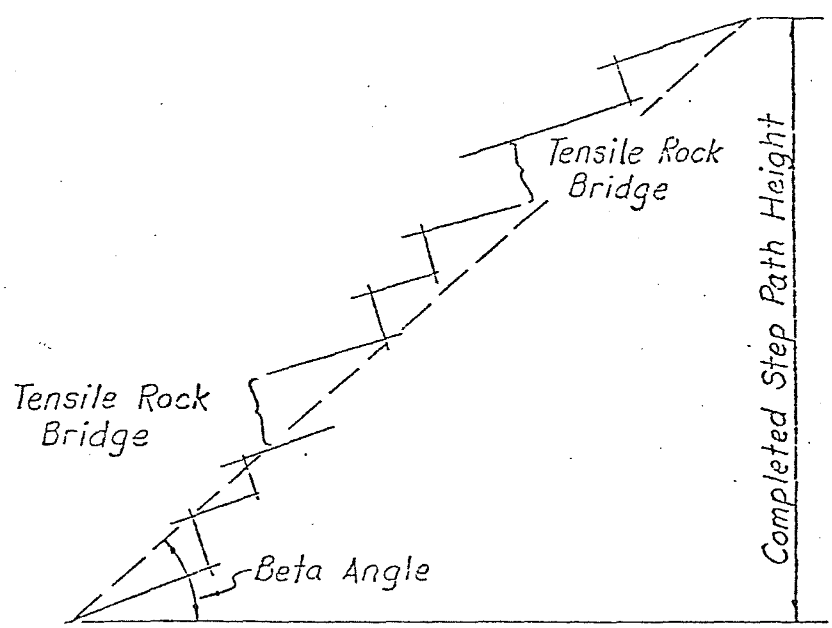

Figure 7. Step Path Geometry Including Tensile Rock Bridges

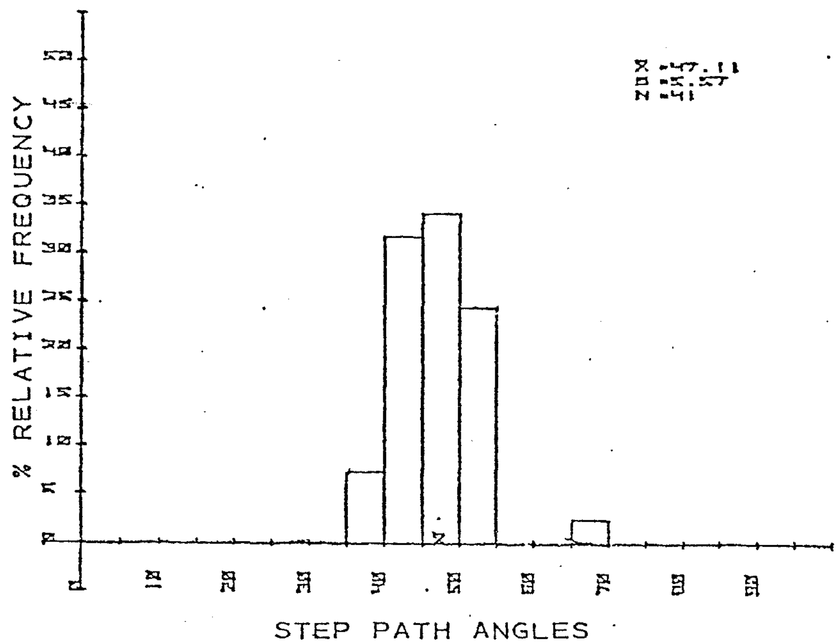

Figure 8. Example Distribution of Step Path Angles

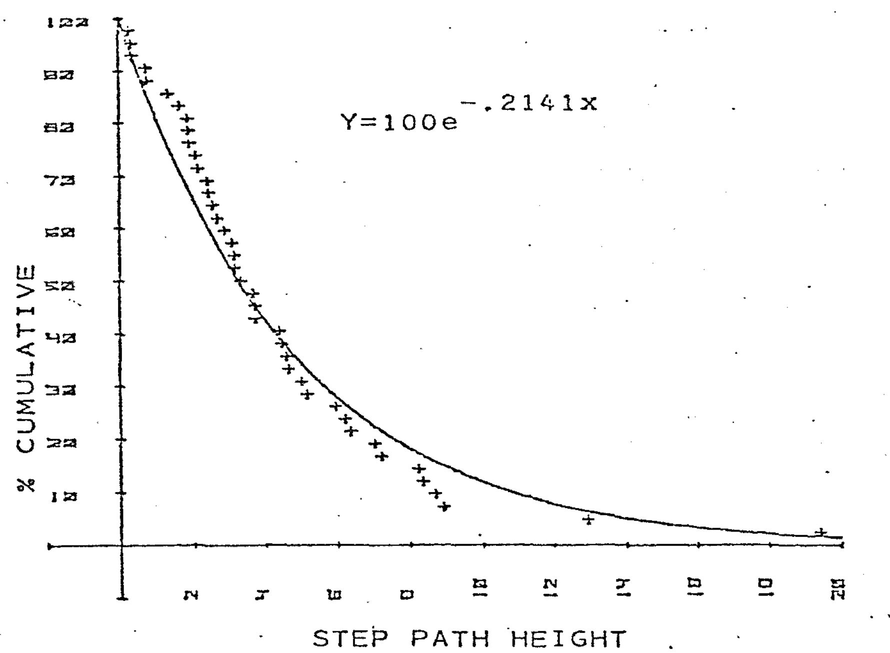

Figure 9. Example Distribution of Step Path Heights

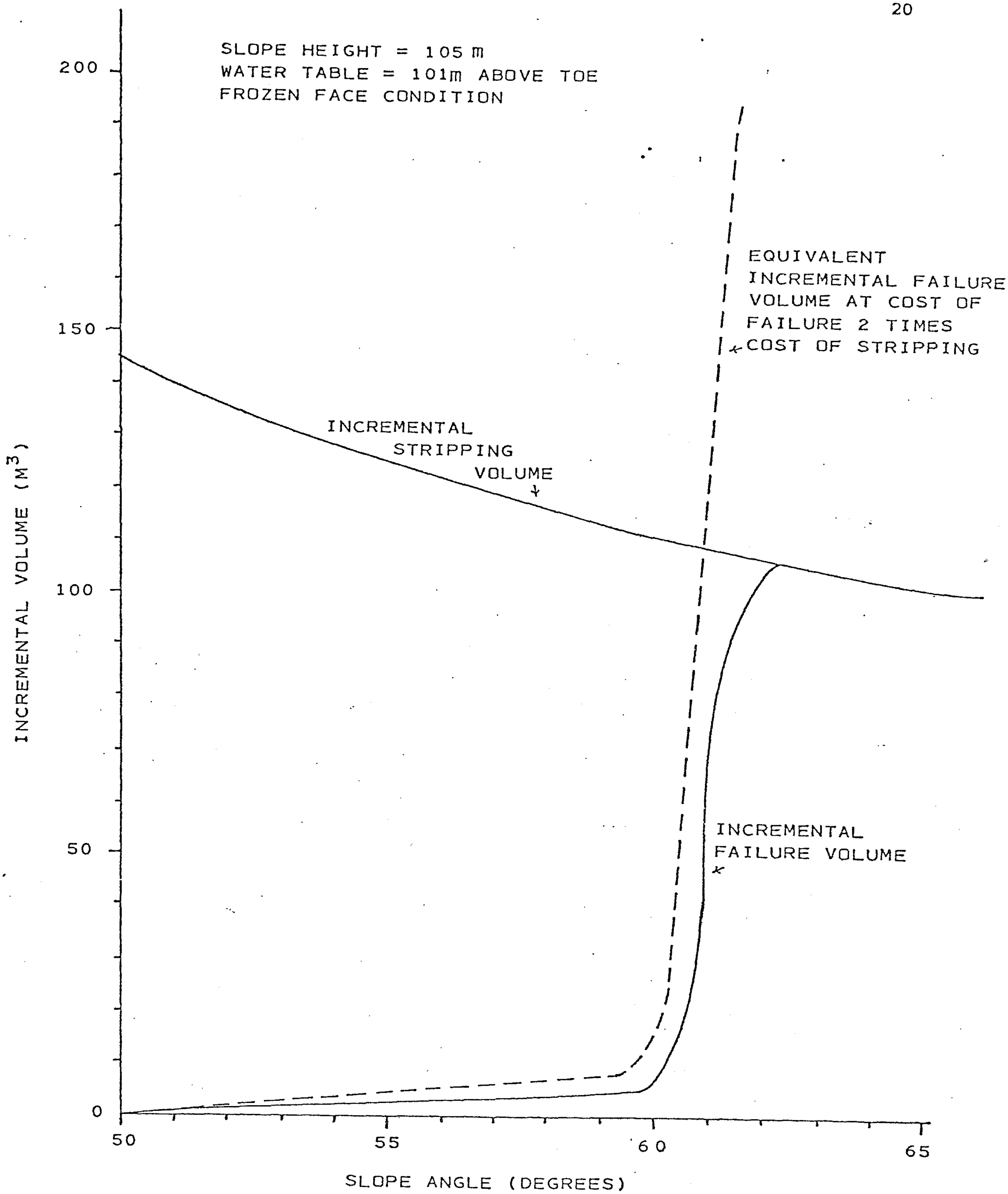

Figure 10. Typical Interramp Slope Angles Incremental Volume Curves for Stripping and Instability

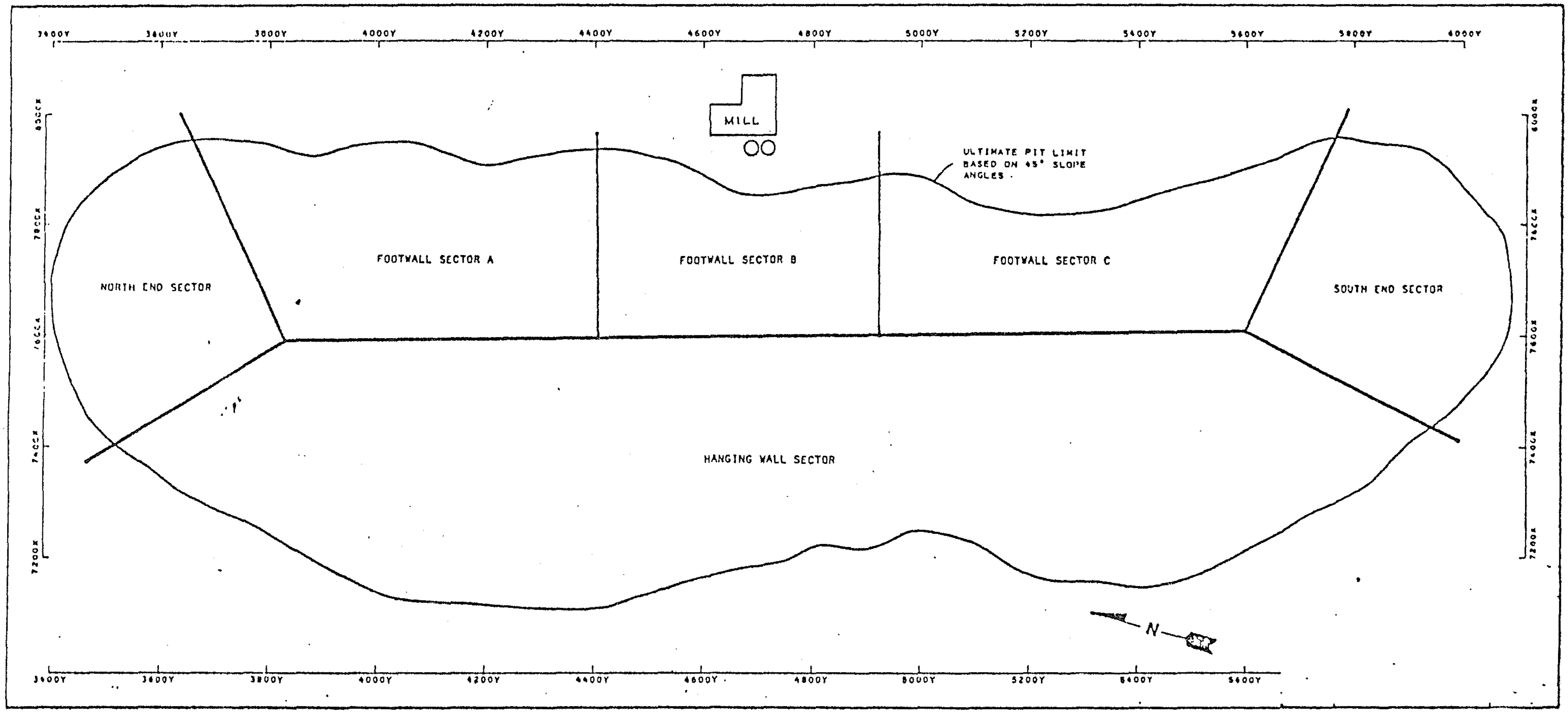

Figure 11. Aitik Mine Design Sectors.

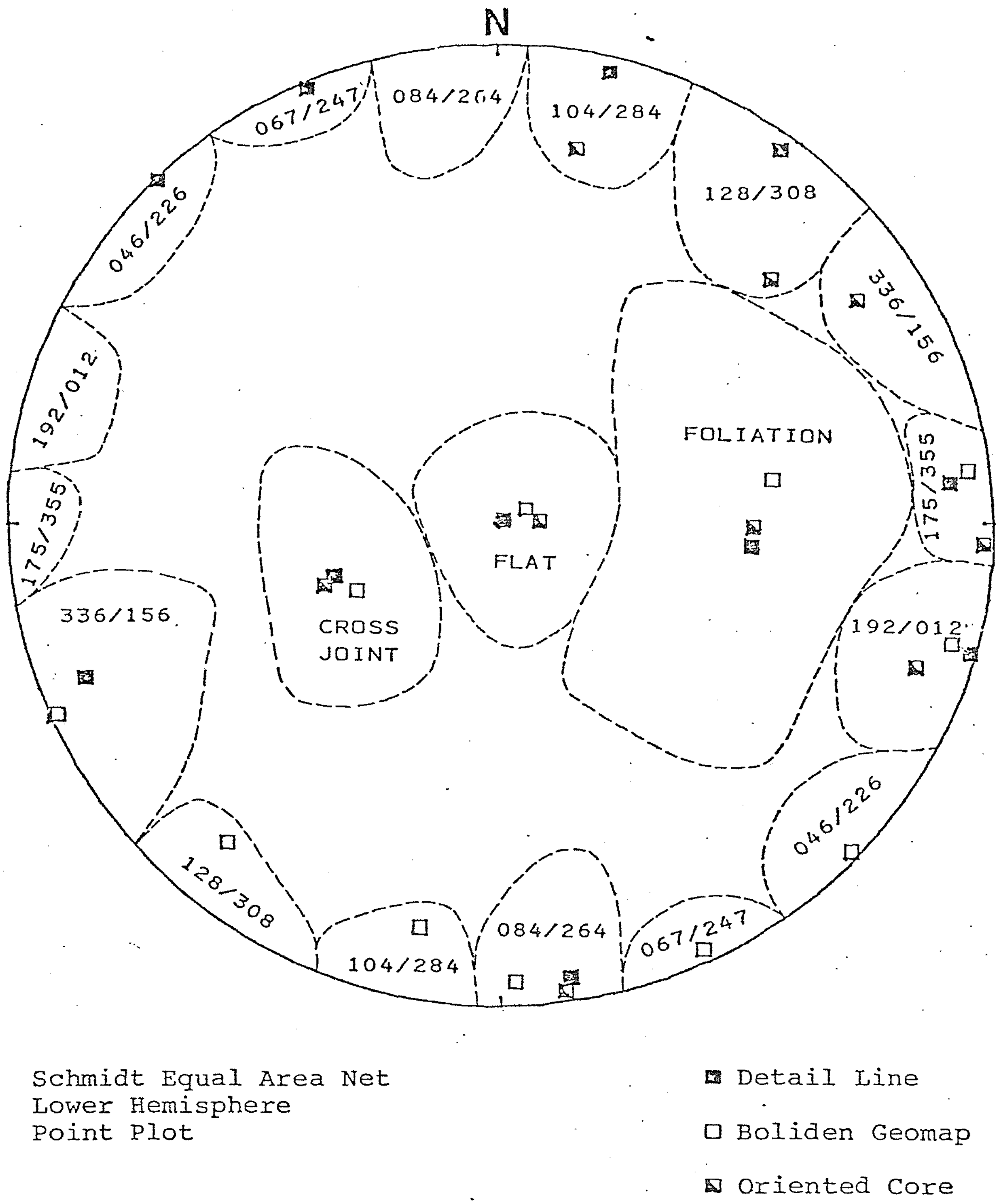

Figure 12. Mean Vectors of Fracture Sets

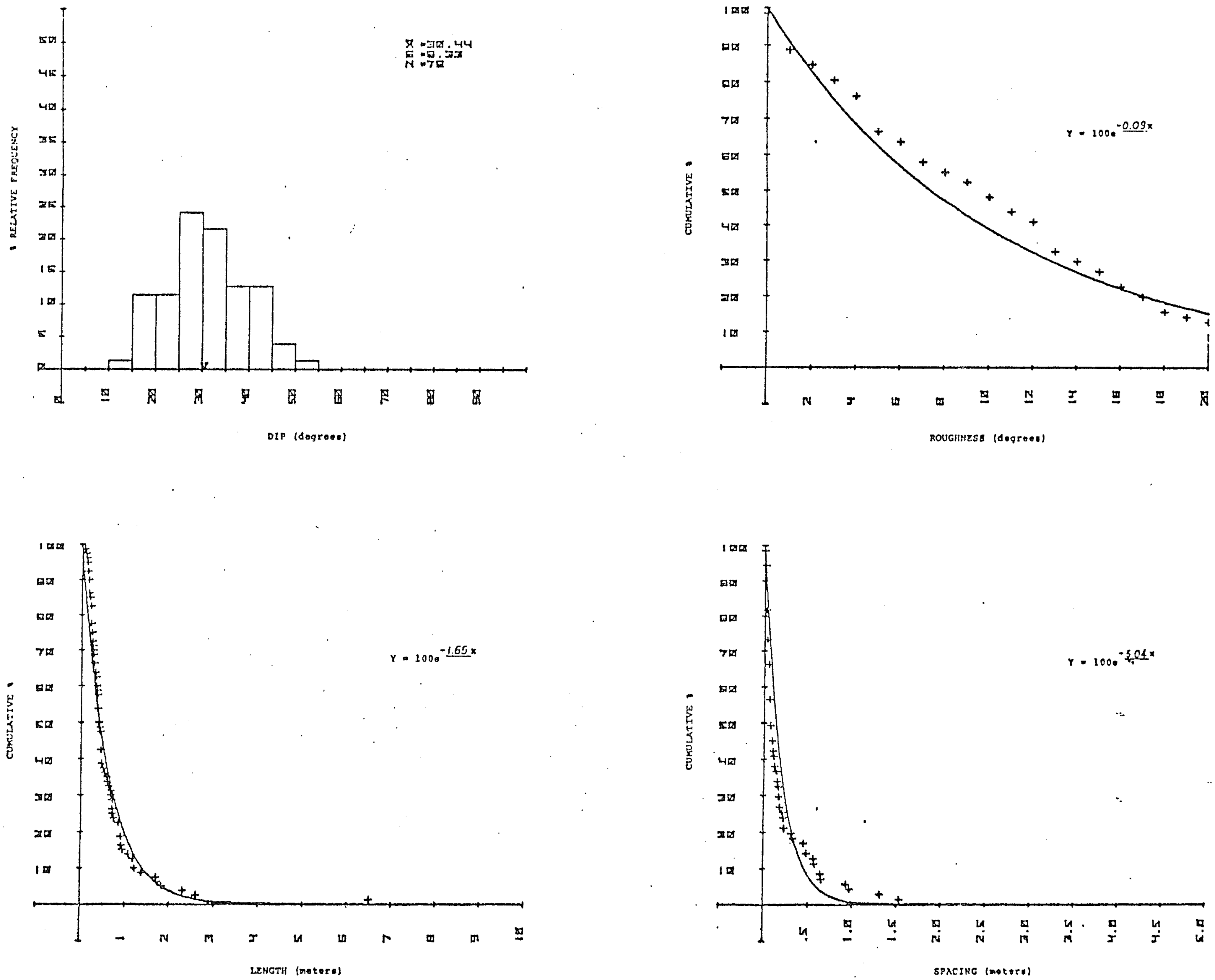

Figure 13. Typical Distribution of Fracture Characteristics

SLOPE HEIGHT (METERS)	ROCK GROUP	WATER CONDITION	LENGTH CONDITION	INCREMENTAL PROBABILITY OF FAILURE FOR THE FOLLOWING SLOPE ANGLES 40	50	55	57	60	65	70	75
			CROSS JOINT - 192/012 STEP PATH								
30	Schist	Sat-Froz	Measured	0	0	.20	.53	.97	1.00	1.00	1.00
30	Schist	Sat-Norm	Measured	0	0	0	0	0	0	.01	.03
30	Schist	Dry	Measured	0	0	0	0	0	0	0	.01
30	Schist	Dry	Continuous	0	0	0	0	0	0	.01	.02
100	Schist	94m-Froz	Measured	0	0	.01	.50	1.00	1.00	1.00	1.00
100	Schist	94m-Froz	Continuous	0	0	.01	.50	1.00	1.00	1.00	1.00
100	Schist	94m-Norm	Measured	0	0	0	0	0	0	0	0
100	Schist	Dry	Measured	0	0	0	0	0	0	0	0
100	Schist	294-Froz	Measured	0	0	0	.04	1.00	1.00	1.00	1.00
			CROSS JOINT - 175/355 STEP PATH								
30	Schist	Sat-Froz	Measured	0	0	.03	.15	.55	.98	1.00	1.00
30	Schist	Sat-Froz	Continuous	0	0	.04	.21	.67	1.00	1.00	1.00
30	Schist	Sat-Norm	Continuous	0	0	0	0	0	0	0	.01
30	Schist	Dry	Continuous	0	0	0	0	0	0	0	.01
100	Schist	Sat-Froz	Measured	0	0	0	.01	.03	1.00	1.00	1.00
100	Schist	Dry	Continuous	0	0	0	0	0	.02	.21	.65
			CROSS JOINT - FOLIATION STEP PATH								
30	Schist	Sat-Froz	Measured	0	0	0	0	.01	.16	.78	1.00
30	Schist	Sat-Norm	Continuous	0	0	0	0	0	0	0	0
30	Schist	Dry	Continuous	0	0	0	0	0	0	0	0
100	Schist	94m-Froz	Measured	0	0	0	0	.04	.69	1.00	1.00
100	Schist	Dry	Measured	0	0	0	0	0	0	0	0

EXPLANATION OF TERMS:

Sat = Saturated; when a number occurs this is the height of water above the toe of the slope

Froz = Frozen Face

Norm = Normal Drawdown

Measured = Analysis used the lengths measured in the field

Continuous = Analysis assumed a continuous joint or in the case of step path there was no intact rock

Table 1. 15 meter Incremental Probability of Failure for Step Paths in the Hanging Wall

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