Cost-Benefit Design of Open Pit Slopes

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Introduction

The objective of an open pit mine plan is to obtain the maximum ore with the minimum of overburden stripping up to the limit where the incremental stripping cost equals the value of the ore recovered. This optimization results in the maximum economic benefit over the life of the mine. In general, increasing the slope angle decreases the stripping and/or increases the recoverable ore (Figure 1). However, as the slope angle increases there is an increase in the number and size of slope failures, hence an increase in the cost of slope instability. As shown in Figure 2, the cost of instability increases more rapidly than the benefits at steeper slope angles. Thus, the net benefit obtained by subtracting the slope instability cost from the benefits has a maximum. The slope angle at which this maximum occurs is the optimum angle as mining at either a flatter or steeper slope angle will result in less benefit.

Since the life of a mine can be 15 to 20 years or more, the time value of money can be significant, and the time at which a slope failure occurs in the mine life will affect mine economics. The cost of slope instability can be distributed over the mine life on the basis of the slope geometry for the time segments of the mine life. These costs can be included in a discounted cash flow analysis to obtain a net present value vs slope angle curve which is analogous to the net benefit curve of Figure 2 (Kim et al., 1977).

The calculation of benefit, which is essentially gross sales less mining and processing costs, is a relatively straightforward mine planning exercise. The calculation of the cost of slope instability involves the application of geotechnical probability analysis to estimate instability and the economic evaluation of the associated risk.

Probabilistic Stability Analysis

Conceptually, the stability of a slope is the ratio of the strength of the material in the slope to the stresses in the slope. If the stress exceeds the strength, the slope is unstable; conversely, if the strength exceeds the stress, the slope is stable. In any real slope a unique value for the stability cannot be determined for the following reasons:

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1) The rock in a slope is heterogeneous. For example, the unconfined compressive strength, a basic measure of rock strength, can vary from 2000 kg/cm^2 for unaltered intact rock to 10 kg/cm^2 for a gouge zone less than a meter away. Since it is not possible to test all the rock, the rock strength must be estimated by testing a limited number of samples of the total and making statistical inferences from the sample values. Since the test values will be the same, the resulting estimate of rock strength will be a statistical distribution rather than a unique value.

2) The rock is anisotropic. Faults, joints, and other geologic features that are weaker than the intact rock tend to be planar and occur in preferred orientations. Thus, the stability of a slope is affected by the orientation of these features relative to the slope. These features can be characterized by attitude, length, and spacing. As with strength, the properties of the structural features are distributions which must be estimated by measurements of a limited sample of the total population.

3) There is a measurement uncertainty. For example, repeat measurements of the attitude of a joint result in a distribution of values for strike with a standard deviation of six degrees. This measurement uncertainty is a part of the dispersion of the sample values.

4) Earthquakes and rainfall, which have an adverse affect on the stability of a slope, are controlled by such a complex interaction of factors that the magnitude and time of occurrence approach a random event. The prediction of their occurrence during the life of an open pit is made primarily by a statistical extrapolation of past events.

Accepting that an exact stability can not be determined for a specific slope configuration, an estimate of the probability of instability can be computed from the distribution of the parameters involved. This method is discussed in the following sections.

Geotechnical Modeling and Data Collection

For stability analysis, the geology of the open pit must be approximated by a model that is amenable to quantification. The model would consist of the following: major structural features, such as lithologic contacts and faults, which have dimensions comparable to the pit and can be located in space; structural domains, which are the volumes of rock bounded by major features; and a population of fractures (joints, bedding, foliation, etc.) within the structural domains. In some geologic environments, particularly porphyry copper, there is a third category consisting of faults with lengths of 50 to 100 m that is referred to as intermediate structures.

Major and intermediate structures can be mapped using conventional geologic techniques. Orientation, length, and spacing of fractures can be obtained by fracture mapping techniques (Call et al., 1976; Baecher et al., 1977). Subsurface techniques, such as oriented core, can be used to supplement surface data and may be required if there are no surface exposures.

The structural domains and major structural features are projected onto a trial final pit geometry and design sectors are determined based on wall orientation, wall height, and structural domains. The term "trial final pit" is used because slope design is an iterative process. Since wall orientation, wall height, and rock type are needed for the stability analysis, the pit geometry is needed. However, the pit geometry cannot be determined until the slope angles are provided. Therefore, a set of angles must be initially selected to develop the trial pit. After the optimum angles are determined, the pit must be redesigned and the new geometry re-evaluated to determine if changes in the pit geometry will affect the optimum angles.

Rock Strength

Direct shear testing will provide the shear strength of fractures and faults. The mean shear strength and the dispersion of the mean for each type of failure surface are required for the stability analysis (Call, 1981). The dispersion of the mean rather than the dispersion of the population should be used since an individual sample has an area orders of magnitude less than the area of a significantly sized failure surface.

Stability Analysis

Slope design involves analysis of the three major components of a mine slope: bench configuration, interramp angle, and overall slope angle (Figure 3). The bench configuration is determined by catch bench width requirements, bench height, and the bench face angle, which is affected by fracture controlled failure geometry and blasting. The interramp slope angles are affected primarily by intermediate and major structure failure geometry. Because of the higher stresses, the overall slope may be additionally affected by rotational shear failure and stress induced failure of intact rock at the toe of the slope, referred to as block flow (Coates, 1978).

Stability analysis requires simplified geometric representations of the actual failure mechanisms that can be treated numerically. The models used are plane shear, wedge, step path, rotational shear, and general surface. These are the common models for rock slope stability (Baecher, 1979).

By plotting the pit wall orientation of a design sector on Schmidt plots of the rock fabric and major structures, the input for stability analysis can be developed (Figure 4). The fractures and major structures are sorted by the failure type orientations and the attitude, distribution length, and spacing distributions computed. These design sets may not correspond to geologic sets, although the orientation boundaries can be adjusted somewhat to avoid splitting a geologic set. We have found that defining sets by visual or mathematical anlaysis, while appropriate for geologic fabric analysis, is less satisfactory for slope design, and it is best to use the wall orientation for determining design sets.

Probability of Instability

In the structurally controlled failure models, the probability of failure (P_f) for a single occurrence of a specified failure model has three parts:

1) the probability that the dip exists (P_d);

2) the probability that the structure is long enough (P_l); and

3) the probability of sliding (P_s).

The probability of dip (P_d) and the probability of length (P_l) are calculated from the statistical distributions of the geologic structures.

The probability of sliding (P_s) is determined by calculating the probability that the shear stress exceeds the shear strength along the failure surface. This probability is calculated from the distribution of safety factors generated either by Monte Carlo or by the application of closed form mathematical modeling.

Using the calculated mean and standard deviation of the distribution of safety factors and assuming a standard normal distribution, the probability of sliding, or the percentage of the total area of the distribution less than 1.0, can be calculated.

The probability of failure (P_f) for a single occurrence of the particular failure mode is the probability that the mechanism is viable and that it will displace.

$$P_f = P_l * P_d * P_s \ .$$

Since more than one potential occurrence of a specified failure mode can occur in a design section, the expected number of failures is the probability of failure times the probability of occurrence of the structures that constitute the failure geometry. Although the actual number of failures that will occur may be more or less than the expected number, it is the best estimate for design.

Utilizing the expected number of failures and the failure volumes calculated in the stability analysis, a probability of failure and expected failure volume curves can be developed (Figures 5a and 5b). The curves for all the potential failure modes can be composited to produce an expected failure volume curve for the design sector (Figure 6).

Earthquake Influence on Stability

Slope displacement will occur if the dynamic forces generated by earthquake-induced ground motion are large enough. The response of a slope to the external forces generated by an earthquake will depend mostly on the ground acceleration, the duration of the event, the rock mass strength, and the slope geometry. Slope movement, if it occurs during the seismic event, is assumed to cease when the event ceases. By calculating the total displacement that occurs during the event, a failure can be defined as that situation where displacement is great enough to disrupt normal mining operations (Glass, 1982). Probabilities of failure are generally increased, but often not significantly, when earthquake forces are included.

Costs of Failure

Given the expected number of failures and expected failure volume, the cost of slope failure can be estimated. The cost of a slope failure is a combination of various factors, each dependent on the type and location of the failure. Failure costs include cleaning up the failure material, reestablishing access on failed ramps and roads, repair of facilities, disruptions in operations, and value of unrecoverable ore.

Cleanup Costs

A cleanup cost (mining cost) is applied to the total expected failure tonnage per sector. This is true even though it may not be necessary to remove all of the failed material. Average mining costs are generally increased by about 20 percent for removing failed material to allow for reduced productivity created by adverse operating conditions in the cleanup area.

Haulroad Repair Costs

Haulroad failures, including rail and conveyor roadways, can be slides which cover the roadway and/or cause the roadway to fail (Figure 6). The greatest total economic impact occurs when a haulroad fails and backfilling results not only in lost production but in lost or buried ore.

Factors which influence haulroad repair costs include:

1) sector height,

2) average failure height,

3) total sector failures,

4) sector width,

5) haulroad length,

6) backfill cost per ton
   - mined waste
   - dump waste, and

7) rail-conveyor repair cost per foot.

If backfill material is available from normal stripping, then only any additional costs associated with diverting this material to backfill are included; however, if other material must be used, the total cost of handling this material will be included in failure costs.

Cost of Lost Production

It is assumed that lost ore production takes place whenever an ore area access ramp or an ore haulage roadway failure occurs. Production is also lost when failures result in damage to critical mine facilities. In a large pit it may be assumed that sufficient working faces are available; so ore production is not affected by other failures.

Cost of lost production is usually determined for various shutdown periods from which an average daily shutdown cost is developed. The daily shutdown cost can be estimated by subtracting fixed costs from the net income.

Costs would be pro-rated for partial cutbacks in production. Fixed costs are those costs which continue when an operation is not operating.

Total days of lost production for a failure is the sum of days lost for the following reasons.

1) Pre-failure road closures for safety reasons.
2) Delay between time of failure and start of repairs.
3) Cleanup and/or backfill times based on expected productivities per shift and whether these functions are scheduled only during daylight hours for safety reasons.
4) Repair time for railroad, conveyor, and other facilities.

Unrecoverable Ore

Backfilling will result in buried or unrecoverable ore when the interramp pit slope angles are greater than the angle of repose for fill material. It is assumed that only backfilled failures will result in lost ore. Cost of unrecoverable ore is estimated in the following manner.

1) If failure occurs in ore, the cost is equal to the benefits attributable to the buried ore.

2) If failure occurs in waste, the cost is equal to the benefits attributable to the buried ore, less the cost of waste not mined.

Surface Facility Repairs

Failure costs include all repair costs associated with damage to structures such as crushers, conveyors, railroads, substations, concentrators, and other miscellaneous structures. This also includes cost for relocating facilities, if required.

Engineering and Monitoring

Probabilistic analysis of slope instability is based on the premise that some slope failures will occur. Engineering and monitoring must be on-going functions to ensure that the effects of any instability are minimized; however, as areas of instability are recognized, increased engineering and monitoring costs will be incurred.

Cost of Failure Summary

Figure 7 represents a format summarizing the expected failure costs previously discussed. Each slope angle represents a final pit plan with individual and total costs. Incremental costs represent the cost per degree of change as slopes are steepened.

Bench Design

Bench design is a special case of geotechnical reliability analysis. Bench faces are normally mined as steeply as possible; as a result, rock falls and raveling are inevitable. Thus, it is customary, and in many cases mandated by mining regulations, that catch benches be left in the pit wall to retain rock falls and raveling.

Analysis of rock fall mechanics by Ritchie (1963) demonstrated that falling rocks impact relatively close to the toe of the slope, but, because of horizontal momentum and spin, can roll considerable distances from the toe. Based on his analysis, Ritchie developed width and depth criteria for a ditch at the toe of a slope to protect highways from rock fall. The concept was that the rock would impact in the ditch and the side of the ditch would stop the horizontal roll.

It is not practical to excavate a ditch in an open pit catch bench, but the same effect can be achieved by casting up a berm (Figure 8). Assuming the berm can be emplaced with slopes of 1.3 to 1, the minimum bench width criteria presented in Figure 8 is recommended for open pit catch benches. For a given bench height and corresponding bench width, the upper limit of the interramp slope angle becomes a function of the bench face angle.

The bench face angle, however, is not a unique value because the variability of the rock fabric produces varying amounts of backbreak. Backbreak is defined as the distance from the design bench crest to the actual bench crest. Figure 9 is an example of the cumulative frequency distribution of measured bench face angles and theoretical bench face angles. The theoretical bench face angle is obtained from stability analyses, assuming a vertical bench face, and is the upper limit of possible bench face angles because it does not include the effects of blasting and digging. Comparison of measured and theoretical face angles at several properties gave a difference of 17° to 20°, except where the bench face was controlled by a strong geologic structure, such as bedding or foliation. In those cases, the measured and theoretical bench face angles were the same.

For an operating property, the measured bench face angles or the adjusted theoretical angles can be used for design. For a new property, the theoretical bench face angle adjusted for the effects of blasting must be used. Rather than choosing the mean bench face angle, which would result in 50 percent of the catch benches being less than the minimum width, or the minimum bench face angle, which would result in unnecessarily flat slope angles, it is recommended that a desired catch bench reliabiilty be chosen based on the potential for rock fall and the exposure of personnel. Catch benches in raveling ground mined by front-end loaders should have a higher reliability than catch benches in massive ground mined with a large rope shovel. A bench face angle should then be chosen to give the desired reliability. For example, if an 80 percent reliability is desired, the bench face angle would be the angle where 80 percent of the bench faces will be steeper than the design angle shown in Figure 8. Using this reliability criteria, 80 percent of the benches will be wider than the minimum width (Figure 9).

In a number of cases where the geology is favorable and economic benefits large, the catch bench criteria limited the slope height and the cost benefit economic optimum could not be achieved.

Financial Risk and Safety

A common objection to using probabilistic slope design and a financial cost benefit optimization results from the assumption that an unstable slope is by definition an unsafe slope and that it is unethical to "gamble" with safety. A response is that, with the inherent uncertainty in predicting the behavior of a slope, there is no such thing as a guaranteed safe slope and it is more ethical to acknowledge the uncertainty and take precautionary measures than to, in effect, deny the uncertainty by saying the slope is safe because it has a safety factor of 1.5.

Major slope displacement is preceded by small, but measurable, displacements and by other indicators of instability, such as tension cracks, rock noise, and changes in groundwater levels; therefore, a monitoring program capable of measuring and assimilating displacement related data will provide evidence of instability so that precautionary measures, such as restricting access, can be taken (Call, 1982).

By estimating the cost of a monitoring system, which will increase with increasing instability, safety can be expressed in economic terms for input to cost-benefit optimization.

References

Baecher, G. B. et al., 1977, "Statistical Description of Rock Mass Properties and Sampling," _18th U.S. Natl. Rock Mechanics Symposium_.

Baecher, G. B., and Einstein, H. H., 1979, "Slope Reliability Models in Pit Optimization," _16th Intl. APCOM Symp._: SME of AIME, New York, New York.

Call, R. D., 1971, "Evaluation of Material Properties," _SME Workshop on Nonwater-Impounding Mine Waste Structures_: SME of AIME short course.

Call, R. D., 1982, "Monitoring Pit Slope Behavior," _3rd Intl. Conf. on Stability in Surface Mining_: SME of AIME, New York, New York.

Call, R. D. et al., 1976, "Estimation of Join Set Chracteristics from Surface Mapping Data," _17th U.S. Symp. on Rock Mechanics_: Utah Eng. Exp. Station, University of Utah, Salt Lake City, Utah.

Coates, D. F., and Yu, U. S., 1978, Block Flow Slope Instability: CANMET Report 78-10, Canadian Dept. of Energy, Mines, and Resources.

Glass, C. E., 1982, "Influence of Earthquakes on Rock Slope Stability," _3rd Intl. Conf. on Stability in Surface Mining_: SME of AIME, New York, New York.

Kim, Y. C. et al., 1977, Pit Slope Manual Supplement 5-3, Financial Computer Program" CANMET Report 77-6.

Ritchie, A. M., 1963, "The Evaluation of Rockfill and its Control": HIghway Record, vol. 17, p. 13-28.

Whitman, R. V., 1984, "Evaluating Calculated Risk in Geotechnical Engineering," 17th Terzaghi Lecture: ASCE Jour. of Geotechnical Engineering, vol. 110, no. 2.

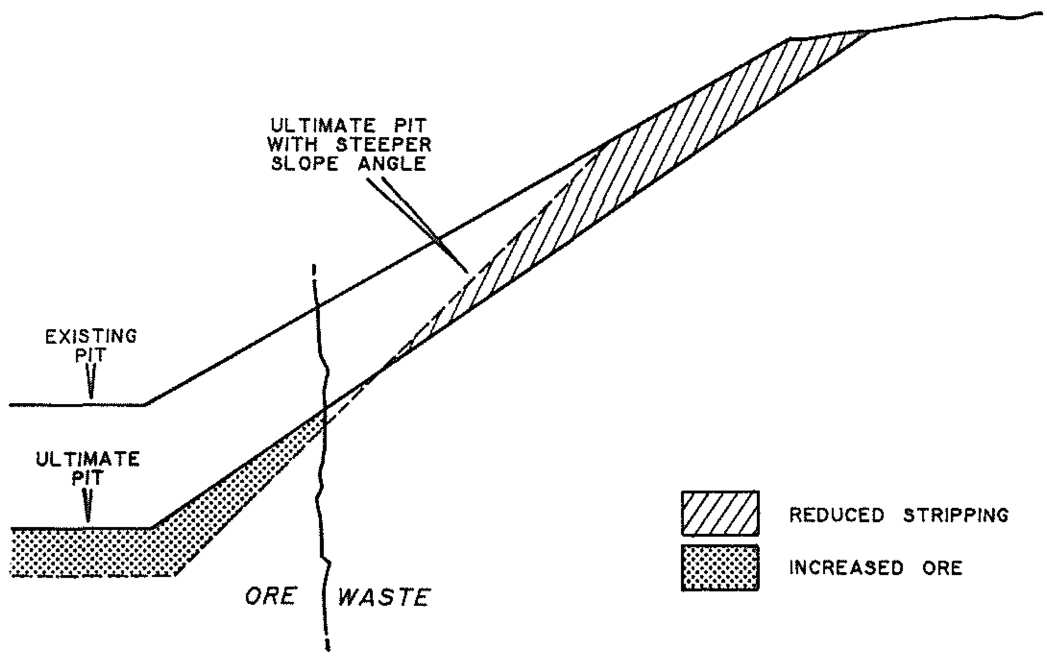


Figure 1: Effect of Changes in Slope Angle.

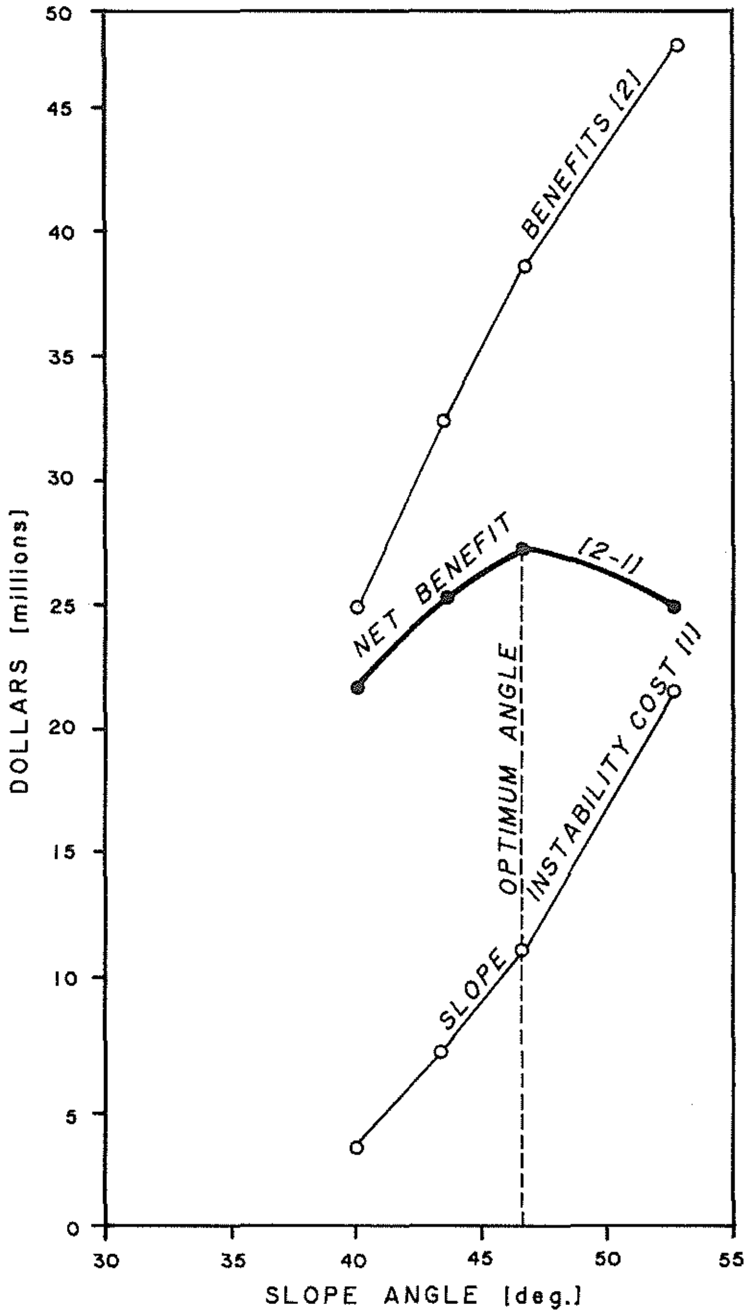


Figure 2: Cost-Benefit Curves.

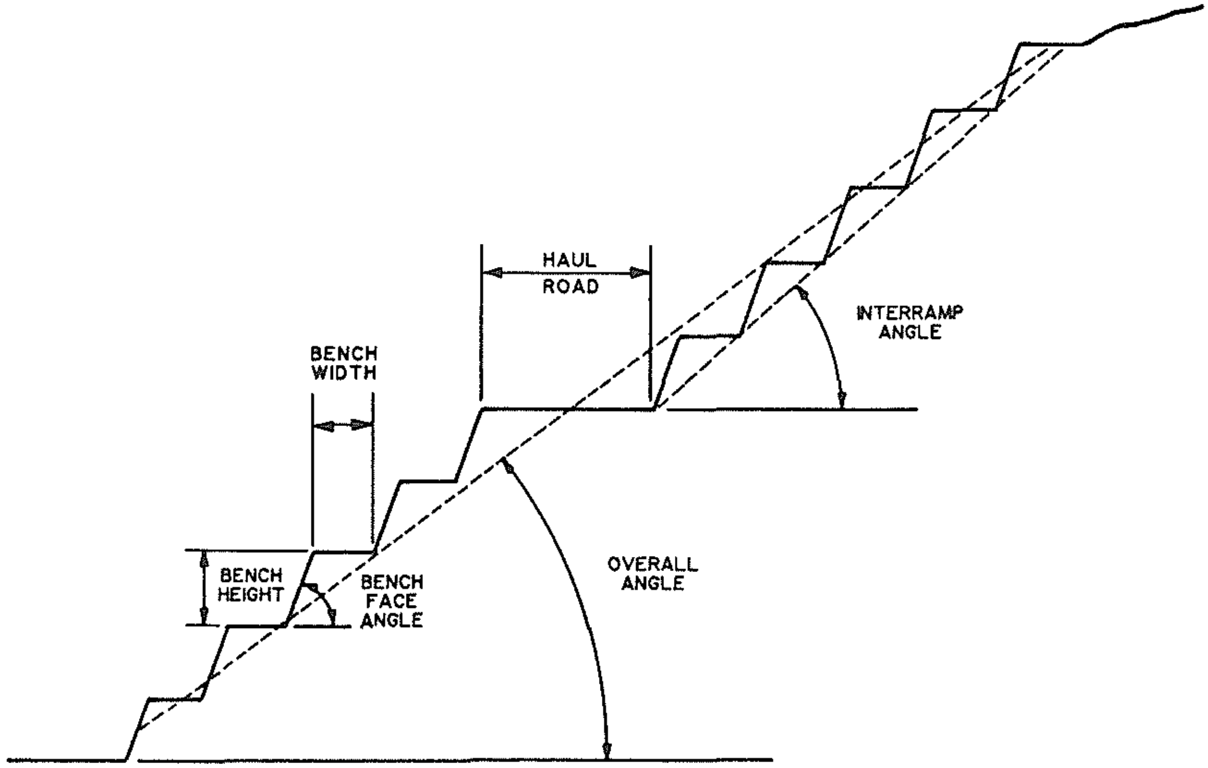
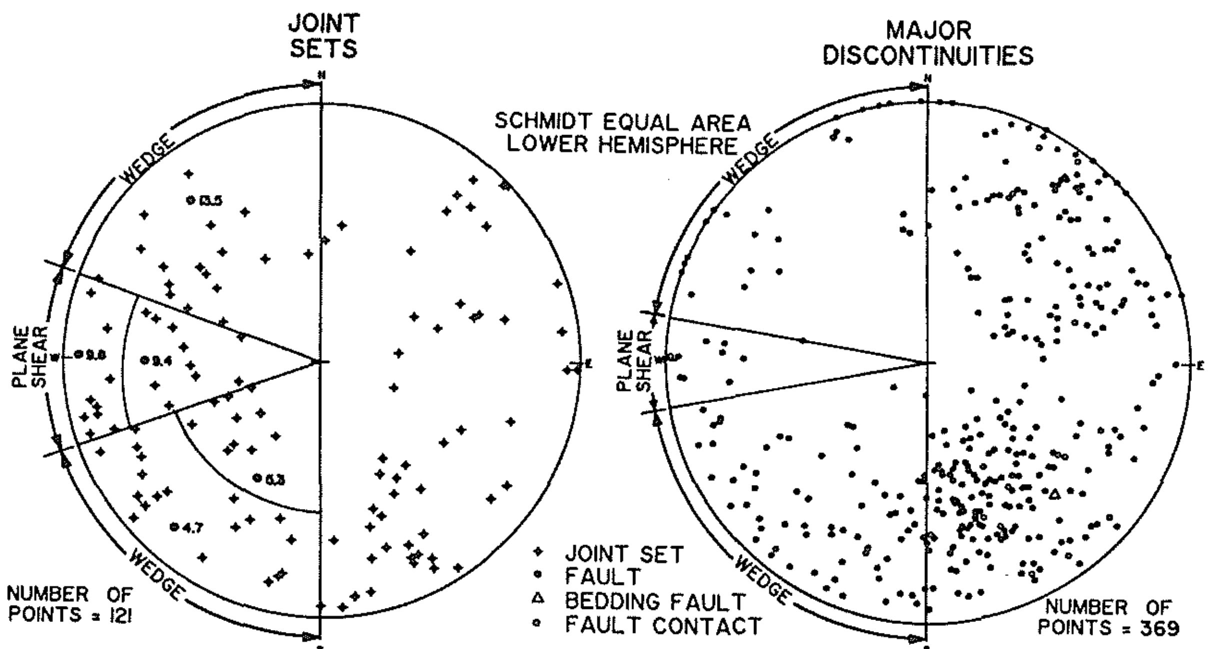


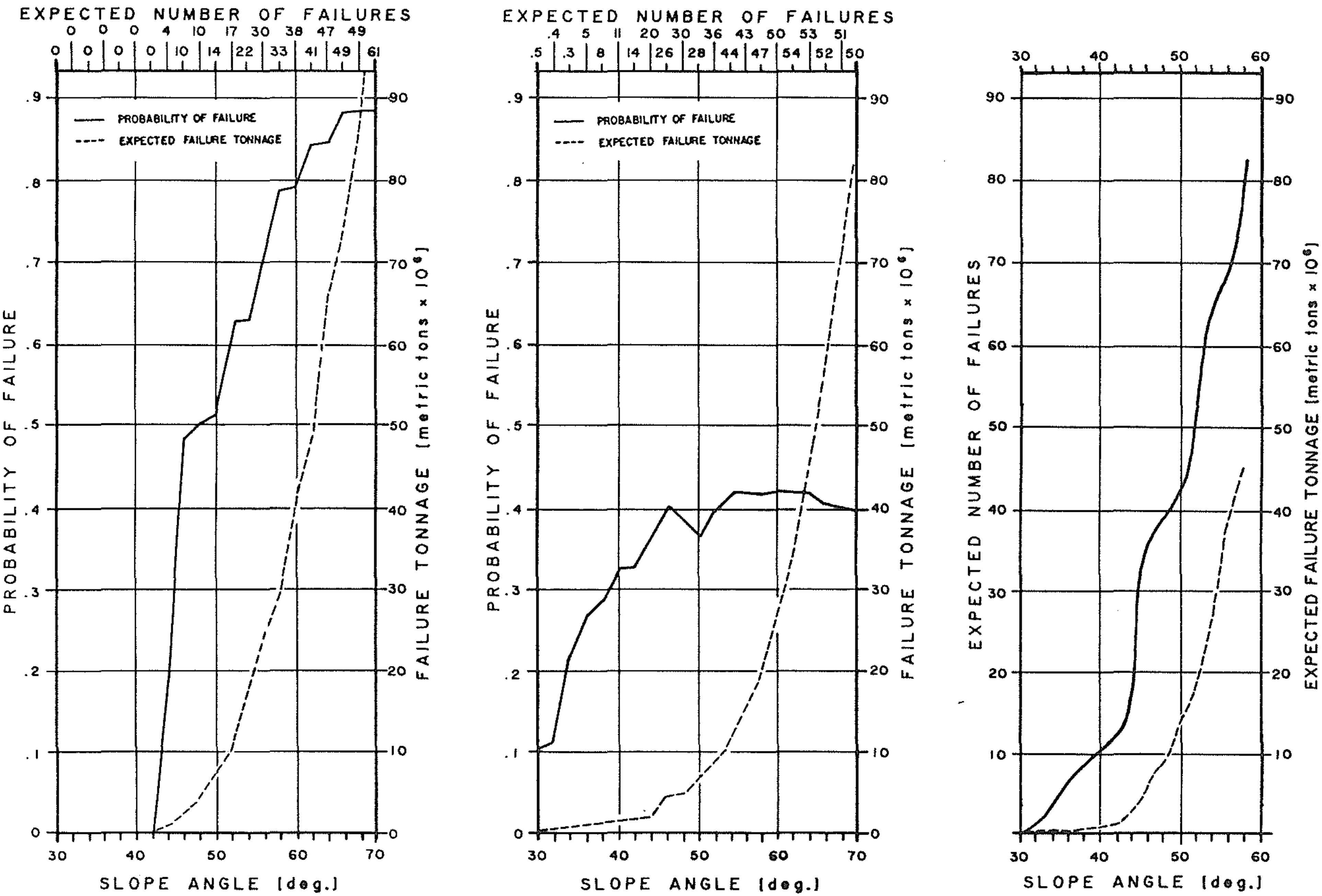
Figure 3: Definition of Slope Geometry.



Design Set	Dip Direction Mean (degrees)	Dip Direction S.D. (degrees)	Dip Mean (degrees)	Dip S.D. (degrees)	Length Mean	Length S.D.	Spacing Mean	Spacing S.D.
4.7	38.1	22.3	71.7	10.2	6.7'	4.1'	.54'	.41'
5.3	52.7	13.5	34.6	7.2	4.8'	5.2'	.65'	.55'
9.8	86.1	13.9	79.6	5.6	3.9'	3.3'	.51'	.34'
9.4	91.1	12.9	45.5	11.5	5.2'	3.9'	.49'	.16'
13.5	133.7	20.3	52.6	13.6	6.8'	5.3'	.61'	.49'

Length Mean	Spacing (Mean) LW	Spacing (Mean) RW	Spacing (Mean) PS
233'	417'	1139'	2963'

Figure 4: Design Set Determination.



(a) (b) (c)

Figure 5: Expected Failure Tonnage Curves.

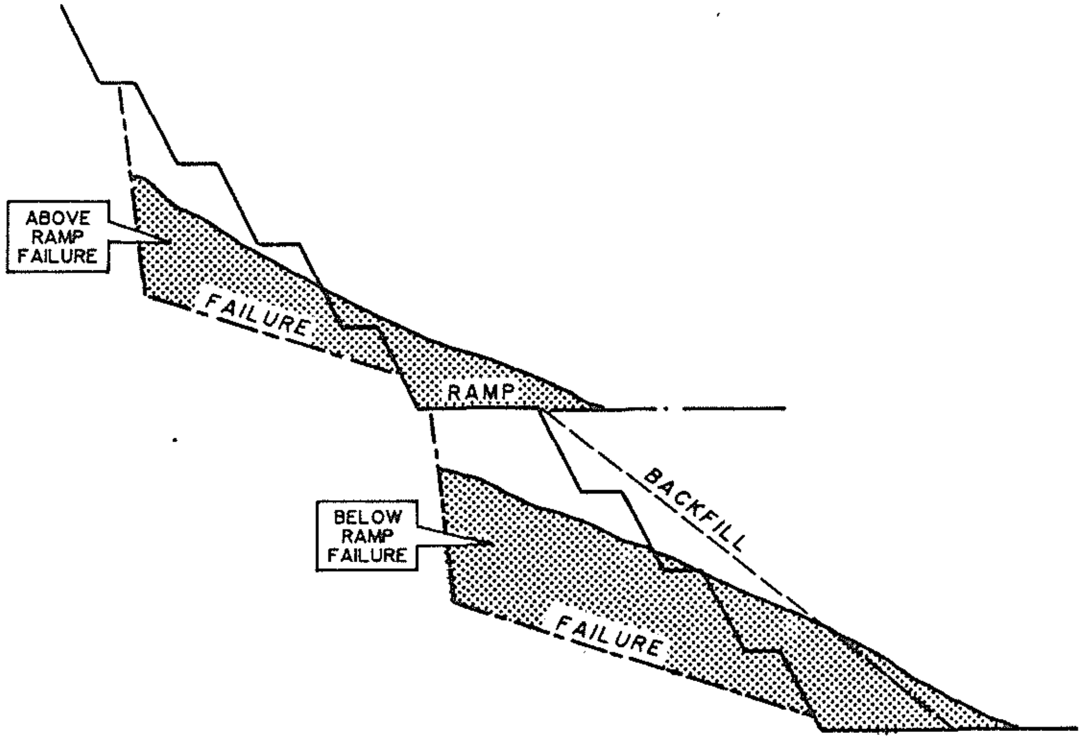
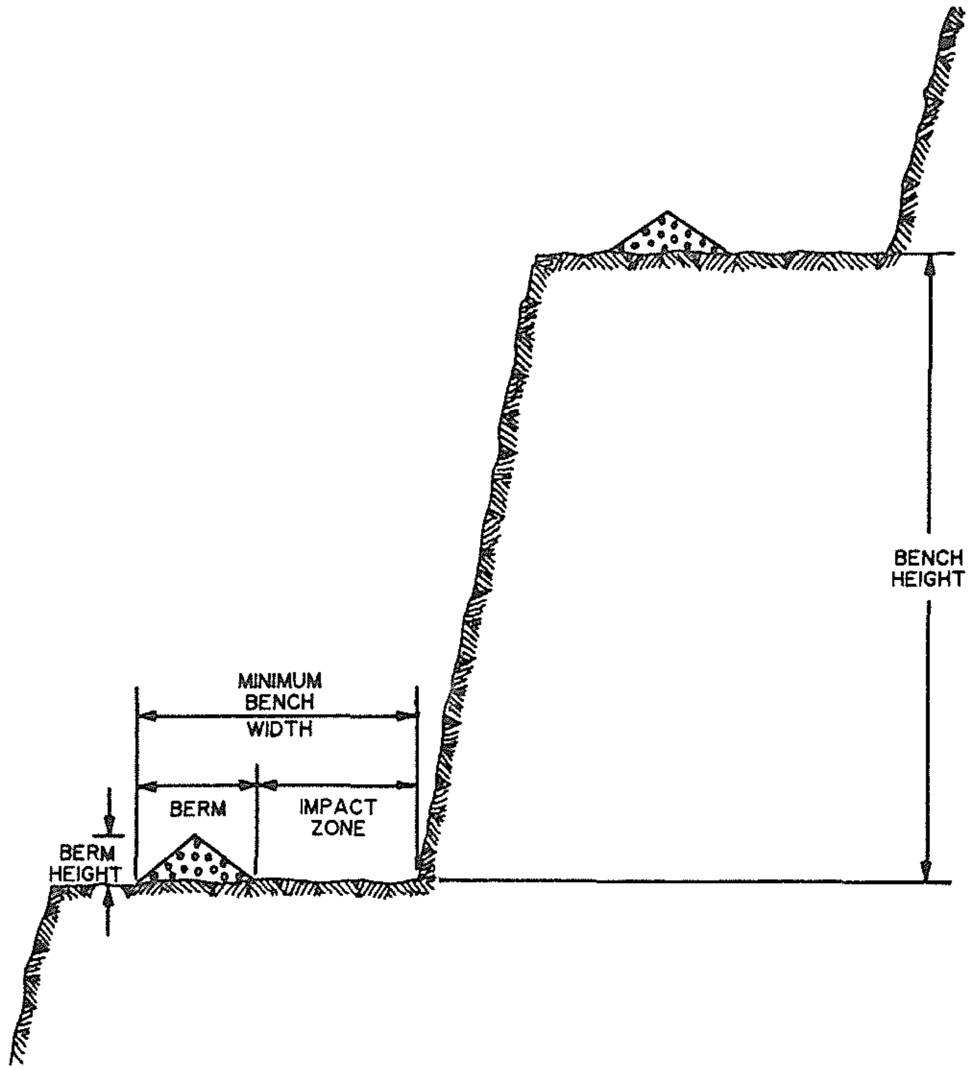


Figure 6: Haulroad Failure Modes.

SLOPE ANGLE	CLEANUP	HAULROAD REPAIRS	SURFACE FACILITIES REPAIRS	LOST PRODUCTION	UNRECOVERABLE ORE	ENGINEERING AND MONITORING	TOTAL COSTS	INCREMENTAL COST/DEGREE
36	$	$	$	$	$	$	$	$
40	$	$	$	$	$	$	$	$
44	$	$	$	$	$	$	$	$
48	$	$	$	$	$	$	$	$
52	$	$	$	$	$	$	$	$
56	$	$	$	$	$	$	$	

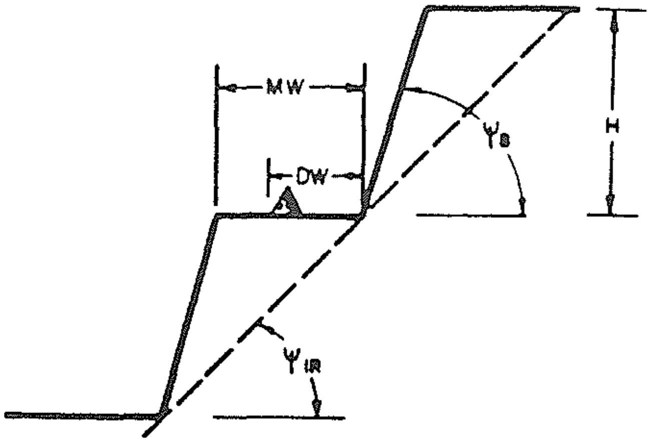
Figure 7: Failure Cost Summary.



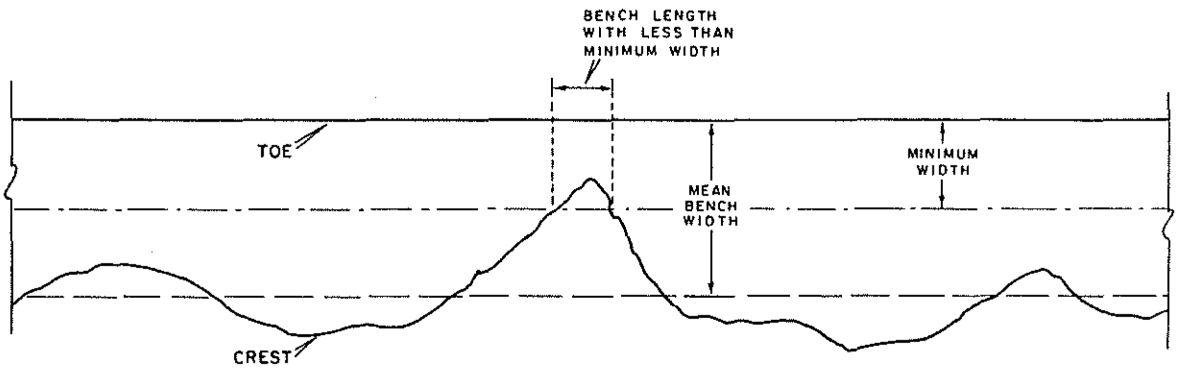
Design Catch Bench Geometry

Bench Height (M)	Bench Width (I)	Berm Height (M)	Berm Width (M)	Impact Zone (M)
15	7.5	1.5	4	3.5
30	10	2	5.5	4.5
45	13	3	8	5

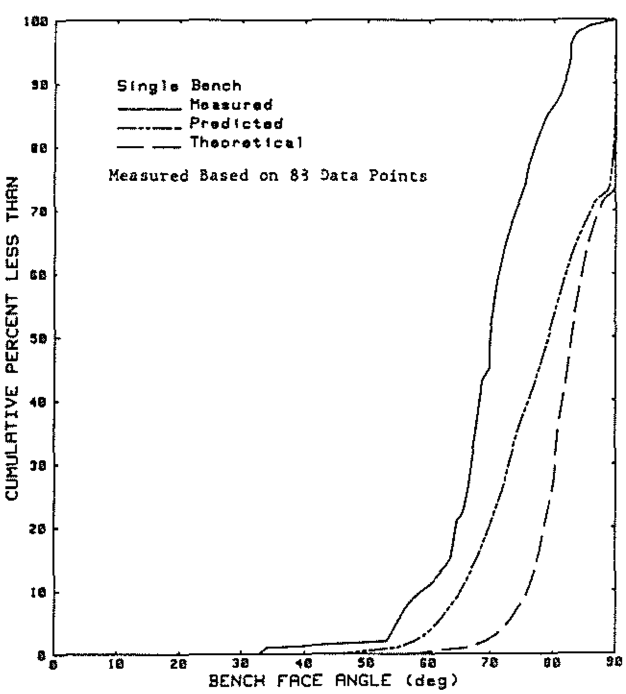
Figure 8: Catch Bench Geometry.



DW	REQUIRED BENCH WIDTH
MW	MEAN BENCH WIDTH
H	SINGLE BENCH HEIGHT
Ψ_B	MEAN BENCH FACE ANGLE
Ψ_{IR}	INTERRAMP ANGLE



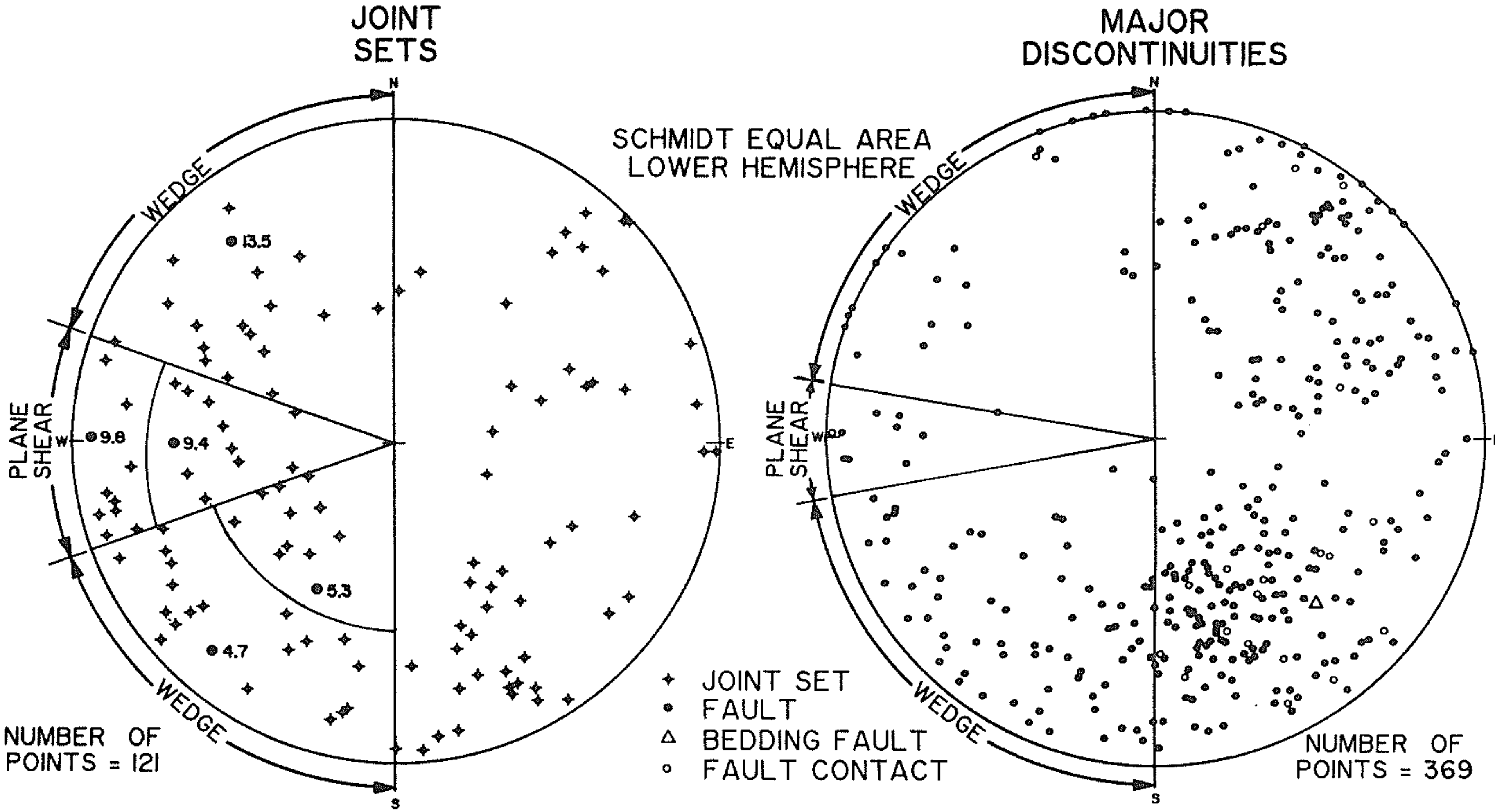
$$\text{RELIABILITY} = \frac{\text{Bench Length Greater Than Minimum Width}}{\text{Total Bench Length}}$$



SINGLE BENCH MEASURED BENCH FACE ANGLES
SB MEAN FACE ANGLE: 69.9
DB MEAN FACE ANGLE: 69.9

SINGLE BENCH (HT=50ft)			DOUBLE BENCH (HT=100ft)		
SLOPE ANGLE deg	% RELIABILITY:MIN. BCH WIDTH OF 25 FT percent	MEAN WIDTH ft	SLOPE ANGLE deg	% RELIABILITY:MIN. BCH WIDTH OF 35 FT percent	MEAN WIDTH ft
30	98	68.3	30	98	136.6
31	98	64.9	31	98	129.8
32	98	61.7	32	98	123.4
33	98	58.7	33	98	117.3
34	98	55.8	34	98	111.6
35	98	53.1	35	98	106.2
36	98	50.5	36	98	101.0
37	98	48.0	37	98	96.1
38	98	45.7	38	98	91.4
39	97	43.4	39	98	86.9
40	94	41.3	40	98	82.5
41	91	39.2	41	98	78.4
42	90	37.2	42	98	74.4
43	89	35.3	43	96	70.6
44	87	33.5	44	93	66.9
45	85	31.7	45	91	63.4
46	78	30.0	46	90	59.9
47	71	28.3	47	89	56.6
48	60	26.7	48	88	53.4
49	55	25.1	49	86	50.3
50	40	23.6	50	83	47.3
51	33	22.2	51	78	44.3
52	29	20.7	52	71	41.5
53	24	19.4	53	61	38.7
54	19	18.0	54	55	36.0
55	15	16.7	55	43	33.4
56	13	15.4	56	36	30.8
57	10	14.2	57	32	28.3
58	3	12.9	58	28	25.8
59	1	11.7	59	23	23.4
60	1	10.5	60	19	21.1

Figure 9: Catch Bench Design.



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