CHAPTER 3

Designing Catch Benches and Interramp Slopes

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3.1 INTRODUCTION

Individual mine bench and interramp slope stability in rock is governed primarily by the rock's geologic structural characteristics. Because of the complex structural variability in most rocks, slope stability cannot be adequately described by a single parameter or index. Instead, rock slopes must be evaluated using probabilistic methods, whereby the full range of geologic variability can be incorporated into the stability analysis. Mine-slope-stability evaluation lends itself readily to risk-management methods, where the criteria (e.g., catch bench width or number of multiple bench failures) are developed using a reliability-based approach. This chapter provides a general overview of the approach that Call & Nicholas, Inc. (CNI) recommends for analyzing structurally controlled failures. The chapter also provides specific details regarding CNI's in-house computer-based analyses.

3.1.1 Applicability of Structure-Based Models

Experience tells us that no single model or form of analysis is applicable in every case. This is particularly true for rock slopes where the materials encountered range from weak soil-like units with very little structure to extremely competent units that are highly fractured. Therefore, a wide range of analyses is available, and each analysis has its own unique areas of application.

Limit-equilibrium methods can be used to look at discrete plane-shear or step-path failures. However, these methods are not suitable for wedge analyses because they underestimate stability based on a two-dimensional analysis of plunge of the intersection. Using limit-equilibrium methods, the number of runs required to account for variations in geologic structure (orientation, length, and spacing) and rock-strength parameters is prohibitive, except where discrete structures are modeled or where failures are controlled by overall rock-mass strength rather than by structure.

Elastic/plastic modeling methods, such as finite element, finite difference, and boundary element, also have their place in slope-stability evaluations (Cicchini and Barkley 2000). These models are very applicable for evaluating high slopes where significant stresses are generated, resulting in strain and yielding in the rock mass. However, these models cannot take the place of structural analyses in situations where potential failure modes are structurally controlled. They do not handle the variation in orientation, length, and spacing of several major structure sets and they do not deal with three-dimensional wedge-failure geometries.

Therefore, for slopes that consist of materials that have reasonable rock substance or rock-mass strengths and a moderate to high degree of fracturing or faulting, three-dimensional structural analyses of potential plane-shear and wedge-failure geometries are critical to understanding the slope behavior.

3.2 CATCH BENCH STABILITY

3.2.1 Individual Bench Configuration

Development of catch bench criteria in mine slopes is necessary in areas of rockfall because the catch benches prohibit rocks from rolling from upper portions of the pit slope to the working areas where personnel and equipment are located. Often overlooked, the bench geometry defines the steepest interramp slope that can be mined while maintaining adequate catch bench widths. The two primary factors that control bench configuration are the type of mining equipment that is used and the bench face angles that can be achieved. The type of mining equipment determines the height at which the bench can be adequately scaled. The achievable bench face angles are controlled by rock strength, geologic structure characteristics, and the mining techniques used to construct the slope (the blasting and digging practices, for example). The local stability of the benches is, largely, a construction issue; the stability condition is created contemporaneously with the mining as the equipment digs toward the final slope limits. It is the one area of the slope where the mining practices exert a large degree of control over slope stability. Conversely, slope stability analysis is needed to optimize the mining methods. It is necessary for the mine operator to know where the final blasthole rows should be located and where to place the flags that define the final digging limit for the shovel and loader operators (the *drop-flag* line).

The final, benched slope configuration is a function of the bench height, the bench face angle, and the required catch bench widths (Figure 3.1).

Bench Heights. Currently, most large mining operations drill and blast on 12- to 15-m intervals (40 to 50 ft), with 15-m intervals being the most common. The mining equipment used to drill and blast the rock determines the bench height. Catch benches can be left either at every mining level (single benching) or at every other mining level (double benching).

Bench Face Angles. In weaker rocks, face angles are often controlled by the equipment and digging technique used while mining. In hard rock, which cannot be mechanically ripped but must be blasted, the stable bench face angles are controlled primarily by the stability of the local geologic structure. In reality, in a large open-pit mine slope, the local bench face conditions are a complex mix of both hard, jointed rock and weaker, altered rock. Bench face angles are not unique; because of the variable geologic character of rock, the stable face angle takes the form of a statistical distribution or a probability density function (PDF). Since the rock structure controls the achievable face angles, bench slope stability also varies as a function of the slope orientation because the mode of sliding for structures is dependent on the wall orientation.

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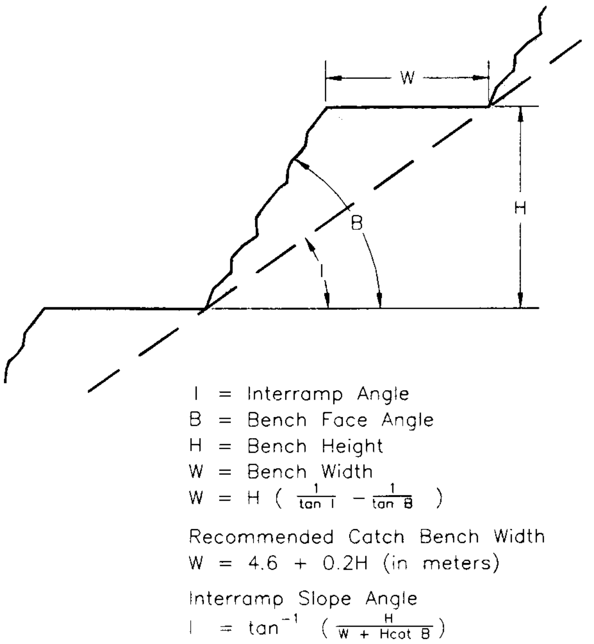


FIGURE 3.1 Relationship between interramp and bench face angles

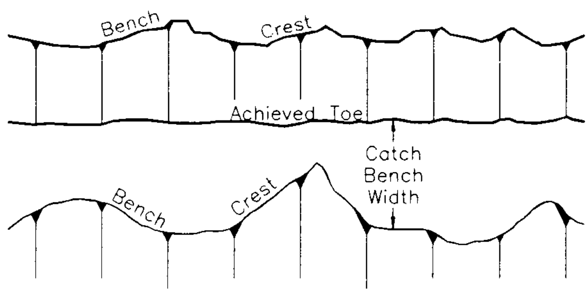


FIGURE 3.2 Catch bench width variability in plan view

Bench Widths. The catch bench widths vary from point to point within the slope because of the variability in geologic structure, which produces varying amounts of *back break* along each mining level. The final face angles are a distribution rather than a single value therefore the achieved bench widths will also be highly variable (Figure 3.2).

Rockfall Mitigation. CNI employs a Modified Ritchie Criteria as a *guide* for developing catch bench width criteria. The Modified Ritchie Criteria evolved following publication of a paper by Ritchie in 1963 on the evaluation of highway shoulders for catching rockfall off excavated and natural slopes. His investigation was limited to a relatively small number of slope-angle/slope-height geometries and therefore required extrapolation for use in open-pit mining. Since slope (bench) height is one of the most important controls on the distance that a rock will travel when detached from the bench, we derived an empirical relationship between the slope height and the average, or preferred, catch bench width as follows:

$$\text{bench width (m)} = 0.2 \times \text{bench height} + 4.5\ \text{m} \qquad \textbf{EQ. 3.1}$$

This equation is published in several papers by Dr. Richard Call and in the *SME Mine Engineering Handbook* (1992). Other researchers have developed rock catchment criteria (Pierson, Davis, and Pfeiffer 1994) from field tests and from computer simulation methods (Colorado rockfall simulation), and most catch bench width criteria include some relationship between block size, bench width, and slope height (Figure 3.3).

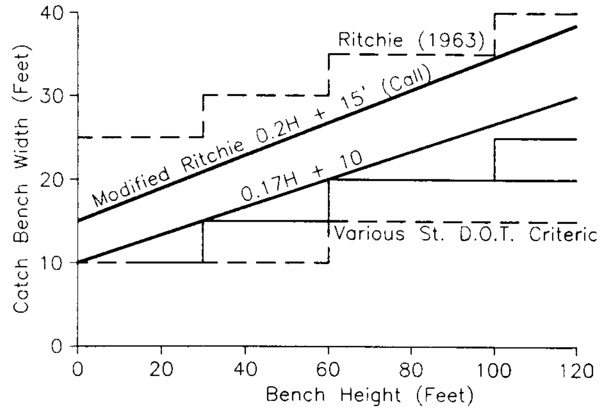


FIGURE 3.3 Catch bench width criteria for rockfall containment

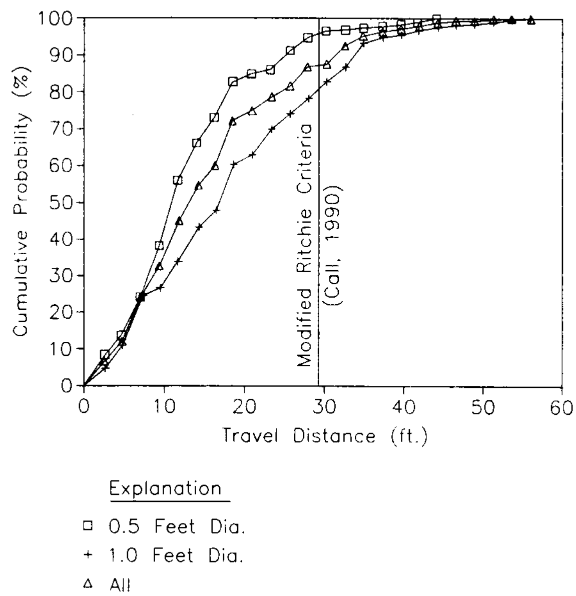


FIGURE 3.4 Rockfall experiment results (Evans 1989)

For many years, we have tried to refine the bench width equations for pit slope stability but have found that the problem is too complex for any single criterion to be 100% effective. This position was supported effectively by the research conducted at the University of Arizona (Evans 1987; Yost 1995). In field tests we have been able to demonstrate that Call's catch bench width criteria is effective in benched mine slopes (Figure 3.4).

In those field tests, the Modified Ritchie Criteria was found to be very conservative for containing rockfall in a limestone slope benched on 40- to 50-ft heights for blocks anywhere from 30 cm to 2 m in size. Because of the complexity of the problem, we have been forced to approach the rockfall problem from a risk-management perspective, and that is why we have increasingly used the reliability-based approach for evaluating benched mine slopes. For the purpose of discussion, the method that follows uses the Modified Ritchie Criteria, as presented by Call, but other catch bench width criteria could be substituted.

3.2.2 Catch Bench Criteria Using a Reliability Approach

In evaluating benched slopes between haul roads, the primary functional requirement is to maintain adequate catch bench widths while optimizing the interramp slope angle on economic criteria. Final bench slope heights (between catch benches) can be any multiple of the drill and blast height. The catch bench widths vary from point to point within the slope because of the variability in

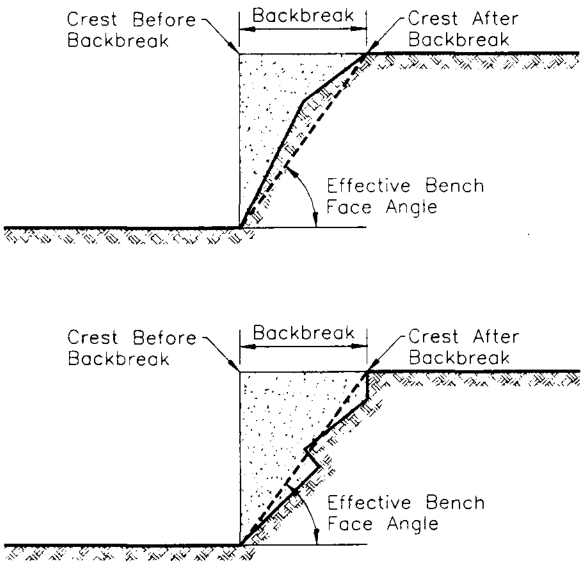


FIGURE 3.5 Definition of back break and effective bench face angle

geologic structure, which produces varying amounts of back break along the bench crest at each mining level. Back break generally is defined as the horizontal distance between the planned toe and the actual mined crest of the final bench slope (Figure 3.5).

This horizontal distance defines the unstable zone within the catch bench. Under ideal conditions (controlled blasting with vertical drill holes in unfractured rock), bench face angles would be nearly vertical. Under actual conditions, however, back break occurs throughout the bench slopes along preexisting jointing and blast-induced fractures. Uncontrolled blasting reduces rock integrity, resulting in further back break.

Because of the variability in geologic structure, namely jointing, catch bench widths vary considerably within the slope for any given interramp slope angle. In such an environment, using modal, median, or mean values for back break (or bench face angle) to evaluate and construct the slope can be misleading if there is a high degree of variability present in the slope. Similarly, it is not appropriate to assume that a *minimum* catch bench width can be incorporated everywhere into the design of the benched slope since, in a highly complex geologic environment, 100% reliability is usually not practical.

CNI has developed a reliability approach in which the analysis is structured to evaluate the percentage of the slope area that meets or exceeds a chosen catch bench width criteria. We have found that this is a more useful risk-management approach for rockfall containment and for slope management. We use a combination of structure modeling, bench face stability analysis, and the Modified Ritchie Criteria to determine the catch bench reliability, which refers to the percentage of benches having final widths equal to or greater than the Modified Ritchie Criteria. The selection of the proper reliability for maintaining a catch bench of a certain width is dependent on many factors including, but not limited to, the following:

- The potential for slope raveling
- The proximity to large slope failures
- The decision to contain overbank from a higher pushback on the benches
- The length of time the benches are expected to be functional
- The climate
- The type of blast control
- The operator's experience

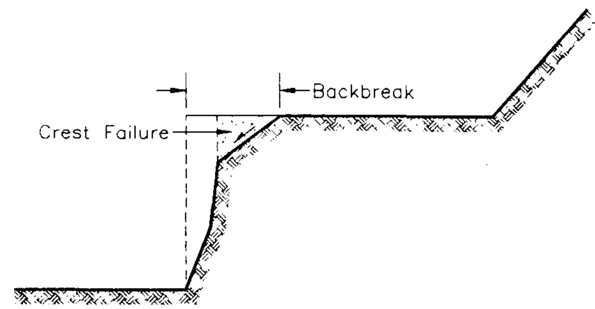


FIGURE 3.6 Crest failures often make up a majority of back break

Since the bench configuration is based on the reliability of the catch bench width, it is the amount of local back break rather than the bench face angle that is of most concern. Although we often use the term "bench face angle" to describe the bench slope, in most cases the bench slope cannot be adequately described by a bench face angle since the actual bench face slope is a complex, three-dimensional surface composed of numerous joint surfaces. Compound bench face slopes composed of several surfaces of different inclinations are not only common, they are the rule in most rock masses, whereas faces formed by one continuous joint face tend to be rare. In fact, practical experience indicates that the majority of back break occurs along short structures near the bench crest. Even where the majority of the bench face slope is quite steep, one flatter joint near the crest can result in significant back break (Figure 3.6).

Therefore, to evaluate the probability of achieving a specified catch bench width, the probability density function for the back break is required. This can be defined by a probabilistic stability analysis of the local rock structures relative to a vertical bench face.

3.2.3 Probabilistic Analysis of Back Break

The local structure data for a region in a mine can be complex. Often, there are many families of structures that show persistence in a region, and these families of structures, or joint sets, must be characterized thoroughly if a probabilistic slope stability analysis is to be conducted for the bench (Call and Savely 1990; Sims and Nicholas 2000). The structural factors that control the stability analysis include the number of joint sets in the region; their probability of occurrence; and their orientation, length, spacing, and frictional shear strength. Each attribute for each structure set must be characterized by mapping if a probabilistic analysis of the back break is to be conducted along the mine benches. From the mapping data, statistical distributions of each parameter for each structure set are developed. The statistical representations are then used in the back break stability model to evaluate bench stability. At CNI, our statistical modeling procedures for the structure can be summarized as follows:

Parameter	Statistical Model(s)	Comments
Occurrence	Deterministic, Cell Method	Probability of occurrence generally is developed using methods similar to CANMET (1977).
Orientation (strike, dip or dip direction, dip)	Normal, Fisher, Spherical	A vector summation is employed more often than simple arithmetic summation.
Length	Neg-Exponential, Weibull, Log-Normal	In cases where structure lengths exceed the outcrop that is being mapped, some extrapolation is required.

(continues)

Parameter	Statistical Model(s)	Comments
Spacing	Neg-Exponential, Weibull, Log-Normal	Spacing must account for length truncation bias (many of the short structures are not usually mapped).
Shear Strength	Deterministic-Gaussian	At low normal stress, shear strength along structures is best modeled as a power curve (Call 1985; Barton 1976).

Statistical models other than those mentioned can be used; however, it should be pointed out that the quality of the final stability model is, to a great extent, determined by the quality of the data that are input.

In the bench slope, stresses are relatively low, and experience shows that the majority of back break occurs as the sliding of blocks along daylighted geologic structure. Numerous papers (Savely 1987) have outlined the essential, sliding-block failure modes for bench slopes. As noted previously, the distribution of back break along each bench is best described by a probabilistic stability analysis. Although there are several different approaches to probabilistic stability analysis, in this case, a Monte Carlo method is used because there is a high degree of complexity to the solution. (There are six to seven key geologic parameters; all key geologic parameters are statistical distributions; several types of statistical models are being simultaneously employed in the solution; multiple failure models are evaluated.) The general procedure for a particular slope orientation is as follows:

1. Determine the potential failure modes in the slope (wedge and plane-shear). Failure modes are defined as structure sets or combinations of structure sets that have a kinematically viable sliding relationship to the slope. (There can be multiple wedge failure modes in a slope, for example.)
2. Determine the maximum back break for each cell or Monte Carlo model *run*.
   a) The following should be determined for each failure mode:
      - Evaluate the probability of occurrence for each structure set in the failure mode.
      - Calculate a theoretical back break for each failure mode occurrence. Sample the statistical model for each key parameter (dip direction, dip, length, spacing) for each structure set involved in the failure mode.
      - Construct the local geometry of the sampled failure mode.
      - Calculate the stability for the constructed geometry. If the geometry is unstable, add it to the theoretical back break model; otherwise, if it is stable, the back break is considered to be zero.

   b) The back break for the other failure modes should be evaluated.

   c) The maximum back break from all of the failure modes should be noted; it should be added to the theoretical back break model for the slope.
3. Combine all of the results once all possible plane-shear and wedge combinations have been evaluated to produce an estimated cumulative PDF for back break. Due to the complexity of the problem, the cumulative PDF for back break tends to have a complicated form that cannot be adequately represented by a statistical model.
4. Convert the back break distance distribution to an equivalent bench face angle distribution.

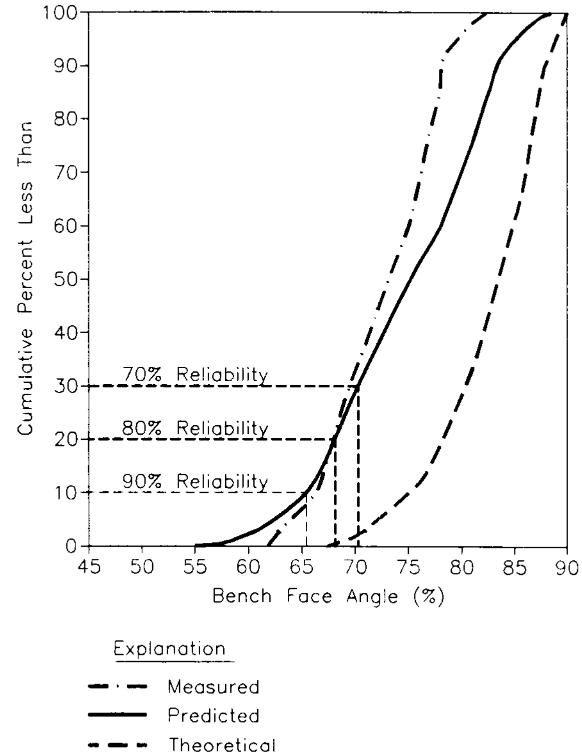


FIGURE 3.7 Effective bench face angle distributions, theoretical vs. predicted vs. measured

This procedure is well suited to computer programming since there are many variables and many model runs are required to achieve a consistent result. A computer-based numerical analysis called *Backbreak* has been in development at CNI since the early 1980s. This computer program uses rock strength and structural fabric data from pit mapping programs to predict back break and bench face angle distributions, based on the size and type of structural failures that can be expected along the benches as they are mined. The output from *Backbreak* is a probability distribution of the back break distance to be expected in a mine sector. The back break distance is then converted to an equivalent bench face angle (Figure 3.5) and is compared to measured values for calibrating the analysis. We like to convert the back break distance to an equivalent bench face angle since it provides us with a benchmark distribution, which we can compare to the face angle estimates made by the geologists who are conducting the routine bench mapping in the pit (Figure 3.7).

Where both the toe and the crest of the bench can be adequately surveyed, actual back break distances can also be measured and compared to the model results. Both the measured benches and the computer analysis are required to assess bench slope stability since the operational practices can reduce the stable face angles that are achieved. An empirical correction factor is then applied to the calculated (theoretically achievable) bench face angle distribution to account for the effects of controlled blasting and excavation. This correction factor is based on experience at mines during the past 15 years; ideally, it would be developed for each rock type in each sector of the mine where controlled blasting has been implemented.

3.2.4 Double or Triple Benching

The bench increment is the distance between mining levels; in most cases, the bench increment is chosen based on either the length of either the blasthole drill rods or the reach of the digging equipment. Most porphyry copper mines currently use a 15-m mining bench increment. In the precious metal, coal, and industrial mineral industries, bench increments vary from 6 to 15 m.

Final bench slopes can be constructed using (1) a single bench technique, where catch benches are left in the slope at every mining level, or (2) a multiple bench method, where catch benches are left in the slope every two to three mining levels. Multiple benching is usually done for the following reasons:

- To provide a steeper interramp or overall slope angle. In a typical scenario, the geologic control to the bench face angle would be strong and the variability in achieved bench face angles would be tightly bounded (10°). With strong geologic control, the bench face angles are relatively fixed and predictable. To steepen the interramp slope angle, the bench height can be doubled and the bench width can be adjusted accordingly. Because bench widths do not have to be doubled for a higher final bench, the interramp slope angle can be steeper while maintaining similar levels of rockfall containment.
- To improve working conditions for the same interramp slope angle. Leaving wider, more reliable catch benches in the slope is often useful where overbank from one pushback may interfere with another pushback lower in the slope. Wider catch benches can also contain overbank material originating from a slope failure higher in the slope.
- To improve the reliability of the catch benches in areas where the geologic structure has short lengths. The majority of bench back break occurs as small failures of the bench crest. These small failures have less effect on the reliability of a wider bench.
- To minimize the occurrence of areas where there is no catch bench left in the slope. When geologic structure has intermediate lengths (less than 60% of the bench height), it is more likely that some catch bench will be available everywhere along a working level if the bench is designed to be wider.

Our experience indicates that single benching is not always safer than double benching. Double benching can actually improve safety in certain areas depending on the local geology. Double benching is not appropriate in the following situations:

- The geologic structures are quite variable in dip. In this case, it is often difficult to get final dig faces to match from one working level to another, creating offsets between working levels, local overhangs, and other safety hazards.
- The rock mass is so fractured that it can be freely dug without the aid of blasting.
- There are many long joints or faults that daylight in the slope, which increase the potential for large failure geometries to develop when mining the lower bench level. Such failure geometries can be readily accommodated in the operation when the slope height, in failure, is 15 m because the equipment can readily scale the full bench height. However, when daylighted structures cause a slope failure of up to 30 m in height, the entire face cannot be adequately scaled, which may pose a safety hazard. For a given sliding plane, the failure volume actually increases geometrically rather than proportionally. The failure volume for a double bench can be three to four times the failure volume of the single bench for the same structures.

3.2.5 Containment Berms

In Ritchie's study, containment ditches cut into the shoulder of the road near the slope toe were found to be a very reliable method of containing rocks that were moving with horizontal momentum toward the road surface. For open-pit mining, excavation of ditches on benches is impractical, but the same effect can be achieved by constructing a berm along the edge of the expected impact zone. The criteria for berm height are based on experience and will vary from mine to mine depending on bench height and rock block size.

3.2.6 Interramp Slope Angle

Once the back break model has been completed, the recommended geometry of the benched slope can be determined by converting the back break distribution (or the equivalent bench face angle distribution) to a catch bench width distribution. The catch bench widths depend on the interramp slope angle; as the interramp slope angle becomes progressively steeper, the catch bench widths decrease proportionally. Once the back break distribution is known, the percentage of the slope that the target catch bench width has met or exceeded can be readily calculated for any interramp slope angle. An example of this is shown on Figure 3.8, where the reliability of achieving the catch bench width is plotted against the interramp slope angle for an estimated back break distribution.

As shown on Figure 3.8, the reliability of achieving a specific catch bench width (in this case a catch bench width of 7.6 m using the Modified Ritchie Criteria) throughout the slope is shown to readily decrease with an increasing interramp slope angle. The average bench width (based on the median back break distance) and a minimum bench width are also shown for each interramp slope angle. The minimum bench width is estimated from the cumulative back break PDF at the 3 to 5% level (3 to 5% of the bench face angles are flatter than this angle). All of these design factors play a role in the design of the benched slope. The average catch bench width is needed for laying out the drop-flag line to help operators assess where to stop digging. The minimum width is useful for evaluating the probability of failure for the entire catch bench for identifying the interramp slope angle at which loss of the entire catch bench will begin to occur consistently. Note that the term "catch bench reliability" refers to the reliability of 7.6-m-wide benches in this example. A catch bench reliability of 90% does not indicate that 10% of the mine benches have been totally lost; instead, it indicates that 10% of the benches are less than 7.6 m wide. All three factors (catch bench reliability, median bench width, minimum bench width) are used to choose the appropriate slope angle for the benched mine slope. There are no unique bench configuration criteria, since each slope is different. Normally, our recommendations are based on 50 to 90% catch bench reliability, depending on the site-specific operational requirements and the geologic conditions encountered. Once the catch bench analysis is complete, the next step in the benched mine slope design is to evaluate larger, structurally controlled failures that exceed the bench scale.

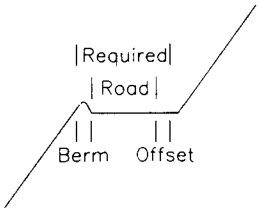
3.3 INTERRAMP SLOPE STABILITY

An often-neglected portion of the process of evaluating slope stability is the analysis of intermediate-size structurally controlled failures that exceed the bench scale but that cannot be modeled well by typical overall slope analyses, such as limit-equilibrium methods or elastic/plastic models (finite element, finite difference, or boundary element). As with the bench-scale analysis discussed previously, the analysis of these potential multiple bench or interramp failures lends itself to a reliability-based approach. This is due to the complexity of simultaneously handling variations in structure orientation, length, and spacing, as well as variations in rock strength. In our experience, a high percentage of failures in open-pit mines are structurally controlled, consequently, it is critical to have a tool to predict the number and size of potential failures that can be expected for various wall orientations and interramp slope angles.

Interramp Angle Based on Catch Bench Reliability

Bench Height = 12m (Single bench)

Bench Height = 12m
Berm Height = 1.5m
Friction Angle = 36 (deg)



DESIGN BENCH WIDTH

Offset	0.0m
Road Width	5.8m
Berm Width	4.2m
Required Width	10.0m

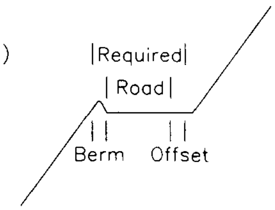
	AVERAGE	MAXIMUM	MINIMUM
Reliability	50%	5%	95%
Face (deg) =	65.0	81.0	39.0
Backbreak (m) =	5.6	1.9	14.8

INTER RAMP ANGLE (deg)	RELIA-BILITY (%)	DESIGN AVERAGE WIDTH (m)	MAXIMUM WIDTH (m)	MINIMUM WIDTH (m)	BFA AT 10.0m WIDTH (deg)
27	93	18.0	21.7	8.7	41.4
28	90	17.0	20.7	7.8	43.6
29	89	16.1	19.8	6.8	45.7
30	86	15.2	18.9	6.0	47.9
31	83	14.4	18.1	5.2	50.1
32	80	13.6	17.3	4.4	52.4
33	76	12.9	16.6	3.7	54.6
34	72	12.2	15.9	3.0	56.9
35	66	11.5	15.2	2.3	59.1
36	60	10.9	14.6	1.7	61.3
37	54	10.3	14.0	1.1	63.5
38	48	9.8	13.5	0.5	65.7
39	41	9.2	12.9	0.0	67.9
40	35	8.7	12.4	−0.5	70.1
41	31	8.2	11.9	−1.0	72.2

Interramp Angle Based on Catch Bench Reliability

Bench Height = 24m (Double bench)

Bench Height = 24m
Berm Height = 2.1m
Friction Angle = 36 (deg)



DESIGN BENCH WIDTH

Offset	0.0m
Road Width	6.6m
Berm Width	5.8m
Required Width	12.4m

	AVERAGE	MAXIMUM	MINIMUM
Reliability	50%	5%	95%
Face (deg) =	71.0	81.0	49.0
Backbreak (m) =	8.3	3.8	20.9

INTER RAMP ANGLE (deg)	RELIA-BILITY (%)	DESIGN AVERAGE WIDTH (m)	MAXIMUM WIDTH (m)	MINIMUM WIDTH (m)	BFA AT 10.0m WIDTH (deg)
27	100	38.8	43.3	26.2	34.7
28	99	36.9	41.3	24.3	36.3
29	99	35.0	39.5	22.4	37.9
30	99	33.3	37.8	20.7	39.5
31	99	31.7	36.2	19.1	41.1
32	98	30.2	34.6	17.6	42.7
33	98	28.7	33.2	16.1	44.4
34	97	27.3	31.8	14.7	46.0
35	96	26.0	30.5	13.4	47.7
36	95	24.8	29.2	12.2	49.4
37	94	23.6	28.1	11.0	51.0
38	92	22.5	26.9	9.9	52.7
39	90	21.4	25.8	8.8	54.4
40	88	20.3	24.8	7.7	56.0
41	86	19.4	23.8	6.7	57.7
42	84	18.4	22.9	5.8	59.3
43	81	17.5	21.9	4.9	61.0
44	77	16.6	21.1	4.0	62.6
45	73	15.7	20.2	3.1	64.3
46	69	14.9	19.4	2.3	65.9
47	63	14.1	18.6	1.5	67.5
48	57	13.3	17.8	0.7	69.1
49	51	12.6	17.1	0.0	70.6
50	45	11.9	16.3	−0.7	72.2
51	38	11.2	15.6	−1.4	73.7
52	32	10.5	15.0	−2.1	75.2

FIGURE 3.8 An analysis of catch bench reliability for increasing interramp slope angle

3.3.1 Data Requirements

While structural analysis of bench-scale stability is typically based on the orientation of joints and other discontinuities that make up the "rock fabric," interramp analysis is generally based on the evaluation of faults and other major structures that have lengths adequate to define failure geometries ranging from double bench to full slope height. Although regional or pit-scale structural features, such as major faults, are generally well defined and are included in the geologic model, faults of intermediate length are typically less well defined. These structures are often part of the bench mapping that is done at most properties; however, a concerted effort must be made to tie structures together from bench to bench to define the overall continuity of each structure. This step is critical to predicting the size of failures that may occur.

Also, while bench-scale analysis is typically based on sliding along rock-on-rock joint surfaces, larger interramp-scale failures are typically assumed to be sliding along fault gouge or clay infilling of the major structures. Therefore, it is necessary (1) to note the filling when mapping each structure and (2) to sample and test representative filling materials as part of the laboratory strength-testing program.

3.3.2 Analytical Approach

Probability Distributions. As indicated previously, because of the number of parameters involved and the variation within each parameter, a reliability-based approach is recommended for structural analysis of interramp slopes. The determination of parameters for the interramp analysis is very similar to the approach used for the bench-scale analysis. The major structures are grouped into sets; the sets are based on orientation relative to the slope face, with left and right wedge-forming sets and with a plane-shear set for orientations nearly parallel to the face. However, since the orientation and length of each major structure are measured individually, there is more flexibility in selecting the type of distribution to be used than there is with the rock fabric data. If enough structures are available, the discrete value of each structure orientation is used rather than a distribution function. This is generally the approach used by CNI. An example of fault

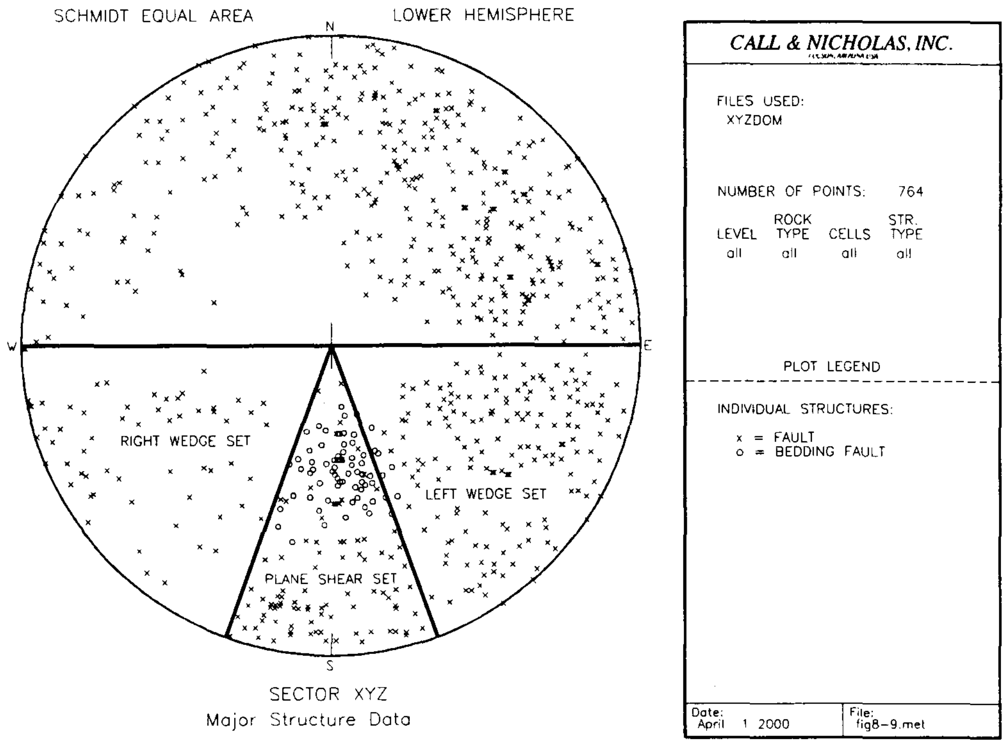


FIGURE 3.9 Schmidt plot of plane shear and wedge sets

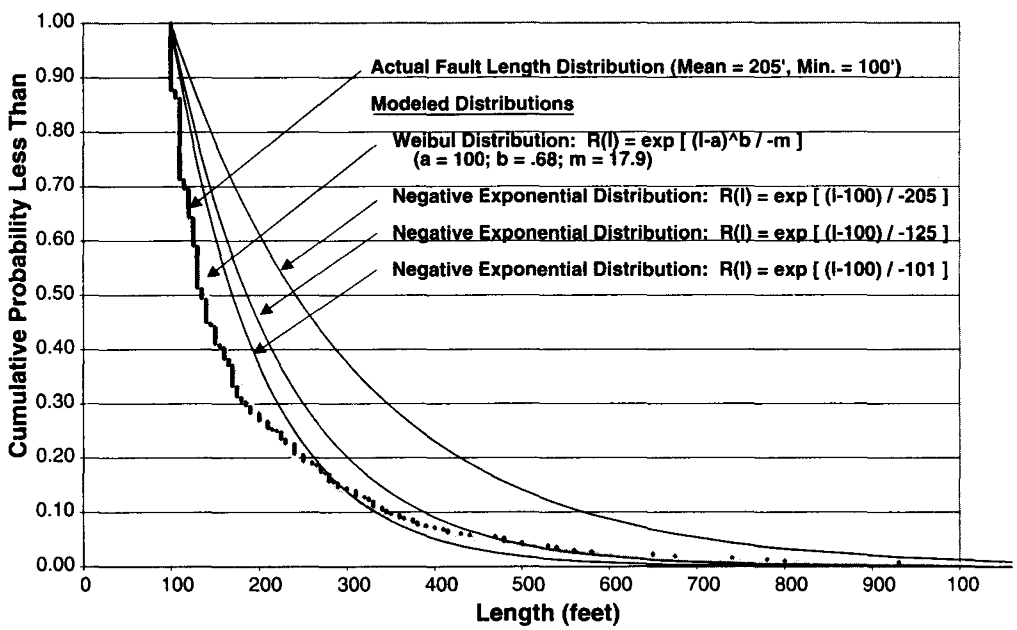


FIGURE 3.10 Probability distributions fit to actual fault data

data divided into simple plane-shear and left and right wedge sets is presented in Figure 3.9.

Lengths and spacings are typically modeled using a Weibull or negative exponential distribution, but a discrete probability distribution based on the values obtained from mapping can also be used. Our experience indicates that the Weibull distribution generally provides the best fit if a discrete probability distribution is not used. The advantage to the Weibull is that it predicts extreme lengths better than the discrete probability distribution, which has a finite upper end. An example of actual data fit with a Weibull distribution compared to several negative exponential distributions is presented in Figure 3.10.

Probability Analysis. In contrast to the bench-scale analysis, CNI uses a closed-form probability analysis rather than a Monte Carlo simulation for the interramp analysis. With a Monte Carlo analysis, the orientation and length of the structure (or structures, in the case of a wedge analysis) are sampled, and the maximum height of the failure is calculated based on the sampled values. However, the interramp analysis is typically performed for a series of heights ranging from the double bench height up to the total slope height, usually in increments equal to the double bench height. Since the various failure heights being analyzed are fixed, it is preferable to calculate the probability (P_l) that a given structure is long enough based on the distribution of lengths, rather than by sampling until the appropriate structure length is obtained. For the wedge analysis, calculate the joint probability that both the left and right structures are long enough. Given the discrete structure orientation(s), required length(s), unit weight of the material, and water conditions, the potential failure geometry is evaluated using the appropriate three-dimensional wedge (Hoek and Bray 1981) or plane shear (Marek and Savely 1978) analysis to determine the minimum strength required for a factor of safety equal to 1.0. The weight of the failure in tons (T_f) for the geometry evaluated is recorded to be used for estimating the expected failure tonnage. The probability (P_s) that the actual strength is less than the minimum value required for stability is then calculated from the distribution of fault gouge strengths. The probability of failure (P_f) is determined by combining the probabilities of the length and strength components as follows:

$$P_f = P_l \times P_s \qquad \textbf{EQ. 3.2}$$

This process is repeated for every orientation in the plane-shear set or, in the case of the wedge analysis, for every combination of left and right orientations. The resulting individual probabilities of failure and failure tonnages are summed and divided by the number (N_d) of daylighted orientations or wedge combinations evaluated to determine the expected probability of failure (P_F) and the expected failure tonnage (T_F), as follows:

$$P_F = (\Sigma P_f) / N_d \qquad \textbf{EQ. 3.3}$$

$$T_F = (\Sigma (T_f * P_f)) / \Sigma P_f \qquad \textbf{EQ. 3.4}$$

As with the bench-scale design, a cell approach is used in which the height and width of the cell are each equal to the height (H) of the failure being analyzed. The number of cells (N_C) within the sector for a given failure height is calculated by dividing the sector area (A) by the cell area (H^2). The probability of occurrence (P_O) of a plane-shear structure within the cell is based on the distribution of fault spacings for structures in the plane-shear orientation. For the wedge analysis, the probability of occurrence of a wedge intersection within the cell is based on the distributions of fault spacing for both wedge sets. The expected total number of failures (N_E) and expected failure tonnage (T_E) of a given type (plane shear or wedge) within the entire sector for a given failure height is calculated as follows:

$$N_E = P_F \times P_O \times N_C \quad \text{or} \quad N_E = P_F \times P_O \times A/(H^2) \qquad \textbf{EQ. 3.5}$$

$$T_E = T_F \times P_O \times N_C \text{ x} \qquad \textbf{EQ. 3.6}$$

To avoid double counting by evaluating small failures that are contained within areas associated with larger failures, the process is started with the largest failure height (H), the corresponding cell area (H^2), and the total sector area (A) and is worked downward. For each subsequent failure height evaluated, the total sector area is reduced by the area of expected failures for the previous height, as follows:

$$A = A - (H^2 \times N_E) \qquad \textbf{EQ. 3.7}$$

The process, which is illustrated in simplified form in Eq. 3.7, is repeated for all heights to be evaluated. For the plane-shear or wedge analysis being performed, this process results in a composited value for the expected number of failures and the expected failure tons in the sector. The expected failure height is also calculated at this time. The entire process described thus far is for the analysis of a single interramp angle. This process is repeated for each interramp angle being considered, with the typical range being from 30° to 60°. When both the plane-shear and wedge analyses have been completed, the results are combined, again discounting to avoid double counting, to obtain the totals for expected number of failures and expected failure tonnages at each interramp slope angle evaluated. The entire process may also be repeated for various groundwater conditions. Typically, if groundwater is present, a wet and a dry case are run to determine the effects of dewatering. The analysis can also be run for various seismic loading conditions.

Obviously, this is not an analysis that you would want to perform by hand. Fortunately, this highly iterative process is well suited for the computer. Sample output from an interramp analysis performed with CNI's in-house software shows the detailed results obtained for each height evaluated for a given sector and interramp slope angle (see Table 3.1).

The total expected number of failures and expected failure tonnages for these sample data for all interramp angles evaluated for both saturated and dry cases are presented in Table 3.2, along with an accompanying graph.

3.3.3 Cost–Benefit Analysis

The results from the interramp analysis do not provide a specific reliability or a factor of safety associated with a given slope angle. What the analysis does provide is an estimate of the number of failures and the total failure tonnages expected for each interramp slope angle evaluated. If there is a sharp increase in either the predicted number of failures or failure tonnage from one angle to the next, it may be possible to select the optimum interramp slope angle using these data alone. Ideally, the optimum interramp angle should be selected (1) by comparing the benefits gained (less stripping, more ore) versus the costs incurred (failure cleanup, downtime, buried ore, etc.) by steepening the slope and (2) by determining the angle that provides the economic optimum. At this point, the slope stability analysis is no longer a standalone process but is an integral part of the mine planning process. This approach is generally referred to as a cost–benefit (or benefit–cost) analysis (Call and Kim 1978), and it has been used successfully by CNI at several large porphyry copper mines to assist the on-site mine planners in maximizing economic recovery.

Two general types of cost–benefit analysis have been used by CNI over the years: a long form and a short form, with the short form being much more frequently used. The long form requires development of a fully sequenced pit, which is then modeled to predict the occurrence of failures at specific times through the course of the mine life. This very detailed process requires significant work by the mine planners prior to the stability modeling process. The short form looks only at the final wall and therefore requires much less work on the part of the mine planners. Since it is often preferable to mine the working slopes at somewhat flatter angles than those of the final walls, the short form is generally all that is required for optimizing the ultimate slopes. If the interramp slope angles in some sectors are constrained by bench-scale or overall stability problems, it may not be necessary to perform the cost–benefit analysis for all sectors of the pit. The remainder of this discussion will be limited to the short-form approach. For a detailed discussion of the long-form approach, refer to the *Pit Slope Manual, Supplement 5-3* (CANMET 1977).

Economic Input Requirements. In addition to the results of the interramp stability analysis, significant economic information is required to perform a cost–benefit analysis. Ultimate pits, including haul roads, must be developed by the mine planners for each of the interramp angles to be analyzed. The net present value of each pit is determined using standard mine planning procedures. Once these plans are available, it is necessary to determine the net present value of the entire sector being evaluated, as well as the value of the last blocks mined on each level. Also required are the failure cleanup costs per ton; the cost per

TABLE 3.1 Sample output from CNI interramp analysis

```
Sector XYZ - DRY
 RESULTS FROM PROGRAM ADD.F

      INPUT DATA:          AVERAGE SECTOR HEIGHT:  1900.
                           AVERAGE DOMAIN HEIGHT:  1900.
                            AVERAGE DOMAIN WIDTH:  2100.
                  EXP. SPACING "PLANE SHEAR" SET:    76.
               EXP. SPACING OF WEDGE SETS - LEFT:    80.    RIGHT:   112.
 <><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><>
 SLOPE ANGLE = 46.
```

HT.	COMPOSITE EXPECTED NUMBER	COMPOSITE EXPECTED TONS	STATIC EXPECTED NUMBER	STATIC EXPECTED TONS	0.060G EXPECTED NUMBER	0.060G EXPECTED TONS	0.160G EXPECTED NUMBER	0.160G EXPECTED TONS	0.400G EXPECTED NUMBER	0.400G EXPECTED TONS
TOTAL	0.3834E+02	0.2853E+06	0.3834E+02	0.2853E+06	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
1900.	0.1160E-13	0.5451E-06	0.1160E-13	0.5451E-06	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
1800.	0.6940E-13	0.2776E-05	0.6940E-13	0.2776E-05	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
1700.	0.4236E-12	0.1424E-04	0.4236E-12	0.1424E-04	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
1600.	0.2586E-11	0.7121E-04	0.2586E-11	0.7121E-04	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
1500.	0.1587E-10	0.3577E-03	0.1587E-10	0.3577E-03	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
1400.	0.9826E-10	0.1823E-02	0.9826E-10	0.1823E-02	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
1300.	0.6152E-09	0.8978E-02	0.6152E-09	0.8978E-02	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
1200.	0.3900E-08	0.4530E-01	0.3900E-08	0.4530E-01	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
1100.	0.2508E-07	0.2198E+00	0.2508E-07	0.2198E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
1000.	0.1642E-06	0.1069E+01	0.1642E-06	0.1069E+01	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
900.	0.1098E-05	0.5208E+01	0.1098E-05	0.5208E+01	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
800.	0.7526E-05	0.2477E+02	0.7526E-05	0.2477E+02	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
700.	0.5316E-04	0.1138E+03	0.5316E-04	0.1138E+03	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
600.	0.3883E-03	0.5333E+03	0.3883E-03	0.5333E+03	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
500.	0.2948E-02	0.2246E+04	0.2948E-02	0.2246E+04	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
400.	0.2345E-01	0.8930E+04	0.2345E-01	0.8930E+04	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
300.	0.2009E+00	0.2954E+05	0.2009E+00	0.2954E+05	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
200.	0.2033E+01	0.8215E+05	0.2033E+01	0.8215E+05	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00
100.	0.3608E+02	0.1618E+06	0.3608E+02	0.1618E+06	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00	0.0000E+00

```
  EXPECTED HEIGHT =   107.
 <><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><>
```

TABLE 3.2 Sample summary results for all angles, dry and saturated

INTERRAMP RESULTS

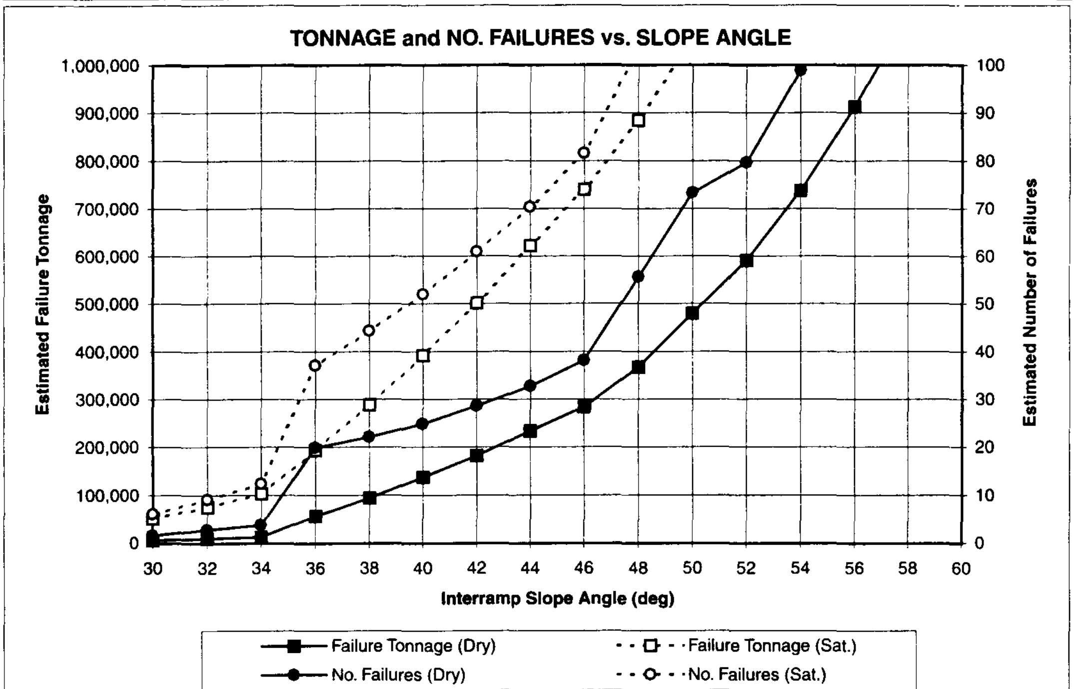
SECTOR XYZ

Structural Domain: ESDDOM(I) Wall DDR: 250

Average Sector Height (ft): 1900
Average Domain Height (ft): 1900
Average Domain Width (ft): 2100

	Plane Shear	L. Wedge	R. Wedge
Fault Spacing:	76	80	112
Fault Length:	152	151	152

INTERRAMP ANGLE	DRY		SATURATED	
	No. Failures	Tons	No. Failures	Tons
60	186.0	1,341,216	262.5	2,166,336
58	140.0	1,110,912	219.1	1,886,208
56	132.1	912,576	201.8	1,654,848
54	98.8	737,856	166.1	1,436,544
52	79.6	591,264	142.2	1,239,072
50	73.3	479,904	127.1	1,057,920
48	55.7	367,142	103.3	884,352
46	38.2	284,266	81.6	740,064
44	32.8	233,827	70.4	621,888
42	28.7	181,891	61.0	501,888
40	24.8	137,222	52.0	391,296
38	22.2	93,965	44.4	288,384
36	20.0	55,930	37.2	193,690
34	3.8	12,374	12.3	103,104
32	2.7	8,893	9.1	74,438
30	1.6	6,086	6.0	51,590



day of lost production due to downtime; the cost per ton of backfilling to restore failed haul roads; and the costs associated with repairing utility lines, dewatering collection systems, etc. Other facilities that may be impacted by failures and that should be considered might include rail lines, in-pit crushers, conveyors, access to underground workings, or any other structure that will be required for ongoing operations.

Cost Calculations. Once this information is compiled, it is combined with the interramp results in a spreadsheet; the spreadsheet is customized for each sector to account for the various haul roads and facilities that may be impacted by slope failures. The

TABLE 3.3 Interramp failure cost calculations

SECTOR XYZ					DATE: 4/1/00	HAUL ROAD												
S HEIGHT: 1900 D HEIGHT: 1900 WIDTH: 2100			CLEANUP CLEANUP COST/TON: $1.20			FAILURES LENGTH: 3900		LOST PRODUCTION FIX COST: FIX DAYS: 3 COST/DAY:				LOST ORE NET VALUE BELOW ROAD WIDTH: 1250			BELOW ROAD FAILURES TOTAL			TOTAL HAUL ROAD
								ON ROAD		BELOW ROAD								
ANGLE (deg)	NO.FAIL	FAIL HGT (ft)	TONS (x1000)	AVE TONS (x1000)	CLEANUP COST (x1000)	ANGLE (deg)	NO.FAIL	COST/FAIL (x1000)	TOTAL (x1000)	BACKFILL TONS	COST/FAIL (x1000)	THICK (ft)	WIDTH (ft)	COST/FAIL (x1000)	TOTAL PER FAIL (x1000)	NO.FAIL	TOTAL COST (x1000)	TOTAL COST (x1000)
42	40.55	115	181.9	4.5	218	42	4.56	$4,805	$21,901	15	$4,805	45	173	$6,102	$10,907	4.56	$49,714	$71,616
40	31.30	111	137.2	4.4	165	40	3.40	$4,805	$16,317	8	$4,805	35	167	$4,337	$9,142	3.40	$31,044	$47,362
38	22.25	108	94.0	4.2	113	38	2.35	$4,805	$11,286	3	$4,805	25	162	$2,843	$7,648	2.35	$17,964	$29,250
36	15.03	105	55.9	3.7	67	36	1.54	$4,805	$7,412	0	$4,805	20	158	$2,078	$6,883	1.54	$10,617	$18,029
34	13.78	103	12.4	0.9	15	34	1.39	$4,805	$6,666	0	$4,805	20	155	$1,958	$6,763	1.39	$9,382	$16,048
32	12.76	102	8.9	0.7	11	32	1.27	$4,805	$6,113	0	$4,805	20	153	$1,913	$6,718	1.27	$8,546	$14,658
30	11.65	101	6.1	0.5	7	30	1.15	$4,805	$5,526	0	$4,805	20	152	$1,868	$6,673	1.15	$7,674	$13,201

LAST BLOCK							
	LENGTH (ft)	NET VALUE (x1000) 30 Deg	34 Deg	36 Deg	38 Deg	42 Deg	WIDTH (ft)
ALL RDS	3900						1
UPPER RD	3900	$77,050	$79,200	$82,450	$87,450	$98,500	1250
CONVEYOR	500	$78,150	$79,850	$80,500	$85,600	$97,600	1100
CRUSHER: (1 OR 0 FOR WIDTH)							1

CONVEYOR										CRUSHER & TUNNEL			SECTOR XYZ					
FACTOR: 1.00 LOST PROD $/FAIL: $20,900 LENGTH: 500			LOST ORE NET VALUE WIDTH: 950			BELOW CONVEYOR			TOTAL CONVEYOR	AREA: FAIL $:	CRUSHER 70000 $95,800	TUNNEL 60000 $175,000	FAILURE COST SUMMARY					
ABOVE CONVEYOR																		
ANGLE (deg)	NO.FAIL	TOTAL COST (x1000)	THICK (ft)	WIDTH (ft)	ORE COST (x1000)	PER FAIL COST (x1000)	NO. FAIL	BELOW COST (x1000)	TOTAL COST (x1000)	NO.FAIL	TOTAL COST (x1000)	ANGLE (deg)	CLEANUP (x1000)	HAUL ROAD (x1000)	CONVEYOR (x1000)	CRUSHER & TUNNEL (x1000)	SECTOR TOTAL (x1000)	INTERRAMP ANGLE (deg)
42	0.58	$12,213	45	173	$7,955	$28,855	0.58	$16,862	$29,075	0.71	$192,648	42	$219	$71,616	$29,075	$192,648	$293,558	42
40	0.44	$9,099	35	167	$5,622	$26,522	0.44	$11,547	$20,646	0.55	$148,702	40	$165	$47,362	$20,646	$148,702	$216,876	40
38	0.30	$6,294	25	162	$3,662	$24,562	0.30	$7,396	$13,690	0.39	$105,707	38	$113	$29,250	$13,690	$105,707	$148,760	38
36	0.20	$4,133	20	158	$2,669	$23,569	0.20	$4,661	$8,794	0.26	$71,406	36	$67	$18,029	$8,794	$71,406	$98,296	36
34	0.18	$3,717	20	155	$2,597	$23,497	0.18	$4,179	$7,897	0.24	$65,467	34	$15	$16,048	$7,897	$65,467	$89,427	34
32	0.16	$3,409	20	153	$2,545	$23,445	0.16	$3,824	$7,233	0.22	$60,621	32	$11	$14,658	$7,233	$60,621	$82,523	32
30	0.15	$3,082	20	152	$2,493	$23,393	0.15	$3,449	$6,531	0.20	$55,348	30	$7	$13,201	$6,531	$55,348	$75,087	30

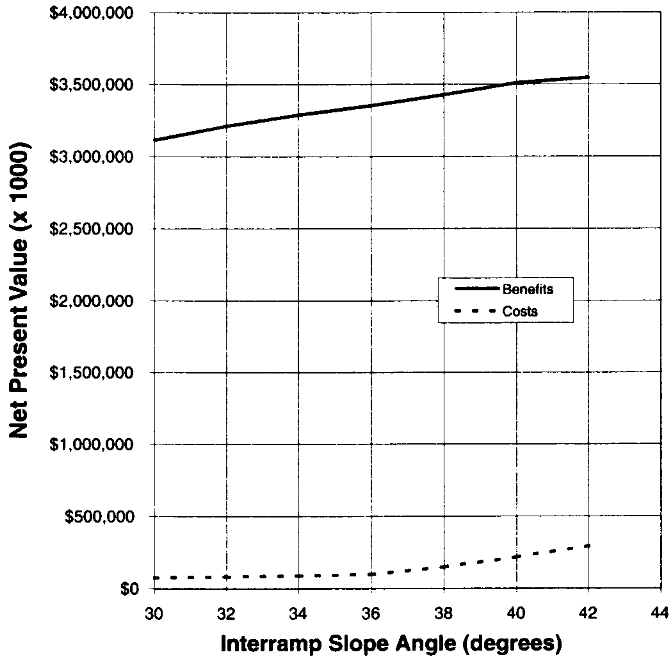


FIGURE 3.11 Benefits and costs versus slope angle

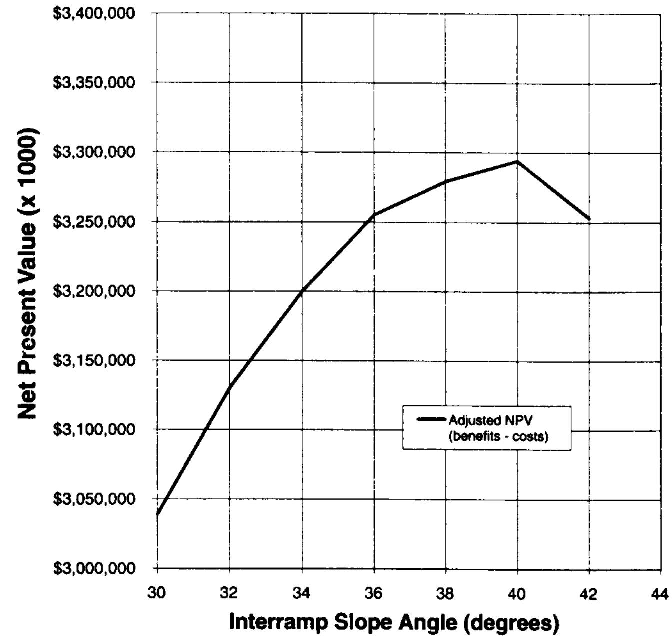


FIGURE 3.12 Adjusted net present value versus slope angle

expected number of failures, failure tonnages, and failure heights are used to estimate the total cleanup costs, as well as the number of failures on haul roads, below haul roads, and on or below other critical structures. Failures on haul roads result in blocked haulage and lost production in addition to cleanup costs. Failures below haul roads require backfilling to restore the haul roads and may require stepouts, which result in lost ore. Failures on or below other critical structures would result in similar consequences and would have specific costs associated with replacing or restoring the impacted structure. A sample spreadsheet showing the cost calculations is presented in Table 3.3.

Cost-Benefit Curves. The net present value versus slope angle is then plotted along with the total cost of failure versus slope angle (see Figure 3.11).

The difference between these two curves represents the adjusted net present value incorporating the economic consequences of slope instability into the mine planning process. A plot of the adjusted net present value versus interramp slope angle (see Figure 3.12) illustrates that this curve peaks at the economically optimal angle.

3.4 CONCLUSIONS

Interramp analysis is a critical, if often neglected, tool in the evaluation of pitward-dipping major structures that are too small or too numerous to model individually and too large to be evaluated using bench-scale analyses. As with all other analytical methods used for slope stability evaluation, interramp analysis is not applicable in all situations because either the structures do not exist to make this a viable failure mode or the database is inadequate (due to limited exposure or economic constraints) to justify an analysis of this magnitude. However, in large open-pit mines, where the economic implications of minor changes in slope angle are significant, this analysis can be extremely valuable for determining the economically optimal interramp slope angle, particularly when combined with a cost-benefit analysis.

3.5 ACKNOWLEDGMENT

Although the Backbreak and Interramp analyses discussed herein have been modified and updated through the years, Call & Nicholas, Inc., acknowledge Dr. R.D. Call and Paul J. Visca for their work at CNI in developing the original framework for both analyses.

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