PIT SLOPE MANUAL

chapter 5

DESIGN

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PIT SLOPE PROJECT

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THE PIT SLOPE MANUAL

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The Pit Slope Manual consists of ten chapters, published separately. Most chapters have supplements, also published separately. The ten chapters are:

1. Summary
2. Structural Geology
3. Mechanical Properties
4. Groundwater
5. Design
6. Mechanical Support
7. Perimeter Blasting
8. Monitoring
9. Waste Embankments
10. Environmental Planning

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FOREWORD

Open pit mining accounts for some 70% of Canada's ore producti With the expansion of coal and tar sands operations, open pit mining wi continue to increase in importance to the mineral industry. Recognizing this, CANMET embarked on a major project to produce the Pit Slope Manual, which is expected to bring substantial benefits in mining efficiency through improved slope design.

Strong interest in the project has been shown throughout its progress both in Canada and in other countries. Indeed, many of the results of the project are already being used in mine design. However, it is recognized that publication of the manual alone is not enough. Help is needed to assist engineers and planners to adopt the procedures described in the manual. This need for technology transfer will be met by a series of workshops for mine staff. These workshops will be held in various mining centres during the period 1977-81 following publication of the manual.

A noteworthy feature of the project has been its cooperative nature. Most organizations and individuals concerned with open pit planning in the country have made a contribution to the manual. It has been financed jointly by industry and the federal government.

Credit must be given to the core of staff who pursued with considerable personal devotion throughout the five-year period the objectives of the work from beginning to end. Their reward lies in knowing that they have completed a difficult job and, perhaps, in being named here: M. Gyenge, G. Herget, G. Larocque, R. Sage and M. Service.

D.F. Coates
Director-General
Canada Centre for Mineral and
Energy Technology

SUMMARY

INTRODUCTION

Purpose and Scope

The purpose of this chapter is to prescribe practical procedures for designing pit walls. Mining engineers are familiar with many of the techniques that might be used; therefore, the chapter is primarily concerned with telling "what to do" and, to some extent, "why". Describing "how to do" is included in the appendices and supplements.

The scope of the chapter is to describe analyses that use the information provided by the investigations for structural, groundwater and mechanical properties. Many optional procedures can be used or not depending on the mine and staff. Stability calculations are shown to provide data to be used in the financial analyses. These in turn provide the basis for optimizing the mine design. Examples of these analyses are included in the supplements.

Mine Planning

One of the objectives of an ultimate pit plan is to ensure optimum recovery of the mineral resource being mined. The pit layout must integrate information on mineable reserves, topography, wall design, dump location, service corridors and plant facilities.

Wall design consists of determining location and slope angles. Design can be thought of as precedent modified by experience, testing and analyses. The process requires input from exploration, stability investigations and financial analyses.

In selecting wall slopes, the main benefit of steeper angles comes from reduced waste excavation for ultimate walls. Delayed stripping for interim and working slopes is also a benefit.

The principal cost of steeper slopes arises from the increased cost of instability. Other costs can arise from decreased operating efficiency. Being constrained to use fewer working benches can be another source of increased cost as are using single instead of double spotting of trucks and various other sources of inefficiencies in drilling, blasting, loading and transporting the muck. There are practical limitations to the steepness of wall angles.

Probability

Reliability theory is used in many other fields of engineering. This approach recognizes the variability of design factors (see Fig. 3). Mine planners use this approach by including the effects of variability of commodity prices, labor costs, ore reserves and grade when making a feasibility study. It is now appropriate to extend this approach to slope design.

The impossibility of knowing all the pertinent structural and strength factors affecting stability of 1000's of feet of wall must be recognized. It is possible to use data on the variability of the various factors to estimate wall reliabilities (ie the probability of stability). The full integration of wall design into the financial analyses requires the use of reliability theory.

Design Stages

There are three main stages in the life of a mine when wall design is required. These are (1) feasibility, (2) mine design and (3) operating stages.

The feasibility stage occurs when the results from exploration are being analyzed. A tentative pit outline is required to determine if the potential orebody is economic. In some cases, slope angles make the difference between an attractive and a low prospective rate of return. This stage often merges with the next stage.

The mine design stage occurs when it has been established that an orebody exists and financing has been arranged for production. Many of the original assumptions must be confirmed. Wall slopes must be analysed more thoroughly. Investigations must be of greater intensity than at the first stage.

During the operating stage of the pit, the assumptions made for the original design are often superseded. Prospective commodity prices change, grade information is amplified and reserve volumes may be governed by new criteria. Redesign of the mine at some point becomes imperative. By this time valuable experience with slopes in some of the wall formations may have been obtained. Relocated walls may turn out to be in new formations, which provides additional reason for redesign of the slopes.

In general, the following steps should be followed at any stage in analysing the stability of the walls.

a. Establish boundaries of the design sectors.
b. Determine design requirements for each sector, eg ramp locations, catch bench specifications, etc.
c. Determine potential modes of instability.
d. Identify design variables such as bench height and angle, etc.
e. Check that required investigation data and analyses are included in reports.
f. Determine maximum feasible slope angles.
g. Analyse stability of wall elements.
h. Determine effects of instability.
i. Determine optimum slope angles through the Benefit-Cost program.
j. Rationalize optimum slope angles into a practical pit plan.
k. Plan field trials for perimeter blasting.
l. Design monitoring systems.
m. Design mechanical support systems if necessary.
n. Design trial slopes where decided.

FEASIBILITY AND MINE DESIGN STAGES

Investigations

Investigation requirements vary with the type of open pit and the stage of development. A small or shallow orebody may not need an intensive investigation. Indeed, many of the procedures prescribed herein should be abridged in some cases. The waste/ore ratio and the existence of geological or assay boundaries will also influence investigations. Production rate, particularly the sinking rate, also can be pertinent. Complex geology and widely varying rock properties usually require special investigations. The differences between mines in bedded deposits and mines in igneous/metamorphic formations can influence requirements.

Information can and must be gathered on structural geology, groundwater and mechanical properties of the wall materials. A mean value for all design factors is required as well as a measure of dispersion such as standard deviation. It is important that all information be recorded in a series of reports.

Data on previous slopes in the same formations, where available, are of great value. Appropriate analyses of such data may permit either design confirmation or indicate that a re-examination of the analyses is required.

Stability Analyses

The pit is divided into design sectors based on the results of the field investigations. These sectors require separate stability analyses owing to different rock conditions or operating factors. At the feasibility stage, slope angles may be based on previous experience and type of mining equipment to be used. At the mine design stage, analyses are required for determination of optimum angles for berms, working faces, inter-ramp slopes and overall slopes. Computer programs for detailed stability analyses are provided. These programs calculate reliability for all slope angles and heights. Approximate procedures suitable for manual calculation of reliability are also included.

The locating of ramps is important. Trade-offs may exist between minimizing haulage costs and placing the ramp on a weak wall so that the overall slope angle is automatically reduced without extra waste excavation. The maintenance of safety berms in benches can be a problem. Each mine usually has some special problem that must be incorporated into the wall design.

Financial Analyses

Input for the financial analyses comes from a set of feasible slope angles and their reliabilities. Costs of instability are incorporated in the financial analyses to provide management with a schedule of prospective rates of return and risk. As the quality of input data improves through the various stages in the development of the mine, the accuracy of predicting the prospective rates of return improves.

Good design ensures safe operations. Current mining has a good record, fatalities occurring at a very low rate. Furthermore, the main causes do not include rock instability. Consequently, steepening of walls can often be considered a reasonable option. For particularly steep angles, the mine planner will want to examine the need for catch benches, special monitoring and perimeter blasting.

OPERATING STAGE

During operations a large amount of information can be gathered on structure, groundwater and mechanical properties of the wall rock. Additional exploration boreholes may have been drilled. These are an important source of supplementary information. The exposure of the wall rock by benching provides the main opportunity to obtain structural information at relatively low cost.

Bench mapping must be conducted regularly. It is essential to detect variations so that unusual behaviour can be anticipated and controlled.

Observations on face seepage and quantity of flow are valuable. Data from piezometers installed at the exploration and mine design stage will have a long period of record. Recording the location of wet blastholes together with the depth below collar of such water levels may be useful. Data should now be available to provide a good estimate of the maximum annual groundwater elevation around the mine.

Large test samples of rock become available as a result of exposed faces. Strength properties of faults, joints and other discontinuities can be tested. Also the opportunity to examine in detail any slides, however minor, is of great value.

If a monitoring system has been installed at the start of mining, deformation measurements would be available. These could be related to the excavated geometry of the walls and possibly to the passage of time. A basis is thus provided for judging if abnormal deformation foreshadowing instability occurs.

ASSOCIATED DESIGNS

Monitoring

As a result of stability and financial analyses, optimum slope angles might be selected that are rather steep. To implement these designs, it is often necessary to pay an insurance premium. Monitoring ensures safety of personnel and equipment.

Perimeter Blasting

The feasibility of having steep slopes depends on the elimination of loose rock. This might be achieved with appropriate perimeter blasting and good scaling.

Mechanical Support

Situations can arise where the use of mechanical support of the walls is economically advantageous. For example, it might be feasible to mine ore whose removal without using support would jeopardize surface plants. Also, in some pits, it would be cheaper to use mechanical support and a steep slope than to excavate the extra waste.

Trial Slopes

Trial slopes can be useful. Expensive, conservative assumptions might have been made in the initial wall design. It is sometimes possible to excavate trial slopes in an area where production would not be significantly affected and thus provide a basis for less conservation.

ACKNOWLEDGEMENTS

D.F. Coates was responsible for producing the chapter; address enquiries to him at: 555 Booth St., Ottawa, K1A 0G1, Canada.

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CONTENTS

FIGURES

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INTRODUCTION

Purpose and Scope

1. The purpose of this chapter is to prescribe practical procedures for determining pit wall geometry. Some or all of the procedures may be used at the designer's option. Only the sections pertinent to the design problem in hand need be read.

2. Mining engineers are familiar with many of the techniques that might be used. The chapter is primarily concerned with telling what to do and, to some extent, why. Describing how to do certain activities is included in the appendices and supplements.

3. The scope of the chapter is to describe analyses that can be used in wall design. The data comes from the economic geology and investigations for structural, groundwater and mechanical properties. Stability calculations provide data to be used in the financial analyses. Examples are included in the supplements.

4. The main benefit of steeper slopes comes from reduced waste excavation for ultimate walls and from delayed stripping for interim slopes. One of the principal costs of steep slopes arises from the cost of instability. Other costs can arise from operating inefficiency. It may be necessary to use few working benches and to use single spotting of trucks instead of double spotting. Various other sources of inefficiencies in drilling, blasting, loading and transporting the muck can arise.

5. Reliability theory explicitly includes the unreliability of materials and of load predictions. Mine engineers include the effects of variability of such factors as commodity prices, labor costs, ore reserves and grade in financial calculations. This approach can be extended to slope design, recognizing the impossibility of knowing all the structural and strength factors affecting the stability of great lengths of wall

(see Fig. 1). It is possible to use data on the variability of the various factors to estimate reliabilities, or probabilities of stability. This is comparable to insurance companies discovering long ago that, although impossible to predict the future for any one person, it is possible to predict statistical averages and dispersions.

6. Extension of the use of reliability to stability analyses is valuable. Prospective rates of return with related probabilities of occurrence are already calculated for investment decisions. Extension of this approach requires integration of wall design into mine design and financial analyses.

Design Stages

7. There are three main stages in the life of a mine when wall design is required. These are shown in Fig. 2: (1) feasibility, (2) mine design and (3) operating stages.

8. The feasibility stage occurs when the results from exploration are being analyzed. A pit outline is required to determine if the potential orebody is economic. In some cases, slope angles can make the difference between an attractive and an unattractive prospective rate of return.

9. At the mine design stage, when it has been established that an orebody exists and financing has been arranged for production, many of the original assumptions must be confirmed. Wall slopes must be determined with more certainty. The amount of investigation and analyses at this stage will be of greater intensity than at the first stage.

10. Finally, during the operating stage of the pit, assumptions made during the original mine design are often superseded. Prospective commodity prices change, grade information is amplified and reserve volumes may be governed by new criteria. Valuable experience may have been obtained with regard to wall stability. Relocated walls may be cut in new formations. Redesign of the mine at some point becomes jmperative.

Fig. 1 - Variability in a pit wall. A slide occurred at a section to the right of the photo that was not visually different from adjacent sections. The slide was impossible to predict with certainty, but it had been shown in the design analysis that there was a 10% probability of instability in the entire wall.

Investigations

11. The chapters on Structural Geology, Groundwater and Mechanical Properties describe information to be gathered. Unique major features such as faults are most important and require specific investigation. Appropriate analyses of hydrostatic pressures and effects of drainage schemes may be needed. Mean values of all design parameters are required as well as measures of their dispersion such as standard deviations.

Analyses

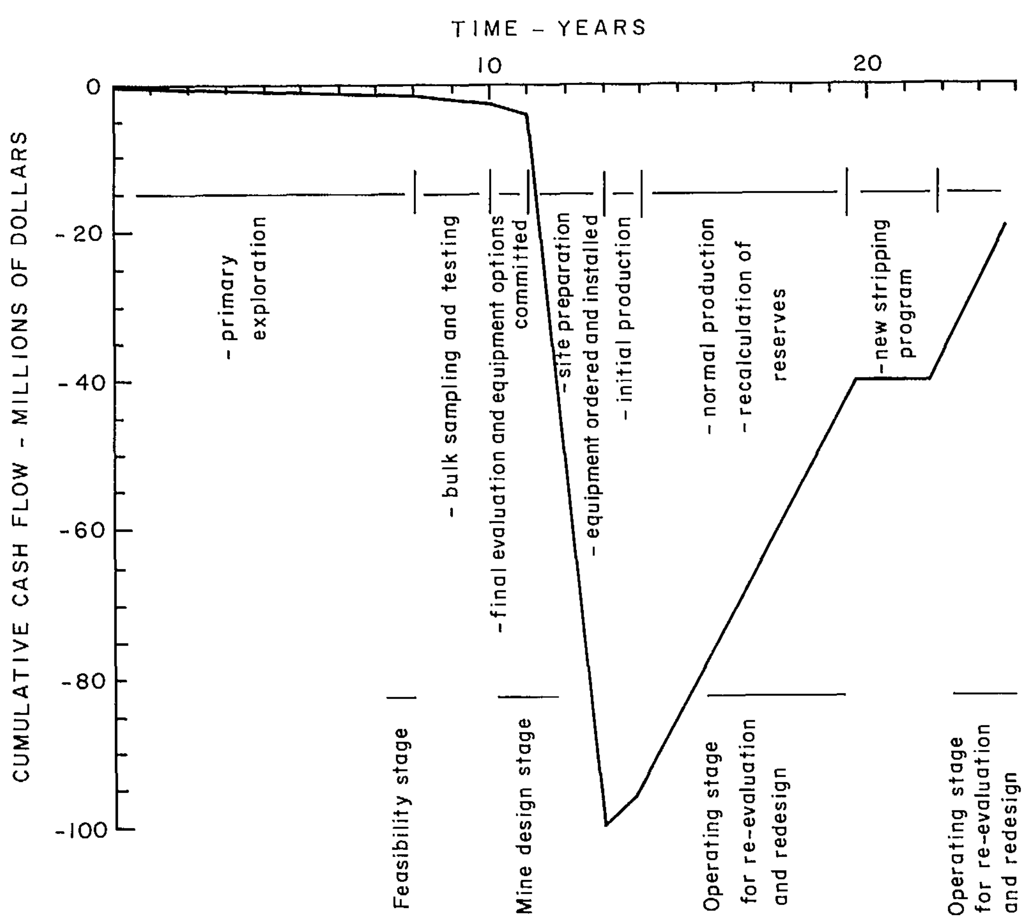
12. Walls are divided into design sectors based on the results of the field investigations. These sectors require separate stability analyses. Results are expressed in terms of reliability or probability of instability, as shown in Fig. 3.

13. These results are input for the financial analyses, where optimum pit slopes are determined. Slope reliability for each depth of excavation is determined. The minimum reliability only occurs

when the pit bottom is reached at the end of the mine's life. In this way, stability information is combined with data on grade, tonnage, prospective prices and other relevant parameters, to provide management with a basis for final decisions. Costs of instability must also be incorporated in the risk analysis as described in Supplement 5-3. In addition, the location of the toes of the walls can depend on grade variations and geometry (see Fig. 4) and may require an additional optimization analysis (1).

14. All slopes are designed to be safe. Like light bulbs and bridges, their reliability is less than 100%. The corollary to recognizing a probability of instability, no matter how small, is to examine the consequences of instability. Aside from the economic effects, safety must be ensured. Fatalities in open pit mining occur at a very low rate as shown in Table 1. Furthermore, the main causes do not include instability. Consequently, steepening of walls can often be considered a reasonable option. Mine planners will want to examine the need for catch benches, special monitoring and possibly perimeter blasting. Figure 5 shows a large slide that was predicted by deformation measurements, which typically provide adequate warning.

Fig. 2 - Cash flow diagram showing the slope design stages of: feasibility, mine design and operating.



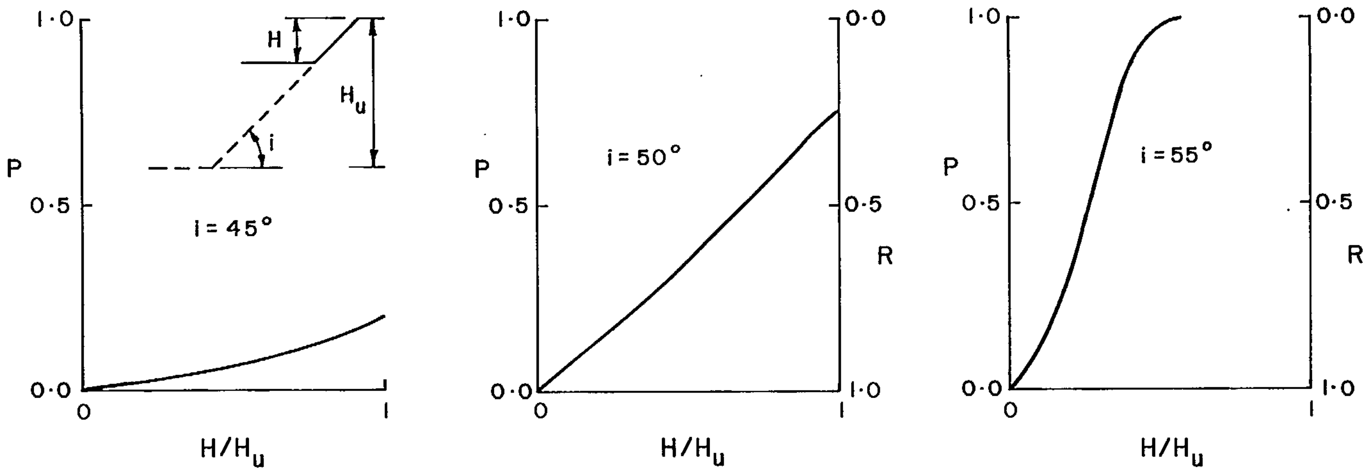


Fig. 3 - Possible variation of probability of instability, P, with slope height, H, for the different slope angles of 45°, 50° and 55°. H_u is the ultimate slope height. R, the reliability, equals (1 - P).

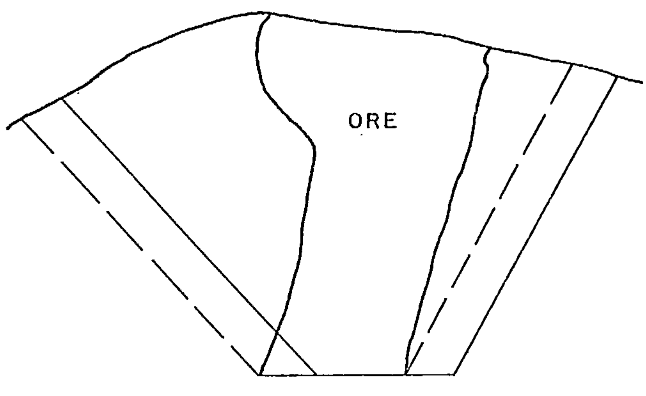


Fig. 4 - Separate optimization analyses are often required to determine the location of the toe of the slope; the best location of the toe at one section may not be the best for an adjacent section owing to the cost of obtaining the last wedge of ore as well as to the changing location of the ore boundary (2).

Table 1: Comparative approximate fatality rates (per 10^6 hours of exposure)

Highway travel		1.8
Air travel		2.4
Motorcycle Travel		4.4
Cigarette smoking		2.6
Open pit mining		0.42
Falls of rock	0.01	
Runs of muck, stockpiling, etc	0.03	
Fall of person	0.05	
Vehicle accident	0.10	
Miscellaneous	0.23	
Logging		0.94
Construction		0.26

Fig. 5 - A large slide that was well predicted by monitoring deformation of the crest.

Associated Designs

15. Optimum slope angles might be selected that are rather steep. It may be necessary to pay an insurance premium in the form of an intensive monitoring system to ensure safety of personnel and equipment. The design of such a system is dealt with in the Monitoring chapter.

16. The feasibility of having steep slopes may depend on the elimination of loose rock from bench faces. This might be achieved with scaling and appropriate perimeter blasting. The design of such a system is dealt with in the Perimeter Blasting chapter.

17. Situations can arise where the use of mechanical support of the walls is economically advantageous. The design of such systems is dealt with in the Mechanical Support chapter.

18. Trial slopes can be useful to eliminate expensive, conservative assumptions made in the initial wall design. It is sometimes possible to excavate such slopes in an area where production would not be affected. Factors which should be considered in the design of any such trials are described in Appendix C.

Report Specifications

19. Reports should be written recording the bases of design for each stage - feasibility, mine design and operating. These are in addition to the individual reports required on structural geology, groundwater and mechanical properties. It is often difficult to recognize when a design has been completed. No sooner is a decision made about the location of a wall or slope angle than modifications occur. However, the recording of design data, particularly when fresh in everyone's mind, is an important contribution to subsequent decisions.

20. The following report specifications are relatively brief, leaving detail as a matter of discretion for the responsible companies and individuals.

a. A Title Page must include the name of the organization, the report title, the author, date, and possibly the number of the particular copy if close control of distribution is desired.

b. The Summary must be a condensation of the information contained in the report. It should not be an expanded title, merely telling what information can be found in the report. Non-specialist language must be used to communicate with mine managers.

c. A Contents page must be included showing the page numbers of the various headings and sub-headings. This supplements the summary as well as being useful to those who must refer to various sections repeatedly. Lists of illustrations and maps are optional.

d. The Terms of Reference or Purpose and Scope must include a statement of the date and authorization of the work. It should be clear whether the design is for a feasibility study, an initial pit plan, interim wall positions or a new ultimate pit plan.

e. Definition of Design Sectors must explain the bases for determining the sector boundaries. Lithology, mechanical properties, structure, wall orientation, topography and such operational requirements as mining equipment, haulage, plant location and waste embankments can all influence the definition of the design sectors.

f. Sector A will describe information used in design and references to the source reports. Lithology and mechanical properties, major structural features and structural fabric, groundwater regime, previous slides are the sort of details that should be described. The assumed potential modes of failure and the range of slope angles considered feasible are recorded. Computer inputs together with output data should be included. Similarly, the internal cost of capital, the cost of instability, the price of the product, grade and reserves that have been used for the financial optimization should be mentioned. Recommended overall, interramp and bench slope angles for ultimate and interim wall locations are recorded. Similar kinds of information are provided on any associated designs that are utilized such as perimeter blasting patterns, monitoring systems, mechanical support and trial slopes.

g. Sector B should be described in a separate section, as should each design sector.

h. Conclusions and Recommendations present the results obtained for each of the design sectors in a consolidated form. They indicate decisions that may have to be made by management. Proposals for additional work may be included. For example, improved structural data on critical joint sets or faults can be collected after the mine design stage when operations start. Similarly, the occurrence of expected face seepage in certain sectors can be confirmed or otherwise. Also, easily obtained information may be recommended to be gathered on mechanical properties, perimeter blasting and possibly monitoring.

i. Acknowledgements provide information on the assistance given by individuals and organizations.

j. Glossary or definition of terms should usually be included to ensure that ambiguities are eliminated from the text (distinguishing particularly between such terms as bench and berm). Abbreviations and symbols, no matter how common, should also be defined in this section.

k. A List of References to reports and possibly outside publications should be included to permit tracing of the data used. The format should be: author, title, publication volume, number, inclusive pages, and date.

l. A Distribution List is used if close control is to be exercised. Such a list normally is the last page of the report.

PRELIMINARY MINE LAYOUT

Ultimate Pit Plan

21. An ultimate pit plan is a layout for the maximum economic recovery of the mineral resource. It is the target of all interim mining.

22. Wall design is the determination of the geometry of the mine boundaries, ie, the location and slope angles of the walls. Design can be thought of as precedent modified by testing and analyses. The process requires input from exploration, experience, stability investigations and financial analyses. At any stage it represents the best effort of the planning engineers, based on the information and time available. Designs can always be improved with more time and/or information, modifications and redesign continuing until implementation.

23. Wall design is an iterative process that must reflect changing conditions as mining progresses. A preliminary layout study is presented in Appendix B to illustrate the context into which rock mechanics work must fit. Part of the input includes the results of site investigations conducted according to the chapters on Structural Geology, Groundwater and Mechanical Properties.

24. The preparation of a preliminary ultimate layout must progress through a series of steps :

a. stating the planning objectives,
b. stating the fixed constraints,
c. stating assumptions and estimates,
d. calculating the economic design criteria,
e. designing the pit,
f. designing the appurtenant facilities and services, and
g. reviewing the design.

The layout in Appendix B is developed through these steps.

25. Feasible slope angles must take into account operating constraints. Ramp and catch bench locations can be influenced by and in turn themselves influence overall slope angles. Figure 6 shows a 60° x 275 ft (84 m) slope where it was difficult to maintain catch benches, and, of course, it would have been impossible to carry a ramp on this wall. Inter-ramp slope angles will be steeper than the slope angle from toe to crest. The locating of surface plant, particularly where restricted by topography, can influence the wall designs. Also, modifications must be made in slope angles for the effects of significant horizontal wall curvature.

Fig. 6 - A 275-ft (84-m) high 60° slope where it was found difficult to maintain adequate catch benches.

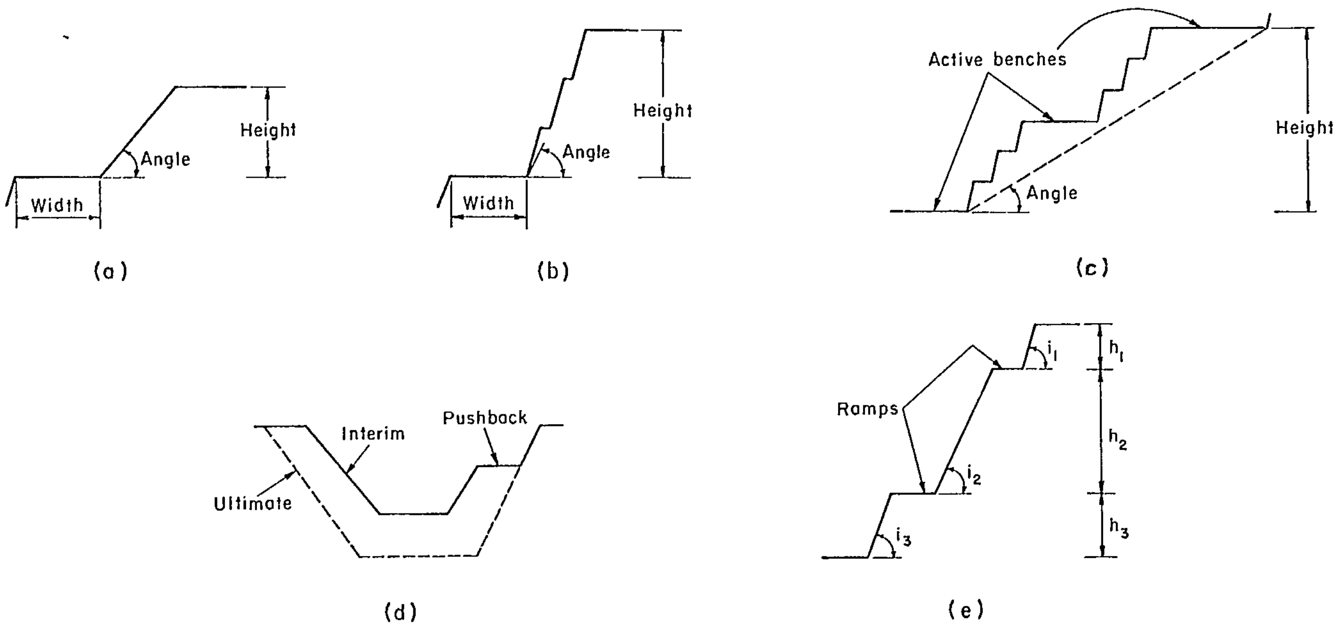


Fig. 7 - Some terms used in describing slopes: (a) bench (ie a single cut); (b) berm (ie combining two or three cuts); (c) working slope (several active benches); (d) interim and ultimate overall slopes; and (e) inter-ramp slope, i_2, h_2.

Wall Geometry

26. Pit walls consist of many slope elements, which require separate design decisions. For example, the basic unit is the bench formed by a single cut or lift. Other elements are berms, working faces, interramp slopes, interim overall slopes connected with a pushback and ultimate walls.

27. Bench slope angles are one of the basic determinants of working face, interramp and overall slope angles. Bench width, bench height and bench angle are defined in Fig. 7(a). For single benching operations, a remanent bench width of approximately one third of the bench height, would usually be needed, which limits the overall slope angle to 72° (assuming a bench angle of 90°). However, without perimeter blasting, it is often not easy to obtain a consistent bench angle of more than 60°, which means that the overall slope angle is limited to 48° (see Fig. 8(a)). Somewhat in contrast to this conclusion, Fig. 9 shows the case of a 55° x 680-ft (207 m) slope that was mined without serious instability although bench widths were rather narrow.

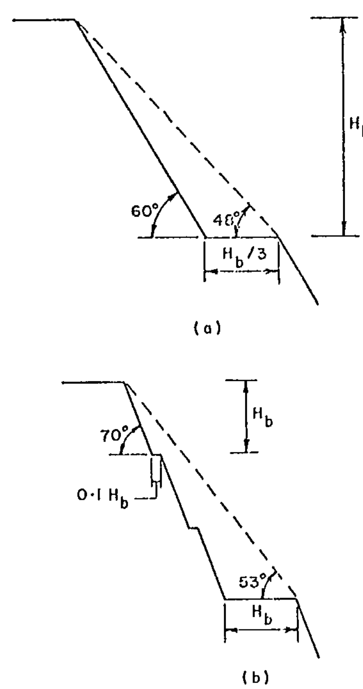


Fig. 8 - Influence of bench slope angles on overall slope angles: (a) single benching with maximum bench angle of 60°, and (b) combining three benches into a berm together with achieving a 70° bench angle gives almost the maximum slope angle that can be obtained, 53°.

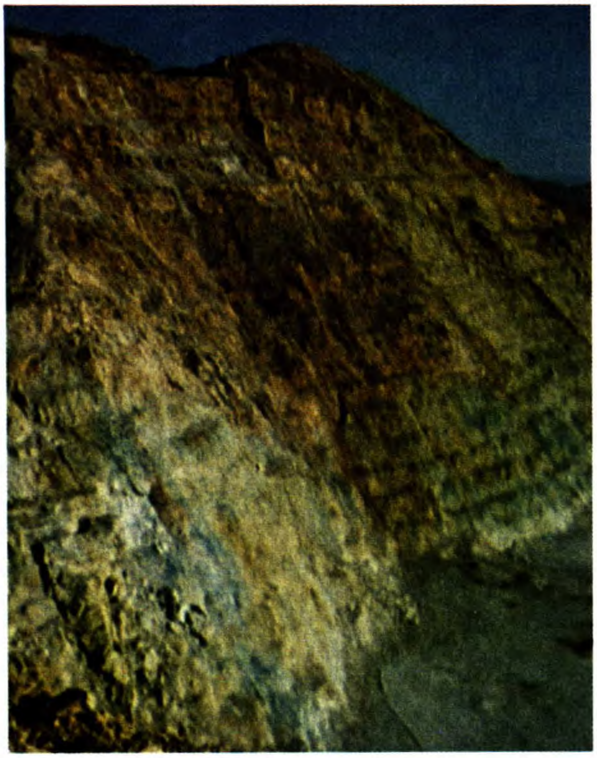


Fig. 9 - A 55° slope 680 ft (207 m) high, which proved to be quite stable.

28. The average bench angle will depend on structure and perimeter blasting. If layering or a strong joint set dips outward normal to the face from about 55° to 80°, this dip angle will substantially determine the maximum bench angle. Where such structural features do not strike parallel to the face, the degree of loosening of 3-d-wedges affects the average bench angle.

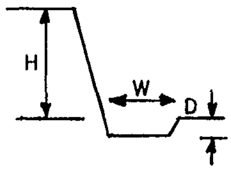
29. There is an optimum bench angle, but it is not likely to be predicted by analysing the problem mathematically. At the operating stage, the optimum angle will probably be determined by experience. The optimum will depend on the effects and costs of various blasting methods, the cost of extra waste excavation with lower angles (if the wall is in waste) and the cost of scaling of steep faces.

30. To overcome the constraint on overall slope angles by bench geometry, two or three benches are often combined into what can be called a berm. Berm width, berm heights and berm angle are defined as shown in Fig. 7(b). This procedure was tried on the wall shown in Fig. 6, which is 275 ft (84 m) high at an overall angle of 60°, but it was found not possible to maintain adequate berm widths. Drills cannot get exactly at the toe of a bench face so that the faces usually cannot be closer than about 2 ft (0.6 m) and 10 ft (3 m) is more common. In this case, even using perimeter blasting to obtain bench angles of 70°, the overall slope angle would only be about 53° (see Fig. 8(b)). On some properties, although difficult it has been possible to drill the full depth of three benches for the perimeter blast, eliminating the remnant benches which increases the overall slope angle by 2° as well as decreasing the amount of potential loose rock that might fall. Berm angles, are governed substantially by the same factors as for bench angles. However, the height of berms makes scaling more difficult and hence a more important cost factor to consider in design. The maintenance of catch berms also becomes more important.

31. Rock falls are difficult to predict. Faces that develop loose rock, particularly above ramps, can be troublesome. An extensive scaling program may be required but may not be completely successful. Mechanical support (such as bolting or retaining walls) and surface treatment (such as mesh, shotcrete or gunite) are described in Chapter 6. An approach developed for highways may be practical for some ramps. The digging of a ditch trap at the toe of the face has been used. This would be a modification of existing mining practice of leaving extra space on a ramp to avoid damaging haulage units. Table 2 shows the appropriate dimensions of such a ditch for various face angles and heights.

32. Where a wall is being actively worked on single, succeeding benches, the assemblage of benches or berms will form a distinct unit which can be called a "working slope or face". If the face coincides with the ultimate wall, this is also the ultimate wall angle. With multiple benching operations, the working slope angle will be much lower owing to the incorporation of the width of the active benches, as shown in Fig. 7(c). This width is determined by such practical

Table 2: Ditch dimensions to contain rock falls



Slope	H ft(m)	W ft(m)	D ft(m)
near	15(5) -30(9)	10(3)	3(0.9)
vertical	30(9) -60(18)	15(5)	4(1.2)
	over -60(18)	20(6)	4(1.2)
0.25:1	15(5) -30(9)	10(3)	4(1.2)
and	30(9) -60(18)	15(5)	6(1.8)
0.3:1	60(18)-100(30)	20(6)	6(1.8)
(73°-76°)	over -100(30)	25(8)	8(2.4)
0.5:1	15(5) -30(9)	10(3)	4(1.2)
(63°)	30(9) -60(18)	15(5)	6(1.8)
	60(18)-100(30)	20(6)	6(1.8)
	over -100(30)	25(8)	8(2.4)
0.75:1	0 -30(9)	10(3)	3(0.9)
(53°)	30(9) -60(18)	15(5)	4(1.2)
	over -60(18)	15(5)	6(1.8)
1:1	0 -30(9)	10(3)	3(0.9)
(45°)	30(9) -60(18)	10(3)	5(1.5)
	over -60(18)	15(5)	6(1.8)

(after Ritchie, A.M., Highway Research Record No. 17, Highway Research Board, Washington, 1963)

considerations as the amount of horizontal swell of the muck, the cutting and dumping radii of the shovel, truck width and bench height.

33. The working face with a single active bench, can consist of several successive benches producing a high slope. Stability and optimization analyses must be conducted for this slope. The Benefit-Cost program has to include the cost effects on operating efficiency of the steep slopes. All of the factors cited above for consideration in determining bench angles are included in the design of the working slopes.

34. Some pits are worked by mining a series of expanding shells or pushbacks. The definition of the terms ultimate, interim and pushback are shown in Fig. 7(d). The design of the interim slopes may not be as important financially as those of the ultimate walls because they may not be as high or they may contain ore. It is usually worth the limited expense of conducting an optimization study. No matter when it occurs, the postponement of the excavation of a ton of waste can be equivalent to the avoidance of such excavation, ie if the postponement is long enough.

35. The prediction of the costs of instability in interim walls is somewhat complex. Depending on the number of working places and the production schedule, instability on an interim wall can be mined out during the next pushback at little extra cost. There are indirect costs due to the changes in scheduled ore production and in the postponment of high grade ore.

36. Ramps affect the overall wall angle. In fact, they are often placed on weak walls to lower the overall angle. The relatively steep angle then is only used for the wall between the ramps, as shown in Fig. 7(e). Stability analyses must usually examine both the overall angle and the interramp slope angle.

37. Overall slope angles are defined as shown in Fig. 10. Where the slope includes ramps, the overall slope angle is defined by a line from the toe to the crest. Where a concave overall slope is designed, the overall slope angle for the full height is assumed to be equal to that of the lower part for purposes of the subsequent computer analyses described in Supplement 5-3. The steeper upper part is treated separately. Where the overall slope is convex, the slope angle for the full height for the purposes of the computer analyses is assumed to be equal to that defined by a line from toe to crest. The steeper lower part of the slope is analysed separately.

Design Sectors

38. Lithology, structure, rock properties, curvature in the wall, ore distribution and oper-

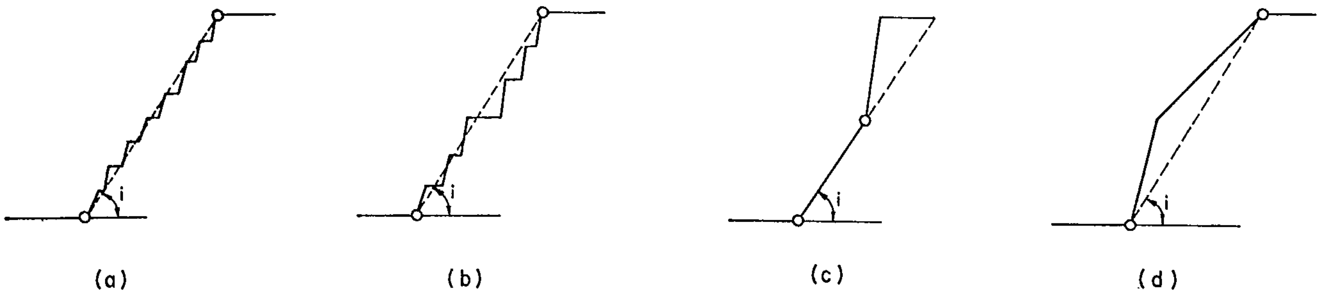


Fig. 10 - Definitions of overall slope angles for the purpose of the Benefit-Cost program: (a) uniform slope from toe to crest, (b) including a ramp, (c) a concave slope face, and (d) a convex slope face. In the case of (c) a separate evaluation of the stability of the steeper upper part of the slope must be made. In the case of (d) a separate evaluation of the stability of the steeper lower part of the slope must be made.

ational factors (eg level of reliability required, support of a main haulage, etc) all can influence the design of a pit wall. It was not uncommon in the recent past to assume that the same slope angle should be used for all walls of a pit. Now it is recognized that, where conditions are sufficiently different, sectors should be differentiated and designed separately as shown in Fig. 11(2).

39. The hangingwall and footwall of an orebody are often of different lithology and hence have different mechanical properties. For this reason, most pits have at least two design sectors.

40. Relative structural attitudes are pertinent. Layering in the walls usually means that the mechanical properties of the rock change with direction. Consequently, in the hangingwall and footwall the layering with respect to the slope surface will be in opposite directions. In the case of the footwall, strong layering dipping between about 45° and 60° could mean that combining several benches into a berm would be desireable to catch loose rock.

41. Similarly, major structural features such as faults, folds and intrusions will have different relative attitudes to the faces of the various walls. Joint fabric may be homogeneous throughout

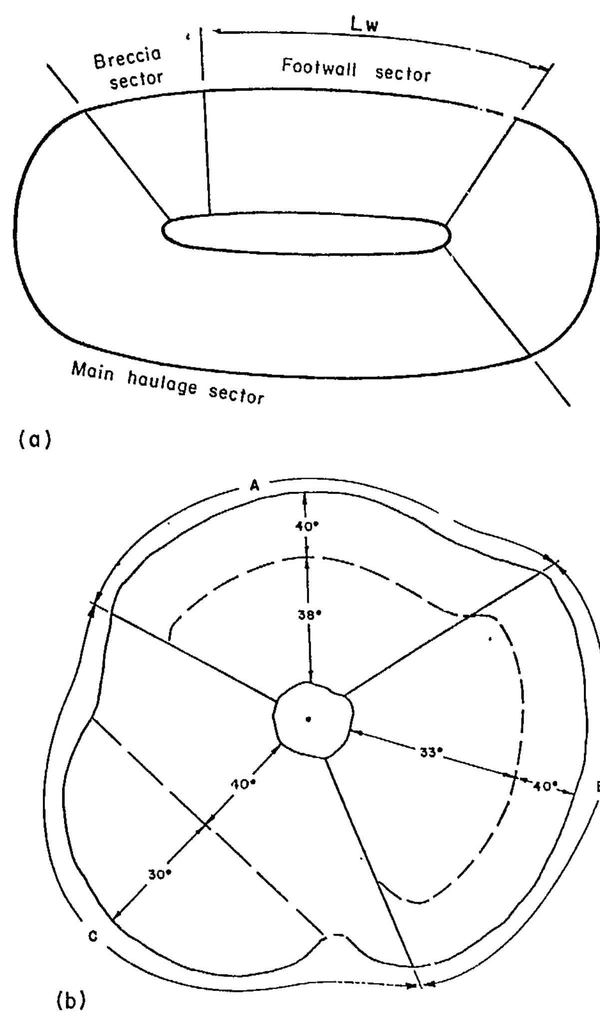


Fig. 11 - (a) Possible factors governing slope design sectors. (b) A large open pit divided into slope design sectors (2).

the mine, but the difference in geometry relative to the various slope faces again might make it necessary to have more than one design sector. Major features, such as faults or dikes, often occur in only one wall, clearly making it a distinctive design sector.

42. Modes of instability that would govern the critical slope angle will be influential in defining sectors. These modes in turn are governed by the rock properties. Figure 12 to 14 show the modes of instability that can be used for the purpose of defining the sectors. The mechanics of the various modes are dealt with below.

43. When the horizontal radius of curvature of a wall is less than about twice the wall height, the effect on stability can be significant depending on the type of possible slide. Sharp curvature, such as may occur in the end walls, can be the basis for defining a separate design sector. If curvature is the only difference between various sections along the wall, such as with an embayment of the ore or a nose of waste, the entire wall can be considered as one design sector and appropriate modifications made at those sections where the degree of curvature is distinctly different. (How to do this is explained in Supplement 5-2).

44. The effects of depth can also cause changes. Lithology might vary with depth. Mechanical properties within a formation might be different in a weathered zone relative to fresh rock at depth. Structural properties also can rotate or otherwise vary with depth. These changes might require the delimiting of a design sector with a horizontal boundary. However, it is usually more practical to extend vertical sector boundaries to the ultimate pit bottom so that the entire wall height is designed at once. The slope analysis would take the changes into account, possibly using different slope angles at different elevations, as shown in Fig. 32(b).

45. Surcharge on the ground surface at the crest of a slope, such as from a waste embankment and similarly from a mountain peak, would create special loading conditions. Adjacent sea coasts and lake shores could create special seepage

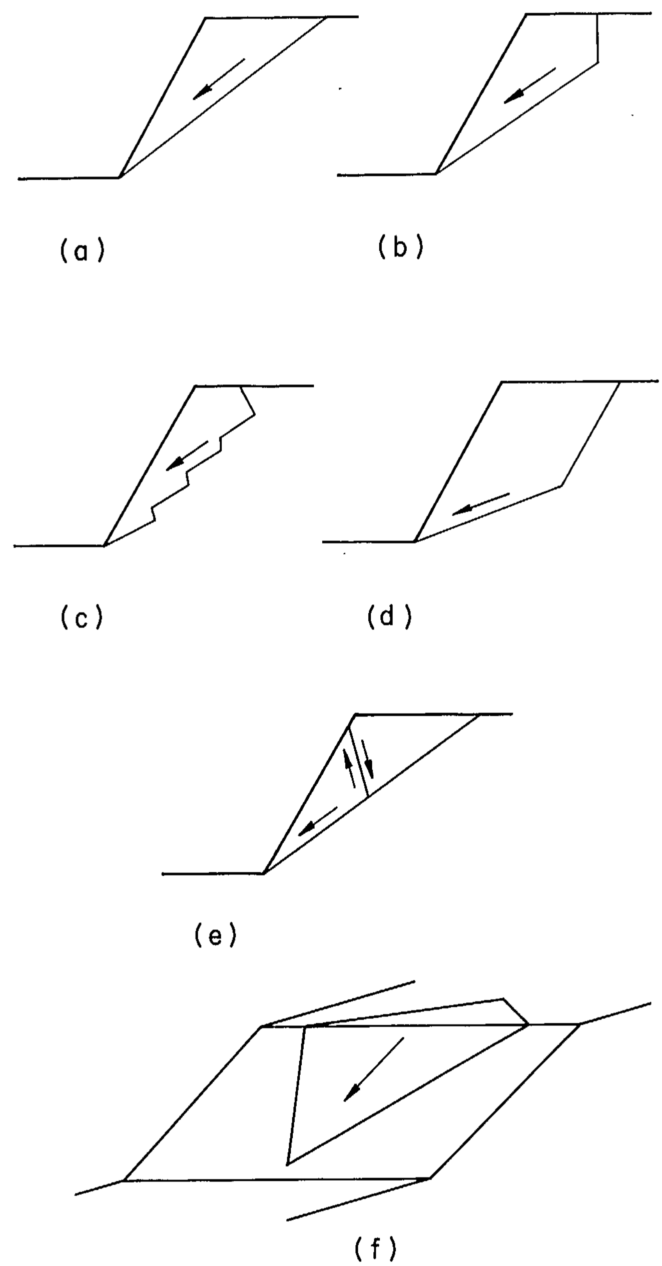


Fig. 12 - Simple and multi-block plane shear sliding modes: (a) one sliding plane and one block, (b) one sliding plane and a tension crack, (c) a series of short sliding planes with connecting cross joints, (d) two sliding planes, (e) a single sliding plane with the sliding block shearing into several blocks, and (f) two oblique sliding planes and a 3-d-wedge.

conditions. The presence of underground workings would require special studies. Stability analyses would have to reflect these conditions, and distinct design sectors would be required.

46. Operational factors might provide the basis for defining a design sector. If a wall is carrying a haulage ramp from the pit or from an adjacent operation, then the reliability required may be higher than for the other walls. Similarly, any plant installation at the crest of a wall, whose location might be dictated by the shortage of space as indicated in Fig. 15 by the location of the crusher in mountainous terrain, could be significant. Nearby houses of a community would have a similar influence. Different modes of transportation, such as skips, rail, truck and conveyor, can dictate different working slopes.

47. In summary, the major questions to be answered when establishing design sectors are:

a. should the sector be defined by the structural domain with respect to the slope orientation?
b. should the sector be defined by the strength of the rock substance where it is sufficiently weak and yielding to control the slope design?
c. is the sector required to have a particularly high reliability because of the presence of a haulage system or surface installations?
d. should the sector be defined by the distinctly different loading conditions occurring at some locations?

Stripping Ratios

48. It is appropriate to use a manual method for the preliminary ultimate layout. The planner can interact with his design as it develops. The use of the criterion of cut-off stripping ratio (COSR) is practical for preliminary layouts. This is the ratio of the quantity of waste material which must be removed from the mine in order to obtain a given unit of the desired mineral.

49. The stripping ratio may be expressed as: tons per ton, cubic yards per cubic yard, or cubic yards per ton. There are three different stripping ratios: (a) the overall stripping ratio, (b) the cut-off stripping ratio (also known as the break-even stripping ratio), and (c) the

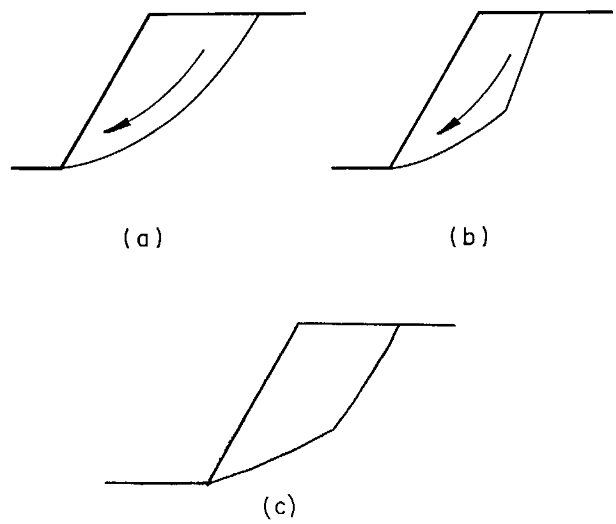


Fig. 13 - In ductile rock without the critically oriented planes of weakness, the potential mode of instability would be by rotational shear as shown in (a), which sometimes occurs in combination with a sliding plane as shown in (b) and (c).

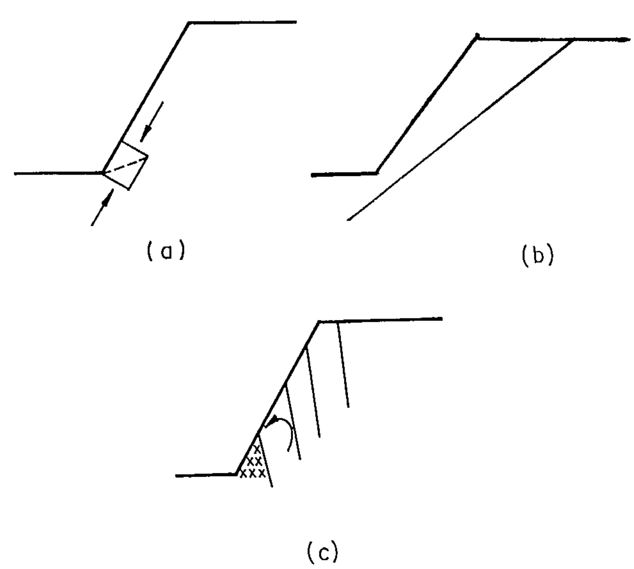


Fig. 14 - In brittle rock without a critically oriented plane of weakness instability would be initiated by crushing or shearing at the toe of the slope as indicated in (a), possibly combined with plane shear as in (b) or leading to toppling of slabs as in (c).

instantaneous stripping ratio (also known as the current stripping ratio).

50. The overall stripping ratio is based on the total amount of waste material which must be removed in order to obtain the total tonnage of desired mineral. The overall stripping ratio is not an operating parameter. It can be used to compare different designs of the same pit or different pits. It is dependent on such factors as the shape and attitude of the orebody, slope angles and topography. It is used to determine waste dump space requirements and capital equipment requirements. It is obtained only after the ultimate pit design is completed. The overall stripping ratio is always less than the cut-off stripping ratio.

51. The cut-off stripping ratio occurs when the cost of the required volume of waste is just equal to the increase in mine revenue from the unit of desired mineral uncovered by the stripping. It states how many tons of waste can be economically removed from the pit in order to mine one ton of ore. Mining at the cut-off stripping ratio results in a break-even operation, showing neither profit nor loss. The COSR is sensitive to changes in operating and waste stripping costs, grade, and product price. It is not usually affected by capital costs since in most cases they do not affect the incremental mining costs of stripping the last slice of waste.

52. The instantaneous stripping ratio applies to a limited period of time such as one month. The instantaneous stripping ratio is a production scheduling tool. The correct stripping ratio at any time is that which will maximize the present value of total future profits. It may range anywhere from infinity, as at the start of a mine, to zero at the end of the life of the pit.

53. Within the ultimate pit outline all ore above the cut-off grade can be mined at a profit. At the limit of the ultimate pit, the cost of stripping is balanced by the net income from the ore mined and processed. The cut-off stripping ratio (COSR) is calculated as follows:

Fig. 15 - A crusher and office located with some difficulty on the mountainside where an open pit is in production and level ground is at a premium.

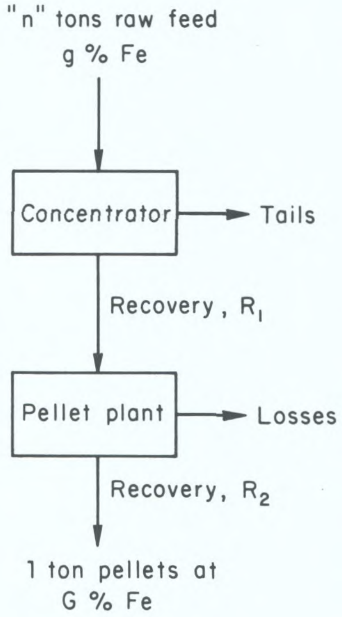


Fig. 16 - Concentration ratio or processing recovery definition.

$$COSR = \frac{R_g - C_m}{C_w}$$

where R_g is the gross value per ton of ore, C_m is the incremental cost per ton of ore for production and processing, excluding stripping, and C_w is the incremental cost of stripping per unit of waste.

54. The first step is to obtain the concentration ratio C_R (tons of raw feed to make 1 ton of final product) as illustrated in Fig. 16. By mass-metallurgical balance, $C_R = G/(gR_1R_2)$, where G is grade of product, g is grade of raw feed, R_1 is the recovery ratio from the first stage of processing and R_2 is the recovery ratio from the second stage of processing. The incremental cost/ton of ore is the cost/ton of product (including the variable costs of production, transportation and royalties) divided by C_R.

55. The second step is to calculate the gross value per ton of ore, R_g. For example, for metals the price/ton, p, can be multiplied by the recovered metal per ton of ore $R_g = pgR_1R_2$. Inflation is ignored assuming both revenue and costs escalate equally. The incremental cost of production does not include depreciation, interest on borrowed capital, and fixed costs. As there is no profit on the last ton of ore, no profit-related taxation is applicable.

56. This procedure entails designing the mine on cross-sections using prescribed slope angles and certain constraints such as a minimum width of pit bottom. Bulges into the pit on some sections must be smoothed out when the sections are put together for the layout. The apparent overall stripping ratio for each section provides an estimate of reserves. When a preliminary layout has been completed, computer analysis for mine design can be used.

FEASIBILITY AND MINE DESIGN STAGES

Wall Design

57. Ensuring stability of pit walls is a necessary part of mine design. The most common cause of rock slope instability is the occurrence of critically oriented planes of weakness such as faults, bedding and joints. Slides can occur on such planes. Small displacements and creep threatening major movement can also be the nature of the instability. Where no critically oriented planes occur, a high, steep slope (probably over 2000 ft for most unaltered rocks) would cause a breakdown in the rock substance leading progressively to the development of a sliding surface. In altered rocks breakdown of the rock substance leads to rotational slides. Certain structural regimes can produce, after sliding is initiated, rock slabs that topple off a relative steep face. These are the major types of wall instability that are experienced.

58. It is difficult to analyse rock slopes with the certainty enjoyed designing building structures. This situation has been resolved by trying to incorporate a safety factor against sliding greater than one. However, such calculations require determination of strength properties, geometry of structural features and groundwater levels in the future slope. All these factors cannot be predicted with certainty.

59. The new approach is, rather than using average or conservative values, to recognize variability, measure it and include it in the design. Reliability is calculated based on measured variation in strike, dip and length of the critical structural features, in strength properties and in the maximum annual height of the groundwater table. Sensitivity studies still are useful in educating designer's judgement. In this context the question becomes - how sensitive is the reliability of a given slope to different means or standard deviations of groundwater, dip of the critical joints and other factors.

60. A new question that arises is - what is the appropriate or acceptable reliability for a slope? The answer is found partly in the economics of the financial analyses. The effects of instability on ramps, surface plant, ore reserves and mining operations are converted to costs. The slope with the minimum ratio of cost/ revenue indicates the appropriate reliability. The answer is also affected by the appraisal of the effectiveness of monitoring. Predicting instability,

the consequences of instability and the possibility of providing quick remedial action - all influence the level of reliability that is acceptable regardless of economics.

61. When optimum slope angles are less than about 45° and dry, the consequences of instability are easily managed. Hence financially dictated optimums can usually be accepted. Also, in this range changes in slope angle usually cause relatively large changes in stripping volumes.

62. When the slopes are to be more than 45° and particularly when they are wet, slides and rock falls are more difficult to correct. Hence, where the corresponding reliability seems lower than desirable, deviating from the financial optimum might be judged appropriate. More experience in the use of this approach may make it possible to provide general guidance on reliability numbers.

63. The amount of analytical work that must be done at each mine development stage will depend on the quantity and quality of field data available together with the effect of the slope angle on stripping. With limited information typical of the feasibility stage, the design sectors of the walls are delineated as well as possible. Rock mechanics analyses of the slopes may or may not be appropriate. Slope angles for the various design sectors might be selected based either on previous slopes in the same rock or on general experience. At the mine design stage, for deep pits much information must be obtained and corresponding analyses are required.

64. In general, the following steps should be followed at any stage in analysing the stability of the walls.

a. <u>Establish boundaries of the design sectors</u> based on the factors described above.

b. <u>Determine design requirements for each sector</u>, eg ramp locations, catch bench specifications, toe location; optimum slope angles for berms, working faces, interim and ultimate walls; scaling methods; erosion control methods; protection of critical ramps or surface plant; possible future reclamation.

c. <u>Determine the potential mode of instability</u> in each sector by asking the following questions:

   i Will planar structural discontinuities striking parallel to the wall daylight in the face? If yes, go to Supplement 5-1 for details on analysing potential plane shear instability.

   ii Will intersections of planar discontinuities daylight in the wall face? If yes go to Supplement 5-1.

   iii Is the rock substance soft or yielding as shown by past experience or strength testing? If yes, go to Supplement 5-2 for details on analysing rotational shear instability.

   iv Are stress concentrations at the toe or between rocks of different deformation properties likely to exceed the strength of the rock substance? If yes, go to Supplement 5-4 for details on analysing the stresses with the finite element method.

d. <u>Identify design variables</u> and separate them into (a) uncontrollable, eg rock structure and shear strength, and (b) controllable, eg bench height and angle, groundwater drawdown, blasting, monitoring, ground support, ramp locations and the like.

e. <u>Check that all required data and analyses are included</u> in the geological, groundwater and mechanical properties reports.

f. <u>Determine maximum feasible slope angles</u> (ie ignoring stability for the moment, considering only operating constraints and possibly reclamation requirements) for the benches, berms, working faces, interim and ultimate wall for each sector taking into account the controllable variables.

g. <u>Analyse the stability</u> of each significant wall element (such as the overall ultimate wall, interim walls, interramp slopes and the like) by selecting two or three slope angles (including the maximum feasible) and determining the reliability of each for a set of slope heights from 100 ft (30 m) to the maximum planned height, in 100 ft (30 m) to 200 ft (61 m) steps.

h. Determine the effects of instability in the various wall elements (eg berms, interramp slopes, overall slopes, etc) on the cost of operations, on critical features such as main haulages, surface plant and future shafts, and on the ore schedule either by using the standard calculations (eg for cost of cleanup, unloading or postponed ore) included in the Benefit-Cost program or by composing appropriate cost estimates for the particular mine conditions.

i. Determine the optimum slope angles by running the Benefit-Cost program with inputs from g and h.

j. Rationalize optimum slope angles for all sectors into a practical pit plan, determining the geometry of transitions from one sector slope to another and eliminating where possible potentially troublesome bulges into the pit.

k. Plan field trials for perimeter blasting where it is judged necessary for acceptable maintenance costs.

l. Design monitoring systems for sectors where optimum slope angles make surveillance neccesary.

m. Design mechanical support systems where the cost of waste excavation would be reduced more than the cost of support or where insurance is required for a surface installation.

n. Design trial slopes where improved data can be obtained to permit redesign of the walls with significant cost saving. Such trials must be in areas that would not affect operations.

65. The main design requirements for each design sector are the determination of the optimum angles for the berms, working faces, interramp and overall slopes. The maximum feasible angles may minimize waste excavation, depending on the ore occurrences. However, the increased cost of operations resulting from increased scaling, clean-up of slide debris, possibly unloading and possibly inefficient mine operations may make a lower angle optimum. The maintenance of safety berms can be a dominant problem. The locating of ramps can be a major question with tradeoffs possibly existing between minimizing haulage costs and placing the ramp on a weak wall so that the overall slope angle is automatically reduced without extra waste excavation. Each mine usually has other special problems that must be incorporated into wall design.

66. The division of the design variables into those that cannot be controlled and those that can, makes explicit the decisions that have to be made. Clearly the geological features are given, eg fault locations, attitudes and composition; similarly joint set domains, geometry and composition. Also, regional groundwater levels and their variations, high permeable zones and the principal directions of permeability, and the mechanical properties of the rocks such as their strength and density are all uncontrollable.

67. On the other hand, the drawdown of the groundwater in the walls is, to some extent, controllable if drainage systems can be effective. Surface erosion effects can be controlled by design.

68. Within the constraints of the pit equipment bench height, widths and angles are somewhat controllable. Perimeter blasting is an option that can be exercised, albeit at a cost, to affect bench angles which in turn control maximum working face angles and overall wall angles. Similarly the use of mechanical support is an option or controllable variable.

69. Monitoring can also be considered a design variable. It is quite reasonable to accept a steep wall with a somewhat lower reliability than normal due to constraints, such as a high mountain scarp or the location of pre-existing plant, and then compensate by using an intensive monitoring system. For example, in one pit where a large slide had prevented mining an important part of the orebody it was decided, with the inspector's approval, to mine at the toe of the slide while measuring the movement every two hours on targets placed from top to bottom of the slide segment. (This is actually not an example of slope design but of compensating for instability with monitoring). Also, many special devices can be used in unusual circumstances such as catch structures and automatic warning devices developed

by the European railroads (see the Monitoring chapter).

70. When the maximum feasible operating slope angles have been determined ignoring stability, the next step is to select one or two lower slope angles that are also feasible. For example, if 47.5° has been determined to be the maximum feasible interramp slope angle, then 45°, 42° and 39° could also be considered. Stability analyses provide a schedule of reliability versus height for each angle.

71. Table 3 shows some typical cost figures for budget purposes for the various elements of work required for wall design. Owing to the manifold variations from mine to mine such figures might be considered meaningless; however, the orders of magnitudes are realistic and as such can be of use to planners. For deep pits, the $100,000 cited arises typically from the time taken by about six men over the best part of a year. The figures are based on the experience of a number of companies. The amounts are expressed in 1973 dollars so that an inflation index (essentially a salary index) should be used to make them applicable to the pertinent year. They are intended to represent both direct and overhead costs and hence be applicable to work done by company staff and by contractors.

Table 3: Budget guidelines (1973 $)
(Typical range shown in brackets)

Depth	Feasibility	Mine Design
100 ft	$ 10,000	$ 15,000
(30 m)	(1000-50,000)	(1000-50,000)
100-1000 ft	20,000	50,000
(30-300 m)	(5000-200,000)	(5000-200,000)
1000 ft	100,000	100,000
	(10,000-500,000)	(10,000-500,000)

Note: In general, estimates based on 2 x salary cost.

Field Investigations

72. Field data are required for mine design. The amount of information to be gathered depends on the potential problems and on the stage of development of the property. Usually less expenditures are warranted at the feasibility stage than at the mine design stage. Judgement must be exercised on the appropriate quantity and quality of investigations.

73. Feasibility stage. Figure 17 shows the integration of field investigations and other activities in wall design. A certain amount of information can and must be gathered at the feasibility stage on structural geology, groundwater and mechanical properties of wall materials. Much information is available on drill hole logs, core logs, outcrop and trench maps, and maps of exploration drifts. Other elements, as described in the investigation chapters, will require extra work. It is strongly recommended that such data be recorded on forms specified in DISCODAT as described in the chapter on Structural Geology.

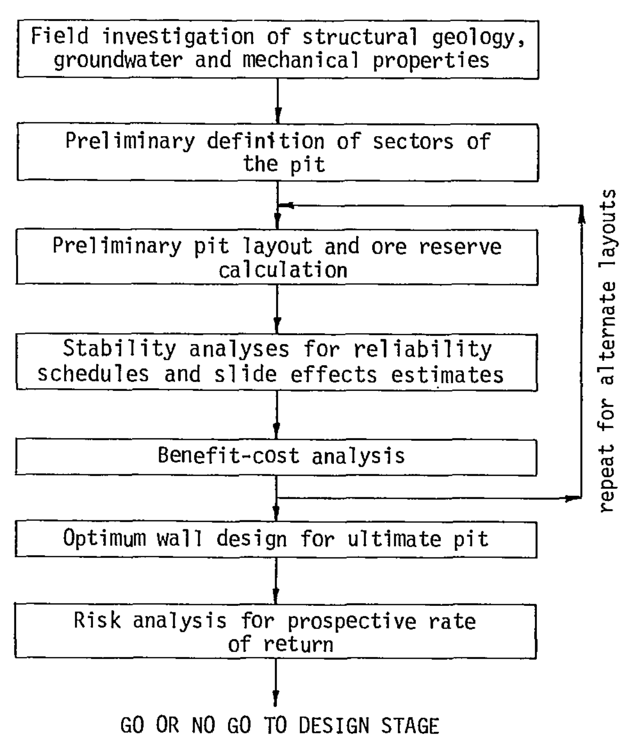


Fig. 17 - Activity flow chart for wall design at the feasibility stage.

74. All pertinent information obtained during the investigations must be recorded in reports. There is often a time lag between the feasibility stage and the mine design stage, and change of personnel can occur during such an interval. In addition, change of ownership or introduction of new partners can make such records of particular value.

75. Specifically, a structural report must be written to provide design information on the lithology and geometry of the formations, on the geometry and composition of major structural features, such as faults, and on the domains, geometries and compositions of the sets of discontinuities.

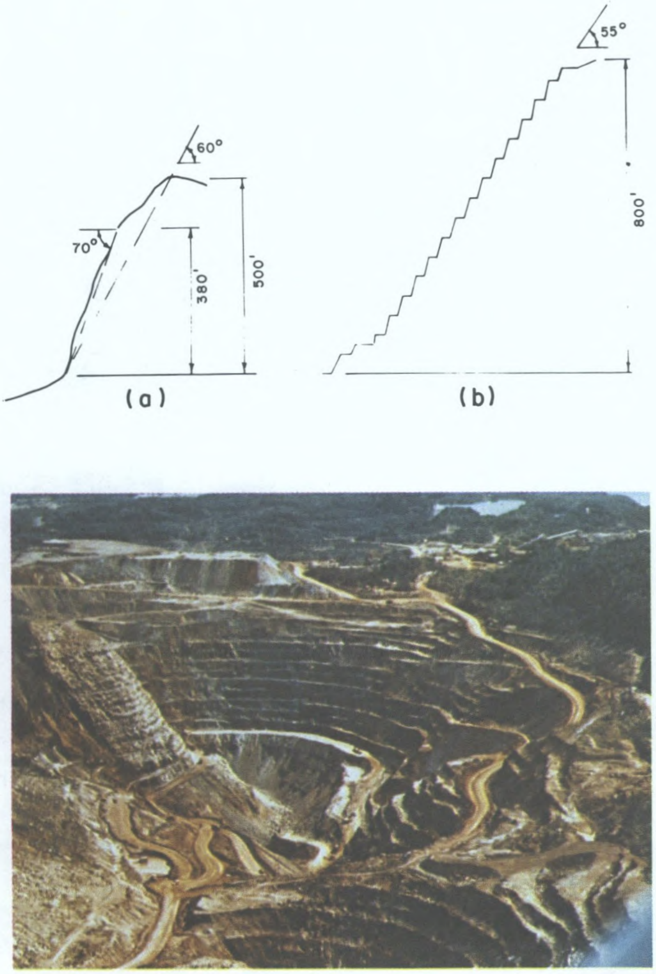
76. The report on mechanical properties of the rocks can be combined with the structural report. It must include the qualitative appraisals obtained from the various logging operations together with the quantitative data obtained from drill penetration rates, from core recoveries and possibly from P-wave velocities obtained within the hole or between holes. The rock mass density might be available from geophysical logging, and the rock substance densities must be measured in the laboratory from core samples. For soil overburden, mechanical analyses and consistency limits are required. Either direct or indirect measurements of the uniaxial compressive strength of the different materials are also needed. (The same information gathered for the ore should be included in the report to assist in appraising the production requirements of drilling, blasting and milling.)

77. A groundwater report must be written. Information must be gathered on the water levels and artesian flows encountered during the drilling of exploration holes. Piezometers must be installed in an appropriate selection of these holes to provide a maximum period of record of groundwater behaviour. Statistics on precipitation from the nearest observation stations, on pertinent stream flow and on any pre-existing groundwater wells must be obtained. From this information and that of the structural study, the location of highly permeable zones where drainage and pumping may be required are predicted. The principal directions of permeability in the rock mass, the seepage line and hence water pressures in the pit walls are analysed. An indication of the feasibility of internal slope drainage is obtained. Also, erosion of the slope surfaces and pumping requirements are appraised.

78. If particularly weak discontinuities are detected during the field investigations, it may be necessary to do some supplementary structural drilling. Such information may be required to delineate the slope design sectors of the ultimate pit and to predict the mode of instability that would apply to each of these sectors. In other words, there must be sufficient structural information to recognize the possibility of plane shear slides, and there must be sufficient information on the mechanical properties to identify sectors where rotational sliding can occur. A minimum amount of strength and groundwater data is needed to conduct stability analyses for all pertinent modes of failure.

79. Appendix C contains the specifications for gathering information from previous slopes in formations to be encountered in the projected open pit. Data on joint fabric, groundwater and effective strength parameters in either excavated slopes in prior open pits or natural slopes on mountain sides and gorges can be used taking into account significant differences between these and the pit slopes. For example, in Fig. 18(a) natural slopes of 60° and 70° are shown. These were used to design the 800 ft (244 m) wall in the projected pit. Fig. 18(c) shows the actual slope that was successfully excavated.

80. During the feasibility, stage heights of previous slopes, average angles and directions (normal to strike) should be collected. In the structural report experience curves (average slope angle versus slope height) must be plotted for the various formations and slope directions. If time permits, information on the joint fabric in these slopes together with the groundwater regime would be useful. In soils, mechanical analyses plus consistency limits is essential. With this supplementary information, better judgement can be exercised on the pertinence of the experience curves to the projected slopes.



(c)

Fig. 18 - (a) Natural slope data used to extrapolate to the 800 ft (244 m) pit wall in (b). (c) The actual slope, shown on the left, was excavated at 58°, 1070 ft (326 m) high. Vugs encountered at the bottom made the modifications necessary.

81. Mine design stage. After the feasibility studies have shown that a potential mine exists and it is decided that development should proceed forthwith, new field investigations are required. For the mine design stage these must include the gathering of additional information on the geology, groundwater and mechanical properties of the rocks to permit the re-examination of the preliminary ultimate slope designs and the design of interim slopes. Figure 19 relates the field investigations to the other activities in the design of the walls. Reports must be written on this work substantially as for the feasibility stage as shown in Table 4. Detailed consideration must now be given to bench slope angles and inter-ramp angles. The required information may be obtained from the additional exploration holes that usually will have been drilled for this stage. In addition, the drilling of some purely structural holes may be warranted. Mean values and a measure of the variability of all geometric properties of critical structural features are required for design.

82. At the mine design stage, a somewhat longer period of record of groundwater levels at

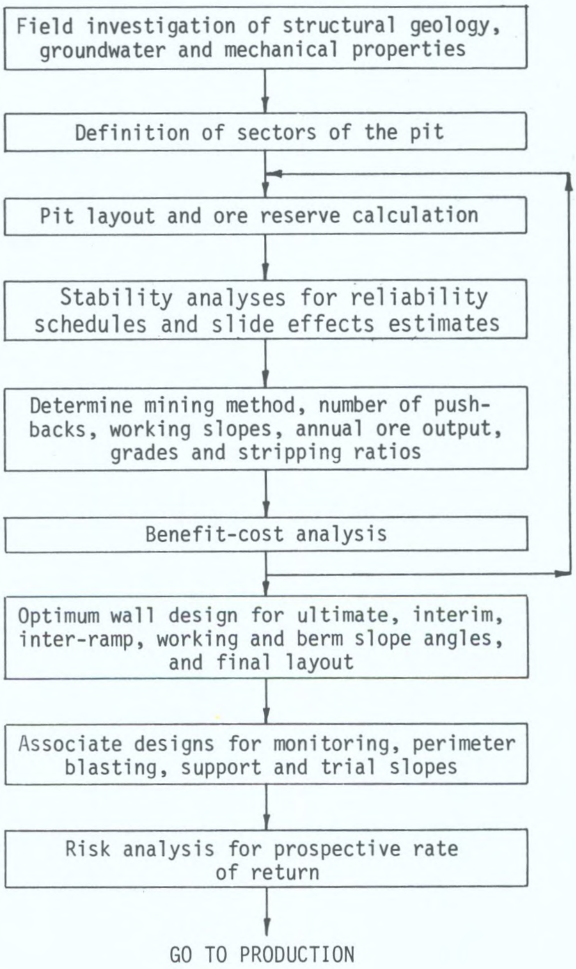


Fig. 19 - Activity flow chart for wall design at the mine design stage.

and around the orebody will be available from piezometers installed in exploration holes. A more thorough examination of the records of precipitation, stream flow and regional groundwater wells must be conducted at this stage to develop improved estimates of future groundwater levels. Using the additional structural information, a better determination of the principal directions of permeability can be made for use in considering drainage schemes and predicting the drawdown curves into the pit. Such predictions based on theoretical analyses are usually rather idealized. In this area, judgement based on empirical data should be used to override such analyses where necessary.

83. More field and laboratory testing must be done to determine mechanical properties of the critical materials. For example, samples must be obtained for shear testing of faults, layers, joints and other critical planes. Additional laboratory testing is required at this time on soft formations that could cause rotational sliding. Enough data is required to provide measures of both mean values and variability of the strength parameters.

84. Previous slopes must be fully exploited. Analysis of previous slides will provide information on strength parameters. Slides such as shown in Fig. 20 were utilized in this way to design the subsequent pit in the same formation, which turned out very successfully (3). Structural investigations on the previous slopes must be conducted if this information is not available from the feasibility stage. In some cases, structural drilling may provide valuable information. Information pertaining to the groundwater regime in the previous slopes must also, at this stage, be collected. Testing of the rock in the previous slopes may be required if it seems that the strength properties of the formations are somewhat different from those in the projected mine.

Stability Analyses

85. Supplements 5-1 to 5-4 describe the computations that can be made to produce the input data

Fig. 20 - A slide in a 215 ft (66 m) wall at 51°, which was used along with similar slides in back-analyses to design the subsequent pit in the range.

Fig. 21 - Potential plane shear sliding in a coal mine on bedding planes on one wall and on laminations on the other wall.

for financial analyses. Different wall elements, eg berms, interramp slopes, ultimate walls, etc, may have different potential modes of sliding. All possibilities must be included in the design appraisal. Reliability schedules according to each mode can be used in the Monte Carlo sampling procedure of the benefit-cost analysis.

86. Plane shear sliding may result when structural discontinuities make it possible for blocks to slide into the pit. Such conditions are shown in Fig. 21 by bedding in one wall and extensive joints in the other. Insufficient shear strength is the second condition for instability. The discontinuities may be bedding planes, fault planes, layering and laminations, and extensive or connecting joint planes. The presence of a single, large plane striking parallel to the wall and dipping into the pit, as shown in Fig. 12(a), is the simplest case. Frequently, the sliding block is truncated by a tension crack, as shown in Fig. 12(b). A set of joints, with the help of cross-joints, can form a sliding surface, as shown in Fig. 12(c). Variations in shear resistance along a sliding plane can lead to an upper block pushing a lower block as shown in Fig. 12(d), which can induce shearing between the blocks and possibly lead to rotation and toppling of the lower block. The toppling is a consequence of the plane shear sliding and does not require a separate analysis. Two discontinuities at different attitudes can combine to form a bi-linear sliding surface, as shown in Fig. 12(e). Usually in such cases the block will be sheared into two or more parts. When the discontinuities strike obliquely to the wall, two or more can combine to provide surfaces on which sliding can occur, as shown in Fig. 12(f). The domains of the attitudes of such discontinuities provide a basis for defining the boundaries of the design sectors. Lengths, spacing and quality of the discontinuities should be examined to confirm the boundaries of the structural domain.

87. When potential sliding is by simple plane shear, as represented by Fig. 12(a), 12(b) and 12(c), a simplified analysis can be used for preliminary design purposes. The critical slope angle can be calculated by an equation given in Supplement 5-1. It shows the influence of cohesion, rock mass unit weight, slope height, surcharge pressure on the surface at the crest, friction on the sliding plane, and water table elevation behind the crest. These parameters are random variables within certain ranges. They cannot be precisely determined. To calculate the expected standard deviation of the critical slope angle requires the use of the computer program PLAFAM of Supplement 5-1.

88. Variations of the input parameters follow no rule and should be determined by measurement. Discontinuity dip, length, cohesion and friction are the variables whose dispersions are most influential; dispersions of rock density, slope height, surcharge pressure and water table usually affect the calculated standard deviation of the critical angle to a lesser degree. It is usually convenient to assume that dip angles, cohesion and friction vary according to a normal distribution as shown in Fig. 22(b). Joint lengths usually follow an exponential distribution as shown in Fig. 22(c)(4). Other variables can follow other distributions. However, it is not essential to force the data representing dispersions into a mathematical form. Histograms of the actual measured values can be used in the stability and financial computer programs described in Supplements 5-1 and 5-3.

89. The dispersion of the dip angle is obtained through the procedure described in the chapter on structural geology. One way is to use the plot of poles on an equal area net, as shown in Fig. 22(c). A spherical normal distribution is fitted, which produces a concentration factor K. From this distribution the standard deviation of the dip is calculated as $1/\sqrt{K}$. Alternatively, a histogram of dip angles can be plotted from the field data. The dispersion of strike also can be treated in either of these two ways.

90. From the structural investigations, it may be known that a major structural feature such as a fault, a dike or layering exists at a critical attitude. Alternatively, a joint set may be known to have such an attitude. An implicity assump-

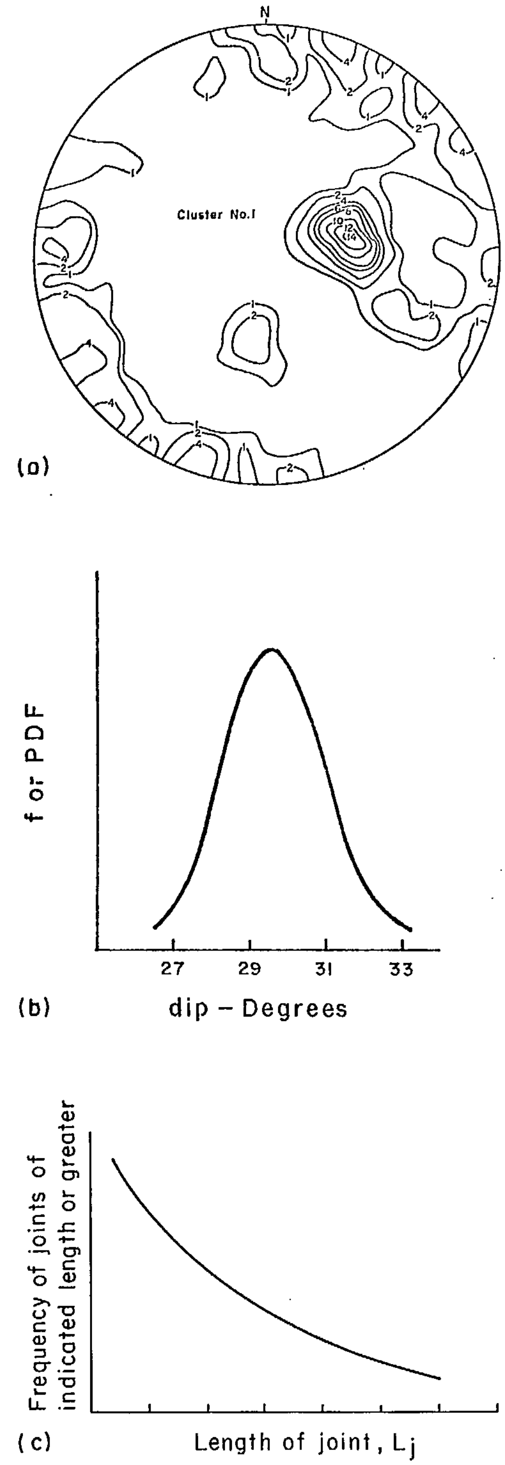


Fig. 22 - (a) Schmidt plot of contours of poles to joint planes showing the identification of sets. (b) A probability density function, PDF, describing the variations in dip angle of a set of joints. (c) A cumulative distribution function describing the variation of joint lengths in a set.

tion is then made that there are cross-fractures present that will eliminate any shear resistance on the ends of the sliding block. It has been shown that if, as shown in Fig. 23(a), the dip of the discontinuity, D, is undercut by the slope, S, sliding can occur. A two-dimensional analysis parallel to the direction D, is then appropriate (5). If there are two discontinuities neither of whose dips, D_1 and D_2, as shown in Fig. 21 (b), are undercut by the slope, S, 2-d sliding cannot occur. If their intersection, I, is undercut by the slope, S, a three-dimensional wedge can slide down the intersection.

91. Example. The height of a pit wall is to be 500 ft (152 m) with topographic variations giving a standard deviation of 20 ft (6 m). The mean annual maximum height of the regional groundwater table is 400 ft (122 m) with a standard deviation of 50 ft (15 m). The mean rock mass density is 173 pcf (2.77 t/m^3) with a standard deviation of 17 pcf (0.25 t/m^3). There is no surcharge on the crest. Bedding planes dip into the pit at a mean angle of 40° with a standard deviation of 3°. The mean apparent angle of friction on the bedding plane is 35° (including dilatency) with a standard deviation of 5°. The mean apparent cohesion is 1440 psf (69 kPa) with a standard deviation of 432 psf (21 kPa). (The terms 'apparent cohesion, C' and 'apparent angle of friction, ϕ' are defined in the chapter on mechanical properties. They take into account the effects of dilatency, infilling material and aperture of the discontinuity.)

92. The mean critical slope angle corresponding to the mean values of the above parameters is calculated to be 45.5°. Thus one cell, ie a breadth of slope equal to the height, of a slope 500 ft (153 m) high at 45.5° would have a 50% probability of sliding (P_s).

93. The standard deviation is calculated to be 4.98 or 5°, which is approximately equal to that of the angle of friction - the most influential variable in this case. Using a slope angle of (45.5-5) or 40.5° would reduce the probability of sliding to 16% (the area of the probabil-

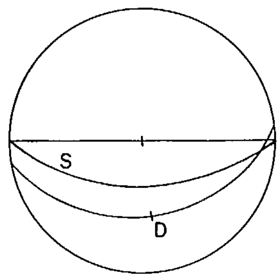


Fig. 23 - A Schmidt plot showing a slope plane, S, and a discontinuity plane striking obliquely to the slope and dipping at a flatter angle. If the dip angle, D, is undercut by the slope, as occurs in this case, then plane shear sliding can occur. When the point D is not undercut by the plane projection S, then simple plane shear sliding cannot occur.

ity (PDF) curve beyond one standard deviation). Figure 24 shows the relation between P_s and slope angle. The volume of any such sliding wedge would be small, the mean dip of the sliding plane being 40°.

94. The above analysis includes the assumption that the length of the structural plane, L_j, extends from toe to crest. From the structural investigations the approximate relation between the intensity of jointing in one set and length will be known. A typical relationship of this nature is shown in Fig. 22(c)(4).

95. The probability of instability, P_f', for one cell of a wall is the joint probability of the probability of occurrence of the critical geometry, P_o and the probability, P_s, that sliding will occur with that geometry, ie

$$P_f' = P_o \cdot P_s \qquad \text{eq 1}$$

96. P_o is the probability that a joint of length equal to or greater than L_j will occur in the face exposed during the period of time, usually one year, used in the Benefit-Cost program. Where only one discontinuity can occur, the probability of having the required length is assumed to be the probability of occurrence, P_o, of the critical geometry. The exponential curve of Fig. 22(c) can be represented as follows:

$$P_L = e^{-L/M} = P_o \qquad \text{eq 2}$$

where P_L is the probability that a discontinuity has a length equal or greater than L and M is the average length measured in the field assuming a representative set of lengths (6).

97. The probability of stability is $(1-P_f')$ for one discontinuity and $(1-P_f')^n$ for n discontinuities. Therefore:

$$R = (1 - P_f')^n \qquad \text{eq 3}$$

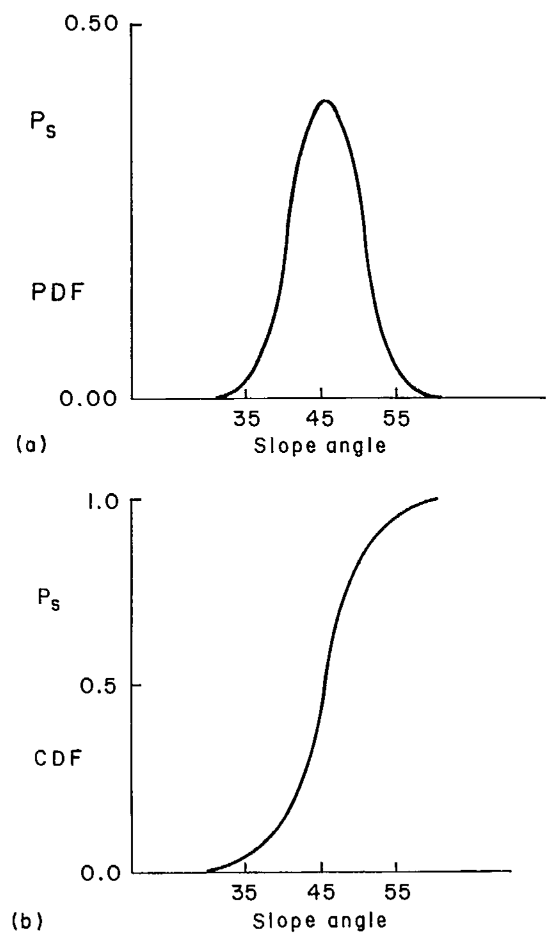


Fig. 24 - (a) The probability density function, PDF, describing the variation of the probability of sliding, P_s, with slope angle, i. (b) The cumulative distribution function, CDF, describing the variation of the probability of sliding, P_s, with slope angle, i.

where R is the reliability of the slope, (1-R) = P_f, the probability of instability for the full height of the slope increased by an increment in height of h (typically the increment for one year), and n is the number of discontinuities intersecting the face in the height increment h (6). (Note that no consideration is given to the spacing of the discontinuities parallel to their plane; techniques for including its effect are not yet well established.) n is calculated referring to Fig. 25 as follows:

$$n = V/S_V$$
$$V = h - h \tan \beta / \tan i$$
$$S_V = S/\cos \beta$$
$$n = h/S \,.\, \cos \beta \,(1 - \tan \beta/\tan i) \quad \text{eq 4}$$

98. A schedule of P_f versus H, the slope height, constitutes the input for the Benefit-Cost program. For example, if the mean length of the discontinuities is 16 ft (4.9 m) and the required length L = 500/sin 40 = 778 ft (237 m), $P_L = e^{-778/16} = 7.7 \times 10^{-22} = P_o$. Hence for one cell of a 45° slope in the above example, which would have $P_s = 0.5$, $P_f' = 0.5 \times 7.7 \times 10^{-22} = 3.85 \times 10^{-22}$. This means $(1-P_f')$ is approximately 1. If the increment in height to obtain the 500 ft (152 m) is h = 100 ft (30.5 m) and the average spacing is 6 ft (2 m), n = 100/6. cos 40(1-tan 40/tan45) = 2.05. Therefore, the reliability $R = (1 - P_f')^n = 1^{2.05} = 1$. From this result, it can be seen that the probability of full height instability of high slopes subject to plane shear sliding will tend to be low. And even with the costs of instability tending to vary as H^3, the greatest costs will usually arise from slides involving low heights because of their low reliability and large number of cells. Supplement 5-3 explains the effect of the breadth of the wall sector, which governs the number of cells that must be recognized for each wall element.

99. The reliability of a bench or berm face will be less than that of the full height of the ultimate wall of the pit. The probability of occurrence of critical geometry, P_o, for a bench height is much greater than for the full wall height. Also, the relative length of slope (ie length of slope parallel to the face divided by height in a given sector is much greater for the lower heights than for the full height, ie the

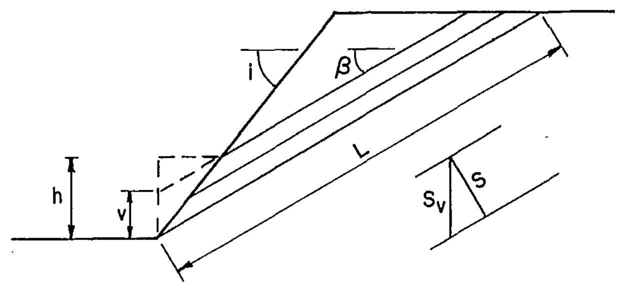


Fig. 25 - The number of discontinuities, dipping at β, that can be undercut will be determined by the vertical distance v. The increase in slope height in one year is h.

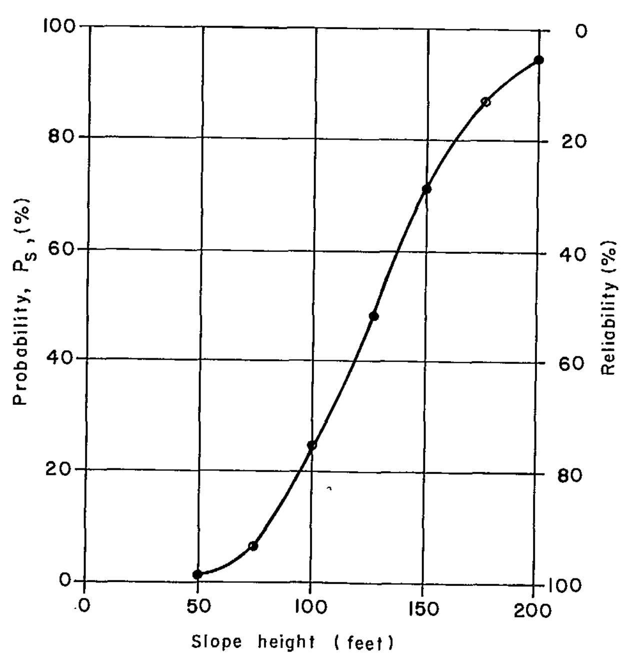


Fig. 26 - Variation of reliability, R, with slope height for a 3-d-wedge in a 65° slope. The probability of sliding, P_s, is equal to (1 -R).

number of cells (1 cell length = slope height) is inversely proportional to the height considered. Each cell is considered to be independent with respect to potential instability. The Benefit-Cost program includes calculating the cost of instability for each slope height of a design sector.

100. A series of such calculations would be done for selected intermediate heights. The resultant schedules of probability of instability versus height for each of two or three trial slope angles would provide input data for the Benefit-Cost optimization program.

101. Many plane shear slides occur, even though the probability of a continuous plane from toe to crest is low, because of the combination of discontinuities into a stepped-path as shown in Fig. 12(c). The distributions of lengths, dips and spacings of the sliding discontinuity and the cross joints are input into PLAFAM. From this information through Monte Carlo simulation a series of stepped-paths are generated. For those that are completed from toe to crest, the mean attitude of the sliding plane and its standard deviation are determined. The probability of sliding, P_s is then determined as for the conventional 2-d-wedge. From the number of paths that are completed, expressed as a ratio of the total number of simulations, the probability of occurrence, P_o, is calculated. Hence using eq 1, 3 and 4, the reliability of the slope against stepped-path slides can be calculated.

102. An example is given in Supplement 5-1. Two sets with dip angles of 40° and 80° for the sliding and cross joints have mean lengths respectively of 6 ft (1.8 m) and 3 ft (0.9 m) and mean spacings of 4 ft (1.2 m) and 3 ft (0.9 m). The program gives β for the sliding plane with a mean of 46.8° and standard deviation of 1.6°. For a slope of 300 ft (91.4 m) and 60° the probability of sliding, P_s, was computed to be 0.305 and the probability of occurrence was 1.000, hence P_f' = 0.305. For an increment of height, h, of 40 ft (12.2 m), the number of sliding joints intersecting the face from eq 4 would be 1.76. Thus from eq 3 for one cell the reliability, R, is 0.53.

103. For the two block mode of sliding, Supplement 5-1 includes a program, TWOBAM, which yields a probability of sliding, P_s. The reliability of the slope can then be determined following the procedure described for the 2-d-wedge.

104. Where it is anticipated that the potential mode of instability is by sliding on two different planes striking parallel to the slope, as shown in Fig. 12(d), or by the sliding block shearing into segments, as shown in Fig. 12(e), computer programs are provided in Supplement 5-1 for the determination of probabilities. Instability often produces slabs that topple, but this happens only because sliding occurs in the two-block mode, which could be imagined as occurring in Fig. 12(e) when the front block is pushed forward.

105. Where intersections of two or more discontinuities are undercut by the slope, a 3-d-wedge, as shown in Fig. 12(f), must be analysed. A program is provided for this case in Supplement 5-1 that includes the effects of groundwater seepage and tension cracks. Probability of sliding is determined by the Monte Carlo method.

106. Example. The means and standard deviations for two sets of joints are as follows:

	A-plane	B-plane
Dip (mean, standard deviation)	45°, 2°	70°,2°
Dip direction	105°, 2°	235°, 2°
Apparent angle of friction	20°, 2°	30°, 1.5°
Apparent cohesion	500, 150 psf (24, 7 kPa	100, 300 psf 48, 15 kPa)

The mean and standard deviation of the dip direction of the slope is 185°, 2°.

107. One thousand simulations using the program of Supplement 5-1 produced the curve of Fig. 26, showing the variation of probability of sliding, P_s, with slope height for a slope angle of 65°. For a height of 100 ft (30 m) there would be a reliability for one cell of 0.775 or a probability of sliding of 0.225. Each simulation was calculated with a different set of parameters

selected randomly according to their means and standard deviations. To obtain the probability of instability, P_f, using eq 1, it is then necessary to determine the probability of occurrence, P_o.

108. To determine P_o of the critical geometry for a 3-d-wedge raises problems that have not yet been resolved in the field of structural geology. An approximation can be used following the procedure for the 2-d-wedge. The joint probability of one discontinuity of each set having the required length is assumed to be the probability of occurrence of the critical geometry, ie $P_o = P_1 . P_2$ where $P_1 = P_L$ for set-1 and P_2 is P_L for set-2, which are determined from eq 2. The probability of instability for one wedge is $P_f' = P_o . P_s$, and the reliability or probability of stability, is $(1-P_f')$.

109. Following eq 3, the reliability of the slope must take into account the number of wedges, n, that can occur. Again following the procedure for the 2-d-wedge, the probability of stability of n wedges in $(1-P_f')^n$ (5). Hence the reliability of the slope is $(1-P_f')^n$ as expressed by eq 3. The simplest way of obtaining an approximation for n is to replace in Fig. 25 the angle β with a, the dip of the intersection of the two discontinuities on which the 3-d-wedge would slide, and replace the angle i with c, the plunge of the slope face in the direction of the intersection. Equation 3 then becomes:

$$n_1 = h/S_1 \,.\, \cos a\ (1 - \tan a/\tan c) \qquad \text{eq 4}$$

where n_1 is the number of discontinuities of set-1 that would intersect the vertical centreline of the cell in the height interval h. n_2 for set-2 is calculated in the same way. The number of potential wedges, n, then is equal to $n_1 n_2$.

110. In the example above for H = 200 ft (61 m), P_s = 0.96. The probability of occurrence, P_o, is determined using the average spacing of set-A, S_A = 8 ft (2.4 m), the average spacing of set-B, S_B = 12 ft (3.7 m), the average length of set-A, L_A = 10 ft (3.0 m), and the average length of set-B, L_B = 16 ft (4.9 m). The height increment, h, is 40 ft (12.2 m). From the Schmidt plot of Fig. 27, the dip of the wedge intersection, a = 31°, and the average plunge of the slope face in the direction of the intersection, c = 62°.

111. From eq 4:

$$n_A = 40/8.\ \cos 31\ (1\text{-}\tan 31/\tan 62) = 2.92$$
$$n_B = 40/12.\ \cos 31\ (1\text{-}\tan 31/\tan 62) = 1.94$$
$$n = 2.92 \times 1.94 = 5.66$$

From eq 2 for set-A dipping at 45°:

$$L_A = 200/\sin 45 = 282.8 \text{ ft}$$
$$P_L = e^{-282.8/10} = 5.23\text{x}10^{-13}$$

In set-B dipping at 70°:

$$L_B = 200/\sin 70 = 212.8 \text{ ft}$$
$$P_L = e^{-212.8/16} = 1.67\text{x}10^{-6}$$
$$P_o = 5.23\text{x}10^{-13}\text{x}1.67\text{x}10^{-6} = 8.73\text{x}10^{-19}$$

For one wedge the probability of instability, knowing the probability of sliding, P_s, is 0.96, using eq 1 is as follows:

$$P_f' = 8.73 \times 10^{-19} \times 0.96 = 8.38 \times 10^{-19}$$
$$\text{and } 1-P_f' = 1.00$$

hence using eq 3 the reliability of one cell of the slope with respect to slides for the full height is:

$$R = 1.00^{\,5.66} = 1.00$$

The influence of the required lengths, L, dominates the result.

112. Rotational sliding would be the appropriate mode to analyse where structural discontinuities do not dip into the pit in rock that is ductile, or yielding. This is common in soils and occurs in rocks where the rock properties approach those of soils, eg in highly altered or very soft formations.

113. The ground must be sufficiently plastic to yield without substantial loss of strength at locations of high stress. In this way the load is more evenly distributed along the potential sliding surface. Because of yielding, shear strength is maintained, even though some zones have been overstressed. Homogeneous slopes in such yielding materials typically fail with a segment describing rotational motion as indicated

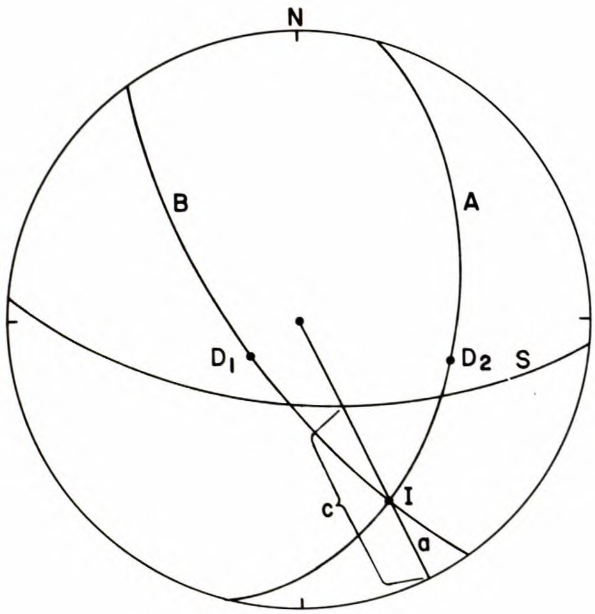


Fig. 27 - A Schmidt plot showing the projection of a slope plane, S. Two joint planes are also shown with dip angles, D_1 and D_2, that are not undercut by the slope. Their intersection, I, is undercut by the slope making possible a 3-d-wedge slide.

(a)

(b)

Fig. 28 - (a) A narrow rotational shear slide in altered rock. (b) A shallow segment rotational shear slide.

by Fig. 13. It is common to assume that the curved surface is circular and passes through the toe. Fig. 28(a) shows one such case in altered quartzite, which produced a shallow slide segment. Figure 28(b) shows another case that is curiously narrow. Figure 29 shows the development of a rotational slide in altered slate and quartzite, which was moving in spite of sub-zero temperatures.

114. To identify critical formations test results must show that the ductility ratio of the ground, ie the ratio of residual uniaxial compressive strength to the maximum strength, is greater than 0.6. This degree of ductility will normally only occur for materials with a uniaxial compressive strength less than about 75 psi (10,800 psf or 517 kPa).

115. Potential sliding partly on a weak plane and partly shearing through the rock substance, as shown in Fig. 13(b) and (c), can be analysed with little loss of accuracy by passing a circular arc close to the plane and through the toe. Alternatively, the surface through the toe can be assumed to be a plane, located by trial and error to find the critical attitude.

116. Supplement 5-2 contains an approximate solution for rotational sliding. It also contains computer programs for detailed analyses of both stability and deformation. The proximate solution makes use of an empirical equation which shows that the critical slope angle is a function of the independent variables: cohesion, rock mass unit

Fig. 29 - A large area involved in a rotational shear slide moving in spite of sub-zero temperatures.

weight, slope height, surcharge pressure on the ground at the slope crest (eg due to a waste dump), angle of internal friction, and maximum annual height of the water table behind the slope crest. These parameters are regarded as random variables within certain ranges, because they cannot be precisely determined. The variation of each is assumed to follow a normal distribution.

117. Example. A pit wall in highly altered rock is to have a height of 500 ft (152 m), variations in topography causing a standard deviation of the height of 20 ft (6 m). The average density of the rock mass, is 173 pcf (2.77 t/m³); the standard deviation is 9 pcf (0.14 t/m³). The average annual maximum height of groundwater above the ultimate toe of the wall, is 400 ft (122 m); the standard deviation is 50 ft (15 m). There will be no surcharge on the ground surface at the crest. Cohesion in the wall rock is 10 psi (69 kPa); the standard deviation is 3 psi (21 kPa). The angle of internal friction is 35°; the standard deviation is 5°.

118. From the proximate solution, the critical slope angle using the mean values of the above parameters is 34.0°. One cell of a slope 500 ft (153 m) high at 34° would have a 50% probability of instability, ie for half the cases strength would exceed stress and half would have strength less than stress.

119. The standard deviation of the critical slope angle is 5.4°. Clearly, the standard deviation of the friction angle, 5°, has the dominant influence in this example. The area under a normal bell-shaped distribution curve between the mean and one standard deviation is approximately 34% of the total area. Hence the area in one tail beyond one standard deviation is 16%. See Table A-3 in Appendix A: at a point one standard deviation from the mean, $z = 1$, P_f is 0.1586. Thus for a 16% probability of instability for one cell, the slope angle would have to be (34.0 - 5.4), say 28.5°. For $P_f = 0.10$ $z = 1.283$; hence $x = 34.0 - 1.283 \times 5.4 = 27.1$. Thus for a 10% probability the slope angle would have to be 27°. Figure 30 shows a probability curve drawn by a desk calculator/plotter using such an analysis (6).

120. Where structural discontinuities do not provide potential sliding surfaces in brittle rock, other forms of instability can occur. For example, some formations, such as shown in Fig. 31, produce abnormal amounts of loose rock, which limit the feasible range of slope angles. Although this is often an economic rather than a stability

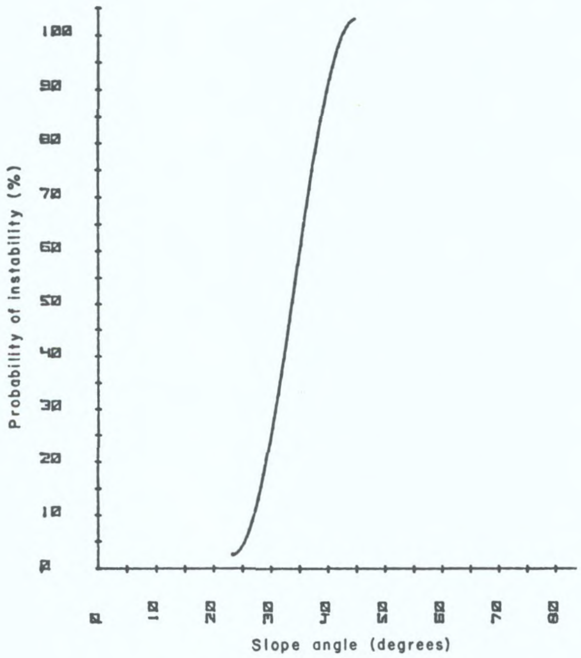


Fig. 30 - A probability of instability, CDF, vs slope angle curve drawn by a desk calculator/plotter using the approximate slide rule analysis (5).

problem, in extreme cases such mechanical properties of the rock govern the slope angle.

121. Block flow sliding may be analysed for high pit walls where neither plane nor rotational shear sliding is possible. Although lacking scientific substantiation, it is envisaged that stresses can develop that produce crushing in the solid rock (see Fig.33). Progressive breakdown of the rock could start in zones of either high stress or low strength. The stress concentration at the toe of a slope, as indicated in Fig 14(a), may be the point where such progressive action starts, which might then work its way through the slope ultimately causing a slide. Figure 32 shows a 51° slope 445 ft (136 m) high in somewhat brittle rock which probably broke up in this manner. Supplement 5-2 includes a finite element program that can be used to analyse stresses in these cases.

122. Brittle rocks, ie with ductility ratios less than 0.6, can produce rotational sliding. However, stress concentrations should be analyzed to compare with the strength of the rock substance. If strength is less than stress, local fracture will occur. An appraisal would then be required to determine if such a fracture would progress into a general breakdown of the slope. Such an appraisal might be qualitative, possibly based on previous experience with instability in the same formations, or it might be aided by computer modelling techniques. No standard procedure is hereby specified. The mine planner must use his judgement in deciding how far to go in his analysis.

123. Where weak planes exist in the slope but do not daylight, instability can occur as a result of crushing of the rock substance between the sliding plane and the toe, as shown in Fig. 14(b). Figure 33 shows such a case. In another mine, it is possible that the cracking beyond the toe shown in Fig. 34 and the resultant heave of some 4 ft (1.2 m), as shown in Fig. 35, were the result of some such mechanism. Special analyses are required in such cases.

124. Shearing or crushing of the rock at the toe of a slope can lead to the toppling of slabs

Fig. 31 - A 500 ft (153 m) high slope where the slope angle was limited to 45° because of the development of loose rock on the bench faces.

Fig. 32 - Slope instability caused by toe crushing. A slope 445 ft (136 m) high at 51° failed in a manner suggesting initiation was by toe shearing followed by a general breakdown of the rock mass.

Fig. 34 - Cracking 100 ft (30 m) beyond the toe of a 300 ft (91 m) high wall.

Fig. 33 - A minor slide caused by wedge geometry. The wedge, which was bounded by three planes that did not daylight into the slope, crushed at its bottom point.

Fig. 35 - Some 4 ft (1.2 m) of heave beyond the toe of a 300 ft (91 m) wall.

and blocks as illustrated by Fig. 14(c). Just the measured seasonal expansion and the partial contraction of the bedding apertures of the case shown in Fig. 36 could ultimately lead to crushing at the toe.

125. The stress concentrated at the toe of a slope, which might initiate these types of insta-

bility, is dependent on the premining state of stress in the formations. If high horizontal stresses exist, high toe stress will occur. The effects of such horizontal stresses has been demonstrated in the field. Figure 37 shows the results of such stresses during stripping operations. The trucks and shovels were lifted some 8 ft (2.4 m) by heaving and cracking of the floor.

126. Whereas constant slope angles are normally used in stability analyses, detailed design may include some variation in slope angle with depth. For example, if groundwater seepage is expected to influence the selection of the ultimate slope angle, steeper angles would be appropriate at levels above the groundwater table, as indicated in Fig. 38(a). Similarly, in the case where cohesion is effective steeper slope angles in the upper portions of the wall, as shown in Fig. 38(b), may be suitable. Separate analyses would be required for the steeper upper slopes and for the overall slope.

127. The modes of instability described above are models of reality, the actual geology and rock properties usually creating more complex situations. Furthermore, there have been cases of instability that have defied explanation. In these situations, it is normally assumed that if all the pertinent information could be determined the breakdown could be analysed and explained. However, for some of these cases it has been difficult even to imagine the explanation. Consequently, the mine planner must still keep his mind open to other possibilities and temper analyses with judgement.

Financial Optimization

128. At the feasibility stage, detailed financial analyses may not be required to determine optimum slope angles. A real impediment is the requirement to prepare at least two complete sets of annual mine plans for walls at different angles. However, at the mine design stage, the complete analysis described in Supplement 5-3 must be conducted. This approach makes it necessary to prepare annual mine plans and to analyse reliabilities.

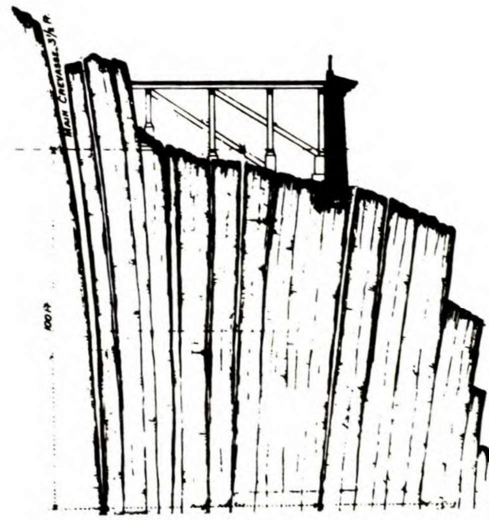


Fig. 36 - The separations of the vertical bedding planes in the shale in this terrace have been shown to be expanding each year and could in time produce toppling failures.

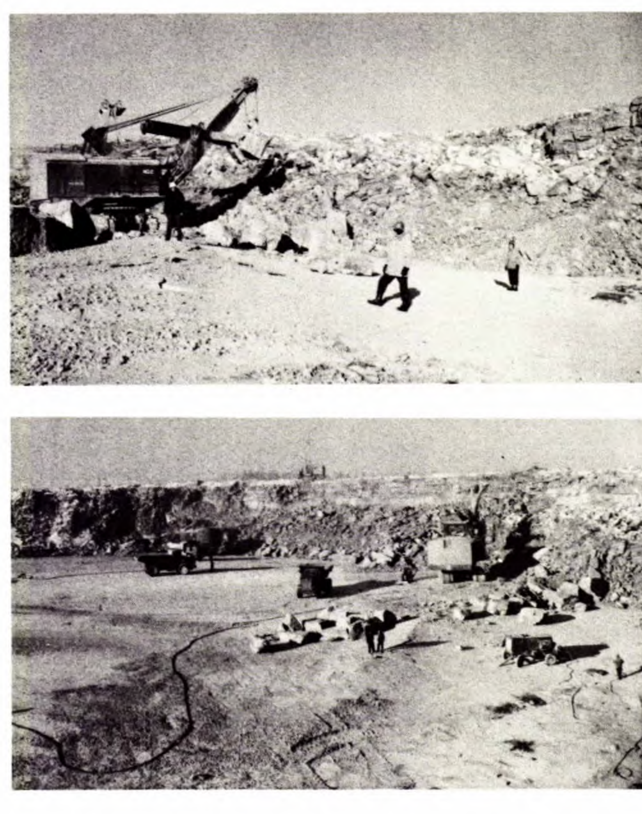


Fig. 37 - An upheaval of 8 ft (2.4 m) and cracking of the floor in an open pit during stripping indicated the presence of high horizontal stresses in the formation.

129. It is sometimes felt that probability calculations are of little value because of inaccuracies in the estimates of the input variables. Estimates of most likely future values and dispersion of metal prices, royalty rates and volume of reserves are often somewhat crude. Production rates and costs, plant costs, average grade and stability of walls are usually predictable within closer limits. However, it is when input data are uncertain that probability analyses are most useful. In designing pit slopes, the calculation of reliability for each trial slope angle is required for the complete mine risk analysis. Owing to the imperfect sampling of data, the accuracy of the probability estimates of stability and prospective rate of return are usually less precise than desired. Instead of assuming certainty in the expected values of the variables, the risk approach treats the measure of dispersion as certain - an improvement but not a perfect representation of reality.

130. Figure 39 shows the cumulative distribution functions for rate of return that are likely to occur at the various mine design stages (7). The decrease in the variation of the predicted rates of return reflects increased knowledge. The volume of the orebody, its grade, production costs, mill recovery, wall stability and the like become more predictable as more investigations are conducted for the mine design stage and as experience is obtained in the operating stage. The main argument for conducting such calculations, even in the relative ignorance of the feasibility stage, is that it is imprudent to ignore pertinent information when making decisions at any stage. Calculated values, as in all engineering, never provide the final answer, the exercise of good judgement is essential.

131. A cut and try procedure is followed for selecting the slope angle for each sector. Two or more feasible slope angles are selected for the various wall elements. Each is subjected to appropriate stability analyses. A schedule of reliability versus slope height is compiled for each angle, see Fig. 3, which becomes input data for the optimization analysis. If more than one mode

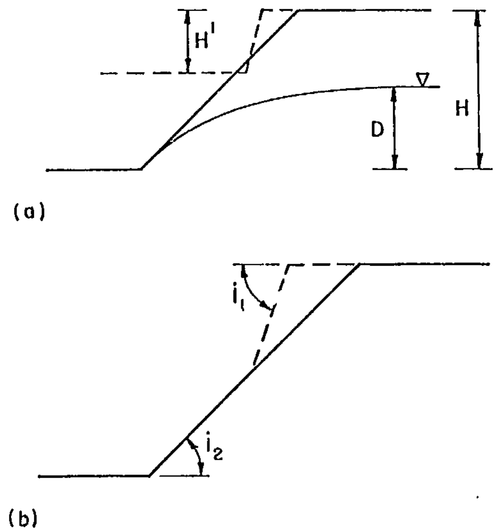


Fig. 38 - Steeper than average wall angles are sometimes reasonable near the crest. (a) For depths H' stability is not affected by groundwater and hence a steeper angle might be feasible. (b) Where cohesion is acting, steeper slope angles, i_1, should be stable for heights less than ultimate.

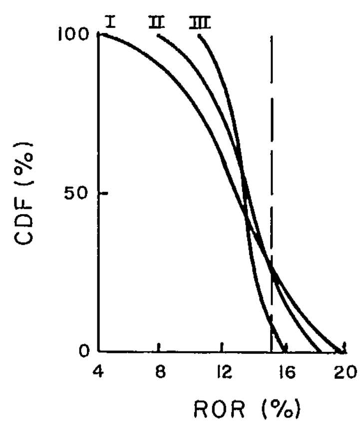


Fig. 39 - Typical cumulative distribution functions, CDF, for prospective rates of return, ROR, on a mine investment; the dispersion of the estimates decreases from I, the feasibility stage, to II, the mine design stage, to III, the operating stage, reflecting the improvement in the quality of the estimates of the components of the calculation (the y-axis indicates the estimated probability that the ROR will be more than the x-value) (10).

of failure is possible, then the effects must be added statistically, which is done by the Monte Carlo sampling procedure.

132. In the Benefit-Cost program, the mining of the projected pit is simulated, proceeding year by year to the interim or ultimate pit bottom for one set of slope angles (see Fig. 40). Using the Monte Carlo method, each year each sector is examined to determine whether a slide occurs or not. The overall slope height for the year is tested first for instability by whatever modes are considered possible. Then any segment of the wall that has either a weaker section of rock or a steeper slope angle (such as an interramp slope, a working face, or above and below a pushback) is also sampled for instability for the year. If instability occurs, the cost is determined and added to the cash outflow. Mining to the pit bottom is simulated 30 to 100 times, to determine the average financial results and their variation. In this way recognition, or appropriate weighing, is given to the fact that maximum wall heights only occur for a few years and only at a time in the distant future.

133. The length of wall parallel to the face must be included in the appraisal of instability, ie there will be more probability of instability in a long wall than in a short one. It is assumed that the breadth of a slide (parallel to the face) is on the average equal to the height of the slope. The length of the sector is therefore divided by the height of slope being examined to give the number of cells or separate slides that could occur. This number then dictates the number of times per year the slope is tested in the Monte Carlo procedure.

134. The main objective in the Benefit-Cost analysis is to assess the incremental benefits and costs of steepening either overall, interramp, working or berm slopes. One design will be a base case against which designs with steeper slopes are to be compared. The optimum slope angle is approximately the angle that produces the greatest surplus of benefits over costs. The actual selection of the optimum is made by examining the most likely rate of return together with the

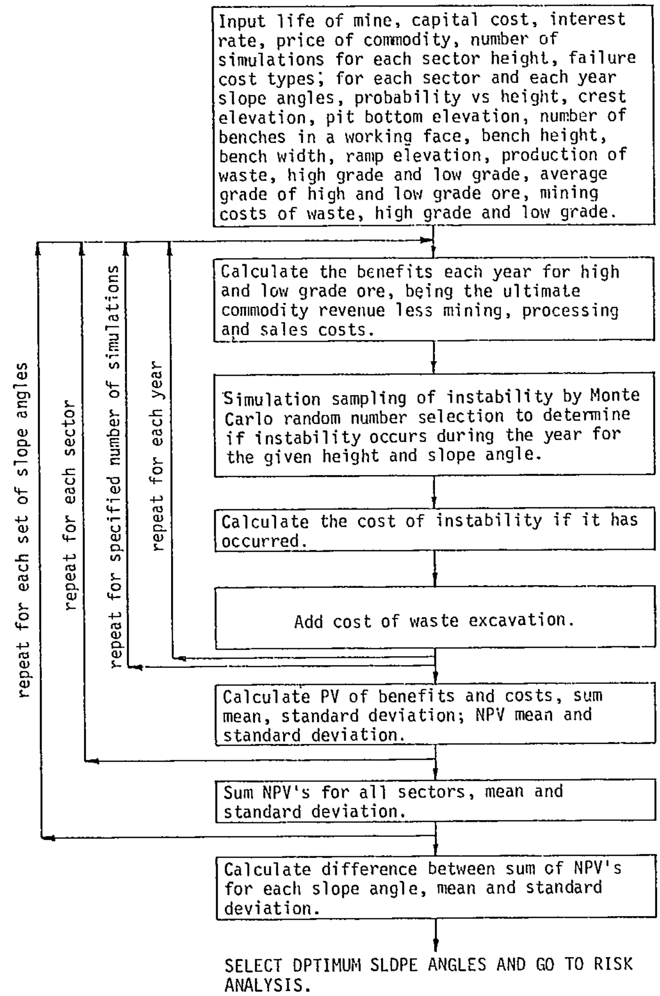


Fig. 40 - Activity flow chart for the benefit-cost program (see Supplement 5-3 for details).

probabilities of lower and greater returns - all of which influence the decision. In other words, the sensitivity of the financial effects to changes in slope angle is important.

135. The difficulty in assessing the incremental benefits and costs associated with steeper slopes lies in quantifying all the benefits and costs. The benefits of steepening a wall may arise not only from increased recovery of ore but also from reduced capital requirements for equipment. The task of quantifying the costs of instability requires prediction of not only the slide probabilities throughout the life of the mine but also the operational impacts of each slide. It should be noted that the selected slope angles are substantially based on economic optimization. There is no explicit use of a concept of 'minimum reliability' because all slopes are designed to be safe - if necessary through monitoring, scaling, etc - hence such considerations only have a place in exercising final judgement on the validity of the calculated results.

136. Instability will normally give rise to the extra costs of either clean-up, unloading, postponed production, ramp relocation or surface plant removal. Each design sector must be examined to determine the appropriate selection of such costs. Generalized methods for estimating some of these costs are based on the concepts shown in Fig.41, 42, 43 and 44. The assumptions contained in these figures are somewhat arbitrary although studies indicate that they provide reasonable estimates. For example, the backbreak at the crest is assumed to be equal to 0.2H, where H is the height of the wall at the time of the slide. Experience shows that this factor can be more or less than 0.2H. A mine planner can select a factor according to his own judgement. It could also be treated statistically.

137. In Fig. 45(a) the cracks in the crest of the wall are out to 100 ft (30 m) for a height of sliding of 500 ft (152 m) in a wall 1000 ft (305 m) high, part of which is shown in Fig.45(b). Other cases have shown cracks out to 60 ft (18 m) for H = 300 ft (91 m), ie 0.2H and out to 50 ft (16 m) for H = 175 ft (53 m), ie 0.29H.

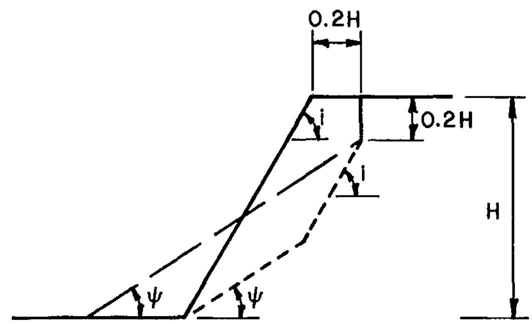


Fig. 41 - Estimating the cost of cleaning up a slide. ψ is the angle of repose of the broken rock. The volume to be excavated lies between the dotted and dashed lines.

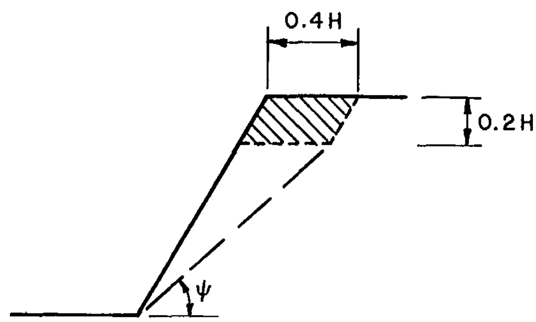


Fig. 42 - Estimating the cost of unloading a wall to eliminate instability. The volume is represented by the hatched area.

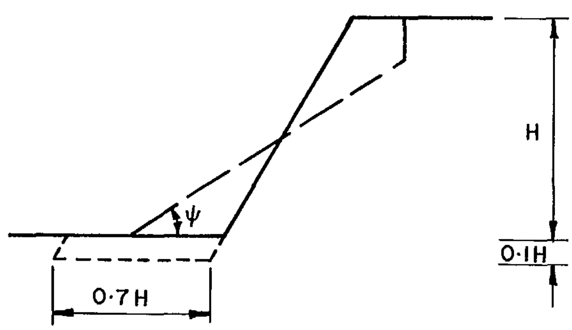


Fig. 43 - Estimating the amount of ore to be lost from planned production by not cleaning up. The volume is within the dashed lines.

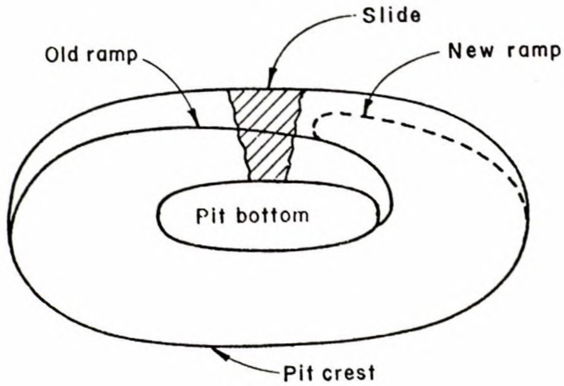
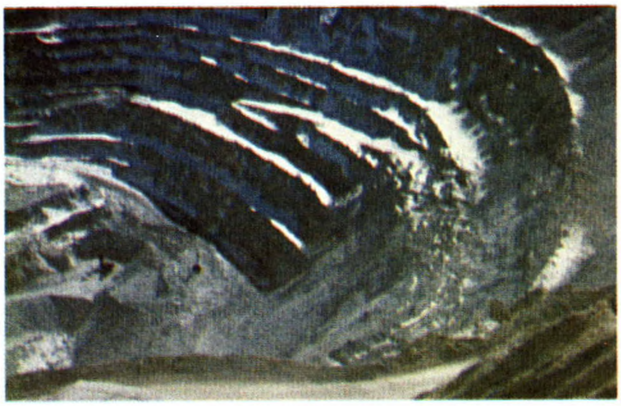


Fig. 44 - Estimating part of the cost of instability when a ramp is undermined by a slide. Extra excavation is required to cut a new ramp. The other part of the cost may arise from extra haulage distance.

(a)



(b)

Fig. 45 - (a) Crest cracks associated with a large volume of moving rock in a wall more than 1000 ft (305 m) high. (b) Part of the wall under the crest.

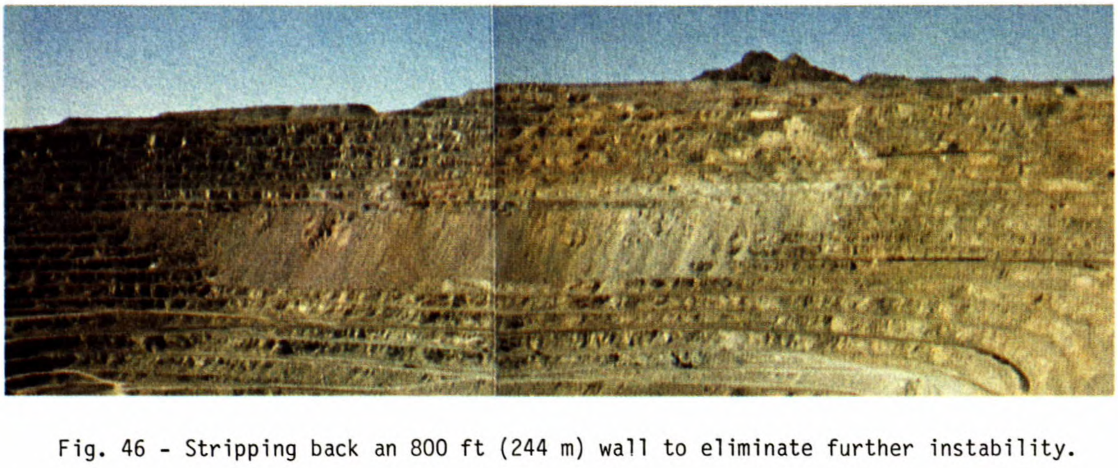


Fig. 46 - Stripping back an 800 ft (244 m) wall to eliminate further instability.

138. Similarly, the breadth of the slide (parallel to the face) can be assumed to be 0.5 of the height of the wall. Figure 46 shows a wall where instability had occurred. The decision was made to mine out the broken material and to decrease the probability of further instability. Originally the interramp slope angle was 45° for a height of 800 ft (244 m). Instability, as a result of the above decision, affected a breadth of some 2000 ft (610 m) of wall, ie 2.5 times the height. The backbreak at the crest was 0.275H. The new wall angle was 37°. This case demonstrated that the standard ratios could be exceeded.

139. If a slide occurs, it may be appropriate, to clean up the debris immediately and resume mining as planned. In one extreme case, instability affecting 750 ft (230 m) of height of wall gave rise to the decision to excavate the slide plus additional stripping for the entire wall to ensure that highgrade ore near the pit bottom would not be jeopardized in the future. The increased stripping was more than 40 million tons.

140. Alternatively, for instability that has not developed into a slide but is clearly working, the decision can be to stabilize the slope by unloading the crest and then to resume mining. In one pit where unloading was necessary, it was calculated by the equation developed for Supplement 5-3 that the cost would be $360,000 compared to the actual cost of $400,000. However, in this case ore was also lost at the toe. In another pit where a plane shear slide occurred on two intersecting faults, it was decided to unload the entire wall, which amounted to 4 million tons of extra stripping compared to an estimated amount using Fig. 39 of 1.5 million tons. (Actually, the remedial action of unloading is often of questionable benefit; its effect on stability should be analysed before making such a decision.)

141. Further alternatives are either to postpone production at the toe of the section until the next push-back occurs or to write off the ore if an ultimate wall is being mined. An example of postponed mining involved a large slide in a pit affecting the entire wall which was 1000 ft (305 m) high. Owing to the constraints at the pit bottom, it became impossible to mine as planned. The difference in the present value of the profit on the ore was calculated to be about $3 million for a ten-year postponement, which is very close to what would have been calculated using the approach of Fig. 40. Another case of instability resulted in the postponement of ore production from the toe of one wall for seven years. Ore production was maintained by mining somewhat lower grade ore in another area where stripping had been largely completed. The volume of ore affected probably exceeded 50 million tons, which is much greater than a calculated prediction using Fig. 40 of 3 million tons for a wall height of 800 ft (244 m). In another case, a large slide caused the abandonment of 2 million tons of ore at the toe of an 850 ft (260 m) high wall where a loss of 3.4 million tons would have been predicted.

142. Volumes of predicted slides can be calculated from the data of the stability analysis. Plane and rotational shear modes involve definite slide segments. Also, the slide scar from rotational shear may be at an angle that will produce further slides. The scar can be reanalysed, and the probabilities and succeeding volumes summed. A volume-probability curve can be developed for a given slope angle by examining surfaces for both rotational shear and plane shear other than the most critical. In two-dimensional analyses an assumption is required for the breadth of slide, which can be as above 0.5 H. Slide volumes for 3-d-wedges are available from the computer programs used for the stability analysis.

143. Special situations must also be recognized. Walls carrying a ramp system that become unstable, such as shown in Fig. 47, cause extra transportation costs arising from increased haulage distances. In one case where the spiral haulage was intersected and switchbacks had to be constructed, the increased in-pit length of transportation added to the operating costs some $200,000 pa for seven years. The capital cost of the switchbacks was small by comparison. The predicted discounted cost of instability using a standard model based on Fig. 43 would have been $3.5 million

compared to an actual discounted total cost of $800,000.

144. Where surface plant would have to be relocated in the event of instability, the costs of such actions must be estimated for each particular situation. Figure 48 shows a crusher ending up close to the crest of a wall (as a result of mining lower grade ore than originally planned) and being threatened with undermining of its foundations by slope instability. The cost of moving the mill would be one of the costs of the instability.

145. Besides using one or more of the general procedures for calculating costs the designer can specify explicitly, by detailed examination of the mining operations, the potential effects of pit wall instability. In addition, recognition can be given to the expectation that slides in steep slopes are likely to be more costly than slides in slopes at angles only slightly greater than the angle of repose of the broken rock. Such predictions are not easy, but the use of best judgement is far superior to ignoring the problem. Using these inputs, the benefit-cost program selects the most appropriate cost prediction and then computes the cost for each slide that occurs in the simulation.

146. The benefits of steeper ultimate slopes are either the reduced excavation of waste material for the same ore reserve or the increased ore reserve for the same amount of waste excavation. On the other hand, the benefit of steeper working slopes lies in the delay of excavation of waste.

147. Mining costs in the computer programs are related to the operation. Those for ore are different from those for waste mining. Also, different production costs may be specified for each sector. As the pit gets deeper, certain changes in unit mining costs can be anticipated due to the increased haulage distance. Labor costs and technological advances may change future costs. Therefore, different costs can be input annually.

148. The output from one computer run (ie 30-100 simulations) consists of the annual mean

Fig. 47 - A main ramp broken up by wall instability. The haulage system had to be redesigned to route the ore at extra cost up the opposite wall.

Fig. 48 - A crusher uncomfortably close to the edge of the pit, after many years of expanded mining, where it could be affected by wall instability.

and standard deviation of the benefits and costs for each sector of the mine. The present values of total benefits and costs for the entire pit for each simulation is computed, together with the mean and standard deviation of the net present value. Supplement 5-3 provides a detailed explanation.

149. The incremental benefits and costs of the alternate layouts are used in the risk analysis program. A flow chart is shown in Fig. 49. The impact of steeper pit slope angles on the rate of return, and other financial criteria, is determined. In examining each slope angle, other stochastic or statistical variables such as future prices are treated as single or deterministic parameters so that the effect of slope angles alone can be seen.

150. Finally, taking into account the important intangible factors that have not been included in the calculations, the wall angles are selected by harmonizing the results obtained for each sector. Smoothing of the pit contours may require modification of optimum slope angles to provide a practical layout. A report should then be written on all the studies incorporated in the pit wall design.

Associated Designs

151. The associated design requirements for monitoring systems, perimeter blasting, support and trial slopes normally do not arise until the operating stage. On the other hand, at the mine design stage, monitoring systems might have to be designed for steep walls. Similarly, plans may be required for measuring blasting vibrations owing to the proximity of surface plant or tunnels. Special situations might be anticipated requiring support analyses at the mine design stage. Also at this stage, trial slopes might be envisaged as being useful.

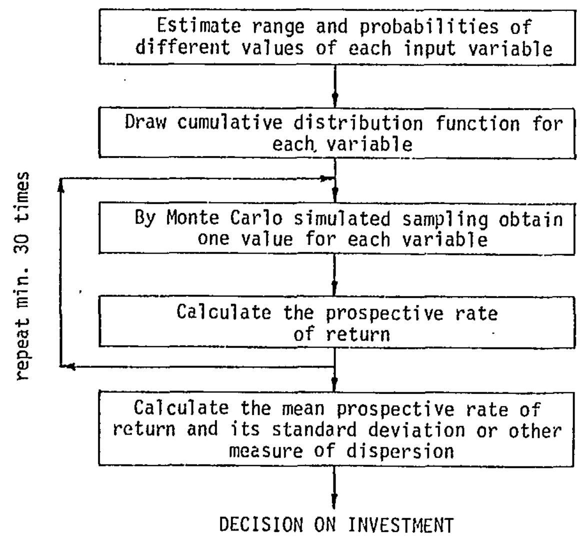


Fig. 49 - Activity flow chart for the integrated risk analysis (see Supplement 5-3 for details).

OPERATING STAGE

Field Investigations

152. During operations a large amount of information can and must be gathered on structural geology, groundwater and mechanical properties of the wall formations. Additional exploration boreholes may have been drilled; these are an important source of supplementary information. Exposure of the wallrock by benching provides the opportunity to obtain more and better structural information at relatively low cost.

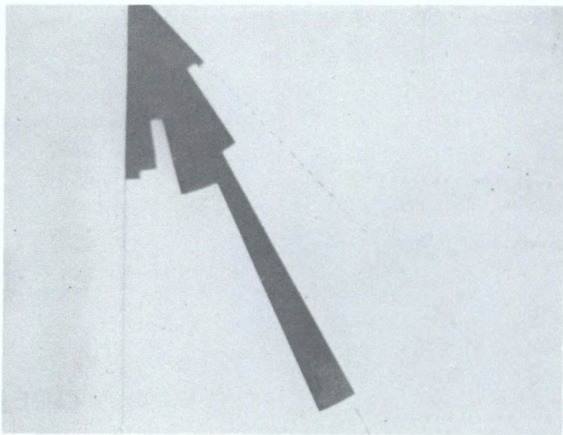
153. As described in the chapter on structural geology, appropriate bench mapping must be conducted at this time. Often bench faces are only available for a short period of time as a result of either deterioration of the faces or subsequent advances in a fast expanding pit. Improved quality of structural information provides a basis for either confirming or modifying the previous design. In addition, bench mapping is essential to detect the inevitable variations from the average conditions assumed in the design so that unusual behaviour can be anticipated and controlled.

154. Fig 50 shows part of a 45° 500 ft (153 m) wall that was found, as a result of structural investigations during operations, to be controlled by the structural fabric. A dip rosette showed that the majority of structural features dipped at angles greater than 59°. Accordingly, the wall was successfully mined back to 59° as shown in the figure.

155. Careful judgement is required to determine how much information is worth gathering. In addition, having incurred the cost of data gathering it is essential that it be stored in a manner that makes retrieval and analysis easy. For this reason, a system has been developed, called DISCODAT, that can be used for this purpose. The system includes computer programs for storage, retrieval and analysis of the geologic data and is described in the chapter on structural geology.

156. Groundwater observations must be gathered. Location of face seepage, such as shown in Fig 51 and approximate quantity of flow should be noted. Data from piezometers installed at the exploration and mine design stage will have a longer period of record and thereby will be of much greater value for redesign of the mine. Such information can usually be supplemented by observing and recording location of wet blastholes and depth below collar

(a)



(b)

(c)

Fig. 50 - (a) A 45° 500 ft (153 m) wall in a pit where structure controls stability. (b) The dip rosette shows the majority of the planes dipping more than 59°. (c) The wall after it had been mined back to 59°.

of water levels. At this stage, data is available to provide a good estimate of the average annual maximum groundwater elevation around the mine. Also, local variations in the permeability of the walls resulting from different geological structure might be evident. Principal directions of permeability should be apparent permitting improved design of drainage systems.

157. Block and large size core samples, such as shown in Fig. 52, become available for laboratory testing of strength properties of faults, joints and other discontinuities. The opportunity to examine in detail any slides, however minor, that have occurred would be of great value to improve knowledge of the strength characteristics of slide surfaces. Appendix C deals with this subject. Similarily, if a monitoring system has been installed at the start of mining, deformation measurements would be available that could be related to the excavated geometry of the walls and possibly to the passage of time. This information could then be analyzed to deduce the effective deformation properties of the rock mass comprising a wall. These properties provide a good basis for predicting further deformation as walls become deeper, which in turn provides a basis for judging when abnormal deformation foreshadowing instability starts to occur.

158. The exposure of the benches and the drilling of blastholes make it possible to use geophysical measurements to obtain information on the variation of the mechanical properties of the rock mass. Refraction seismic measurements can be useful for this purpose. Inter-hole or hole-to-surface direct velocity measurements might be appropriate. In both cases, useful information can be obtained for anticipating local instability and for providing guidance on ground variations with respect to production and perimeter blasting. Detailed procedures are provided in the chapter on structural geology.

159. Any new exploration holes drilled during the operating stage must be fully exploited for the information that can be obtained for stability considerations. In addition, bench mapping may show a need for structural drilling to delineate

Fig. 51 - Seepage on a pit face can be observed from time to time, providing useful information on groundwater flow.

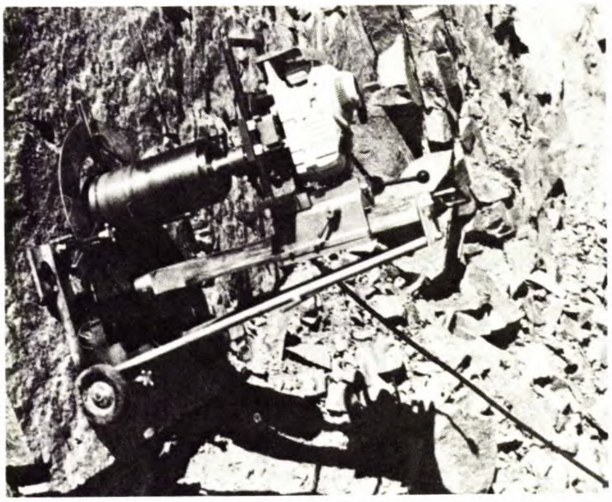


Fig. 52 - A portable sampling drill at the slope face for obtaining large size core for direct shear testing (up to 12 in. (0.3 m) in diameter).

or explore serious anomalies that have been detected.

160. The drilling of blastholes provides a challenge and possibly an opportunity to gather a large amount of strength information on both the ore and the wall rocks. By measuring penetration rate and hold-down pressure, a rock quality index might be obtained that can be used as an indication of relative rock mass strength. However, case studies have shown that there are practical difficulties to overcome.

Previous Slopes

161. Examination of previous slopes at all design stages can be an important source of information for stability analyses. At the operating stage, the slopes that have already been excavated in the wall formations can provide data. These slopes may be in rock of either the ultimate or interim walls. Any slides that have occurred, even on bench faces, provide the opportunity for analysis of strength properties as described in Appendix C. For example, Fig. 53 shows a 3-d-wedge slide that could be used to obtain information on the frictional properties of the structural features bounding the wedge.

162. Stable slopes at relatively steep angles also provide the opportunity for a statistical analysis to confirm or to modify probability calculations made at the mine design stage. Such procedures are described in Appendix C on the utilization of information from previous slopes. Figure 54(a) shows an initial compilation of experience with previous slopes in one mining area (8). Figure 54(b) shows the selection for one sector, ie from all the south slopes, of four of the points representing extremes of slope angle and height plotted as described in Appendix C. Such plots together with normal investigations and analyses can provide the bases for selection of feasible slope angles.

163. If it had been decided at the mine design stage to cut a trial slope during operations, the results of this trial would be available during the operating stage for either the redesign of the walls or the re-evaluation of the original design.

Fig. 53 - A 3-d-wedge slide that could be used to obtain information on the frictional properties of the joints.

Such a trial slope would have been planned to test either a specific structural or lithological condition or to obtain stability data for an extensive length of slope that could be analyzed statistically.

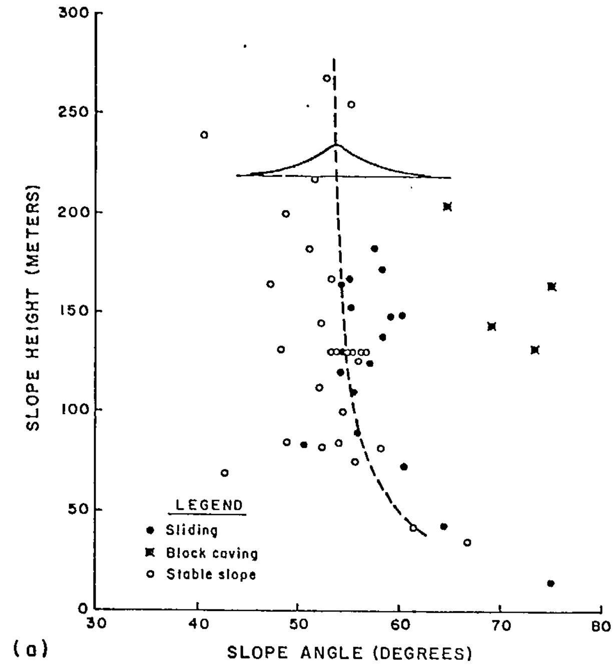
Stability Analyses and Financial Optimization

164. The design sectors of the walls may require redefinition. Improved structural information may indicate substantial differences from the picture obtained during the mine design stage. New boundaries may define the orebody. Changes in surface plant location may have occurred, and many other reasons can lead to the need for a new layout.

165. Instability can develop that does not immediately produce a slide. As a result of detailed investigations and analyses, a diagnosis of the cause can be made. Then the appropriate remedial steps can be considered. A common action is to unload the crest; the probable cost and financial impact are treated in Supplement 5-3. An alternate step, if groundwater has been shown to be influential, is to install a dewatering system, which is dealt with in the chapter on Groundwater. Mechanical support might be feasible, technical details and cost data being presented in the chapter on this subject. In some cases, perimeter blasting to minimize the shock hitting the rock might be helpful. This would require an examination of costs; guidance is provided in the chapter on perimeter blasting.

166. The appropriate stability analyses and financial optimization for redesign purposes are essentially as described above for the mine design stage. The only difference is that the input data for these analyses will be of a higher quality. Much better information on structural conditions, groundwater and mechanical properties of wallrock will be available. Potential instability modes will be more evident as a result of experience with actual slopes in the wall rock. Similarly, estimation of costs resulting from instability would be more accurate.

167. The Benefit-Cost program simulates mining each year considering instability as the only



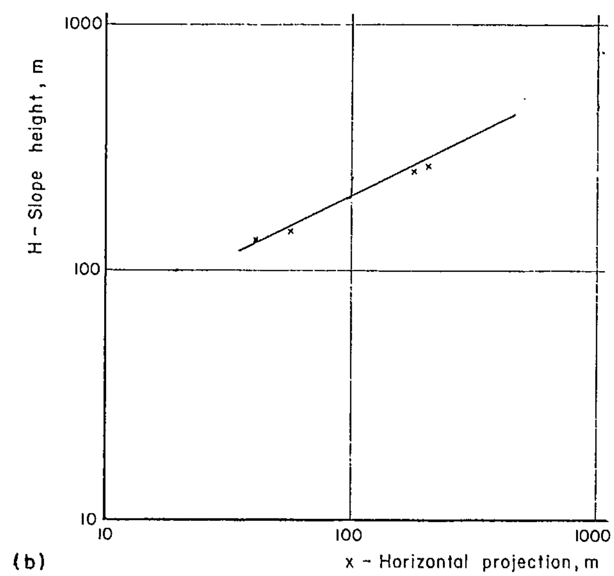


Fig. 54 - (a) A compilation of stable and unstable slopes showing the dispersion about some mean curve of critical slope angles with respect to slope height (7). (b) From the slopes dipping to the north a selection is made of those representing extremes of height and slope angle plotted on a log-log graph to provide an envelope curve for comparative purposes (Appendix C provides more details).

random variable, ie all other parameters are considered fixed. Whether instability occurs or not is determined by the Monte Carlo procedure. In each simulation, benefit is defined as the value of production net of the cost of mining, which is calculated by using the price of the commodity and the recovery rate. Cost is defined as the expense of excavating the waste plus that resulting from any instability. Yearly averages, resulting from a number of simulations, together with their standard deviations are calculated. The present values of the average yearly benefits and costs are then calculated. A second and possibly a third overall slope angle would be simulated in the same way providing benefit-cost comparisons and perhaps indicating an optimum angle. Among the feasible angles, the selection would be made on the basis of the output from the Risk Analysis Program, on the financial risks involved for each angle.

168. An example of slope design for one part of a copper pit for a pushback is given in the Supplement 5-3. The Benefit-Cost analysis for Sector No. 1 in this pit proceeded in the following manner. The mining period for the pushback was 10 years. The internal cost of money was assumed to be 15%. The recovery of metal was assumed to be 86% and the price of copper to be \$0.64/lb. The approximate average grade of ore was input for each year, varying from 0.44 to 0.52. Ultimate top and bottom elevations for the sector were input, giving a maximum height of 1500 ft. Yearly crest and pit bottom elevations were input. Bench geometry was defined: height 50 ft, width 20 ft and face angle 59°. Ramps were specified as 130 ft wide; four were envisaged as being required in this sector, their yearly elevations being input. The first trial was for an overall slope angle of 45°, the program calculating interramp and working face slope angles. Probability of instability schedules were determined for heights from 200 ft to 1800 ft, interpolation being used for intermediate values occurring in the wall. The yearly quantities of ore and waste excavation were input, together with their mining costs. The discounted results gave \$198.0 million for the benefits and \$108.8 million for the costs with a standard deviation of \$1.3 M. A second slope of 38° was analysed, and the benefit-cost results examined. In this case, the lower slope angle was found to be more economic.

Associated Designs

169. Perimeter Blasting. The increase in size of open pit operations in recent years has resulted in higher bench heights, larger diameter blastholes, more powerful explosives and frequent use of contractors for stripping. All these measures make backbreak and scaling problems more severe. Where slopes are designed to be relatively steep, the condition of the faces can be important.

170. If back-break and subgrade fracturing are not controlled, they will decrease the width of the catch benches and necessitate a decrease in the overall slope angle. In turn this may result in decreased recoverable ore reserves resulting from the increased waste/ore ratio. Remedial measures, such as scaling, perimeter blasting and use of mechanical support, are expensive.

171. An economic optimum may exist where some of the money saved by using larger blasts is spent to maintain pit wall quality. The effects of blasting can be controlled to some extent by modifications at the interim or ultimate wall. In the chapter on perimeter blasting various techniques are described, how they work, the kind of results they give and their costs. The procedures, tests and technical details to develop wall control procedures at each stage of operations are given.

172. There are four approaches to perimeter blasting that might be feasible in any particular case:

a. pre-splitting,
b. cushion blasting,
c. buffer blasting and
d. line drilling.

All of these techniques are designed to create a low concentration of explosive energy at the perimeter of the pit.

173. The properties of the rock being blasted

influence the success of perimeter blasting. The most important properties are:

a. the strength of the rock substance and

b. the nature, frequency and orientation of structural features.

These properties must be determined. The perimeter blasting pattern is designed using controllable variables such as spacing, burden, hole diameter, and explosive type. Perimeter blasting can usually be incorporated into the mining cycle without undue disruption to operations. The costs and benefits must, however, be evaluated to determine feasibility.

174. For the prediction of costs, it may be appropriate to set up a series of field tests using holes at various spacings and, if feasible, of various diameters. These trial patterns can be oriented at different angles to the major structural features. The results may be used to design full-scale blasting patterns.

175. At the operating stage, the location of faults and weak ground, and the behaviour of the various formations are evident. Experience should have established: (a) the best of the four types of perimeter blasting to be used, (b) the effects of orientation of the structural fabric with respect to the face, (c) the importance of variations in explosives and (d) the relation between backbreak distance and charge size.

176. Figure 55 shows the results of perimeter blasting in an iron ore pit where plans were being made to mine to an overall slope angle of 60° on a wall that would not include a ramp.

177. Monitoring Systems. The design of the walls might lead to the need for monitoring. Routine visual examination by appropriate personnel is important and must be the first element of a program. However, the gathering of quantitative data is also valuable.

178. The justification for any type of monitoring system is that it will provide information which can have a bearing upon the stability of the slope. Variations in pore water pressure or changes in intensity of micro-seismic events indicate changing conditions, although they do not necessarily indicate the onset of instability.

Fig. 55 - Two faces, 60 ft (18 m) and 120 ft (36 m), created by preshearing in an iron ore pit.

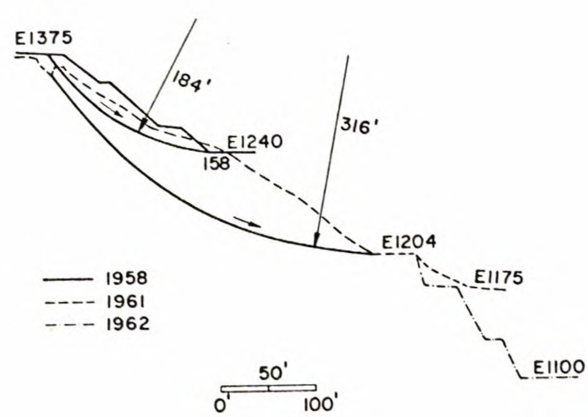


Fig. 56 - A section of a wall that started to slide when only 85 ft (26 m) high and continued to develop new slides until the pit bottom was reached at a depth of 275 ft (84 m).

Excessive displacement, on the other hand, is instability, and its measurement gives data indicating the extent and degree of the problem.

179. A system for monitoring the stability of slopes may consist of three units: (1) the sensors measuring deformation or some other effect, (2) a data transmission system, and (3) a data storage and processing unit. A fully integrated system would collect information, transmit this information to a convenient point and present it in an immediately intelligible form (such as graph of movement vs time).

180. Surveying techniques are the simplest to use for measuring displacements, either relative or absolute. However, their cost may be greater than for semi-automated systems, particularly where frequent readings are required. For precise location or checking of primary and secondary monuments surveying must be used.

181. Movements around a pit have a variety of patterns. If no significant displacements occur in the vicinity of a slope, it is, by definition, stable. Major movements are always preceded by minor ones, although the occurrence of minor displacements does not necessarily mean the onset of serious instability. In some cases, months and years can elapse between the commencement of the displacement and ultimate sliding. In others, it may be only a matter of days, or it may never happen.

182. During the operating stage, some sections of the walls may display signs of instability giving rise to a distinct requirement for monitoring. New information on critical structural features may indicate wall sections that should be watched. A section where a slide has occurred often is worth instrumenting because the first slide may represent an inherent weakness at this section that will lead to subsequent slides. The section in Fig. 56 shows such a series of slides. Mining excessively close to surface plant may be required by new ore boundaries, such as will be the case in Fig. 57, making it important to monitor the plant structures.

183. Where measurements have been taken over a period of time and movement has occurred, the

Fig. 57 - In a copper pit, after the milling rate had been increased, the ultimate pit was expanded so that the smelter is quite near the ultimate crest, which will create a requirement for monitoring.

Fig. 58 - A rock slope adjacent to a dam where some 2000 solid steel anchors and 1000 cable anchors were used to insure stability.

question frequently arises on the magnitude of movement that indicates developing instability. Three sources of guidance are available. A finite element analysis using the deformation properties of the rock mass will provide information on the deformations that are not serious - greater movement being suspect. Empirical observations on unstable slopes provide information on the types of movement that can be related to instability. Thirdly, supplementary measurements, such as microseismic, pore pressures and internal strains, might clarify what is happening. As a practical measure, the development of cracks at the crest should be mapped and their aperture monitored.

184. Mechanical Support. Support such as rock anchors and mesh as shown in Fig. 58, can be used to provide additional stability. Applications can be for overall slope or localized face stabilization. This approach is only feasible in special situations. For example, particularly high-grade ore under a high wall could make a support system economic. Where there is limited surface area for plant, support may be required to guarantee stability of a slope under such a structure.

185. Support methods for rock slopes include deep rock anchors, shallow rock bolting, shotcrete or other surface coatings, retaining walls and buttresses. Rock bolting can be used for bench stabilization and for the prevention of minor rockfalls. For overall support of large slopes, the only practical method is the use of deep anchors; these constitute a major engineering system, which requires detailed design and continuous inspection during installation.

186. Coatings applied to the rock face such as gunite, shotcrete and epoxy resins may be practicable to preserve the rock face and prevent spalling. Such coatings can be used to seal the rock surface preventing weathering, unravelling or erosion of the rock face. It may be advisable to include conventional reinforcement or wire fibres to counter effects of inadequate drainage and frost action. Drains through the coating are advisable to diminish these effects.

187. Concrete buttresses can be used to support specific overhanging major rock blocks, and retaining walls can be used to support sliding loose rock.

188. At the operating stage, a need for support, might have become quite evident. The frequency of slides might be turning out to be greater than predicted. The ore boundaries might be found to extend under surface plant. Loose rock on the faces might be excessive. Detailed information on how to make preliminary designs on which to base cost estimates is given in the chapter on Mechanical Support.

189. Trial Slopes. In some situations it is appropriate to plan for the excavation of trial slopes. For example, if it is suspected that the assumptions about the strength properties of critical discontinuities have been very conservative, trial slopes could be designed with the objective of being able to establish that such conservatism was unnecessary. The specific objective might be to determine the effective length of critical joints or whether joints combine to form stepped failure patterns, or it might be to test the effective shear strength of such discontinuities. The potential benefits of the trial would be a reduction in the large sum of money required to excavate waste.

190. The duration of the trial should be planned in some detail, and the output data and its use explicitly established. For example, a trial could be planned to determine the effective strength properties of a major structural feature such as a fault or bedding plane, which would only require a limited area. On the other hand, to obtain appropriate statistical data, a trial might have to be planned to encompass a considerable length of wall.

191. During operations, the question of appropriate slope angles for interim pit walls may be considered. On one hand, having overall interim slope angles low enough to provide the maximum amount of working space so that the greatest working efficiency can be obtained is advantageous. On the other hand, the planners might like to have relatively steep interim slope angles to provide a test of the rock strengths.

In all mines, production considerations are given top priority. However, there are pit configurations where an interim slope can be used as a trial that will not unduly affect operating efficiency. Such a trial should be in an area where instability, should it develop, would not affect haulageways or surface installations.

192. If the rate of return of the mine investment is particularly sensitive to the design slope angles and if operations would not be unduly affected, in the early years it might be advantageous to conduct a trial. Under these circumstances, it becomes the responsibility of the mine planner to identify all pertinent factors to establish the benefits and costs for the trial. Utilization of such information will be essentially along the lines described in Appendix C for the use of information from previous slopes. A report must be written describing purpose, details and output from such an experiment.

Costs

193. Table 4 provides some typical cost figures for the redesign of pit walls. The range in costs that can occur, depending on the site conditions and situation at the time of the redesign, is also provided. Being based on the experience of a number of companies, these guidelines are realistic, although the wide variety of conditions that can prevail might tend to lead to the assumption that they are meaningless. Amounts are expressed in 1973 dollars so that an inflation index (essentially a salary index) should be used to make them applicable to the pertinent year. Being based on both direct and overhead costs the figures should be equally applicable to work done by company staff and by outsiders on contract. For guidance on calendar time, the $100,000 figure tyically represents a design study conducted over approximately one year.

Table 4: Budget guidelines (1973 $)
(Typical range shown in brackets)

Depth	Operating Stage
<100 ft	$10,000
(30 m)	(1000-50000)
100-1000 ft	50,000
(30-300 m)	(5000-200000)
>1000 ft	10,000
(300 m)	(10000-500000)

Note: In general, estimates based on 2 x salary cost.

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GLOSSARY OF TERMS

(Dimensional symbols in brackets eg [D] indicate the physical nature of the term, ie L stands for any unit of length. M for mass, F for force, T for time, and D is used if the term is dimensionless. Descriptive terms, eg bench, have no symbol)

ANGLE OF INTERNAL FRICTION ϕ [D]
The maximum angle of obliquity between the normal and the resultant stress acting on a surface within a soil of rock.

ANGLE OF REPOSE [D]
The angle with a horizontal plane at which loose material will stand on a horizontal base without sliding.

BENCH
The entire geometry of the ground resulting from one production blast, which includes such aspects as the bench height, bench width, bench face and bench angle.

BENCH ANGLE [D]
The angle of inclination of the bench face measured from the horizontal.

BENCH FACE
The vertical, or near vertical, surface of rock exposed by the blasting of one production cut.

BENCH HEIGHT [L]
The difference in elevation between the grades of two succeeding downward production blasts.

BENCH WIDTH [L]
The horizontal ground left between each succeeding lift or blast.

BERM
The entire geometry resulting from the combination of two or more benches at succeeding levels, which includes such aspects as berm height, berm width, berm face and berm angle.

BERM ANGLE [D]
The angle of the berm face from the toe of one berm to the crest of the berm above.

BERM FACE
The vertical, or near vertical, surface of rock exposed by the successive benches that have been combined into the berm.

BERM HEIGHT [L]
The difference in elevation between the grade of two successive berms.

BERM WIDTH [L]
The horizontal ground left between each succeeding berm face.

BREADTH [L]
A short dimension usually parallel to the plane of the orebody, unlike length, width and thickness.

CATCH BENCH
A bench designed to provide a sufficient width to catch loose, falling rock.

COEFFICIENT OF INTERNAL FRICTION ϕ [D]
The tangent of the angle of internal friction.

COEFFICIENT OF PERMEABILITY k $[LT^{-1}]$
The rate of discharge of water under laminar flow conditions through a unit cross-sectional area of a porous medium under a unit hydraulic gradient and standard temperature (usually 20°C).

COHESION c $[FL^{-2}]$
The portion of the shear strength, S, indicated by the term c, in Coulomb's Equation, $S = c + \sigma \tan \phi$, where ϕ is the angle of internal friction. It has the nature of an intergranular binding force. Sometimes referred to as apparent cohesion when it is known that the straight-line relationship is not valid but is assumed for a limited range of normal stress.

CREST
The top of an excavated slope or the summit land of any eminence.

CRITICAL CIRCLE (CRITICAL SURFACE)
The sliding surface for which the factor of safety is a minimum in an analysis of a slope in ductile ground where average stresses can be used.

CRITICAL DIP β_c [D]
The minimum dip of a discontinuity striking parallel to a slope face on which sliding can occur taking into account the frictional resistance of the discontinuity, seepage and earthquake forces.

CRITICAL SLOPE ANGLE i_c [D]
The maximum angle with the horizontal at which a slope of a given height will stand unsupported.

CUMULATIVE DISTRIBUTION FUNCTION CDF
A function, or curve, expressing the probability of a parameter, or random variable, having a value equal to or less than a specified value, or argument of the function.

DEFLECTION [L]
The movement of a point on a body.

DEFORMATION [L]
The change in a linear dimension of a body, or the absolute movement of a point on a body.

DENSITY (MASS DENSITY) $[ML^{-3}]$
Mass per unit volume.

DETERMINISTIC
Describes a parameter whose value is known with certainty.

DIP [D]
The angle of a slope, vein, rock stratum, or borehole as measured from the horizontal plane downward. Where applicable, the dip is measur-

ed normal to the strike.

DISPERSION

The variability of observed data of a simple property.

DISPLACEMENT [L]

The straight line distance between two points or positions.

DOMAIN

The areal extent of a given environment (10).

DRAWDOWN [L]

The vertical distance the free water elevation is lowered or the reduction of the pressure head due to the removal of free water.

DUCTILE

Pertaining to a substance that readily deforms plastically (10), arising from metal that is capable of being drawn through the opening of a die without breaking and with a reduction of the cross-sectional area.

DUCTILITY RATIO

The ratio of residual strength, ie after large strain, to peak strength.

ELASTICITY

The property or quality of being elastic. An elastic body returns to its original form or condition after a displacing force is removed.

EQUIPOTENTIAL LINE

Line along which water will rise to the same elevation in piezometric tubes (9).

EXPERIENCE CURVE

A curve plotted from actual slopes of height versus either slope angle or the horizontal projection of the slope face.

EXPONENTIAL DISTRIBUTION

A probability density function characterized by an exponential curve defined by one constant, which is equal to the mean value when the range is from zero to infinity. The cumulative distribution function is then also an exponential function.

EXTENSOMETER

Instrument used for measuring small deformations, deflections, or displacements.

FABRIC

The orientation in space of the elements of a rock mass (10).

FACE

The vertical, or near-vertical, surface of rock exposed by mining operations.

FACTOR OF SAFETY [D]

See Safety Factor.

FIELD STRESS

See Stress.

FLOW LINE

The path that a particle of water follows under laminar flow conditions (9).

GEOPHONE

A device to detect seismic waves transmitted through ground.

GROUND SHOCK

Suddenly initiated ground motion. It can be caused by an earthquake, explosion or rockburst.

GROUND WATER LEVEL

The level below which the pores and fissures of the rock and subsoil, down to unknown depths, are full of water (10). Or elevation at which the pressure in the water is zero with respect to the atmospheric pressure (9), see Line of Seepage.

HISTOGRAM

Measured values of a parameter are grouped into classes and the number of occurrences, or frequency, plotted as a vertical bar over the range, or class, of the parameter's value. The relative area under the bar represents the probability of occurrence of that dip for that population.

HYDRAULIC GRADIENT i [D]

The loss per unit distance of elevation plus pressure head. Critical Hydraulic Gradient: Hydraulic gradient at which the intergranular pressure in a mass of cohesionless soil is reduced to zero by the upward flow of water (10).

IDEOGRAM

A vertical bar graph showing the frequency of occurrence of various safety factors in a Monte Carlo analysis.

INTERRAMP SLOPE ANGLE [D]

The inclination of the line joining the toe of the slope above one ramp to the outermost crest point of the ramp above.

INTERIM SLOPE

The slope of the wall of an interim pit before a final pushback occurs to the ultimate wall.

INTERNAL COST OF CAPITAL

The interest rate representing the appropriate charge for the use of capital obtained from within the company.

JOINT

In geology, a plane or gently curved crack or fissure, which is one of an approximately parallel set of fissures ranging from a few inches to many feet apart. Joints occur in rocks of nearly all kinds and generally in two or more sets that divide the rocks into polyhedral blocks.

JOINT SYSTEM

Consists of two or more joint sets or any group of joints with a characteristic pattern, such as a radiating pattern, a concentric pattern, etc. (10).

KURTOSIS [D]

A measure of the peakness, or flatness, of a probability density function. A normal distribution has a kurtosis of 3.0.

LAYER

A bed or stratum of rock separated from the adjacent rock by a plane of weakness (10).

LENGTH [L]

Usually a long dimension parallel to the orebody, unlike width and thickness.

LINE OF SEEPAGE (SEEPAGE LINE) (PHREATIC LINE)

The upper free water surface of the zone of seepage (9).

MODULUS OF ELASTICITY (MODULUS OF DEFORMATION) E $[FL^{-2}]$

The slope of the tangent (hence 'tangent modulus') of a stress-strain curve. The use of the term Modulus of Elasticity is recommended for materials that deform in accordance with Hooke's Law, the term Modulus of Deformation for materials that deform otherwise (9).

MOHR ENVELOPE (RUPTURE ENVELOPE) (RUPTURE LINE)

The envelope of a series of Mohr Circles representing stress conditions at failure for a given material. According to Mohr's Strength Theory, a failure envelope is the locus of points the coordinates of which represent the combinations of normal and shearing stresses that will cause a given material to fail (9).

MONTE CARLO

Referring to the process of random sampling from a defined distribution of values of a parameter. It simulates the occurrence of various values of strength and geometric properties in nature.

NORMAL DISTRIBUTION, STANDARD NORMAL, NORMAL CURVE, GAUSSIAN

The distribution of numerical data, x, about an average value, m, that follows the gaussian equation:

$$y = \frac{1}{s\sqrt{(2\pi)}} \exp \left[\frac{-(x-m)^2}{2s^2}\right]$$

where y is the frequency of occurrence and s is the standard deviation of the data.

OVERALL SLOPE ANGLE [D]

The angle measured from the horizontal to the line joining the toe of a wall to the crest of the wall.

PARAMETER

A quantity constant in a special case but variable in different cases, eg angle of internal friction, modulus of deformation, etc.

PIEZOMETER

A device for measuring the hydrostatic pressure at a point in the ground. Simple piezometers are open groundwater wells.

POISSON'S RATIO μ, v [D]

The ratio of the transverse normal strain to the longitudinal normal strain of a body under uniaxial stress.

PROBABILITY

It is concerned with unpredictable individual events but which are predictable in large numbers. It is the frequency ratio of occurrence of one event that can be expected in an infinitely large population of events. For less than an infinite population, it has the meaning of relative likelihood of occurrence, and as such it can be a personal opinion instead of being based on observations or analysis, eg 'I believe there is a 1 in 3 likelihood of hitting ore at this location'.

PROBABILITY DENSITY FUNCTION

It is the relation between the relative likelihood of occurrence of a parameter or event and the numerical value of the event.

PROBABILITY OF INSTABILITY

The probability that the variations in length and spacing of discontinuities will combine with those governing the probability of sliding to permit instability.

PROBABILITY OF SLIDING

The probability that, given the critical geometry, the variations in strength, groundwater and dip will combine to produce sliding.

PROGRESSIVE FAILURE

Failure in which the ultimate resistance is progressively, rather than simultaneously, mobilized along the ultimate failure surface.

RATE OF RETURN

The interest rate that is equivalent to the money, net of expenses, earned by invested capital.

RELIABILITY

The obverse of the probability of instability, P, ie 1-P, or the probability that strength exceeds stress.

ROCK MASS

The in situ rock made up of the rock substance plus the structural discontinuities.

ROCK SUBSTANCE

The solid part of the rock mass, typically obtained as drill core.

ROCKFALL

The relatively free fall of a detached rock of any size from a steep slope.

RANDOM SAMPLE

A sample taken in such a way that there is an equal chance of every member of the target population being selected or observed. By contrast, a biased sample is one that is taken in a manner that results in a greater possibility of it being selected or observed than others in the target population, eg a set of dip angles obtained from core will be biased against dips parallel to the hole.

SAFETY BERM

A berm designed to provide a sufficient width to catch loose, falling rock.

SAFETY FACTOR FS [D]

The ratio of the ultimate stress to the working stress at fracture or yield.

SAMPLE POPULATION

The group of data from which actual samples are taken, which may or may not be equivalent to the target population, eg the dip of the joints available for measurement on the faces of the benches may not include all representative joints.

SECTOR

The length of wall, or pie slice, that can be considered sufficiently homogeneous to use one slope angle resulting from a comprehensive stability analysis.

SEGMENT

A vertical interval in a wall of one sector.

SHEAR FAILURE

Failure resulting from shear stresses.

SHEAR STRENGTH [FL^{-2}]

The internal resistance offered to shear stress. It is measured by the maximum shear stress, based on original area of cross-section, that can be sustained without failure (9).

SKEWNESS [D]

A measure of departure from symmetry of a probability density function. A normal distribution has a skewness of zero.

SLIDE

A relatively deep-seated failure of a slope. Three main types can be identified. (1) Block Flow Slide: a slide resulting from internal deformation leading to failure of a blocky rock mass with a strong, brittle rock substance. It is believed that failure is initiated by the concentration of stress on corners of the individual blocks bounded by joint planes and that, when a general breakdown has occurred, a flow of blocks and pulverized material to the bottom of the slope occurs. (2) Plane Shear Slide: a slide resulting from the presence of planes of weakness, eg faults, dikes or soft layers, in critical orientations within the slope. Large segments of the slope move down along these planes. (3) Rotational Shear Slide: a slide resulting from the yielding and redistribution of shear stresses in a soft rock or soil so that a more or less circular surface of failure develops before the cohesion breaks down and permits a comprehensive, circular sector of the slope to fail by rotating.

SPECIFIC GRAVITY G [D]

Ratio of the mass of a body to the mass of an equal volume of water at a specified temperature.

STANDARD DEVIATION S

The square root of the quotient of the sum of the squares of the difference between the arithmetic mean, m, and a number of values of a quantity, x, divided by the number, n. It is a measure of the dispersion of the number of values about the mean, ie,

$$S = \pm\sqrt{[(m-x)^2/n]}$$

STOCHASTIC

Describing a parameter whose exact value either is unknown and can only be measured imprecisely or is subject to variations in nature.

STRESS $[FL^{-2}]$

The force per unit area, when the area approaches zero, acting within a body. Effective Stress (Effective Pressure) (Intergranular Pressure), σ' $[FL^{-2}]$: The average normal force per unit area transmitted from grain to grain in a granular mass. It is the stress that is effective in mobilizing internal friction. Field Stress $[FL^{-2}]$: The stress existing in a rock mass independent of any man-made works. Residual Stress: Stress that exists in a formation owing to previously applied forces or deformations. Neutral Stress (Pore Pressure) (Pore Water Pressure) u, $[FL^{-2}]$: Stress transmitted through the pore water) (water filling the voids of the mass). Normal Stress $[FL^{-2}]$: The stress component normal to a given plane. Principal Stress, σ_1, σ_2, σ_3 $[FL^{-2}]$: Stresses acting normal to three mutually perpendicular planes intersecting at a point in a body, on which the shear stresses are zero. Major Principal Stress σ_1 $[FL^{-2}]$: The largest (with regard to sign) principal stress. Minor Principal Stress, σ_3 $[FL^{-2}]$: The smallest (with regard to sign) principal stress. Intermediate Principal Stress, σ_2 $[FL^{-2}]$: The principal stress whose value is neither the largest nor the smallest (with regard to sign) of the three. Shear Stress (Shearing Stress) (Tangential Stress), $\tau[FL^{-2}]$: The stress component tangential to a given plane. Total Stress, σ $[FL^{-2}]$: The total force per unit area acting within a granular mass. It is the sum of the neutral and effective stresses (9).

STRESS CONCENTRATION k [D]

Ratio of the stress at any point to the applied or principal field stress.

STRIKE [D]

The bearing of a horizontal line in the plane of an outcrop, joint, fault, or the structural plane.

STRUCTURAL FEATURE

In geology, a feature representing a discontinuity of mechanical properties, such as a joint, fault, or bedding plane.

STRUCTURE

(a) In civil engineering, the assemblage of structural elements designed to support and transmit loads to the subgrade of the foundations. (b) In geology, the assemblage of structural features that together with the rock substances make up the rock mass with the emphasis being on the structural features.

TARGET POPULATION

The entire group of data from which representative samples are to be taken, eg the dip of the joints in a set.

THICKNESS [L]

A dimension, together with width, usually referring to the dimension of an orebody normal to its plane.

TOE

The bottom of a slope.

TRIANGULAR DISTRIBUTION

The probability density function characterized by a finite range and linear variations from zero to a maximum.

TRUNCATED NORMAL DISTRIBUTION

A normal distribution with one or both tails terminated abruptly, which is appropriate for parameters that cannot have values beyond this point, eg cohesion less than zero.

UNIFORM DISTRIBUTION

A rectangular probability density function.

UNIT WEIGHT $[FL^{-3}]$

Weight per unit volume. Dry Unit Weight (Unit Dry Weight), $\gamma_e[FL^{-3}]$: The unit weight that, when multiplied by the height of the overlying column of ground, yields the effective pressure due to the weight of the overburden (1). Saturated Unit Weight, γ_{sat} $[FL^{-3}]$: The wet unit weight of a granular mass when saturated (1). Submerged Unit Weight (Buoyant Unit Weight, γ_b $[FL^{-3}]$: The weight of the solids in air minus the weight of water displaced by the solids per unit of volume of mass; the saturated unit weight minus the unit weight of water. Wet Unit Weight (Mass Unit Weight), γ_m, γ_t $[FL^{-3}]$: The weight (solids plus water) per unit of total volume of mass, irrespective of the degree of saturation (9).

VARIANCE

A measure of the dispersion of a random variable, which is equal to the square of the standard deviation.

WIDTH [L]

A dimension, together with thickness, usually referring to the dimension of an orebody normal to its plane.

WORKING FACE OR SLOPE

The series of benches or berms making up a typical working pattern.

YOUNG'S MODULUS $[FL^{-2}]$

See Modulus of Elasticity.

ABBREVIATIONS

β - dip of a sliding plane in a plane shear failure
γ - density
δ - angle between a rock face and a joint plane
eq - equation
FE - finite element
ϕ - friction angle
Rg - gross revenue
in. - inch
k - kilo
kg - kilogram
M - mean
m - metre
M - million
NPV - net present value
N - Newton
N - normal force
σ - normal stress
n - number of years
PBP - payback period
u - pore pressure
psf - pounds per square foot
psi - pounds per square inch
p,P - price
T - tangential force
3-d - three-dimensional
T - tons
γ - unit weight
V - volume
v - volume per unit weight
W - weight
W - width
PV - present value
PVR - present value ratio
R - radius
r - radius of curvature
RN - random number
r - rate of return
R - reliability
RC - reliability coefficient
R - resisting force
FS,S - safety factor
SM - safety margin
S - shear strength
τ - shear stress
S - standard deviation
z - statistical coupling factor, (M-x)/S, where x is a value of the parameter being examined statistically
q - surcharge load on the crest of a slope

SYMBOLS

a	acceleration
c	cohesion; unit cost
g	acceleration due to gravity
i	dip of slope face
i_c	critical slope angle
k	ratio of clean-up or unloading excavation costs and normal unit mining costs
n	number of years
p	price
q	surcharge load on the crest of a slope
r	radius of curvature; rate of return or interest rate
u	pore pressure
v	volume per unit weight
z	statistical coupling factor
C	total cost
CDF	cumulative distribution function
CF	cash flow
D	height of groundwater above the toe of the slope
E	modulus of elasticity
G	grade of a ramp
H	height
I	intensity of jointing per unit volume; correction angle for a slope arising from horizontal curvature
K	ratio of horizontal to vertical premining field stress
L	length
L_j	length of a joint
L_w	length of a wall, typically along strike
M	mean; million
NPV	net present value
N	normal force
P	probability; price
PBP	payback period
PDF	probability density function
PV	present value
PVR	present value ratio
R	radius; reliability; resisting force
RC	reliability coefficient
Rg	gross revenue
ROR	rate of return
FS	safety factor
S	safety factor; shear strength; standard deviation
SM	safety margin
T	tangential force
V	volume
W	weight; width
β	dip of a sliding plane in a plane shear failure
γ	density or unit weight
δ	angle between a rock face and a joint plane
μ	Poisson's ratio
ϕ	friction angle
σ	normal stress
τ	shear stress

APPENDIX A

RISK AND PROBABILITY IN DESIGN

Discounted Cash Flow

1. Open pit mine investment and design decisions involve comparing the various alternatives. Formerly it was normal practice to select single values for the various input factors, such as plant cost, production rate, volume of reserves and the like. These factors were then used in the calculations as if they were known with certainty. More recently, it has been accepted that for proper appraisal of investment alternatives, it is important that the uncertainty in predicting future values of these factors be recognized.

2. Cash flow analyses are usually conducted to determine the optimum mine layout, production rate, mill capacity and cut-off grade. The definition of cash flow is shown in Table A-1 and a sample plot is shown in Fig.A-1. This approach should be extended to determinate wall angles. Three financial criteria can be used in the search for these various optimums: payback period, PBP; present value ratio, PVR; and rate of return, ROR(A-1). ((A-1) indicates a reference listed at the end of Appendix A.) One may be preferred over others, but each provides a different aspect of the prospective investments - hence probably all three should be used.

Table A-1: Cash flow computation procedure

	Gross Revenue
Less:	Operating Cost
	Gross Profit
Less:	Accelerated Capital Cost Allowance
	Income Before Exploration and Development Deductions
Less:	Exploration and Development Deductions
	Depletable Income
Less:	Depletion Allowance
	Taxation Income
Less:	Federal Income Tax
Less:	Provincial Income Tax
Less:	Provincial Mining Tax
	Net Profit
Plus:	Accelerated Capital Cost Allowance
Plus:	Depletion Allowance
Plus:	Exploration and Development Deductions
	Cash Flow

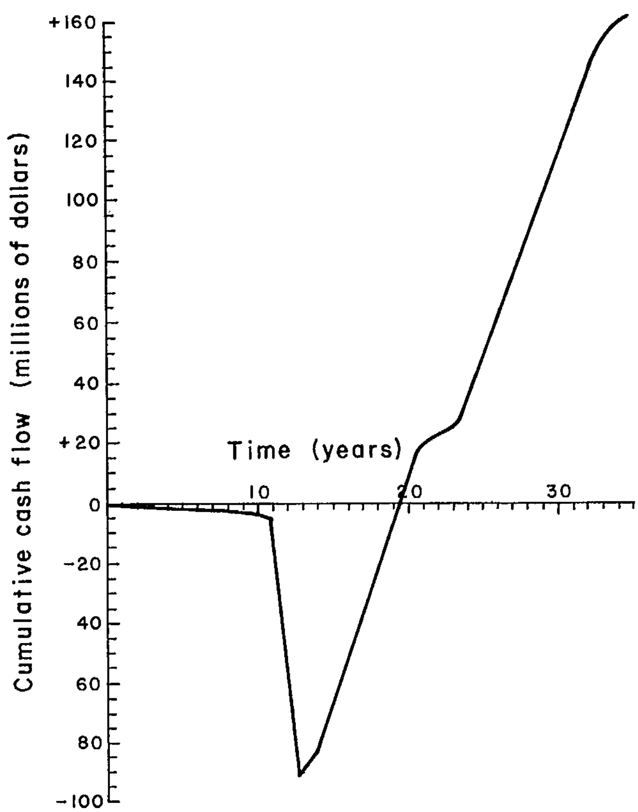

Fig. A-1 - A cumulative cash flow graph showing initial outflow due to exploration, then due to feasibility testing and plant investment, followed by net inflow during production, completed by a closing out period.

3. The rate of return, ROR, on pre-production investments (more commonly called the "discounted cash flow rate of return" or DCFROR, which simply indicates how the ROR is determined) is the most common criterion for comparing investments. On the other hand, present value calculations are readily used for comparative design purposes, eg for determining the benefits and costs of steeper slopes, as treated below. The rate of return represents the discount rate that will make the present value of the negative cash flows just equal to that of the positive cash flows. A simple example of this type of calculation is shown in Table A-2. In this case the rate of return by interpolation is 7-3/4%. All of such calculations can be made assuming single values for the future magnitude of all of the input variables.

Table A-2: Calculating discounted cash flow rate of return

Year	CF $M	$(1+10\%)^n$	PV	$(1+8\%)^n$	PV	$(1+6\%)^n$	PV
1	-10	1.10	-9.09	1.08	-9.26	1.06	-9.46
2	3	1.21	2.49	1.17	2.58	1.12	2.68
3	3	1.33	2.26	1.26	2.38	1.19	2.52
4	3	1.47	2.04	1.36	2.21	1.26	2.38
5	3	1.61	1.86	1.47	2.04	1.34	2.24
			-0.44		-0.05		+0.36

4. The sensitivity of the financial criteria to deviations from the assumed value of an input variable can be examined. For example, repeated calculations of the rate of return can be made assuming one of the variables such as price is 10%, 20% and 30% higher than used originally which may show that the rate of return increases by twice as much as the price. Identifying such a sensitive relationship indicates an area where extra study would be warranted to predict with greater accuracy future values of the variable.

Uncertainty

5. As indicated above, the estimate of each input variable and the calculated prospective rate of return are uncertain. The grade of ore available for mining may prove to be quite different from that expected. Likewise estimated volume of mineable ore, particularly at the exploration stage, is usually a fairly crude estimate. Further exploration expense, plant cost, actual production rate achieved, mill recovery, production costs, actual wall angles, market prices and future royalty rates are all impossible to predict with absolute certainty.

6. Almost all design and analysis procedures in the past, whether in engineering, business or economics, have not incorporated the use of probability to describe the inherent uncertainty associated with the variables in the problem. Instead, these procedures invariably used the deterministic approach, in which a point estimate of each variable is assumed to represent the variable with certainty. Afterwards, the uncertainty associated with the results is implicitly adjusted in some manner, such as the use of safety factor in engineering and the use of a higher discount rate in business.

7. Wall stability is subject to variability. We know that rock strength is variable; considering its geologic nature how could it be otherwise. Groundwater pressures are variable, as are the stresses promoting breakdown. Human error in evaluating these factors adds to the uncertainty.

8. To analyze the uncertainty inherent in the financial calculations, it is necessary to use certain techniques available in the subjects of statistics and probability. With the recognition of uncertainty in stability analyses, mine planners must become familiar with these techniques.

9. The following definitions serve as an introduction to a few useful concepts. The subject of Statistics and Probability is the science of analyzing observations or data to determine valid inferences. Typically it deals with a set of data that have variability due to chance, ie random variation.

10. Target population is the entire group of data from which representative samples are to be taken, eg the dip of the joints in a set.

11. Sample Population is the group of data

from which actual samples are taken, which may or may not be equivalent to the target population, eg the dip of the joints available for measurement on the faces of the benches may not be representative of all the joints in the wall rock.

12. A random sample is taken in such a way that there is an equal chance of every member of the target population being selected or observed. By contrast, a biased sample is one taken in a manner resulting in a greater possibility of some members being selected or observed than others, eg a set of dip angles obtained from drill core will be biased against dips parallel to the hole.

13. Probability is concerned with events that individually are not predictable but that in large numbers are predictable. It is the relative frequency of occurrence of one event that can be expected in an infinitely large population of events. For less than an infinite population and for expressions on the state of nature it has the meaning of relative likelihood of occurrence. It can be a personal opinion on degree-of-belief instead of being based on observations or analysis, eg 'I believe there is a 1 in 3 likelihood of hitting ore at this location'. In fact, most of the estimates of the dispersion of the input variables in mine risk analysis can be considered as subjective, rather than objective, probabilities. In this way, full use is made of all information including expert opinion.

14. Probability distribution is the relationship between the relative likelihood of occurrence of an event and the numerical value of the event, eg if the event is the dip of joints in a set, the probability distribution may be as shown in Fig. A-2. Three axioms apply in this area. The probability associated with any discrete event among all possible outcomes must be between 0 and 1. The sum of all probabilities defined in the probability distribution or function must add up to 1. If A and B are mutually exclusive events, the probabilities associated with A and B must be additive, ie P (AUB) = P(A) + P(B).

15. Histograms, rather than curves, can be used to describe the distribution of a factor such as strike or dip. For example, if the observed

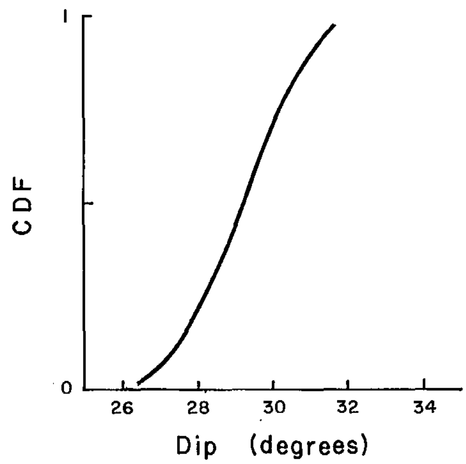

Fig. A-2 - Cumulative distribution function, CDF, of dip angles in a joint set; the y-axis indicates the ratio of the number of joints to the total number with expected dips less than the angle on the x-axis.

dips of a joint set are grouped into degree intervals and the number of observations plotted against the interval, a histogram is obtained as shown in Fig.A-3(a). Regarding the interval to be used, a crude rule is to divide the range into 10 intervals and then round off the intervals; a somewhat less crude rule is to calculate the interval K from the equation: K = 1 + 3.3 log n, where n is the number of observations (A-2).

16. By dividing the number of tests or observations in the interval of a histogram by the total number of observations, the proportion of observations in each interval, or frequency ratio, is obtained as shown on the right ordinate of Fig. A-3(a), ie 10 tests out of a trial of 38 is 0.26 of the total. This figure can then be described as the frequency distribution of the data. If the right side axis were divided by the interval of the bars, ie in this case 1° but it might have been 2° or more, the chart becomes a frequency density distribution. If a curve is fitted to the histogram to smooth out the frequency distribution, as shown in Fig. A-3(b), a better representation of the target population may be obtained. This curve is called a probability density function because it represents the probability of encountering any dip value on the x-axis. The expected probability of occurrence of dips between two x-values is obtained by multiplying the y-value by the interval on the x-axis. Because the sum of all probabilities must equal 1, the area under the curve is equal to unity.

17. The summation of the area under the probability density curve of Fig. A-3(b) up to any x-value represents the probability of occurrence of dips equal to or less than the value of the dip on the x-axis. If this cumulative area is plotted for each dip angle, a cumulative probability distribution curve is obtained as shown in Fig. A-2. A similar curve could be constructed from the original histogram of data by adding the number of observations in each interval up to the specified dip angle, eg with 2 observations at 27° and 8 observations at 28°, in which case a cumulative frequency distribution curve would be obtained.

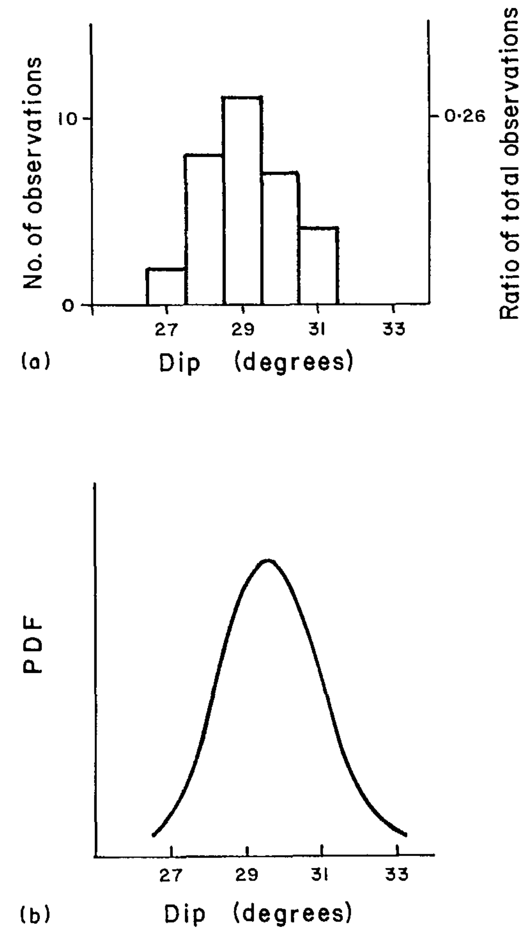

Fig. A-3 - (a) Histogram of observed dips of joints in a set; when expressed as a ratio of total observations, the chart is a frequency distribution. (b) A probability density function, PDF, describing expected variations in dip angles of a set of joints; the y-axis value multiplied by an interval on the x-axis gives the probability of occurrence of dips within that interval.

18. Normal Distribution is a mathematical function that can be used to describe as shown in Fig. A-3(b), variations above and below the mean value. Its main virtue is that it has certain convenient mathematical properties. Hence, in the face of our imperfect knowledge of the variation of the input parameters to the financial calculations, it is sometimes convenient to assume that they have a normal distribution. (One alternate assumption, as shown in Fig.A-8(c), is to assume straight line distributions between some estimated minimum and maximum values and some central value, which is particularly convenient when using the Monte Carlo approach below.)

19. The properties of the normal distribution curve have been well documented. For example, the area under one tail of the curve beyond one standard deviation, S, from the mean, M, is approximately 16% (15.9%) of the total area, making the area under the two tails 32% and the area under the curve within one standard deviation of each side of the mean approximately 68%. The areas under the various portions of the normal distribution curve for limits of x-values other than one standard deviation from the mean can also be calculated. Table A-3 gives areas under a standard normal curve to the left of an x-value expressed in the standardized form (x - M)/S where M is the mean, S is the standard deviation, R is the reliability and P_f is the probability of instability.

20. To establish probability density functions for input variables for financial analysis, these characteristics of normal distributions can be useful. For example, in estimating future mining costs $1/T might be the most likely value, together with the estimate that only a 10% probability exists they could be reduced below $0.85/T and a corresponding 10% probability that they could exceed $1.15/T. Assuming a normal distribution, the data, shown in Table A-3 can be used to construct the entire probability distribution curve, PDF, for the future mining costs. For example, when P_f = 10%, from 0.100 in the third column interpolate z = 1.28 in the first column. Therefore, for P_f = 10% z = -1.28 and for

Table A-3: Standardized normal distribution (A-3)

z = (M-x)/S z = (x-M)/S	$R = 1-P_f$ P_f	P_f $R = 1-P_f$
0.0	0.5000	0.5000
0.1	0.5398	0.4601
0.2	0.5793	0.4207
0.3	0.6179	0.3820
0.4	0.6554	0.3445
0.5	0.6915	0.3085
0.6	0.7257	0.2742
0.7	0.7580	0.2419
0.8	0.7881	0.2118
0.9	0.8159	0.1840
1.0	0.8413	0.1586
1.1	0.8643	0.1356
1.2	0.8849	0.1150
1.3	0.9032	0.0968
1.4	0.9192	0.0807
1.5	0.9332	0.0668
1.6	0.9452	0.0547
1.7	0.9554	0.0445
1.8	0.9641	0.0359
1.9	0.9713	0.0287
2.0	0.9772	0.0227
2.1	0.9821	0.0178
2.2	0.9861	0.0139
2.3	0.9893	0.0107
2.4	0.9918	0.0081
2.5	0.9938	0.0062
2.6	0.9953	0.0046
2.7	0.9965	0.0034
2.8	0.9974	0.0025
2.9	0.9981	0.0018

P_f = 90% z = 1.28. Other probabilities can be determined as follows:

z = (x-M)/ S	Probability, P_f	x
-1.28	10 % less than x	$0.85/ton
-1.00	15.9% less than x	-
0	50.0% less than x	1.00
1.00	84.1% less than x	-
1.28	90 % less than x	1.15
2.33	99 % less than x	-

The distribution is defined by knowing that the area under the curve to the right of $0.85 is 90% of the total, as is the area to the left of the curve from $1.15. It follows that the standard deviation for the mining costs is:

$$S = (x - M)/z = (1.15 - 1.00)/1.28 = 0.117$$

The x-value, or costs corresponding to the probability of 15.9% less than x is thus (1.00 - 0.117) or $0.883/T and similarly for 84.1% is $1.117/T, and that for 99% is (2.33 x 0.117 + 1.00) or $1.273/T.

21. For experimental data, such as the dip angles described in Fig. A-3, assuming that the variations follow a normal distribution, the curve is defined by the mean, M, and the variance, v, or standard deviation, S, according to the following equations:

$$M = \frac{1}{n} \sum_{1}^{n} x_i \qquad \text{eq A-1}$$

where n is the number of values of x being used.

$$v = \frac{1}{n} \sum_{1}^{n} (x_i - M)^2 \qquad \text{eq A-2}$$

$$S = \sqrt{v} \qquad \text{eq A-3}$$

For the data in Fig.A-3, M = 29.7°, v = 1.83 and S = 1.35°.

22. The combining of statistical variables by addition, subtraction, multiplication, etc, has also been studied (A-4,5,6). When the variables x and y are normally distributed and independent (having means and standard deviations of M_x, S_x and M_y, S_y respectively), the following equations for normal distributions can be used for determining means and standard deviations:

for a = (x + y) and a = (x - y);

$$S_a = \sqrt{(S_x^2 + S_y^2)} \qquad \text{eq A-4}$$

for a = xy;

$$S_a \simeq \sqrt{(M_x^2 S_y^2 + M_y^2 S_x^2 + S_x^2 S_y^2)} \qquad \text{eq A-5}$$

for a = x/y;

$$S_a \simeq \frac{1}{M_y^2} \sqrt{[(M_x^2 S_y^2 + M_y^2 S_x^2)]} \qquad \text{eq A-6}$$

for a = f (x, y);

$$S_a = \sqrt{[(\frac{da}{dx})^2 S_x^2 + (\frac{da}{dy})^2 S_y^2]} \qquad \text{eq A-7}$$

(The differentiations being done with mean values.)

for a = x^2;

$$M_a = M_x^2 + S_x^2 \qquad \text{eq A-8}$$

$$S_a = 2M_x S_x \qquad \text{eq A-9}$$

The above formulae for multiplying and dividing were derived for standard deviations of the order of 5% of the means. When standard deviations are as much as 30% of the means, the above formulae can give answers as much as 10% in error, which are considered acceptable in this work. For S_x and S_y greater than about 40% of their means, the Monte Carlo method, as described below must be used to avoid introducing excessive error. The means of the answers are obtained, unless otherwise indicated, by the usual calculations as if the values were certain, eg $M_{(x-y)} = M_x - M_y$ or $M_{xy} = M_x M_y$. Implicit in these procedures is the assumption that true means and standard deviations are being used.

23. Consider as a simple example determining the stress on a failure plane; assume it is obtained by multiplying rock density, γ, by the height of rock over the failure surface, H. The mean and standard deviation of the density are given as 165 pcf (2642 kg/m), and 16 pcf (256 kg/m) and the mean and standard deviation of the

height of rock over the plane are 500 ft (153 m), and 20 ft (6.1 m). Assume densities and heights are independent of each other and are normally distributed. The product of these two variables produces, using eq A-5, the following characteristics:

$$M = 165 \times 500 = 82{,}500 \text{ psf}$$

$$S = \sqrt{(165^2 \times 20^2 + 500^2 \times 16^2 + 16^2 \times 20^2)}$$

$$= 8{,}660 \text{ psf}$$

In other words, the most likely value of the stress is 82,500 psf (3.95 mPa); however, there is a 16% chance that it will be less than (82,500 - 8,660) = 73,840 psf (3.54 mPa).

24. Because several distributions other than the normal distribution can be transformed into the normal form, eg log normal distributions, a design system based on assumed normal distributions has wide applicability (A-7).

25. The applications of probability and reliability from other areas of engineering can be used in evaluating the stability of pit walls. A simplified concept is shown in Fig. A-4, where the variation of the strength of the rock and the stress on the potential failure surface is recognized. The safety factor using the mean values of strength and stress is two. However, it can also be seen it is possible for stress to exceed strength in some cases, ie in the hatched area where the two curves overlap. In other words, on some wall sections where the strength and stress are represented by Fig. A-4 stress could exceed strength causing a slide.

26. Safety margin is an alternative to safety factor, which includes both mean values and distributions (A-8). It is defined as follows:

$$M_{sm} = M_r - M_s \qquad \text{eq A-10}$$

where M_{sm} is the mean safety margin, M_r is the mean strength and M_s is the mean stress. The standard deviation of the safety margin, from eq A-4, is as follows:

$$S_{sm} = \sqrt{(S_r^2 + S_s^2)}$$

The safety margin is variable and, provided both strength and stress are normally distributed, Fig. A-5 shows the probability density function for the safety margin. The area under the curve to the left of the origin, where stress exceeds strength, represents the probability of instability, P_f. The area to the right of the origin represents the probability of stability, or reliability.

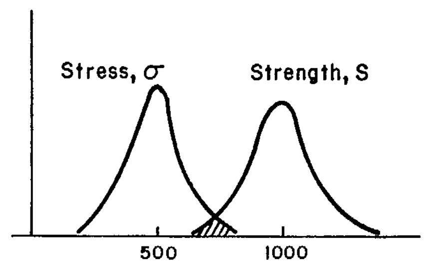

Fig. A-4 - Probability density functions, PDF, for strength and stress on a potential surface of sliding. Although the mean strength is twice the mean stress, owing to the variations in both there can be instances where stress will exceed strength (as shown by the hatched area) and where failure will occur.

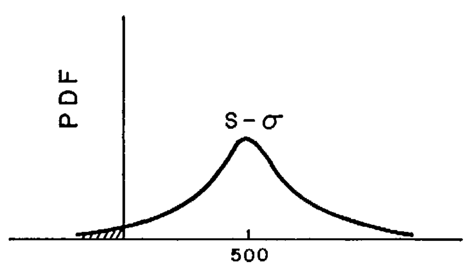

Fig. A-5 - A probability density function for the difference between strength and stress, the safety margin. The possibility of this margin being negative is shown by the hatched area, which represents failure conditions.

27. The safety margin can be made dimensionless by dividing its mean value by its standard deviation, i.e. $M'_{sm} = M_{sm}/S_{sm}$. The difference between this parameter and any other value of the safety margin can then be related to the standard cumulative distribution function, CDF, of the normal distribution curve. For example, for one section of a wall the safety margin, the difference between the strength resisting sliding and the stress causing sliding, is x; for the entire wall the mean difference is M. Normalizing these values as follows: $(x - M)/S_{xm}$, a measure of the probability of stability, or reliability, is obtained. When plotted on scales that linearize the cumulative distribution function, Fig. A-6 is obtained. The reliability, R, is the obverse of the probability of instability, P_f, ie:

$$R = 1 - P_f \qquad \text{eq A-11}$$

(A more formal definition of reliability is the probability that a slope will perform its function within specified limits at a given confidence level for the required period of time under all operating conditions.) In this way, using means and standard deviations of stress and strength of the structure, reliability can be calculated.

28. In slope analysis the safety margin can be defined as the difference between the mean critical slope angle M_i, and other slope angles, i. If a quick analysis of the stability of a pit wall were required without a full financial analysis, the determination of reliability could be omitted. The safety margin could be converted into a reliability coefficient, RC, as follows:

$$RC = \frac{M_i - i}{S_i} + 1 \qquad \text{eq A-12}$$

where i is the slope angle, M_i is the mean critical slope angle and S_i is the standard deviation of the critical slope angle. By calculating RC, the use of Table A-3 or Fig. A-6 is avoided. For example, RC = 1, ie $i = M_i$, would be known to correspond to a probability of instability of 50% (similar to a safety factor of 1); RC = 2, ie $i = M_i - S_i$, would correspond to a probability of instability of 16% and RC = 3, ie $i = M_i - 2\,S_i$, would correspond to a probability of 2%.

29. It is sometimes necessary to recognize variations of some geologic or operating data that are not symmetrical in which case the log-normal distribution as shown in Fig. A-7 is useful. For example, the lengths of the joints in a set is not usually symmetrically distributed, but a log-normal distribution often fits the field observations quite well. By using the logarithm of the x-values, all of the above operations can be conducted as for normally distributed variables.

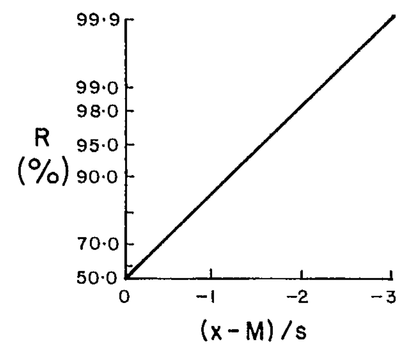

Fig. A-6 - Cumulative area under a normal distribution curve. The variable R is the ratio of the area under a normal distribution curve excluding the tail beyond x to the total area (M is the mean and s is the standard deviation); R can also represent the reliability corresponding to the various values of x, where M is the mean of the safety margin (strength minus stress) (A-7).

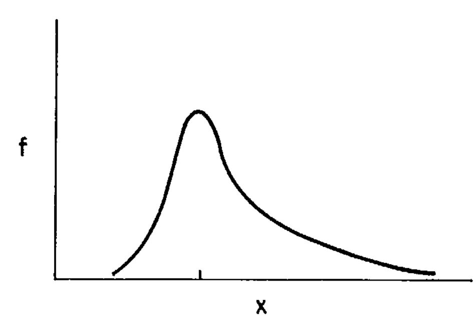

Fig. A-7 - A log-normal frequency distribution curve, which is often a convenient approximation for non-symmetrical distributions. If f is plotted against log x, a normal distribution curve is obtained.

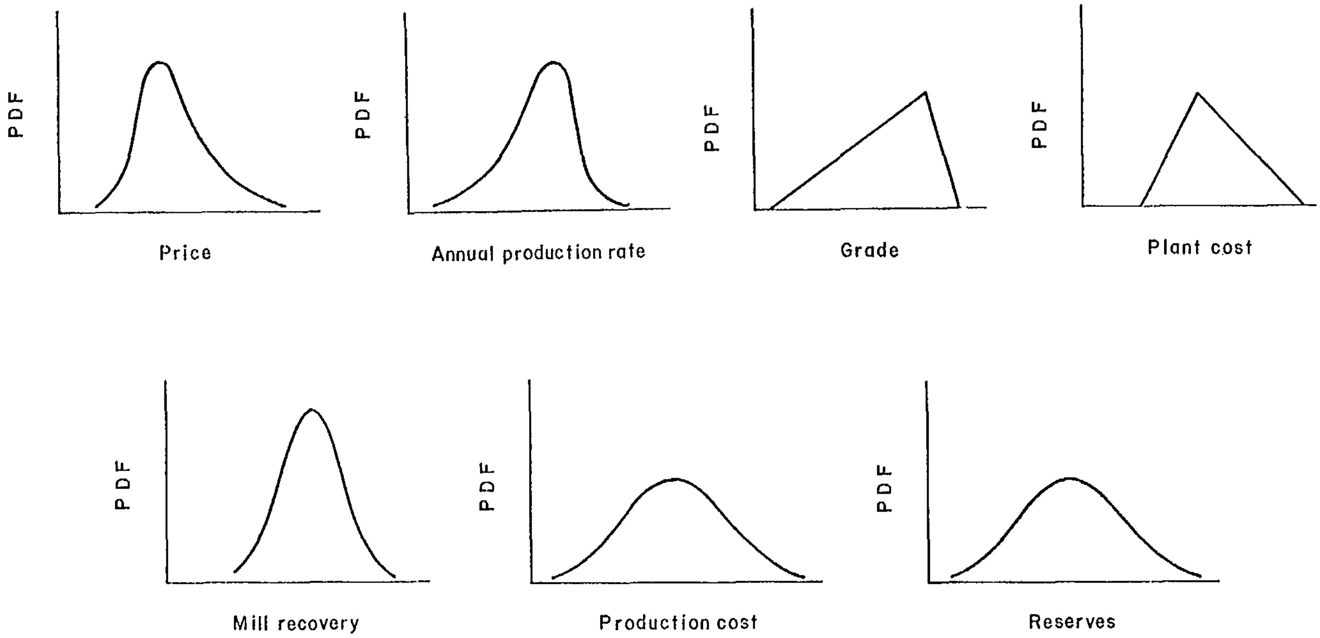

Fig. A-8 - Probability density functions, PDF, for (a) estimated price of mine product, (b) estimated annual production, (c) grade of ore, (d) estimated plant cost, (e) estimated average mill recovery rate, (f) mine production cost, and (g) estimated volume of reserves.

Monte Carlo Simulated Sampling

30. The Monte Carlo approach is an alternative to the algebra of normal distributions for analyzing the effects of variability. It is a simulation method that is particularly useful where variables having distributions other than normal must be combined. For this reason, it is commonly used in financial risk analyses.

31. The general procedure is that a probability density function estimate is made for each variable as shown in Fig. A-8 (A-9). A random number selection is then made of each variable. These selected values are used in conventional calculations to obtain the answer for one particular set of variables. For example, the gross revenue, Rg, for one year will equal the product of the volume of production sold, V, and the price obtained, P, which is assumed to be constant throughout the year; hence Rg = PV. P and V might be predicted as shown in Fig.A-8(a) and (b). Alternatively, one could estimate their most likely values together with minimum and maximum values. For lack of a better hypothesis, it could be assumed that the probability density function is linear between these three values, creating a triangular graph as shown for grade in Fig.A-8(c). Rg is calculated using a random number selection of P and V, and repeating this operation 100 to 1,000 times to obtain the expected distribution of Rg. From these data, the mean or average or most likely value of Rg is obtained together with the probability of values occurring above and below the mean.

32. The selection of values for variables consists of generating one random number for each. This could be done by spinning a roulette wheel or by using a computer program, or a series of random numbers could be obtained by taking, say, the fourth and fifth digits in a column of seven digit telephone numbers. Each random number is then used to select a value of the variable from a table or curve relating the two. In the above example, a probability density function for price could be converted, in effect, into a histogram.

The schedule below uses an interval of $0.10, and the histogram data in the fourth column is related to the random number.

Price	Probability	Cumulative Probability	Random Number
$0.80	0.08	0.08	0 - 8
0.90	0.23	0.31	9 - 31
1.00	0.38	0.69	32 - 69
1.10	0.23	0.92	70 - 92
1.20	0.08	1.00	93 - 99

Hence, for a random number of 29, price would be $0.90, which would be used for one calculation of the gross revenue, R_g.

Rate of Return

33. Willingness to take risk is dependent on the probability of achieving the desired return. Realizing the shortcomings of the conventional investment approach, management has resorted to some additional procedures. Higher than normal acceptable rates of return may be used to compensate for risk. Three-level estimates of pessimistic, expected, and optimistic results may be used. Sensitivity analyses can be made of the effects of uncertain factors. These procedures, however, all suffer the same disadvantage. They do not provide information regarding the extent of risk involved. In addition, estimates that are too conservative result in missing many potential opportunities for making attractive investments. Even sensitivity analyses, performed on one variable while keeping the remainder of variables constant, does not give information about the possible effects of combinations of variations.

34. Using the above tools, the prospective rate of return from a mine investment can be evaluated taking into account the uncertainty of the input variables including wall stability. For example, either using the algebra of normal distributions or the Monte Carlo approach, the mean or most likely rate of return can be calculated together with its standard deviation. Alternatively, a cumulative distribution function can be determined as shown in Fig. A-9 (A-10). There is a 50% probability here that the rate of return will be 20%. There is an 89% probability that the rate of return will be greater than 0, and a 10% probability that it will be 50% or more.

35. It can be appreciated that the array of information provided by Fig. A-9 is more valuable than simply a mean rate of return and perhaps even some extreme values of so-called minimum and maximum figures. The probability of suffering a loss, ie less than 0% rate of return, is an important aspect of the investment appraisal being of more concern for the conservative investor than for the gambler-type. Because there are no

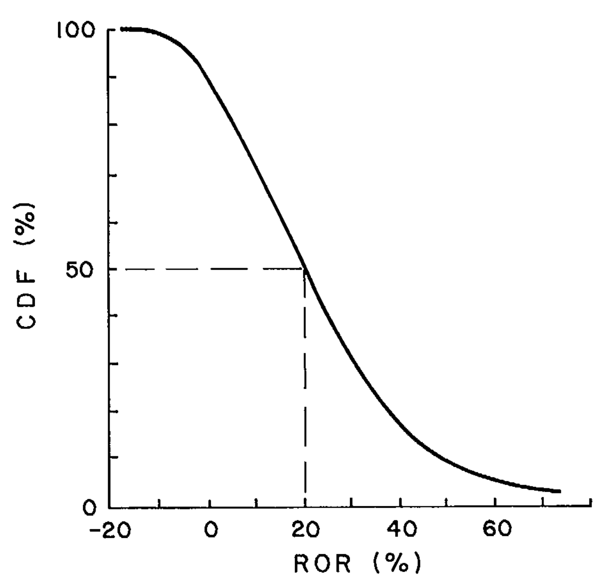

Fig. A-9 - A cumulative distribution function, CDF, for prospective rates of return, ROR (A-9).

absolute criteria for judging investments, the probability estimates are used in different ways by different individuals and companies.

36. One objection to the use of risk analysis is that it is difficult to express input variables in terms of ranges and probabilities. Experience in other industries has shown that the task is not as difficult as may appear. Often information on the degree of variation in factors is readily available from past experience. Even for factors that have no history, a "best guesses" is worth using. The lack of knowledge is, in itself, important information. It is, in fact, this lack of knowledge which may distinguish one investment possibility from another. The less certainty there is in an average estimate, the more important it is to consider the possible variation in that estimate.

37. The concept of conservatism in slope design thus requires modification. Rock mechanics analysis must provide the best estimate of the probability of instability so that appropriate costs can be assigned in the risk analysis for the financial determination of optimum wall angles. The consequences of instability remain under control of the operators, who can recognize its possibility and ensure safety by appropriate monitoring. Any overriding selection of a conservative slope reliability number could produce a financial analysis biased away from the optimum.

38. Steepening the ultimate slope angle will result in increasing reserves and/or decreasing the stripping ratio. Steepening the working slope angle will result in decreased preproduction stripping and/or a different mining sequence. Slope angle is only one among the controllable variables, eg others are stripping ratio or wall angles, milling capacity, discount rate, capital expenditure schedule and mining sequence. The uncontrollable variables are reserves, grade, price, mill recovery, tax and royalty rates, depreciation and depletion, and political changes.

39. Three financial criteria can be used: (1) the net present value (NPV), (2) the discounted cash flow rate of return (ROR) and (3) the wealth growth rate of return. The NPV criterion is the difference between the present value of the positive and negative cash flows, discounted at a predetermined interest rate. This rate of interest is usually the firm's cost of capital. The project under examination is accepted only if NPV is greater than zero. The main appeal of the NPV method is that it takes explicit account of the time value of money, whereas the main difficulty is to determine the firm's cost of capital, ie, the discount rate. Discounted cash flow rate of return is that discount rate at which the present value of positive cash flows equals the present value of negative cash flows. The method produces a rate that may not have any direct relation to the firm's cost of capital, avoiding the difficulty of the NPV method. The method, however, implicitly assumes that the firm's reinvestment rate of the positive cash flow is the same as the ROR. The wealth growth rate is not yet as well known as the previous two methods (A-11). It specifies an explicit reinvestment rate for positive cash flows appropriate to the firm. The wealth growth rate is defined as the rate that equates the sum of the initial negative cash flows, or investment, with the future value of all positive cash flows compounded at the firm's prespecified reinvestment rate.

Benefit-Cost

40. Optimum angles must be selected taking into account the economics of the entire operation. An important step in conducting such an optimization analysis is to establish quantitative relationships between slope angles and various operating costs.

41. A benefit-cost analysis is performed to examine the incremental benefits and costs of steepening wall angles. A comprehensive risk analysis is then made to integrate this information into the investment analysis so that the effects of the various feasible slope angles on the ultimate financial criteria (eg rate of return) can be calculated and the optimum slope angle determined.

42. When probabilities and costs of insta-

bility have been estimated for one slope design sector, the incremental effects on cash flow of a slope angle steeper than some base case can be examined. Using the Monte Carlo approach, a series of simulations of the mining of the ore provide annual increments in benefits and costs for the life of the mine. The net present values are computed. After a series of simulations, the average increment in net present value together with its variability can be determined for each increase in slope angle. Preliminary optimum slope angles can then be selected.

43. The incremental benefits and costs of steeper slopes are then utilized in the risk analysis program to determine the overall financial impact of steeper slope angles. Optimum wall angles are selected using the information on averages, variability and sensitivity, and taking into account intangible factors that have not been included in the calculations.

44. A simple example is as follows:

Wall height feet	Wall height metres	Probability of instability for a working slope angle of 25°	Probability of instability for a working slope angle of 35°
0	0	0.0	0.0
200	61	0.02	0.10
400	122	0.04	0.15
600	186	0.06	0.20
800	244	0.09	0.25
1000	305	0.10	0.30
1200	366	0.105	0.35
1400	427	0.11	0.40
1600	488	0.12	0.45
1800	549	0.14	0.50

45. A benefit-cost analysis is conducted to determine the feasibility of steepening the working slope angle from 25° to 35°. Past operations provide the above data for 25°; rock mechanics analyses are used to determine the probability of instability for 35°. A daily mining rate of 25,000 tons is used. The mining cost is assumed to be \$0.25/T and cost of money 15%.

46. The cost of instability includes the assumption that any cleanup would be done by the mine itself at a cost of only 25% more than normal operating costs. The result of 100 simulations using the Monte Carlo method shows the increment in NPV of using 35° to mean \$1,748,000 with a standard deviation of \$1,048,000. The range in incremental NPV is from -\$545,000 to \$4,673,000.

47. The analysis was done a second time assuming clean-up to be performed by an outside contractor at a unit cost 300% more than normal mining costs. In this case, the average increment in NPV was \$328,000 with a standard deviation of \$2,515,000. The range in results for the 100 simulations was from -\$5,172,000 to \$7,349,000.

48. In the first case, there is a 7% probability of a negative result, and in the second case a 44% probability. In the first case it might be worthwhile to increase the slope angle, whereas in the second case there is little incentive to do so.

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APPENDIX B

MINE PLANNING

A HYPOTHETICAL CASE OF A PRELIMINARY LAYOUT

INTRODUCTION

1. The preparation of preliminary pit layouts is described in this Appendix to illustrate the context into which wall design must fit. Layouts must integrate such details as mineable reserves, topography, pit outline, dump location, service corridors and plant facilities. A hypothetical iron deposit is used as an example. Input includes results of a feasibility stage site investigation.

SITE CONDITIONS

GENERAL

2. The Mining Company's iron ore operations are located in the Canadian Shield and produce 3.0 million long tons (3.05 million tonnes) annually of 65.6% Fe pellets. These are shipped by rail and ship to customers. Mining operations at the present pit are drawing to a close as the deposit nears the end of its economic life.

3. Fairly extensive diamond drilling has outlined a new deposit about 5,000 ft (1,500 m) southeast of the concentrator and pellet plant (Fig.B-1 and B-2). It has geological reserves in excess of 230 million tons (234 million tonnes).

4. Recognizing that the new open pit could eventually be up to 5,000 ft (1,500 m) long, 2,500 ft (750 m) wide and 1,000 ft (300 m) deep, a preliminary layout is required for regulatory agencies.

5. The deposit is a canoe-shaped taconite orebody. The major axis is oriented at 045°. It lies obliquely across the flank of Donald Ridge as shown in Fig.B-1. A normal fault, oriented at 033°, bisects the deposit as shown in Fig.B-2. Reserves were determined using an assay cut-off grade of 16% Fe. Grade variations within the deposit are not extreme and contaminants are minor. Generally, however, mineralization south of the normal fault is of higher grade, where it appears that percolating ground water has leached out and reduced the silica content.

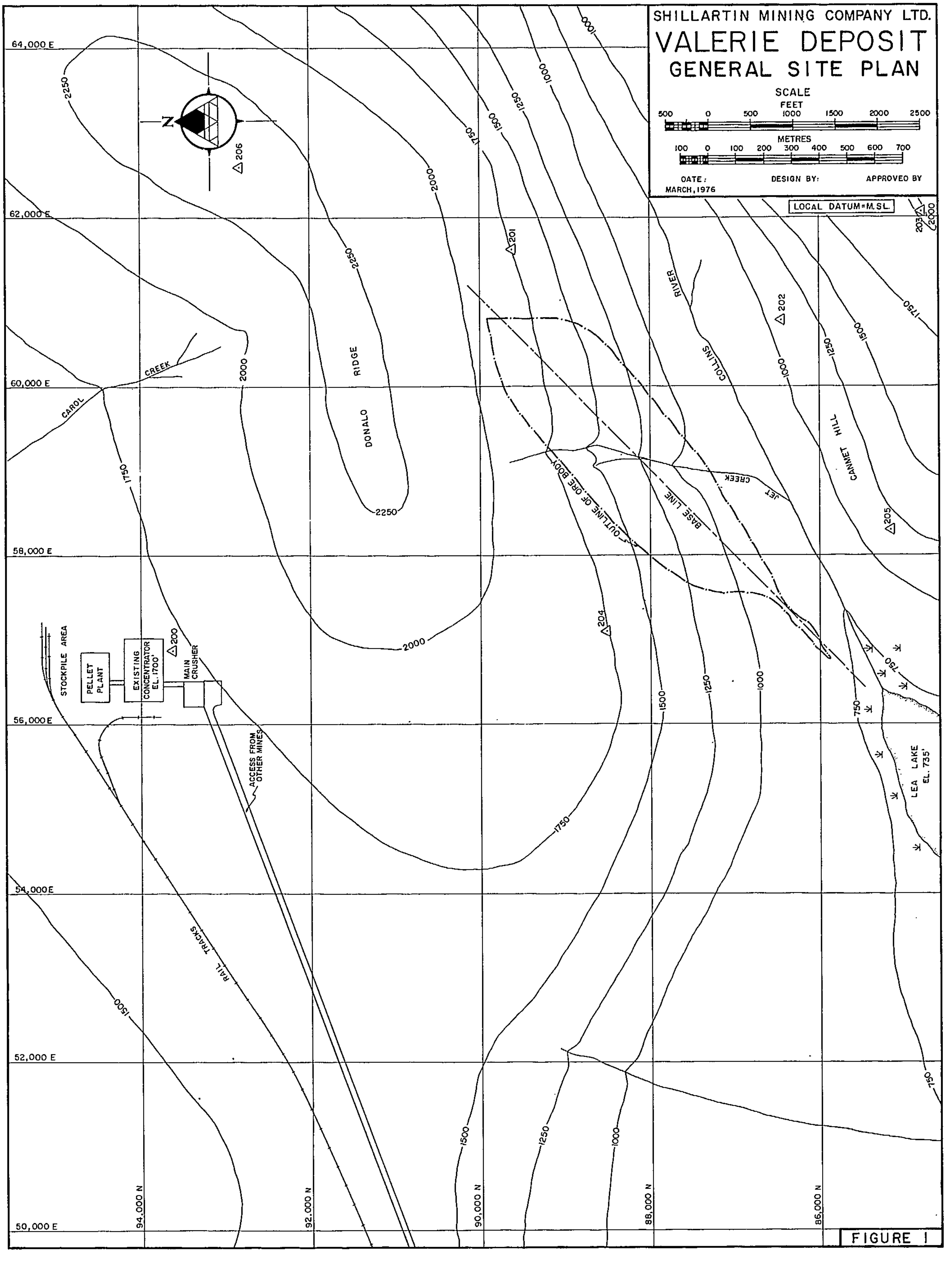
6. Wall rock on the north side of the fault is hard iron formation. There is a minor foliation approximately parallel to the orebody outline. To the south of the fault, the walls consist of leached iron formation, and foliation is rarely apparent.

7. Glaciation has left the terrain gently rounded and blanketed with up to 50 ft (15 m) of uniform silty sandy till. The higher north-eastern end of the deposit is covered with less than 20 ft (6 m) of overburden. Large boulders are rare and do not exceed 12 in. (30 cm) in diameter.

8. Jet Creek crosses the centre of the deposit before joining the Collins River which runs parallel to the southeast wall before draining into Lea Lake. Flow rates show wide seasonal fluctuations. Data from an upstream provincial monitoring station indicate that flows in Jet Creek vary between 25,000 gpm (1600 l/sec) and zero, and that Collins River flows can vary between 60,000 gpm (3800 l/sec) during the spring run-off and 5000 gpm (300 l/sec) in late summer.

9. Vegetation consists mainly of black and white spruce with some jackpine. Local areas have aspen and birch. There is muskeg around Lea Lake. Logging operations ceased in the late 1930's, and damage from forest fires since then has been minor. No large mammals have been sighted in the area for some years. The Collins River-Lea Lake system supports a small commercial fishery and some recreational fishing.

10. The company owns all the land and mineral rights over the entire area shown in Fig.B-1;


SHILLARTIN MINING COMPANY LTD.
VALERIE DEPOSIT
GENERAL SITE PLAN
SCALE
FEET
500 0 500 1000 1500 2000 2500
METRES
100 0 100 200 300 400 500 600 700
DATE: MARCH, 1976
DESIGN BY:
APPROVED BY
LOCAL DATUM = M.SL.
64,000 E
62,000 E
60,000 E
58,000 E
56,000 E
54,000 E
52,000 E
50,000 E
94,000 N
92,000 N
90,000 N
88,000 N
86,000 N
N
DONALO RIDGE
CAROL CREEK
COLLINS RIVER
JET CREEK
CANMET HILL
BASE LINE
OUTLINE OF ORE BODY
STOCKPILE AREA
PELLET PLANT
EXISTING CONCENTRATOR EL. 1700'
MAIN CRUSHER
ACCESS FROM OTHER MINES
RAIL TRACKS
LEA LAKE EL. 735'
200
201
202
203
204
205
206
2250
2000
1750
1500
1250
1000
750
FIGURE 1

there are no houses. Most employees live in Louiston about 12 miles (19 km) away.

11. Present operations are typical of a truck-and-shovel taconite operation. Existing equipment in good condition will be re-assigned. This equipment will include one 15 cu yd shovel and one large blasthole rotary drill. Only two 150 short ton (136 tonne) capacity trucks will be re-assigned; the remainder will be scrapped. The new pit dimensions, operating techniques and equipment must be compatible with those in the existing pit. Benches are currently 40 ft (11 m) high in both ore and waste; haulage roads are limited to an 8% maximum adverse grade and are 100 ft (30 m) wide, including a drainage ditch on one side.

12. The current production rate, which must be sustained is 3.0 million tons (3.05 million tonnes) of pellets per year. The pellet grade is 65.6% Fe, 4.5% SiO_2. Production is not subject to seasonal variations, and there is no provision for stockpiling low grade ore. A royalty of $1.00 per ton ($0.984 per tonne) of pellets is payable to the province.

13. Mining and environmental regulations are integrated into the layout. Chapter 10 on environmental planning provides guidance on work that may be necessary. Requirements in this case include: (a) submission of a mining plan for approval by the mine inspector; (b) height of benches should be no higher than considered safe for the equipment being used; (c) contaminated surface waters should be confined and treated so that upon discharge, suspended solids do not exceed 15 mg/l, and (d) a comprehensive environmental assessment report must be prepared.

Economic Geology

14. Table B-1 lists average assays for each section. Alumina (Al_2O_3) and manganese (Mn) are of minor significance. Moisture varies between 1 and 2% (dry basis). One anomalous block of ground below the 16% Fe cut-off grade is adjacent to the normal fault on sections 33 and 34. Geological reserves are 232 million tons (236 tonnes) at an average grade of 29.8% Fe.

Table B-1: Ore grades by section

Section	North of Fault % Fe	South of Fault % Fe
27	-	30.7
28	-	31.2
29	27.9	31.0
30	28.1	30.9
31	29.0	30.7
32	28.0	31.8
33	27.8	29.8
34	28.2	30.3
35	29.1	-
36	28.7	-
37	29.1	-

Structural Geology

15. The normal fault extends almost the full length of the deposit and contains gouge material. The fault zone varies in width between 6 in. (15 cm) and 4 ft (1.2 m).

16. The design sectors are identified in Fig. B-2. In Design Sector 1, both the ore and waste formations are hard, with uniaxial compressive strengths greater than 25,000 psi (170 mPa). They have similar joint orientations. Joints are tight, have minor infilling and are generally smooth. The most common joint set is parallel to the foliation with a mean strike and dip of 044°, 55°. Other sets are 290°, 47° and 170°, 80°. No information is available on the length of joints or large-scale roughness. The average laboratory-determined residual friction angle is 39°. The joint spacing ranges typically from 6 to 12 in. (15 - 30 cm). Plane shear sliding is possible for 3-d-wedges formed by the 044°, 55° and 170°, 80° sets.

17. An ultimate slope angle of up to 55° seems possible, provided adequate drainage measures to control water pressures are instituted and the dispersion of dip of the critical joint set is small. This figure is used for layout purposes. When stripping operations have exposed sufficient material to assess the structural properties - dip, strength, roughness and lengths - of the

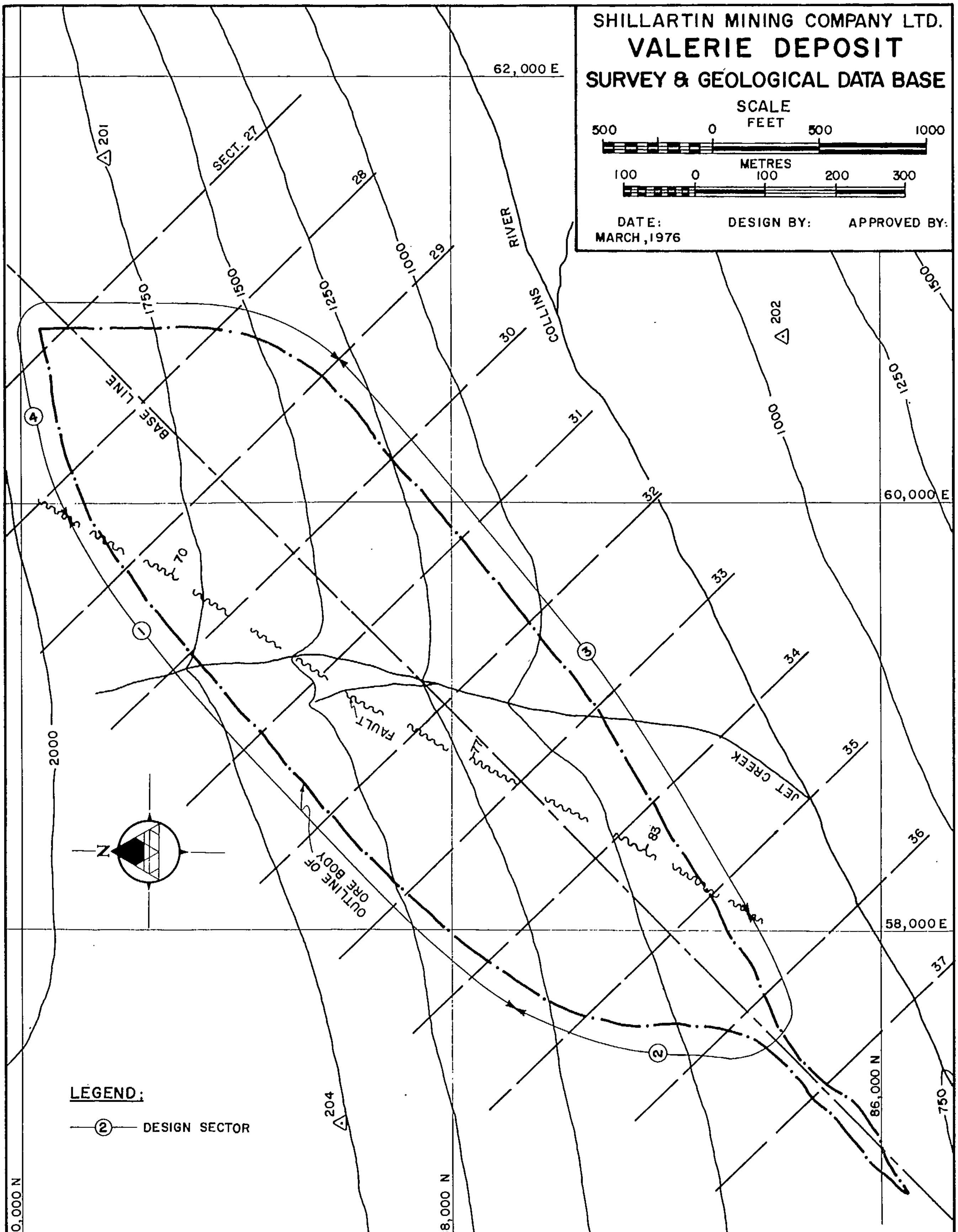

SHILLARTIN MINING COMPANY LTD.
VALERIE DEPOSIT
SURVEY & GEOLOGICAL DATA BASE
SCALE
FEET
500 0 500 1000
METRES
100 0 100 200 300
DATE: MARCH, 1976
DESIGN BY:
APPROVED BY:
62,000 E
60,000 E
58,000 E
90,000 N
88,000 N
86,000 N
SECT. 27
28
29
30
31
32
33
34
35
36
37
201
202
204
2000
1750
1500
1250
1000
750
COLLINS RIVER
JET CREEK
BASE LINE
FAULT
OUTLINE OF ORE BODY
70
77
83
N
LEGEND:
DESIGN SECTOR

FIGURE 2

fractures a stability analysis would be conducted. Safety berms are required for protection against rockfalls because of the relatively close joint spacing. Controlled blasting near ultimate slopes may be required.

18. In Sector 2, potential instability will be structurally controlled. Joints have the same characteristics as the sets found in Sector 1. The only change from Sector 1 is the orientation of the wall. Three-d-wedge slides would be possible. The ultimate slope angle is assumed to be 37°.

19. In Sector 3, both the ore and waste are soft with mean uniaxial compressive strengths of 8000 and 10,000 psi (55 and 69 mPa) respectively. The ore appears to have had a greater proportion of the original silica leached out. Both ore and waste materials are weathered. Poor core recovery of approximately 50% has been experienced in several drillholes. Many joints, especially those which are nearby vertical, are clay-filled. Laboratory measurements of joint friction yielded residual values ranging from 18° to 35°, although the lowest angles were found only on nearly vertical joints. Three joint sets have been tentatively identified which in order of decreasing importance, are 220°, 49°; 320°, 90°; 045°, 40°. Joint spacings average 1 to 2 ft (30 - 60 cm). All three sets show wide dispersions of attitude. The sets appear to be common to both ore and waste rocks. Drill crews estimated that groundwater flow rates into boreholes have varied from 0 to 15 gpm (1 l/sec).

20. Two modes of potential sliding govern stability in this sector. Plane shear sliding on the 220°, 49° set is possible. Rotational shear is also possible. Until the properties of the rock mass can be assessed in greater detail, the ultimate slope angle is not to exceed 36°. This choice of angle is supported by analysis of existing slopes. Drainage will be required.

21. In Sector 4, characteristics of the ore and waste are essentially the same as in Sector 3. The changing orientation of the pit wall as it swings around the end of the pit is the only difference. An ultimate slope angle of 42° is used for this sector. Drainage is to be used in Sector 4. It will be particularly important to control seepage through the vertical joint set.

22. Controlled blasting near the ultimate slope will be required. Light blasting adjacent to individual bench faces is likely to be required to avoid major ravelling problems along the vertical joint set.

Hydrology

23. The hydrology of the area is complex and little information is avaliable. Few exploration holes were logged to ascertain water conditions, and most were sealed on completion. No monitoring has been conducted to determine the groundwater level and its seasonal fluctuations.

24. Some drill samples of overburden were bagged and preserved. Classification tests on these samples show that the overburden is predominantly silty sand. This soil is thought to be uniformly distributed, and no important beds of gravel, sand or clay have been noted.

25. The groundwater table is believed to be 100 ft (30 m) below ground surface, except where it approaches the Collins River. It is known that Jet Creek is not spring-fed. Drilling records from holes near the normal fault suggest that it plays an important role in controlling the ground water regime. The water table appears to drop about 50 ft (15 m) on the south side of the fault.

26. Experience indicates that the run-off coefficient is about 0.15. The recharge zone is assumed to be north and east of the orebody along Donald Ridge. Low porosities exist in the cherty metamorphosed sandstones north of the fault. Because of the close joint spacing, cross structure leakage along joints and fractures may account for 75% of the active ground water regime. Porous rocks are present south of the fault, and joints are somewhat open. Permeability is likely to be high with wide variations.

27. Peripheral wells are to be used, extending an average of 200 ft (61 m) below the ultimate pit floor for draining the orebody before mining commences. Depth of the wells would be as much as 1000 ft (300 m). The alternatives would be either

adits, which would be more costly, or horizontal drains which would require the mine to be partially developed before they could be installed. These factors would offset three major benefits of dewatering, ie (a) increasing operating inefficiency by keeping the pit roads and floor dry; (b) draining slopes; and (c) reducing the weight of material moved. Up to 30% of the dewatering wells can be expected to be dry. Wells are to be cased 10 in. (25 cm) in diameter and spaced 300 to 400 ft (90 - 120 m) apart. Initial pump capacities will be 300 to 600 gpm (19 - 38 l/sec).

28. Initial evidence suggests that wells will not be necessary in the northeast corner of the pit, since excavation is unlikely to penetrate the groundwater horizon. A single well should be drilled to the south, midway between section lines 29 and 30, to evaluate this hypothesis.

29. A graded filter comprising mine waste rock on top of 3 - 6 in. (8 - 15 cm) of sand will be placed on the expanded overburden face along the Collins River. It will be supplemented with a collector ditch at the toe of the face. Seepage through the embankment is to be returned directly to the river.

30. Jet Creek will be completely removed by mining. Any remaining surface run-off around the pit will be collected by a peripheral interceptor ditch and diverted into drainage channels. Water in the pit will be handled by sump and pump arrangements.

MINE ECONOMICS

Operating Data

31. Table B-2 lists data that were either assumed or obtained by analysis of the most recent 12-month records.

Stripping Ratio

32. The stripping ratio is the ratio of the quantity of waste material which must be removed from the mine to obtain a given unit of the desired ore. The units adopted may be of volume or weight. The stripping ratio may be expressed as: (a) tons per ton; (b) cubic yards (metres) per cubic yard; or (c) cubic yards (metres) per ton.

33. Three different stripping ratios must be considered: (a) the overall stripping ratio, also known as the general stripping ratio; (b) the cut-off stripping ratio, also known as the breakeven stripping ratio; and (c) the instantaneous stripping ratio, also known as the current stripping ratio.

34. The overall stripping ratio involves the total amount of waste material which must be removed during the life of the mine. The overall stripping ratio is not an operating parameter. It may be used to compare different designs of the same pit or designs of different pits. It is used to determine waste dump space and capital equipment requirements. It is obtained only after the ultimate pit design is completed. The overall stripping ratio depends on such factors as the shape, dimensions and attitude of the orebody, the geometry of the excavation and the topography of the area.

Table B-2: Operating data

Item	$
Gross revenue FOB	
Port Gibson	0.472 per iron unit
Pelletizing cost	2.94 per ton pellets
Concentration cost	3.24 " " "
Rail freight	2.75 " " "
Yard costs	0.42 " " "
Overhead	0.75 " " "
Royalty	1.00 " " "
Mining cost	0.82 per ton mined
Waste stripping cost	0.B2 " " "
Overburden stripping cost	0.95 per cu. yd. soil.
Contingency allowance	10% of estimated ore production cost
Metallurgical recovery: pelletizing	98%
Metallurgical recovery: concentrating	85%
Ore and rock density (average)	11.5 cu ft/ton (0.23 m /t)
Overburden density	18.0 cu ft/ton (0.50 m /t)
Swell factor, rock	1.3 (bank to dump)
Swell factor, overburden	1.2
Mean pellet grade	65.6% Fe
Minimum pit bottom width	150 ft (46 m)

35. The cut-off stripping ratio is the amount of waste that can be economically excavated to mine one ton of ore. Stated slightly differently, mining at the cut-off stripping ratio results in a breakeven operation, showing neither profit nor loss. The cut-off stripping ratio (COSR) is a tool to determine the economic pit limits. The COSR is sensitive to changes in operating cost, product price, processing and transportation costs, and waste stripping costs. It is based on the extra cost incurred to obtain extra revenue.

36. The instantaneous stripping ratio applies to a specific point in time, or to a given period, such as one month. It is a production scheduling tool. The best instantaneous stripping ratio at any time is that which will maximize the present value of total future profits. This ratio is sensitive to changes such as an increase in mining costs without an equivalent increase in selling price. The instantaneous stripping ratio may range from infinity at the opening of a new mine to zero when mining the final cut.

37. Theoretically, the ultimate pit outline is that within which all ore above the cut-off grade can be mined at a profit. At the limit of the ultimate pit, the incremental cost of stripping is balanced by net income from the ore mined. This is calculated as follows:

$$COSR = \frac{A - B}{C}$$

where COSR is the cut-off stripping ratio, A is the gross value/ton of ore, B is the incremental or variable cost/ton for production and processing, excluding stripping, and C is the incremental or variable cost of stripping per unit of waste.

38. The COSR is not an operating control parameter. Its use is limited to preliminary pit layouts. The COSR may be expressed in any consistent set of units, but for the graphical design method used here it must be in volume units.

39. The first step in the calculation of COSR is to obtain the concentration ratio (the tons of raw feed required to produce 1 ton of final product). The concentration ratio is used to convert costs per ton of product to costs per ton of raw feed. In this case: η tons of raw feed at g% Fe enters the concentrator; R_1 is recovered and the balance is tails; R_2 is recovered from the pellet plant and the balance is in losses. 1 ton of pellets at G% Fe is produced. The concentration ratio, $\eta = \frac{G}{g \; R_1 \; R_2}$. For example, if G = 65.6% Fe, g = 27%, R_1 = 85% and R_2 = 98% then η = 2.92

40. The second step is to calculate the gross value per ton of ore. For example, the gross value per unit of iron sold is equivalent to \$47.20/ton. Hence the gross value per ton of ore = \$47.20 x 0.27 x 0.85 x 0.98 = \$10.62 per ton of iron ore.

41. The third step is to estimate the component costs and calculate the total variable cost per ton of ore mined. For example, when the grade is 27% Fe and the concentration ratio is 2.92 the following figures apply:

Item	Cost per ton pellets/2.92	=	Cost per ton mined
Concentrating	\$3.24		\$1.11
Pelletizing	2.94		1.01
Rail freight	2.75		0.94
Yard costs	0.42		0.14
Overheads	0.75		0.26
Royalty	1.00		0.34
Mining	-		0.82
		Sub total	4.62
Contingency at 10% of estimated operating costs............			0.46
Total estimated variable cost of production (excluding stripping).....			\$5.08

42. The fourth step consists of calculating COSR. For example,

$$COSR = \frac{10.62 - 5.08}{0.82} = 6.76 \text{ tons waste/ton ore}$$

43. In calculating the incremental or total variable cost of production, in this case it is appropriate to use constant concentrating and pelletizing costs because the average grades determined for each section by the geologist do

not vary by more than about 5%. If the range of grades had been much wider, it would have been necessary to know how the operating costs per ton (eg, crushing, flotation, power) varied and make allowances in the calculation. It is assumed that both revenue and costs inflate equal amounts. The variable cost of production does not include taxation, depreciation, interest on borrowed capital and overhead items unless they increase incrementally with mining, such as having to buy an additional truck. At the ultimate pit limit, revenue and cost just balance. There is no profit, and no profit-related taxation is applicable. After the ultimate pit has been outlined and preliminary operating schedules have been determined, tax, interest, depreciation and other overheads are included in the benefit-cost analysis. A contingency allowance is included at this first stage to reflect physical inaccuracies, eg, excessive dewatering could be required or estimated mining costs could be too low.

44. The COSR calculation must be repeated for each grade shown on the pit sections. With the assumption made about operating costs, the COSR has a linear relationship with grade. It is, therefore convenient, to select only a few grades and construct a graph as shown in Fig. B-3.

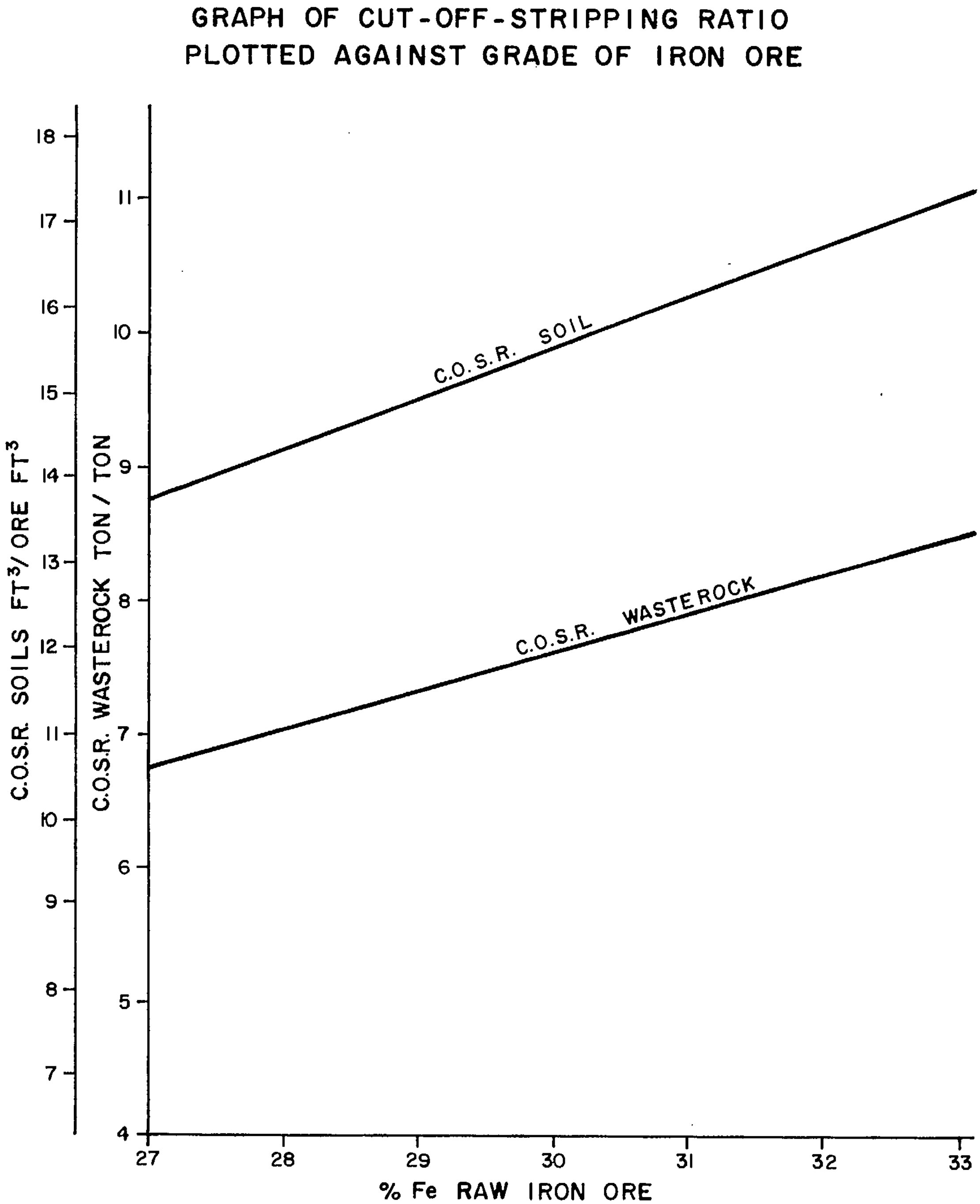

GRAPH OF CUT-OFF-STRIPPING RATIO
PLOTTED AGAINST GRADE OF IRON ORE
C.O.S.R. SOILS FT³/ORE FT³
C.O.S.R. WASTEROCK TON / TON
C.O.S.R. SOIL
C.O.S.R. WASTEROCK
% Fe RAW IRON ORE
18
17
16
15
14
13
12
11
10
9
8
7
11
10
9
8
7
6
5
4
27
28
29
30
31
32
33

FIGURE 3

MINE LAYOUT

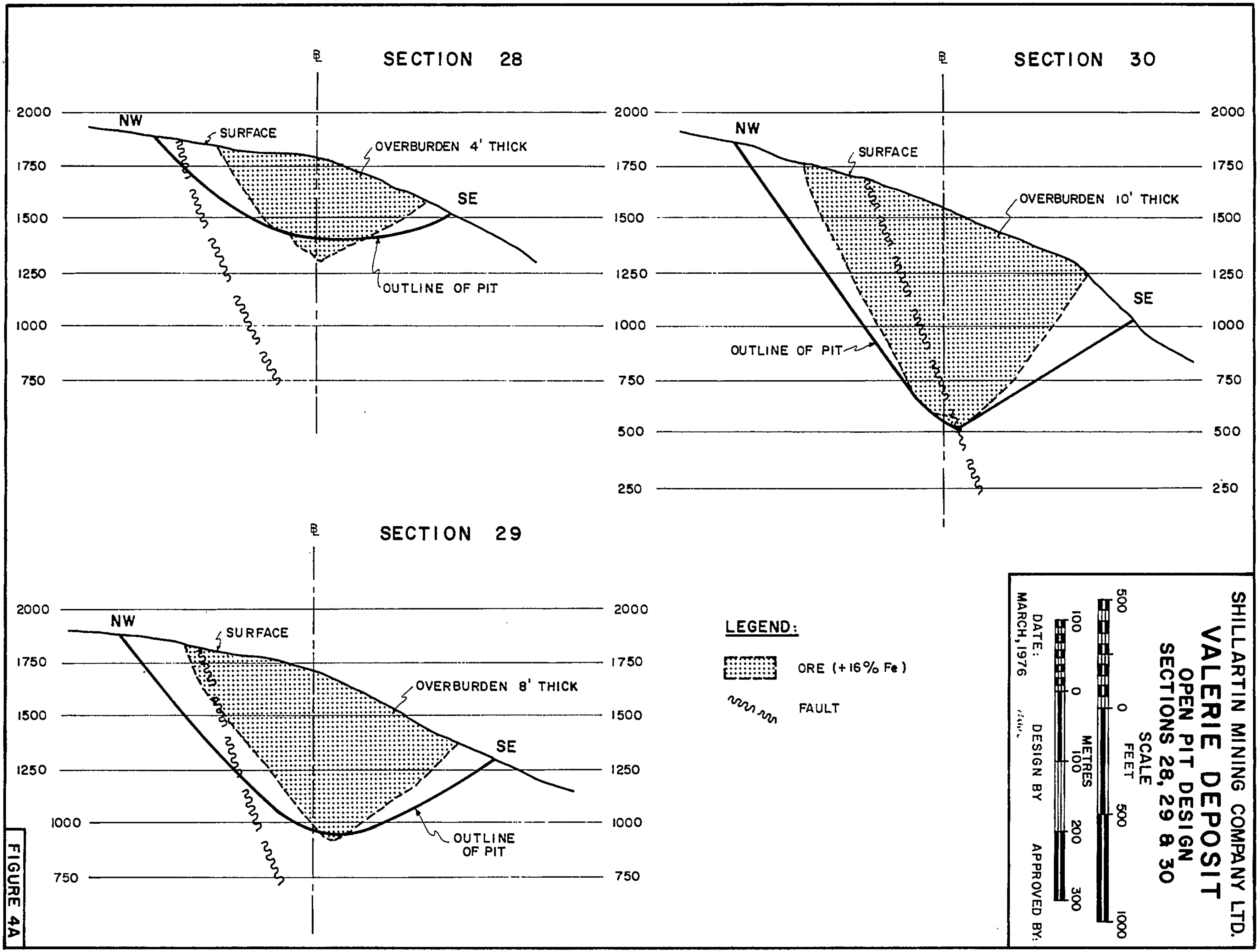
45. The objectives of a preliminary layout are to: (a) establish approximate operating plans for the deposit; (b) select dump and service locations with respect to roads and plant; and (c) provide input for the subsequent pit design using the financial analyses of Supplement 5-3.

Pit Layout

46. Each section drawn through the deposit is examined (Fig.B-4). The initial outline is determined using a minimum bottom width of 150 ft (46 m), the slope angle appropriate to that section and the COSR for the average grade of ore on that section. The specified wall angle is assumed to be in the plane of the section. The cut-off stripping ratio is determined by reference to the graph of grade vs COSR (Fig.B-3). The pit bottom elevation is adjusted up or down until the economic cut-off is determined.

47. The set of cross-sections is then superimposed on a longitudinal section along the baseline (Fig.B-5). The lowest elevation of the pit floor in this case occurs at approximately Section 34 with an elevation of 135 ft (41 m).

48. This initial analysis shows that the mineable extent of the deposit is from Section 28 to 36 inclusive. Sections 27 and 37 do not contain a sufficient width of mineralization to maintain a 150 ft (46 m) pit bottom at any elevation. It is good practice to remove all overburden and topsoil from any open pit area in the initial development stage. It is also good practice to use a 25 ft (8 m) berm width at overburden/rock contacts when the depth of overburden exceeds 20 ft (6 m). All sections to the southwest of Section 31 have over 20 ft (6 m) of overburden and hence incorporate such a berm width. The overburden and topsoil are



SECTION 28
SECTION 29
SECTION 30
NW
SE
SURFACE
OVERBURDEN 4' THICK
OVERBURDEN 8' THICK
OVERBURDEN 10' THICK
OUTLINE OF PIT
2000
1750
1500
1250
1000
750
500
250
LEGEND:
ORE (+16% Fe)
FAULT
SHILLARTIN MINING COMPANY LTD.
VALERIE DEPOSIT
OPEN PIT DESIGN
SECTIONS 28, 29 & 30
SCALE
FEET
500 0 500 1000
METRES
100 0 100 200 300
DATE:
MARCH, 1976
DESIGN BY
APPROVED BY:

FIGURE 4A

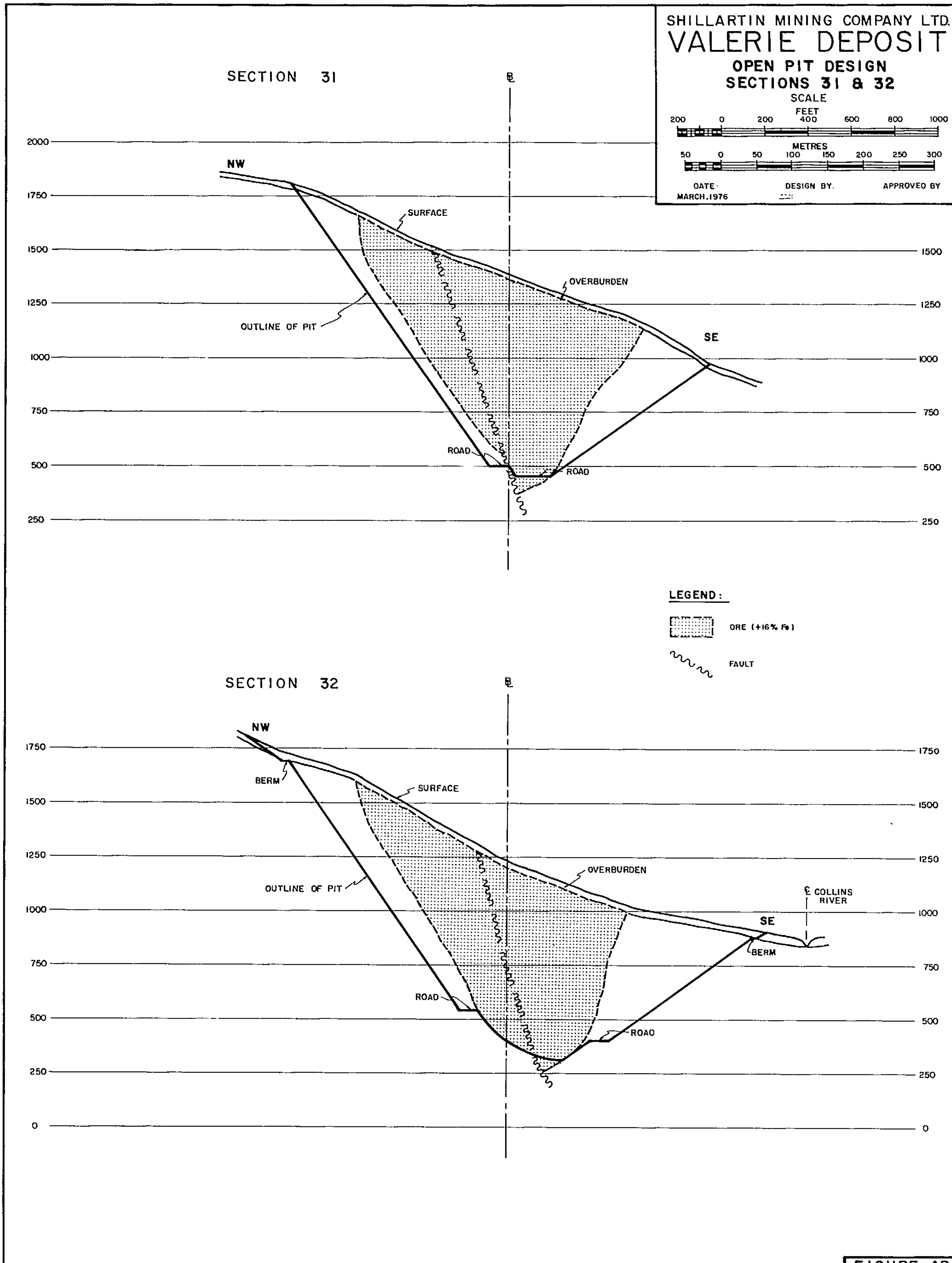

SHILLARTIN MINING COMPANY LTD.
VALERIE DEPOSIT
OPEN PIT DESIGN
SECTIONS 31 & 32
SCALE
FEET
200 0 200 400 600 800 1000
METRES
50 0 50 100 150 200 250 300
DATE MARCH, 1976
DESIGN BY.
APPROVED BY
SECTION 31
NW
SE
SURFACE
OVERBURDEN
OUTLINE OF PIT
ROAD
ROAD
2000
1750
1500
1250
1000
750
500
250
LEGEND:
ORE (+16% Fe)
FAULT
SECTION 32
NW
BERM
SURFACE
OVERBURDEN
OUTLINE OF PIT
℄ COLLINS RIVER
SE
BERM
ROAD
ROAD
1750
1500
1250
1000
750
500
250
0

FIGURE 4B

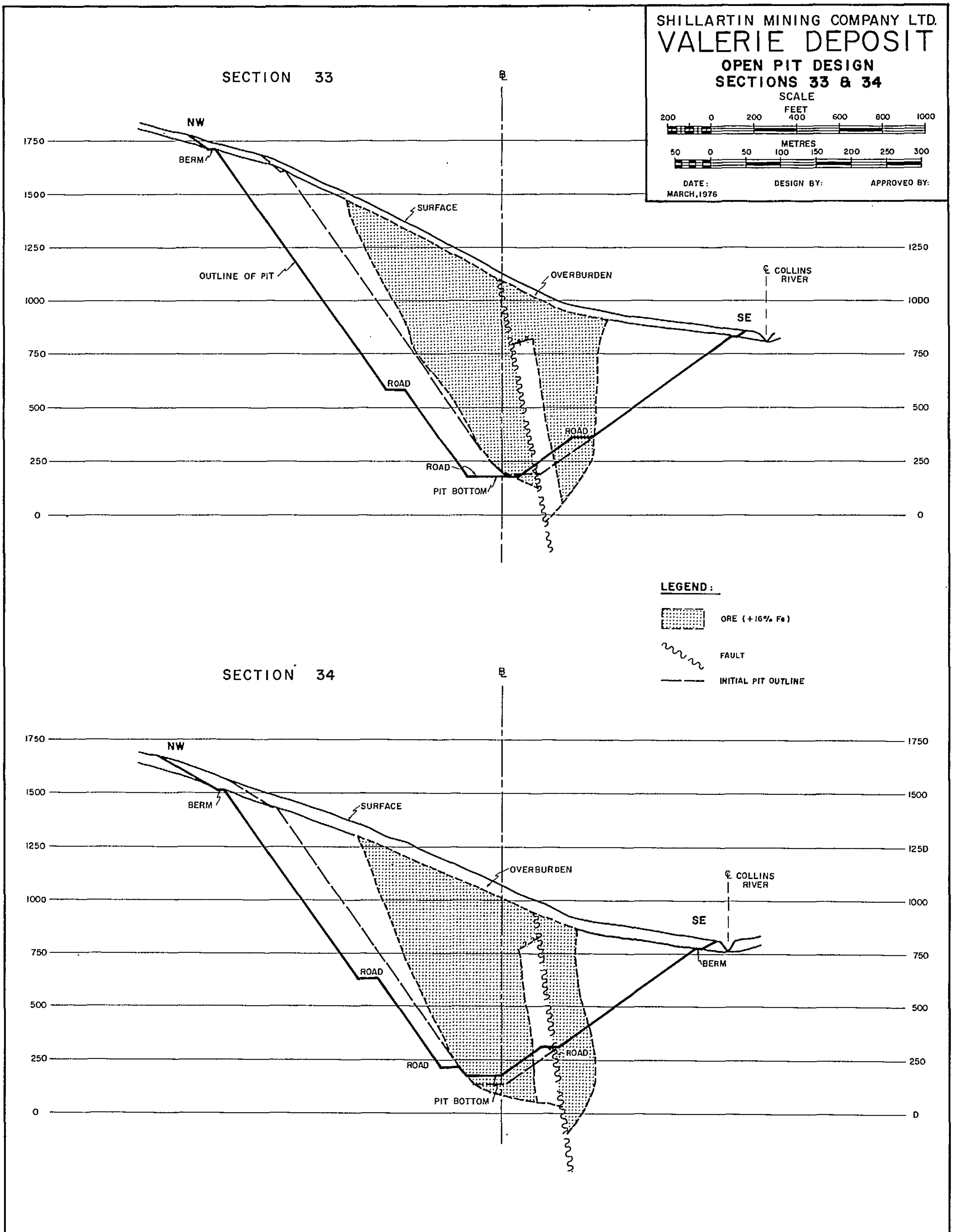

SHILLARTIN MINING COMPANY LTD.
VALERIE DEPOSIT
OPEN PIT DESIGN
SECTIONS 33 & 34
SCALE
FEET
200 0 200 400 600 800 1000
METRES
50 0 50 100 150 200 250 300
DATE: MARCH, 1976
DESIGN BY:
APPROVED BY:
SECTION 33
NW
BERM
SURFACE
OUTLINE OF PIT
OVERBURDEN
℄ COLLINS RIVER
SE
ROAD
PIT BOTTOM
1750
1500
1250
1000
750
500
250
0
LEGEND:
ORE (+16% Fe)
FAULT
INITIAL PIT OUTLINE
SECTION 34
NW
BERM
SURFACE
OVERBURDEN
℄ COLLINS RIVER
SE
BERM
ROAD
PIT BOTTOM

FIGURE 4C

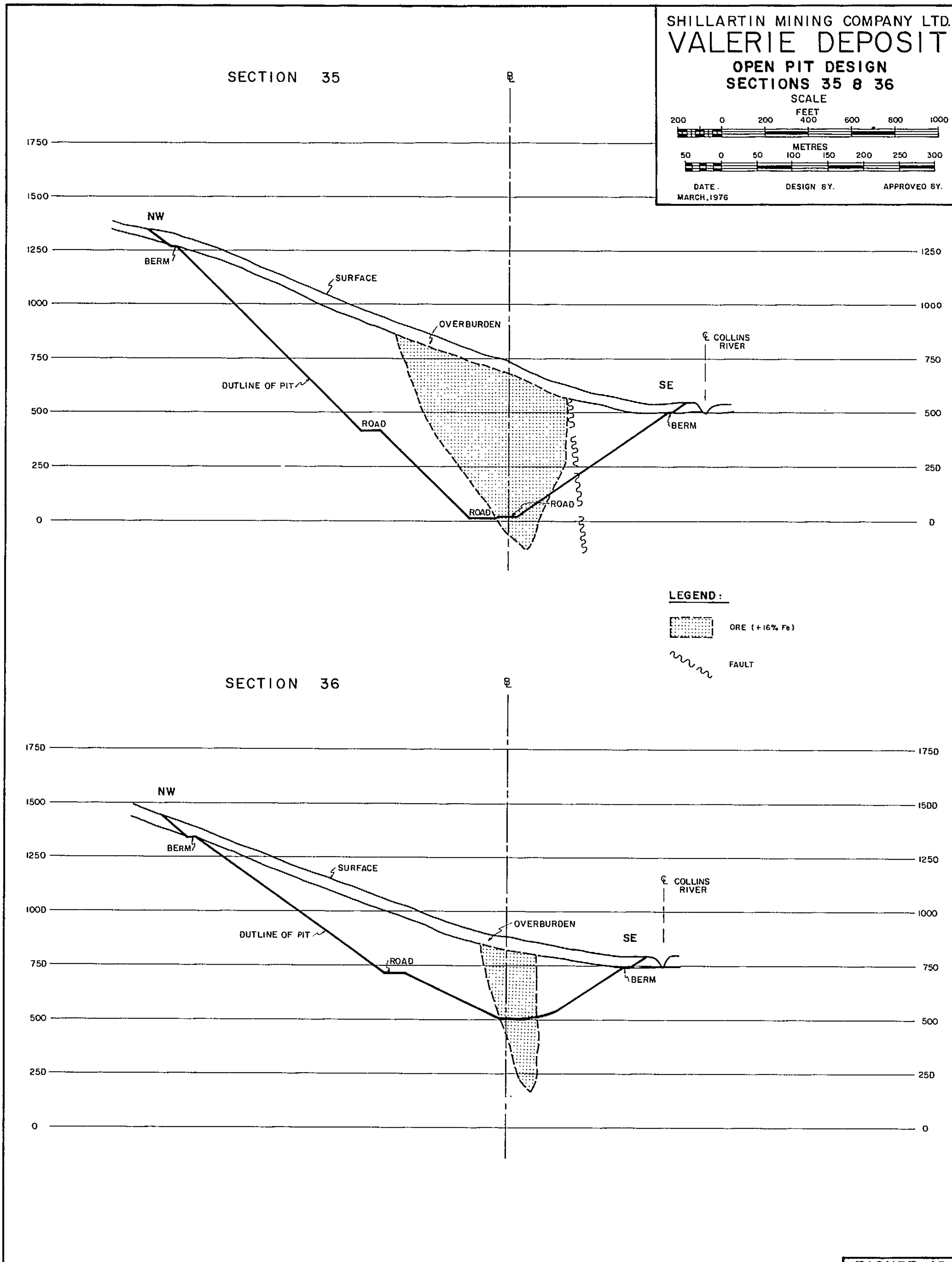
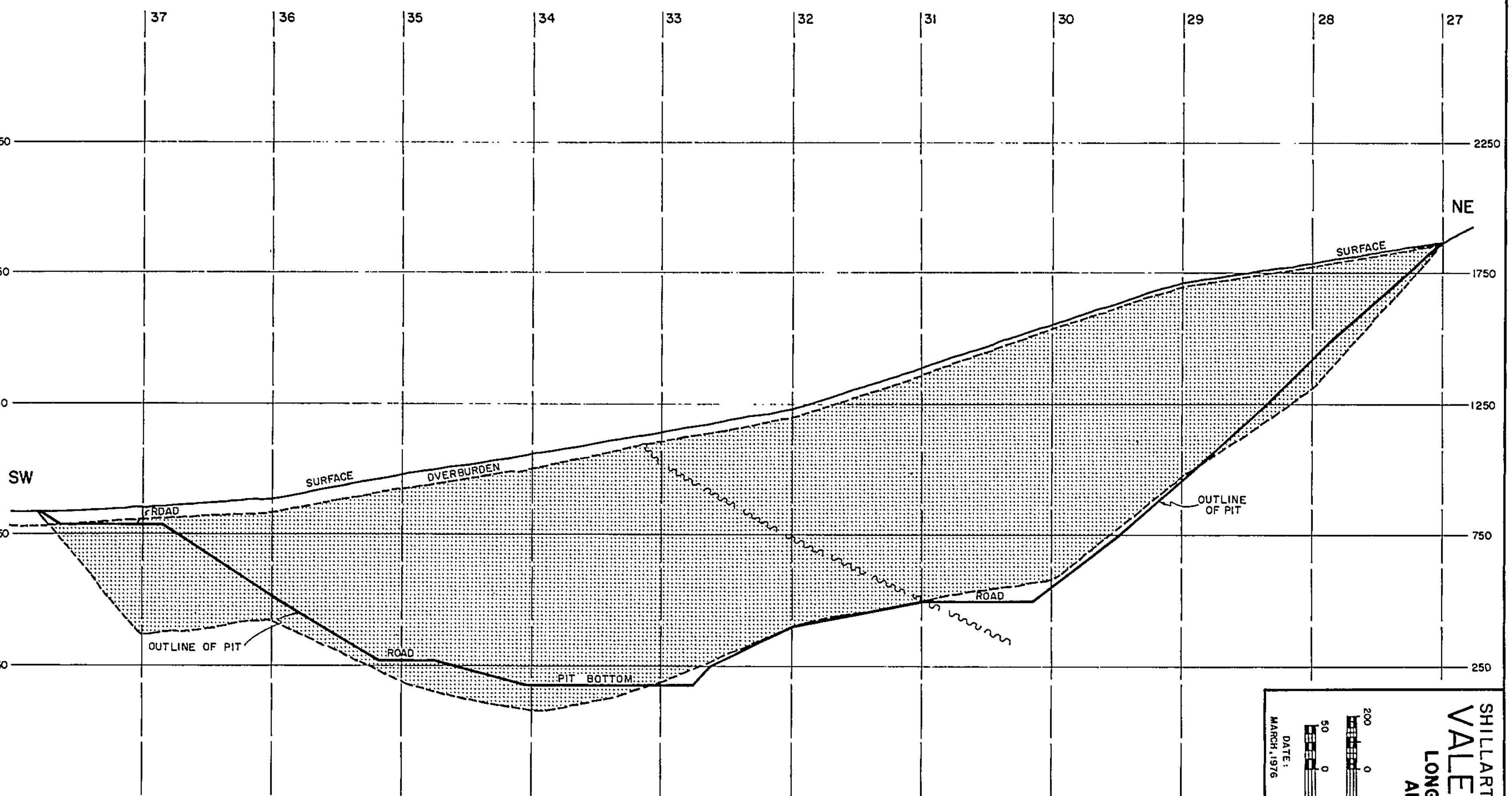

SHILLARTIN MINING COMPANY LTD.
VALERIE DEPOSIT
OPEN PIT DESIGN
SECTIONS 35 8 36
SCALE
FEET
200 0 200 400 600 800 1000
METRES
50 0 50 100 150 200 250 300
DATE. MARCH, 1976
DESIGN BY.
APPROVED BY.
SECTION 35
NW
BERM
SURFACE
OVERBURDEN
OUTLINE OF PIT
ROAD
ROAD
ROAD
SE
BERM
℄ COLLINS RIVER
1750
1500
1250
1000
750
500
250
0
LEGEND:
ORE (+16% Fe)
FAULT
SECTION 36
NW
BERM
SURFACE
OUTLINE OF PIT
ROAD
OVERBURDEN
SE
BERM
℄ COLLINS RIVER
1750
1500
1250
1000
750
500
250
0

FIGURE 4D



LEGEND:

ORE (+16% Fe)

FAULT

SHILLARTIN MINING COMPANY LTD.

VALERIE DEPOSIT

LONGITUDINAL SECTION

ALONG BASE LINE

SCALE

FEET

200 0 200 400 600 800 1000

METRES

50 0 50 100 150 200 250 300

DATE: MARCH, 1976

DESIGN BY:

APPROVED BY:

FIGURE 5

stockpiled separately.

49. The size of the pit is influenced by certain factors. The overall slope of the 16% assay grade contact on sections 28 and 29 is less than the maximum safe slope angle for that portion of the pit, hence total extraction may be possible. At depth, the slope of the contact allows for a progressive flattening on sections 30 to 34, giving minimum stripping. The north bank of the Collins River limits the southeast pit wall. The pit bottom must be determined by reference to the longitudinal section.

50. The river has cut through the overburden to bedrock. The pit should not encroach closer to the river than 150 ft (46 m) measured along the bedrock-overburden contact. Hence the river limits the pit between Sections 33 and 36 for some 1500 ft (457 m). The river is to be regarded as being fixed for this initial study in view of local commercial fishing. The full cost, benefits and environmental effects of such alternatives as moving or by-passing it through a culvert must be assessed in detail later.

51. The economic limits to the open-pit are reached on Sections 35 and 36. The initial layout must be transferred to a plan, where the outline of the crest is examined. The ultimate position of the pit bottom must be reexamined with reference to the ultimate haul road.

Haul Road

52. On the assumption that the highest levels of ore will be mined first, the open-pit workings would begin in the northeast and move progressively down towards the southwest. The ultimate haul road is the one required to mine the lowest portions of the orebody. Other access roads will be necessary during the operating life of the deposit, but none are as critical to mine design as the ultimate haul road, for which allowances must be made in all mine plans.

53. The point at which the ultimate haul road enters the pit is influenced by: (a) proximity to existing and future beneficiating or processing facilities; (b) areas suitable for waste rock disposal; and (c) lowest topographic point on the ultimate pit crest. The road is 100 ft wide (30 m) and has a maximum grade of 8%. The 100 ft (30 m) width allows for double lane traffic and overtaking, as well as for a drainage ditch on the inside edge and a windrow on the outside edge.

54. The road entrance which best suits these criteria is in the southwest, approximately adjacent to the baseline at Section 37. A trial road entering at this point on bedrock has an elevation of 780 ft (237 m) and must drop to elevation 135 ft (41 m), the lowest point in the pit. It would have a total length of 8000 ft (2430 m). The road could be laid out in an anti-clockwise direction, which would eliminate the need for a curve at the mine entrance and would keep it on the sheltered southern pit face. It would, however, pass into overburden before penetrating the rock wall.

55. By choosing a clockwise spiral ramp system, the road stays in rock all the way to the pit bottom. To reach elevation 135 ft (41 m), a corkscrew is necessary with the road running along the north wall from the entrance to Section 30, then back on the south wall to Section 35, on the north wall again to Section 33 and then to the pit bottom at Section 34. The required road widths can be superimposed on the two critical sections, 33 and 34. It is found that the required widths for the ramp pulls the pit bottom upwards by 100 ft (30 m). The extra stripping requirements associated with such a width make the lowest levels of ore more expensive to mine than the revenue they would produce, ie, they exceed the COSR. A new pit bottom of elevation 175 ft (53 m) at Section 33 is required. By rising between Sections 33 and 35 on the north wall, turning and going along the south wall to Section 30, turning again and going towards the road entrance along the north wall, all the constraints are met.

56. The economic limits of the pit are reached on all sections between 31 and 36. The slope angles on the sections used in this layout must be modified where the wall is not normal to the section. Slope angles are checked on both the section along the baseline and radial sections at the northeast and southwest corners. The slope of

the walls is critical to the overall shape of the pit, its depth and maximum recovery of the iron ore.

57. Having determined the ultimate shape of the pit at its economic limit, quantities are calculated as follows:

Iron ore	- 204.7 million tons at 29.7% Fe (208.0 million tonnes)
Waste rock	- 61.3 million bank cubic yards (46.9 million bank cubic metres)
Overburden	- 10.9 million bank cubic yards (8.3 million bank cubic metres)

58. A breakdown of these reserves by section is included in Table B-3. In determining these volumes and tonnages, each parallel section is assumed to have an influence of 250 ft (76 m) in each direction. The end effects of the curving benches must also be incorporated. Later, when more detailed scheduling is required, intermediate sections will be drawn.

Table B-3: Ore reserve summary

Section	Overburden million bank cubic yards	Waste rock million bank cubic yards	Ore million tons grade
28	0.11	1.38	11.02 - 31.2%
30	0.38	6.07	33.91 - 30.1%
31	0.78	4.86	30.85 - 30.2%
32	1.11	7.29	29.67 - 30.0%
33	1.50	12.81	29.38 - 28.4%
34	2.34	10.11	25.67 - 28.5%
35	2.36	9.18	16.55 - 29.1%
36	2.04	5.97	3.07 - 28.7%
TOTAL	11.90	61.35	204.68 - 29.7%
	(9.10)	(46.93)	(207.95)- 29.7%

59. Sections 33 and 34 highlight the use of the COSR together with the physical constraints to obtain the economic pit bottom. The physical constraints are that the pit bottom should be no less than 150 ft (46 m) wide, the south crest should be no closer than 150 ft (46 m) to the Collins River, and the overall wall angles should not exceed 55° and 36° on the north and south walls respectively.

60. The ultimate road design requires that three full road widths of 100 ft (30.5 m) must be incorporated on the north side of the pit. If the wall is pushed back 300 ft (91.5 m), the allowable COSR will be exceeded. If the road is located inside the initial pit limit, economic ore will be lost. In the COSR calculation, the additional 300 ft (91.5 m) width of waste excavation must be balanced by a proportionate increase in the ore intercept. The orebody width does not increase with depth, so that the only choice is to raise the pit bottom. Sections 33 and 34 (Fig.B-4C) show how the initial layout without the road had to be modified to meet the COSR criterion.

61. The benefit-cost analysis procedure of Supplement 5-3, gives a rigorous method of evaluating the benefits and costs associated with variations in slope angles. The relative merits of alternative designs can be evaluated in terms of increased benefits, primarily through increased ore production, or decreased costs, primarily through waste stripping. When used in conjunction with the INRISK financial risk analysis model, it is a powerful tool for the mine planner in optimizing mine design.

Dump Locations

62. Preliminary planning of the rock dumps, overburden and topsoil stockpiles must be an integral part of the ultimate pit plan. All dumps and stockpiles must be explicitly located for the following reasons: they must be clear of all mine workings, pipelines, roads, beneficiating plants and surface drainage features; haulage distance from the pit area to the dump point should be a minimum; the land should be owned by the company; other potential ore reserves amenable to open pit mining should not be affected.

63. From Table B-3, the broken volumes of material that must be moved are:

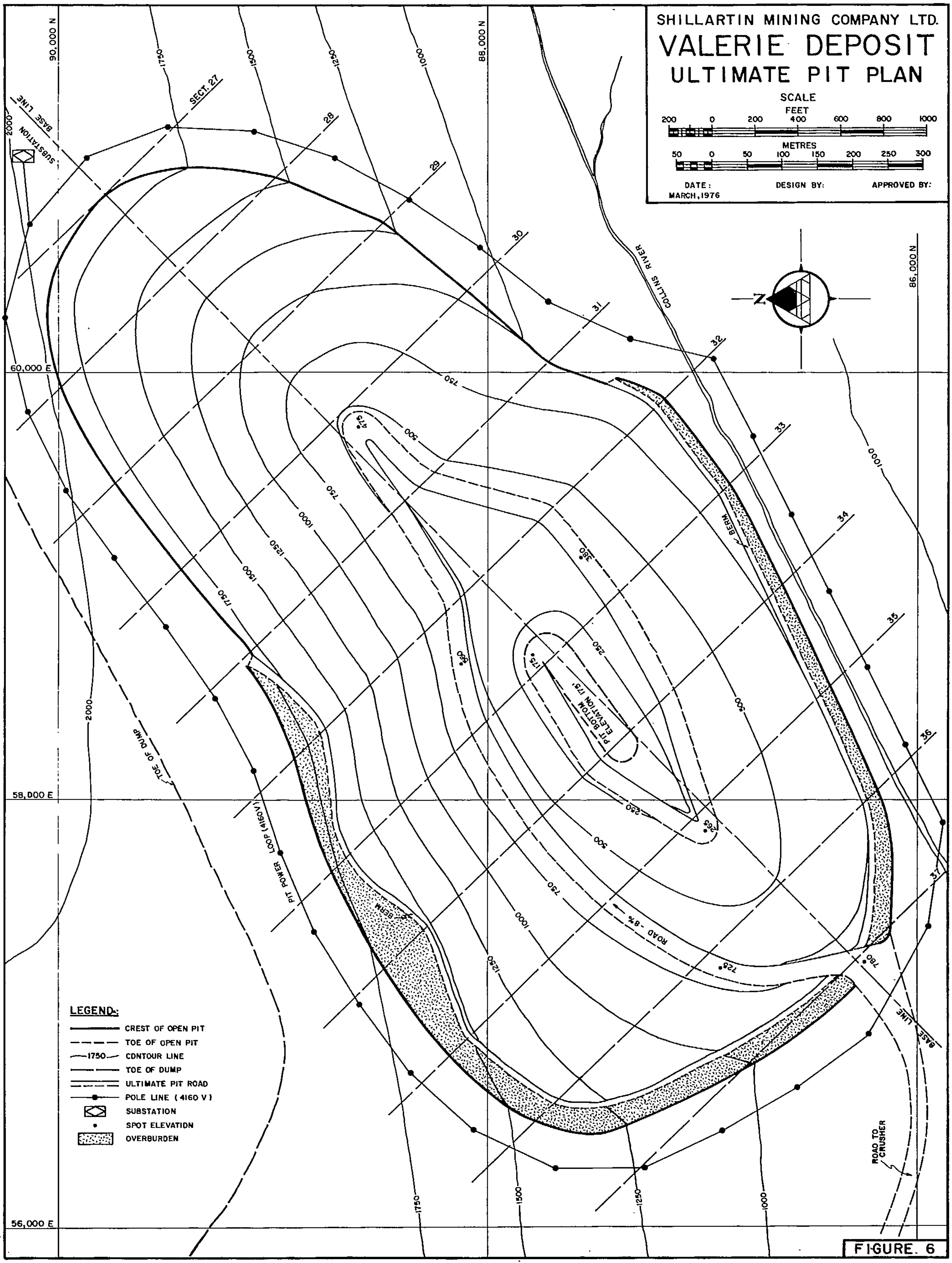

SHILLARTIN MINING COMPANY LTD.
VALERIE DEPOSIT
ULTIMATE PIT PLAN
SCALE
FEET
200 0 200 400 600 800 1000
METRES
50 0 50 100 150 200 250 300
DATE: MARCH, 1976
DESIGN BY:
APPROVED BY:
LEGEND:
CREST OF OPEN PIT
TOE OF OPEN PIT
1750 CONTOUR LINE
TOE OF DUMP
ULTIMATE PIT ROAD
POLE LINE (4160 V)
SUBSTATION
SPOT ELEVATION
OVERBURDEN
90,000 N
88,000 N
86,000 N
60,000 E
58,000 E
56,000 E
SECT. 27
28
29
30
31
32
33
34
35
36
37
BASE LINE
SUBSTATION
COLLINS RIVER
N
BERM
TOE OF DUMP
PIT POWER LOOP (4160 V)
PIT BOTTOM ELEVATION 175'
ROAD - 8%
ROAD TO CRUSHER
2000
1750
1500
1250
1000
750
500
250
475
380
560
175
265
725
780

FIGURE. 6

Soil - 450,000 cubic yards (340,000 cubic metres).

Overburden - 10.9 million bank cubic yards (8.3 million bank cubic metres) x swell factor of 1.2 = 13.1 million cubic yards (10.0 million cubic metres).

Waste rock - 61.4 million bank cubic yards (46.9 million bank cubic metres) x swell factor of 1.3 = 79.8 million cubic yards (61.1 million cubic metres).

64. Initial exploration of the area covered by the general site plan, Fig.B-1, revealed no other mineralization. The southern and eastern edges of the property are adjacent to the limits of this plan. The location of any dumps is further constrained by the Collins River and the need to avoid its contamination by waste material or surface water runoff. The relatively steep topography of the Collins River valley is significant. The route of the haulroad from the pit crest to the crusher and the need to minimize waste haulage distances, particularly uphill, must be taken into account. Separate sites for each class of material are required to preserve topsoil and segregate overburden from rock. No dumping in the pit is considered feasible.

65. Inspection of Table B-3 illustrates the distribution of waste material and highlights the need to move large volumes from the central portion of the deposit. In selecting dump sites, loading the pit wall must be avoided; a minimum of 400 ft (122 m) between the pit crest and any dump is to be provided. This margin accommodates an access road, ditch, dewatering wells linked to the pipeline drainage system and the power line.

66. Screen analysis of soil samples taken during exploration diamond drilling and from a small road cut for drill access showed the soils of the area to be silty sand. They appear adequately drained and should not present a problem for stockpile stability.

67. Two overburden stockpile areas are specified, one at either end of the orebody. Overburden to the northeast of section line 32 will be taken to the smaller dump at the northeast of the pit, Overburden Dump No. 1. Overburden from the remainder of the pit area will be taken to the west of the pit at Overburden Dump No. 2. These dumps have capacities of 3 and 10 million cubic yards respectively (2.3 and 7.7 million cubic metres). Both areas are sufficiently close to the pit so that stripping by scraper is feasible.

68. The topsoil area is designed with sufficient capacity to hold the estimated volume of soil and organic debris covering the orebody. The topsoil will be redistributed over the waste dump as part of the overall reclamation program.

69. To be aesthetically acceptable when laying out the waste dump areas it is assumed that no dumps are to exceed the natural surface elevation of 2250 ft (690 m) and that they are not to be visible above the Donald Ridge. The dump slopes were specified at 30°. Best construction practice using trucks is to dump in successive lifts and to leave a berm every 80 ft (24 m) to reduce the natural angle of repose of 36° by some 2° to 3° to give an overall angle of 33° to 34°.

70. To accomodate the disposal of some 80 million cubic yards (61 million cubic metres), two separate areas adjacent to the open pit are required. To the north an area flanking the west end of Donald Ridge is available, which provides sufficient space for 45 million cubic yards (34 million cubic metres) of broken material. This waste dump, No. 1, will provide sufficient capacity for all workings northeast of section 33. Access will be by way of intermediate haulage roads from the upper levels of the orebody. Before its rated capacity is reached, topsoil could be applied to the final graded surfaces to aid rehabilitation. The balance of the waste rock disposal requirements are met by a second dump, No. 2, to the west of the pit and south of the road to the crusher. The layout incorporates a road at 8% grade rising from the main haulage to the top of the dump. The dump will be built progressively by running along the contours from the pit crest. Only in the final years will a long uphill haul be necessary. The high cost

associated with waste disposal from the lowest elevations are critical to the planning of the pit bottom. The second waste dump has a rated capacity of 35 million cubic yards (27 million cubic metres) and between them, the two dumps supply the total waste dumping requirements with reserve capacity.

71. Ditches are integrated into the overall drainage control system. Run-off can be routed through settling basins and treated, if necessary, at a central location before discharge to the Collins River/Lea Lake system. The main settling pond is initially scheduled to be located just north of Lea Lake, between the pit and No. 2 waste dump. A secondary pond may be necessary on the north flank of Donald Ridge, and a possible site is identified just south of the main plant area, feeding into Carol Creek. Scheduling is beyond the scope of the preliminary plan, but sequential reclamation using the stockpiled overburden and topsoil would be feasible if started on the dump slopes below the ultimate road.

72. Foundation investigations and analyses for the dumps are required in advance of the feasibility study. Environmental assessment will precede dump site clearances. Consequently, a program of soil boring and hydrological testing is required.

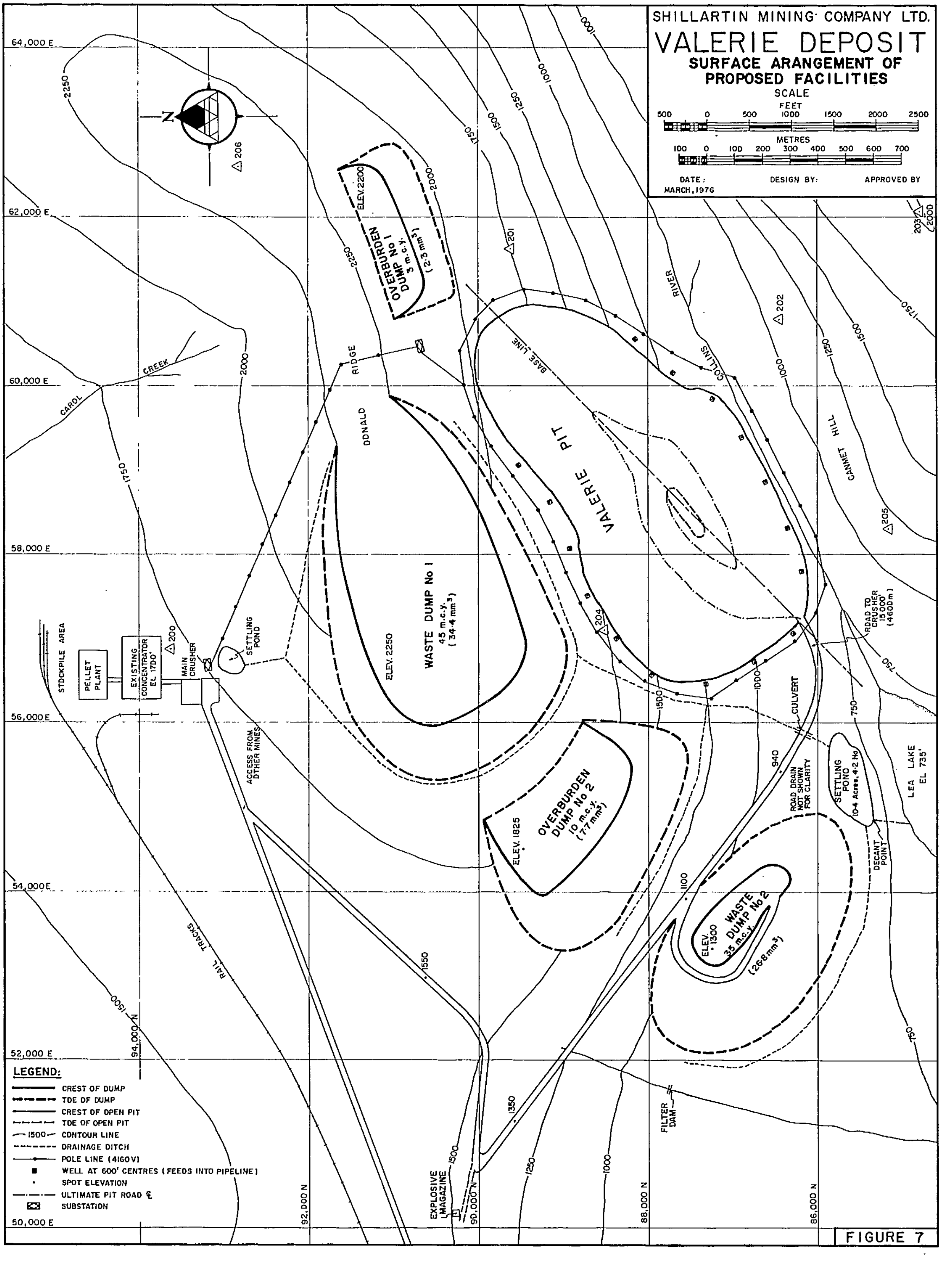

SHILLARTIN MINING COMPANY LTD.
VALERIE DEPOSIT
SURFACE ARANGEMENT OF PROPOSED FACILITIES
SCALE
FEET
500 0 500 1000 1500 2000 2500
METRES
100 0 100 200 300 400 500 600 700
DATE: MARCH, 1976
DESIGN BY:
APPROVED BY
LEGEND:
CREST OF DUMP
TOE OF DUMP
CREST OF OPEN PIT
TOE OF OPEN PIT
1500 CONTOUR LINE
DRAINAGE DITCH
POLE LINE (4160V)
WELL AT 600' CENTRES (FEEDS INTO PIPELINE)
SPOT ELEVATION
ULTIMATE PIT ROAD ℄
SUBSTATION
STOCKPILE AREA
PELLET PLANT
EXISTING CONCENTRATOR EL 1700'
MAIN CRUSHER
SETTLING POND
ACCESS FROM OTHER MINES
RAIL TRACKS
CAROL CREEK
DONALD RIDGE
WASTE DUMP No 1
45 m.c.y.
(34·4 mm³)
ELEV. 2250
OVERBURDEN DUMP No 1
3 m.c.y.
(2·3 mm³)
ELEV. 2200
VALERIE PIT
BASE LINE
COLLINS RIVER
CANMET HILL
OVERBURDEN DUMP No 2
10 m.c.y.
(7·7 mm³)
ELEV. 1825
WASTE DUMP No 2
35 m.c.y.
(26·8 mm³)
ELEV. 1300
SETTLING POND
10·4 Acres, 4·2 Ha
DECANT POINT
LEA LAKE EL 735'
ROAD DRAIN NOT SHOWN FOR CLARITY
CULVERT
ROAD TO CRUSHER 15 000' (4600 m)
FILTER DAM
EXPLOSIVE MAGAZINE
64,000 E
62,000 E
60,000 E
58,000 E
56,000 E
54,000 E
52,000 E
50,000 E
94,000 N
92,000 N
90,000 N
88,000 N
86,000 N
N
200
201
202
203
204
205
206
2250
2000
1750
1500
1250
1000
750
1550
1350
1100
940

FIGURE 7

SERVICES

73. Mine planning must include the services required to support operations. The ultimate road between the pit bottom, the top of the dump and the primary crusher has to be delineated. Drainage must be shown to be feasible. The electrical power supply route must be laid out. All reasonable eventualilities must be considered.

74. The haulage road was described for the ultimate pit. Generally, access roads are not shown until later when operating schedules have been prepared. Individual haulage routes will be developed within the pit to service blocks of ore by the shortest practical routes, and possibly no ore will be hauled up the ultimate road for several years. Access roads outside the pit are planned when operating schedules are known.

75. The alignment of drainage ditches for surface runoff is shown on the site plan Fig.B-7. An estimate of size and concentration of the suspended solids provides the data for laying and a settling and decant lagoon of 10.4 acres (4.2 Ha) discharging into Lea Lake. Roadside ditches are not shown on the site plan. These serve the double purpose of containing road runoff and helping to control surface drainage within the disturbed area. A small filter dam must be constructed on the unnamed stream which crosses the main haulage road west of the pit. This will control suspended solids flowing down from the waste dump and from the road itself. The dewatering wells drilled around the pit crest will pump groundwater into a pipeline for delivery to the settling lagoon. The pipeline would need to be designed to avoid frozen pipes, particularly on gradients and at bends.

76. A gravel filter of sand and mine waste is recommended to control seepage from the overburden between the pit crest and the Collins River. Seepage will be collected in an adjacent ditch and channelled into a sump where it will be pumped into the well pipeline for discharge into the settling area.

77. Power supply must be considered. The shortest route is from the existing substation to where the pit crosses Donald Ridge. This line feeds the pit loop from a substation at the northeastern corner of the pit. Initial mining activity is expected to be in this area. Current practice is to loop the pit with the main high-tension line (4160 V) to provide security of supply.

78. A mine explosives and cap storage magazine is provided. This is conveniently located adjacent to the main haulage road. The magazine is well removed from the beneficiating area, the pit and the power line.

REVIEW

79. The deposit contains mineable reserves of 204.7 million tons (208.0 million tonnes) of 29.7% Fe ore. This compares with the original geological reserve of 232 million tons (236 million tonnes) of 29.8% Fe ore. The overall mine recovery is high at 88%, being largely a function of the physical shape of the orebody. To mine all 204.7 million tons (208 million tonnes), 72.1 million bank cubic yards (55.2 million bank cubic metres) of overburden and waste rock must be stripped and placed on dumps. The overall stripping ratio is 0.35 cu yd/ton of ore.

80. The ultimate pit design requires a thorough understanding of the interaction of the physical and economic design criteria. The Collins River provides an example of an environmental criterion overriding an economic one. Diversion of the river and a push back of the south wall of the pit would release additional ore. Steepening the south pit slope would likewise increase mineable resources. In later studies, which must include detailed field investigations, the costs of a higher probability of slope instability or possible downstream environmental degradation must be weighed against the additional ore that would be released by slope steepening or river diversion.

81. With some 72 million bank cubic yards (55 million bank cubic metres) of waste and overburden to be hauled out of the pit area, dump locations are crucial. Note that if more distant sites are considered stripping costs would increase, thereby affecting the COSR, reducing the size of the pit and making some ore uneconomic. The surrounding topography limits the options for dump location, and by inspection it is clear that no waste can be dumped in the pit. Each of the selected dump sites must be investigated to ensure they are safe and will accommodate the designed volumes of material.

82. The haulage road is 15,000 ft (4600 m) long and climbs at 8% for 9500 ft (2900 m) from the pit crest to the existing crusher. From the ultimate pit bottom, the last increment of ore must be hauled a total distance of 23,000 ft (700 m), against a rise of 1500 ft (460 m). Preliminary calculations show that towards the end of the life of the mine, as many as 14 150-ton (136 tonne) trucks will be required to support the planned production of 9 million tons (9.14 million tonnes) per year over this extended haul distance. Final design studies should consider relocating the primary crusher at the pit crest and installing a conveyor to the concentrator. Also,

once operating schedules have been worked out, simulation studies should be run to compare ore and waste trucking requirements with different pit and waste dump operating sequences.

83. Future rock and soil mechanics studies will be required to meet specific goals. Each wall design sector must be confirmed in location and extent. The greatest effort should be devoted to Section 3 in the south wall; although this is not the flattest slope, it is the longest and controls the greatest tonnage of ore. Its detailed design must be closely coordinated with environmental assessment.

84. The hydrology aspects of future mine planning of the pit are important. Detailed design of drainage measures should be delayed until wall design has advanced somewhat further. Hydrology studies should include a reconnaissance and monitoring program that will assist in defining the local recharge zones and in estimating variations in hydraulic gradients. Investigations should examine the hydraulic conditions around the normal fault.

85. The ultimate pit plan is sufficiently comprehensive to have identified dump locations outside the pit. Soil testing must be performed for the dump locations along the Collins River and also along the route of the main haulage road. Dump site investigations should be given highest priority, because, if any dump is not feasible for the required storage capacity, a more remote position would have to be chosen. This would affect the economics sufficiently to change the size and shape of the mine.

86. Future studies during the operating life of the mine may show that the north wall of the pit (Sector 1) should be less than 55°. This would require a revised ultimate pit plan in which either the No. 1 waste dump would have to be moved back from its present position, or the crest location would be maintained if the dump were already in existence. Such a re-design might require ore to be left at depth. These possibilities are raised to illustrate the interrelated nature of the total planning process.

APPENDIX C

PREVIOUS SLOPE STUDIES

INTRODUCTION

Purpose and Scope

1. The purpose of this appendix is to assist mine planners use information which has become available from experience with previous slopes. Case histories of slopes which have been unstable provide information on strength of various rock types and hence are particularly useful. Data on stable slopes, can provide a statistical basis for examining the probability of instability, even though the degree of stability is not known. The scope of this appendix is to describe the gathering and use of such information.

2. Information on which to base the design of a wall is typically less than the designer would like to have. At the feasibility stage data obtained from a few boreholes and outcrops is used for the entire wall. However, there is a limit to the permissable expenditure of investigation funds owing to the scarcity of pre-production capital. Even at the mine design and operating stages, the probability of producing benefits of greater value than the additional expenditures must exist. Surveying previous slopes is the area where such benefits can usually be obtained.

Natural Slopes

3. Natural slopes may be formed in different ways such as by erosion and weathering, glacial action, seismic activity, changing climatic conditions and progressive breakdown. The profile may be affected by age, rate of weathering, climate and amount of rock face covered with talus. High slopes are usually flatter than low slopes. Fig.C-1 and 2 indicate the variety of shapes that can occur in natural profiles, which makes their representation by an average slope angle difficult (C-1).

4. Natural slopes, even though they have been produced through a combination of mechanisms, can be valuable to the mine planner. The face direction of natural slopes is important. It is usual to find a variation of slope angles for different bearings with respect to the geological structure, ie, slopes facing in the direction of the dip of major sets of discontinuities are normally flatter than those in the reverse direction. Also, climatic conditions can affect south-facing and north-facing slopes differently.

Excavated Slopes

5. The factors affecting short-term stability (ie less than geologic time) for excavated slopes are usually known. Hence the performance of excavated slopes compared to natural slopes is more readily applied to the design of future slopes. Types of potential slides can be identified and analyses of stable slopes may be used to determine the minimum effective strength parameters.

Stable Slopes

6. The stability of natural slopes is related to their method of formation and age. Significant differences might be observed in slopes that have been in existence for decades compared with those of geologic age. These slopes usually have an unknown safety factor but do provide a useful addition to the statistical data base.

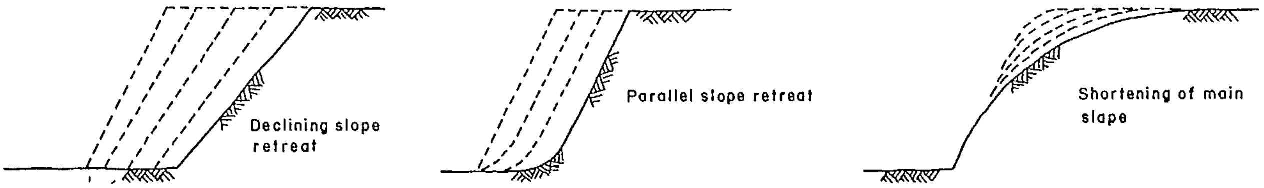


Fig. C-1 - Modes of retreat in natural slopes; the slope angle varies with age in some cases (C-1).

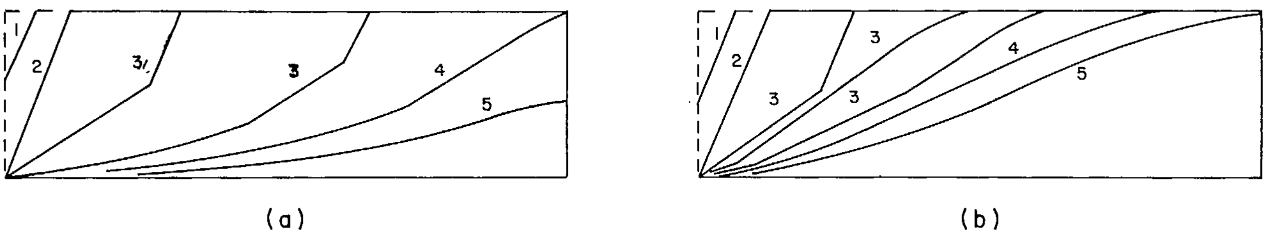


Fig. C-2 - Slope development in hard rock in (a) semi arid and (b) humid climates. 1 and 2 = stream cutting stage. 3 = erosion. 4 = 'stable' slope. 5 = long-term modification into an s-profile (C-1).

7. Excavated stable slopes provide more pertinent data than natural slopes, particularly where deformations are monitored during excavation. Such measurements can be used in elastic analyses for determining deformation properties.

8. If slides have occurred in these slopes, the mechanism of potential sliding in the stable areas is known, and hence calculations of existing safety factors can be made. In any event, such slopes add to the data that can be used in reliability studies.

Unstable Slopes

9. Slopes which have been unstable provide particularly useful data. Analyses may be based on limit equilibrium. Where the possibility of progressive breakdown is to be considered, stress distributions must be examined. The latter method of analysis usually employs finite element techniques. Different rock properties are used when applying these two methods. Supplements 5-1, 5-2 and 5-4 provide information on the data requirements for analyses.

10. Information obtained from slides cannot always be applied directly to the design of new walls. Such a procedure can under-estimate the pre-sliding strength of the rock. Results of the analysis should take into account the deterioration that might have occurred on the sliding surface as a result of large movement.

11. Where movement has been along throughgoing weakness planes within the rock mass, the extent of these slides are governed by the length and orientation of the weakness planes in relation to the slope. In general, the lower the slope height, the greater the incidence of discontinuities that extend from toe to crest; hence, many such slides are observed on bench faces. These slides can occur on one plane (simple plane shear), on two or more planes striking parallel to the slope (multi-block and stepped-path plane shear) or on two oblique planes creating a 3-d-wedge. Procedures for analyzing such slides are contained in Supplement 5-1.

12. Slopes where sliding has taken place through intact material usually behave like soil slopes. Rotational shear slides are common in these cases. Slides can also occur partly along structural planes and partly through the intact rock. Stress concentrations at the toe of the slope can initiate failure in brittle rocks. Appropriate analyses are presented in Supplements 5-2 and 5-4.

INFORMATION FROM SLIDES

Topography

13. Where a slide has occurred, a topographical plan of this part of the slope should be prepared. This should be drawn at a scale such that the slide zone occupies an area of about 8 in. x 12 in. (20 cm x 30 cm). The plan should cover an area around the slide out to a distance of one to three times the height of the slope. It should include information on all surface features, particularly the location of any surface or buried water supply pipes and drainage systems.

14. Groundwater data and geological information, as discussed below, should also be included on the plan. If water is flowing out of the face, the location of these flows should be clearly marked. In particular, the intersection of the phreatic line with the slope face (ie, the highest elevation at which the groundwater intersects the slope) should be shown.

15. All dumps within a distance from the crest of the slope of one to two times the slope height should be located.

16. Key elevations should be noted. They should be recorded either as spot heights from conventional surveys or by contours from photogrammetry. The desired elevation resolution will vary with the size of the slide, but in general, contours should be drawn at intervals not exceeding 5% of the slope height so that a difference of 1% of the height of the slope can be interpolated. If the slope consists of a number of cut benches, the elevation of toes and crests of each bench face should be shown.

17. Terrestrial photogrammetry is generally the most convenient method for surveying a slide. Alternatively, triangulation lines from the ends of a fixed baseline can be used, sighting on objects within the slide zone.

18. Profiles through the slide should be prepared. A single section is appropriate for a small slide such as in a slope less than 150 ft (50 m) high. Three sections, one at either end and one through the centre, should usually be drawn for a large slide. A scale of 1:2000 would be suitable for slope heights up to 1500 ft (500 m), whereas a scale of about 1:200 would be best for a slope height of 150 ft (50 m). The scale of the drawing should be such that all significant dimensions can be read with ease. Drawings with slope heights measuring 12 in. (30 cm) are generally suitable.

19. Information recorded on sections should include the following:

a. original slope and ground surface profile, the slide scar drawn in full line with original slope profile as a dotted line;
b. surface drainage measures;
c. occurrence of groundwater, if any, on the slope surface;

d. borehole location together with drilling, geological and groundwater information;
e. information on geology structural of the slope surface,
f. geologic interpretation of slope materials;
g. position of piezometers in boreholes and water level recordings;
h. location of the plane along which the slide has occurred.

20. A comprehensive set of photographs should accompany the plans and sections. Photos should include the following:

a. for large slides, a panoramic photomosaic of the entire slope containing the slide;
b. a larger-scale frontal view showing only the slide;
c. a view at the top of the slope showing the top of the slide surface and any tension cracks developed in the crest area;
d. if applicable, a smaller-scale comprehensive photo of the top surface showing drainage installations, dumps, etc.;
e. one photo from each side of the slide;
f. any other photographs showing items of interest which could be related to the slide, eg close-up of slide surface, structural detail, infilling materials, etc.

Slide Surface

21. It is important to examine the slide surface. Often only the top portion containing the tension cracks is visible. The rest of the surface is usually covered by slide material. The top portion can be located either by conventional profiling and triangulation or by terrestrial photogrammetry. The major portion of the slide surface is much more difficult to determine. Core drilling may be possible in a temporary slope. Sometimes the slide debris can be excavated allowing the slide plane to be observed and surveyed.

22. The slope is frequently inaccessible, and location of the slide surface has to be determined by other means. The direction of movement will be governed by the slide surface. Therefore, if the pattern of displacement is determined by repeated observations, the shape of the slide surface can be inferred. Because the distribution of movement within the slide segment is not known, such inference may be in error.

23. Where a rotational slide occurs, which usually indicates that the material is similar to soil, a pit 3 ft (1 m) in diameter can be dug to establish location of the slide surface. This method can only be used when movement has reached equilibrium or has been reduced to a small constant rate in the order of 0.1 in. (2.5 mm) per day. The pit can be dug either by hand or, if access is possible, by machine. In either case, the sides have to be shored for safety purposes. The most satisfactory method of establishing the position of the slide surface is visual inspection. At least four such inspection pits are generally necessary before the slide surface can be adequately defined, but more may be necessary for very large or more complicated failures.

24. In soil slopes, provided the slope is accessible and not unduly hazardous, a simple penetrometer test carried out at different depths in the test pit may indicate the depth to which the material was disturbed. Care must be taken to evaluate results since the displaced mass may frequently consist of large boulders of intact material. This problem can normally be overcome since it is often possible to estimate where the failure surface will be encountered.

Structural Mapping

25. A comprehensive description of the geology should include details of lithology, structure and tectonic history. The most important information includes the attitude and spatial distribution of such discontinuities as faults, shear zones, geological contacts, bedding planes and joints. Information should be recorded on the plan and sections. A full description of the rock types should be recorded using the standard procedures described in the chapter on structural geology.

26. Mapping of cracks in the crest area and on benches should be conducted from the time they first appear. Plans should be dated and stored for future reference if instability should recur.

27. A study of the tectonic history of mechanisms causing the various sets of discontinuities is useful for defining sets (C-2). An assessment of the magnitude and direction of residual stresses may also be helpful in explaining unusual instability.

28. A detailed structural study should be conducted on the faces of the slopes adjacent to the slide. This survey must extend for a distance on either side of the slide equal to the height of the slope. Major structural features require special investigation. A set of discontinuities usually requires at least 100 observations, although in some cases a minimum of 20 may be acceptable. These observations are to be carried out according to procedures described in the chapter on structural geology.

29. In addition, a joint survey should be carried out where the slide surface is exposed. The object is to establish which joint sets were involved in the slide. The rock surface should first be hosed down or cleaned by some other method. These surveys should be carried out on horizontal and vertical lines if possible. Any adits within the same structure should be utilized.

30. All joint survey information should be plotted on equal area nets and compared with those of previous structural reports. Boundaries of the domain within which the slide occurred should be examined to determine if they are confirmed or need to be modified. Continuity and joint sizes should be determined and compared with prior information. Similarly, data on joint spacing, in-filling and waviness must be gathered and compared.

Groundwater

31. Observations are necessary to confirm the presence of any groundwater within the slope. If piezometer observations are not available prior to the slide, boreholes should be drilled as soon as possible. Piezometers should be located to measure the head at the slide surface.

32. If feasible, at least two vertical boreholes should be drilled in the crest area beyond any tension cracks. These are ideally located at distances from the face of H/2 and H/4, (as shown in Fig.C-3), where H is the height of the slope. Appropriate depths must be judged but usually would exceed H/2. If strike length of the slide exceeds H, then two lines of two boreholes each should be drilled. In attempting to locate the water table, it is helpful to drill dry until water is encountered. This may be done to depths of about 75 ft (23 m). At least two, but preferably three piezometers should be installed in each borehole. The depth of these are based on structural conditions and encountered water.

33. If necessary, additional piezometers should be installed to determine groundwater distribution within the same structural domain but in an area unaffected by the slide. These piezometers would give an estimate of the groundwater regime within the slope prior to the slide. One piezometer hole should be located on the crest at the centreline of the slide at a distance back from the slope face approximately equal to 2H as shown in Fig. C-4. Two should be placed on the crest at a distance approximately equal to H on either side of the slide zone and H/2 back from the slope face.

34. Rainfall records covering 12 months prior to the slide should be plotted with rainfall to any suitable scale on the vertical axis and time to a scale of 1 mm = 2 days on the horizontal axis. This record should be compared with the annual records for the previous five to ten years, to determine if the climatic conditions have been unusual.

Rock Strength

35. Joint surfaces of sets involved in the slide should be tested. Experience indicates that if approximately 70% of the total area of the slide surface consists of plane surfaces, determining the strength characteristics of the surfaces is important for the evaluation of stability. If it is less than 30% an estimate of strength can be based on testing the rock substance. If the percentage of plane surface to total sliding surface lies between 30% and 70%,

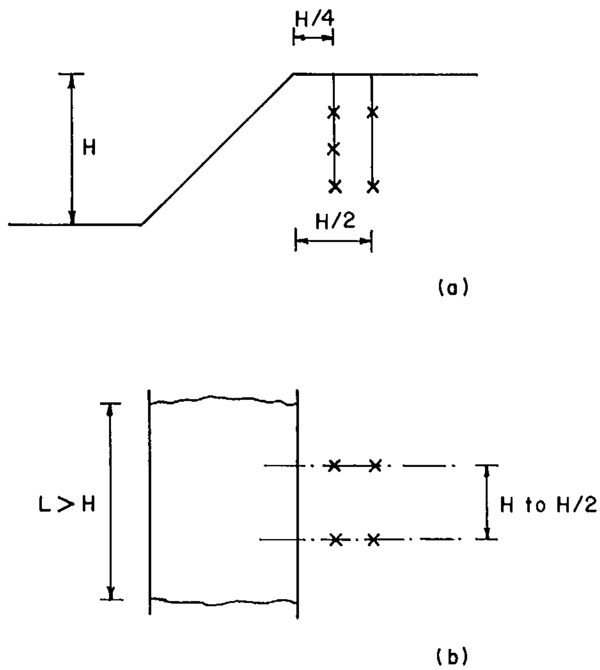


Fig. C-3 - Recommended locations of piezometers to provide groundwater data for estimating water pressures on the surface of sliding. (a) Two holes on the centreline of the slide. (b) Two lines of holes where the length of slide is greater than the slope height.

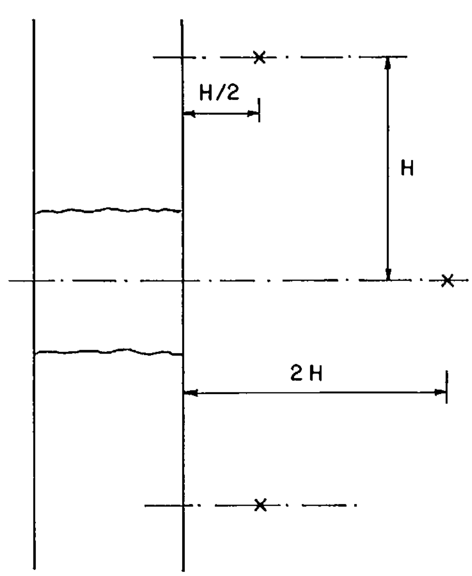


Fig. C-4 - Recommended locations of piezometers to determine groundwater levels in the ground around a slide but unaffected by the slide.

testing must include direct shear tests on the rock surfaces, triaxial tests on any thick, fine-grained gouge and tests on the rock substance.

36. In all cases, classification tests as described in the chapter on mechanical properties should be carried out for all rock strata involved in the failure.

Mining Method

37. The method of producing a slope is sometimes responsible for causing a slide. The rate at which the slope was formed may also be pertinent. In particular, details of blasting practices should be noted, including: (a) diameter of hole, (b) depth of hole, (c) burden and spacing, (d) type of explosive, (e) charge per hole and method of charging, (f) number of holes detonated per blast and per delay, (g) extent of backbreak and (h) perimeter blasting details. An analysis can then be made to see if ground shock might have contributed to the instability. The chapter on perimeter blasting describes such an analysis.

Seismic Activity

38. If available, Richter Scale values should be noted for any recent seismic activity in the area. This would help to determine if ground motion contributed to a slide.

Analyses

39. Valuable information is obtained on actual modes of instability by examining slides. Sometimes expected modes are confirmed. Often slides do not conform exactly to the simple models used in conventional analyses. Sometimes, the actual slide cannot be analyzed with the conventional methods described in the manual. For example, many slides occur on faults or joints that do not intersect the slope face. These require shearing of the rock substance at the toe of the slope between the face and the discontinuity as shown in Fig. C-5 and as represented in Fig. C-6. Also, slides occur that result in face slabs being rotated up to the vertical, then

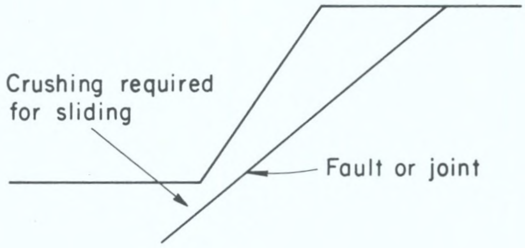


Fig. C-6 - A typical situation where a discontinuity does not daylight into the slope face. Sliding on the discontinuity requires crushing or shearing of the rock substance between the discontinuity and the slope face at the toe.

Fig. C-5 - A 3-d-wedge slide where the bounding planes did not daylight into the slope face, requiring crushing at the bottom point of the wedge to permit movement.

Fig. C-7 - A pit wall that was designed based on analysis of slides in two previous pits in the same formation. The slide that can be seen in the centre of the wall occurred two months after mining was completed and after a scavenging operation had cut the toe back beyond the ultimate wall line.

toppling into the pit. This might be predicted by the 2-block plane shear analysis described in Supplement 5-1 but might require a multi-block analysis developed from first principles.

40. Statistical information may be obtained by surveying the slide. For plane shear instability, it will be of interest to know how the strike and dip of the sliding surfaces compare with population data previously sampled. It will be even more interesting to determine the lengths of the discontinuities, comparing them with the prior frequency distributions. A stepped-path in many cases, will constitute the sliding surface, the details of which, ie dip, strike and length of each step, can be of particular value in confirming or providing data for modifying input data for the stability analysis of Supplement 5-1. Similarly, the waviness of the discontinuities might be measured, and samples containing the surface might be obtained for testing frictional properties.

41. For rotational shear sliding, the effective cohesion in the rock mass is difficult to predict from laboratory testing. Consequently, valuable information can be obtained on its magnitude and variation by analyzing slides. Fig. C-7 shows a pit wall that was designed based on analysis of rotational slides in two previous pits in the same formation (C-3). A slide can be seen in the centre of the wall that occurred two months after mining had been completed.

42. Example. In a western copper pit, a large slide occurred in the southeast wall. The height of wall affected was 430 ft (131 m), average slope angle 45° and length of slide about 700 ft (214 m). It was bound by a steeply dipping fault on the southwest, a steeply dipping joint on the northeast and by a dike back from the crest. The wall rock is somewhat altered quartz diorite and dacite porphyry.

43. The slide was subjected to a special investigation. Mine surveying provided topographic information from which sections through the slide could be drawn. The slide surface was not exposed for examination, and no piezometers were installed to provide detailed information on the groundwater. The mining method was not considered crucial for such a deep-seated slide, nor was seismic activity a contributing factor.

44. The slide occurred over a six-month period. Some 25 ft (8 m) of movement at the crest was recorded. The displacement vector was substantially to the northwest, dipping at various angles to a maximum of 18°. From this it was deduced that sliding had occurred on a joint surface that is of infrequent occurrence and usually has a strike of about 185° and a dip of 30° to the west. Movement on such a surface, not down dip but to the northwest, would be on an apparent dip of 22-1/2°.

45. Tension cracks in the crest were found to contain water 2 ft (0.6 m) below surface. The entire wall was described as wet.

46. Structural investigations showed that the dominant sets of joints and faults all dipped steeply. Exposures in all pits on the property had to be examined to obtain data pertinent to the joint on which sliding seemed to have occurred. Samples were obtained from these various exposures and tested in direct shear. The peak angle of friction was 27° - 36° with apparent cohesion 270 - 820 psf (13 - 39 kPa).

47. A two-dimensional analysis was made for the section shown in Fig. C-8. The density of the rock mass was assumed to be 145 pcf (2.32 kg/m^3), and maximum strength parameters were used. For a full head of water the calculated safety factor was 0.7, with no groundwater it was 2.1. However, the groundwater might have been at full height and the actual sliding surface other than that assumed. In addition, waviness or the dilatency factor might have been operating to increase the total frictional reaction to more than 36°. Similarly, perhaps the sliding surface was discontinuous providing either rock bridges between several joint surfaces or a stepped-path for sliding. If only one joint surface was involved it would have been at least 370 x 700 ft (110 x 215 m). Without surface exposure these factors could not be clarified.

48. The investigation of the slide provided information useful for the design of walls for

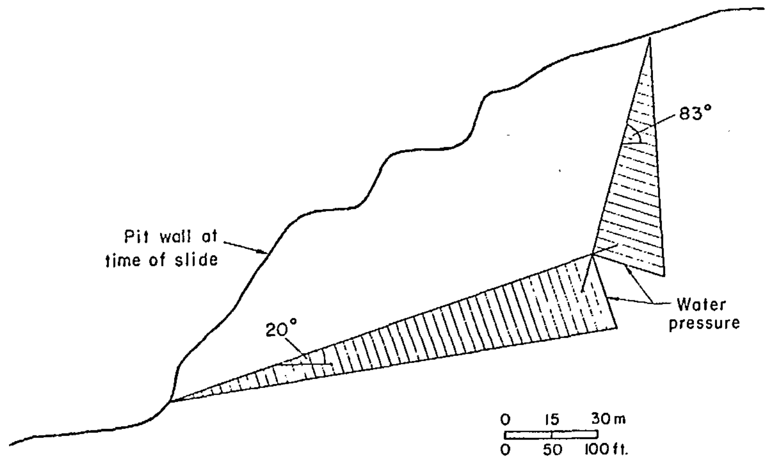


Fig. C-8 - A simplified section through the wall of a pit where a large slide occurred. The height of the slide was 430 ft (131 m) and the average slope angle was 45°. The breadth of the slide was approximately 700 ft (214 m). Sliding seemed to occur on a flat lying discontinuity aided by steeply dipping joints that probably trapped water to provide hydrostatic pressures.

subsequent pits. Two-dimensional plane shear sliding is now known to be possible on an east wall. Steeply dipping joints and faults facilitate such sliding. They destroy end constraints on the segment and provide a nearly vertical cutoff, as shown in Fig C-8 behind the crest. For design purposes, more data on orientation, length and waviness of flat-lying discontinuities would be required to determine reliability schedules. These joints may be quite large in extent.

49. Analysis did not confirm the strength characteristics obtained from the laboratory test. However, as mentioned above, the strength values might be substantiated with more information on details of the discontinuity.

50. It seems clear from the evidence that groundwater can contribute to instability of such walls.

INFORMATION FROM STABLE SLOPES

Natural Slopes

51. Studies indicate that natural slope angles in rock vary inversely with height through some exponential function (C-4). One form of such a relationship is $H = aX^b$, where H is the height, a and b are constants and X is the horizontal projection of the slope face. Plotted on a log-log graph such curves are straight lines with a slope of b. There is some evidence that all empirical lines converge to a point around H = 10,000 ft (3050 m) and X = 75,000 ft (22,800 m) as shown in Fig. C-9 (C-5). Insufficient research has been conducted on this conclusion. It does have considerable empirical basis and is worth using in slope design, retaining as always critical judgement.

52. Natural slopes can be considered to be at the critical angle related to the present stage in their geological history. Such a critical angle is not usually the same as the critical angle with respect to sliding in the modes described in the main text. The natural slope angle is usually flatter than the maximum used in mining. Consequently, when comparing planned pit slopes with natural slopes, the different mechanics of breakdown, different directions and different methods of creation must be taken into account.

53. In spite of these differences, the advantages are worth the expense of collecting data on natural slopes in wall formations. Such information has the value of being about real slopes.

54. The first step is to establish the populations to be sampled. The slopes must be in the same geologic formations as one of the pit wall sectors. The direction of the slope face relative to such major structural feature as bedding, schistocity or a distinctive set of joints must be the same as that of the pit slope. The groundwater regime should be similar although analytical corrections can be made for differences.

55. The next step is taken if the slope population is large. The statistical variation of slope angles for a given height might be obtained by dividing the slopes into cells, equal in length to the height at that section. For each cell, H and X (the horizontal projection) must be determined. These might be obtained from an existing

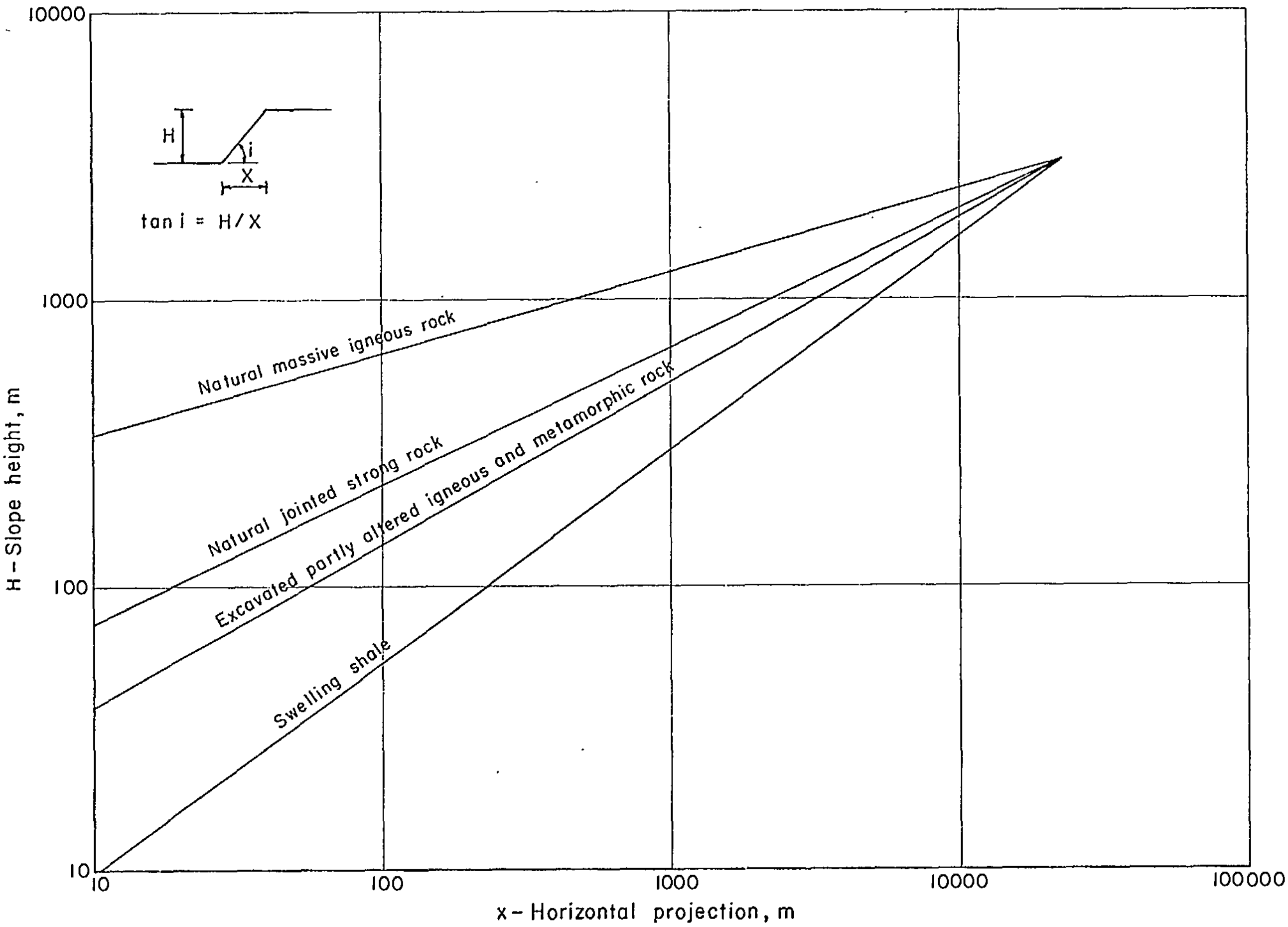


Fig. C-9 - Regression lines developed from actual slopes, both natural and excavated, in various rock types. These curves represent upper envelopes to the data (C-5).

topographic map, from a topographic survey for the purpose, from an aerial survey or from direct measurements of height and slope angle.

56. All the data for the cells can be plotted on a log-log graph. An envelope straight line is then passed through the extreme points giving a curve of maximum H for any value of X (or for any value of slope angle). When only a few points are available, the convergence point might be used to draw the curve. This graph is not normally a design curve for the appropriate wall sector, although it might be so used at the feasibility stage. Also, it is not necessarily an upper bound curve because the mechanisms of formation and potential instability, together with duration of the natural slopes, are quite different from those of the pit walls. Furthermore, the optimum wall angle is a product of financial and operating factors besides stability considerations. However, it has been found on many properties that it is helpful to compare designed slope angles with an experience curve of this nature.

57. Figure C-10 shows natural slopes in a somewhat altered taconite formation with two different orientations that were applicable to two different wall sectors. The north-facing slopes produce a curve directed towards the convergence point that includes three of the four points. The south-facing slopes produce a curve that is above the convergence point. Because these are natural

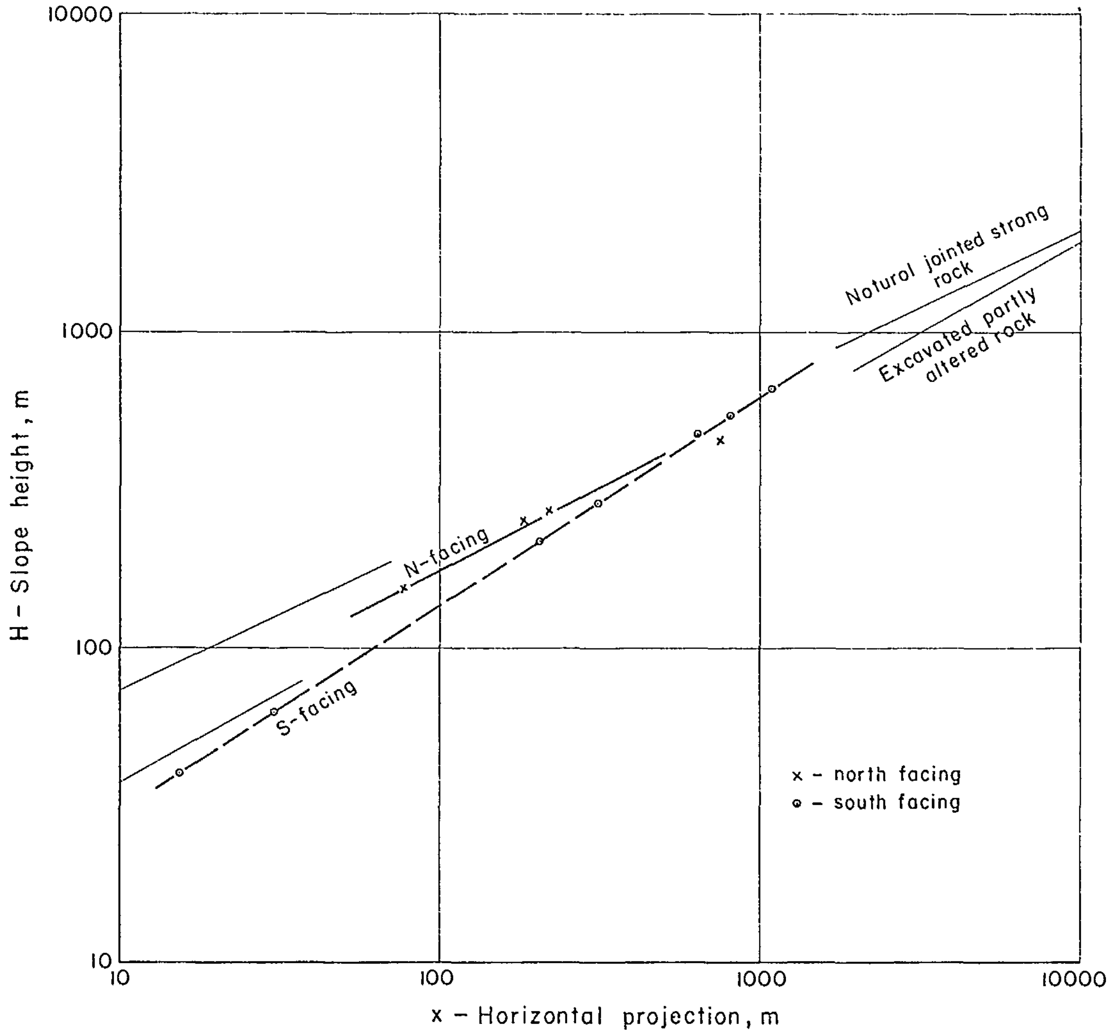


Fig. C-10 - Data on natural slopes in a somewhat altered taconite. Two sets of data representing different orientations were gathered to be used in the design of two different wall sectors in a pit.

slopes, such a regression curve is probably more valid than one connecting the highest point and the convergence point.

58. Figure C-11 shows an example in limestone where only two natural slopes were available to provide some guidance for the design of a wall sector that would be about twice as high. In this case, using the convergence point seemed to produce a better envelope curve than by just joining the two points. In the event, the wall was excavated deeper and steeper than originally designed, which produced a point above the curve. This experience is consistent with the expectation that excavated slopes can stand at steeper angles than natural slopes in the short-term. The designed and excavated wall angles are shown. The wall turned out to be sufficiently stable for mining purposes.

Excavated Slopes

59. Where slopes have been excavated experience curves can be developed in the same manner as above. Such slopes, having been excavated and exposed for decades rather than

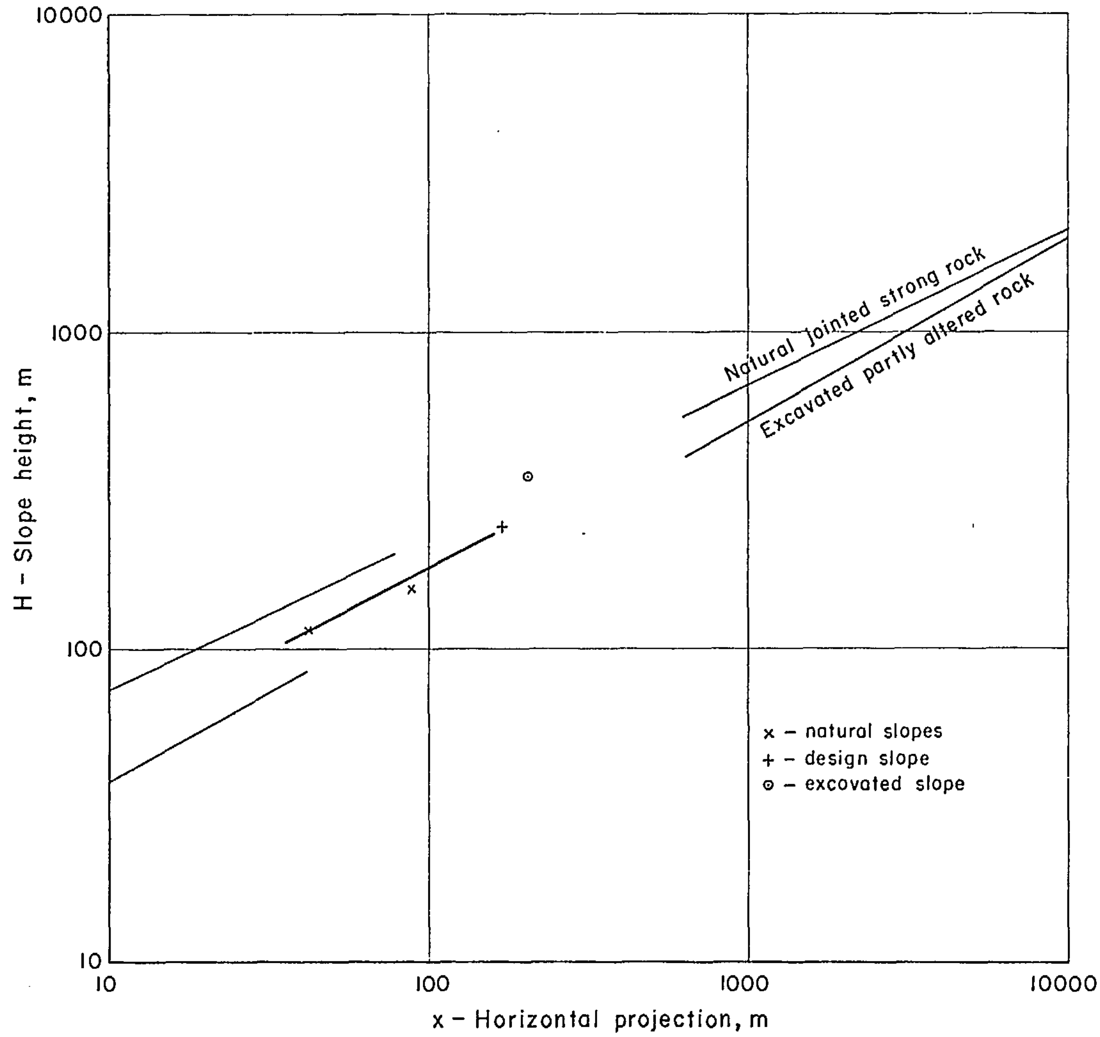


Fig. C-11 - Data from two natural slopes in a limestone are shown compared to the design of a wall slope in the same formation and the actual excavated slope in that pit. It could be expected that excavated slopes would be stable at steeper angles for a relatively short period of time compared to natural slopes that have been developed over geologic time.

geological periods, will be more pertinent to wall design than natural slopes. On the other hand, these slopes might have incorporated a variety of safety factors determined by actual operating conditions. For this reason, as well as when there is a small number of previously excavated slopes available for a study, it may be advisable to use the convergence point to draw the envelope curve as strange envelope curves can otherwise be obtained.

60. These experience curves can be very useful to the planner. Differences between the experience curves and the planned slopes can be examined through stability analyses, possibly resulting in greater insight into the potential modes of instability.

61. Figure C-12 shows the points obtained from prior slopes in a series of open pits on one property (C-6). The data for the south walls include two slopes produced at the surface by underground block caving operations. These slopes were stable for more than 20 years. The curve through the convergence point coincides fairly well with the data points. The points for the north walls would produce a curve much below the convergence point and would probably not be

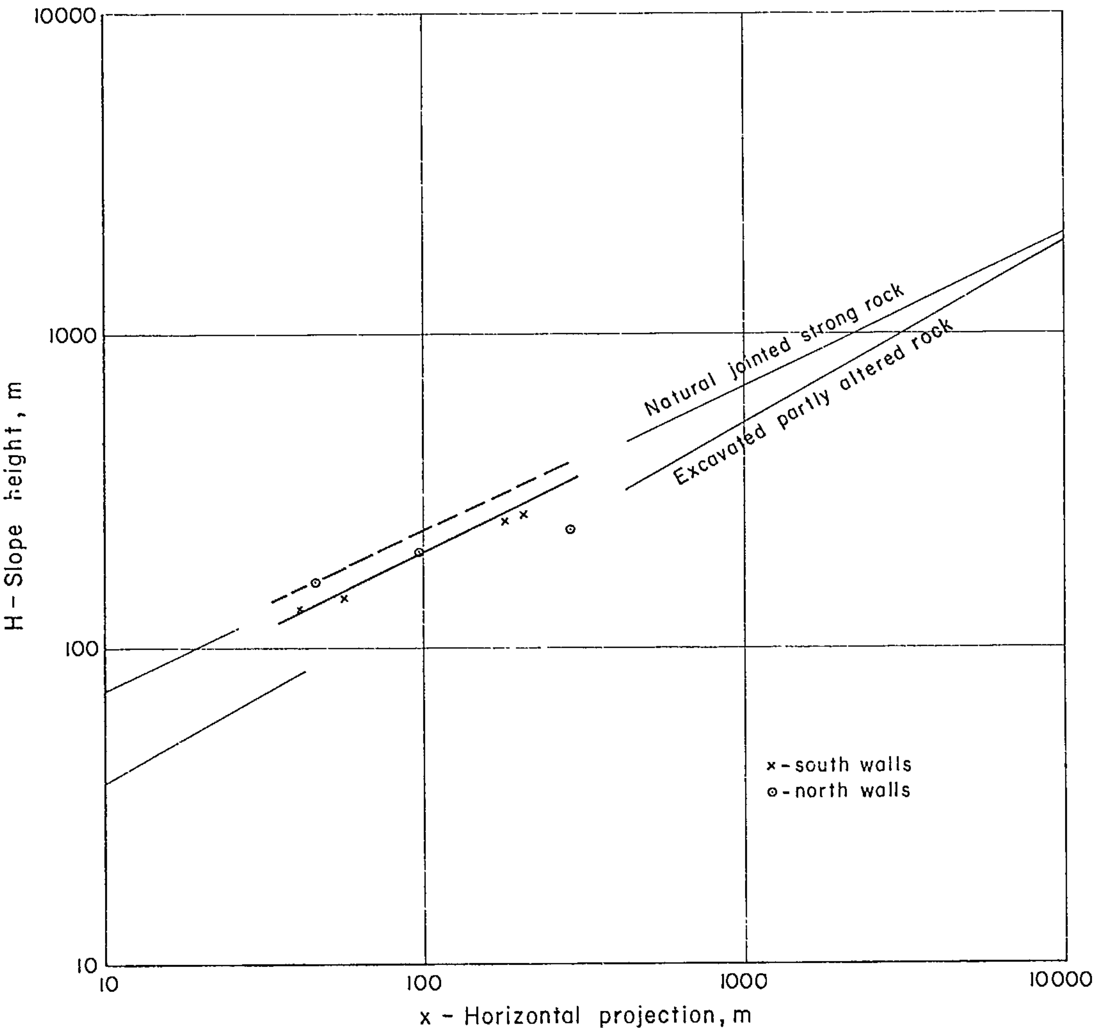


Fig. C-12 - Data from prior excavated slopes in a series of open pits on a porphyry copper property. The points representing the steepest slopes for various heights in the south walls produce a curve that is more consistent with the general regression lines than the data points for the north walls. The excavated slopes undoubtedly, in effect, included different safety factors, which could account for this deviation.

valid for extrapolation. As mentioned above, different safety factors in these slopes could account for their not providing a conventional line. In this case, passing a curve through the uppermost and the convergence point would probably give the most valid curve for extrapolation. Some slopes on this property in which slides occurred would plot below these two curves. This is a common experience and, when thought about in the stochastic terms of the text, is quite understandable. This recognition must be included in exercising judgement in using such curves.

62. Depending on the relevance of the previous slopes and the importance of the wall it may be appropriate to conduct a detailed investigation. Detailed structural geology, mechanical properties and groundwater data would be useful. Some differences from the pit wall might exist in orientation of critical structural features, joint lengths, spacing, mechanical properties and groundwater regime. A new envelope curve could be constructed using an incremental analysis to account for the change in critical slope angle caused by such differences (C-7). This would extrapolate from the experience curve to the conditions applicable to the wall rock.

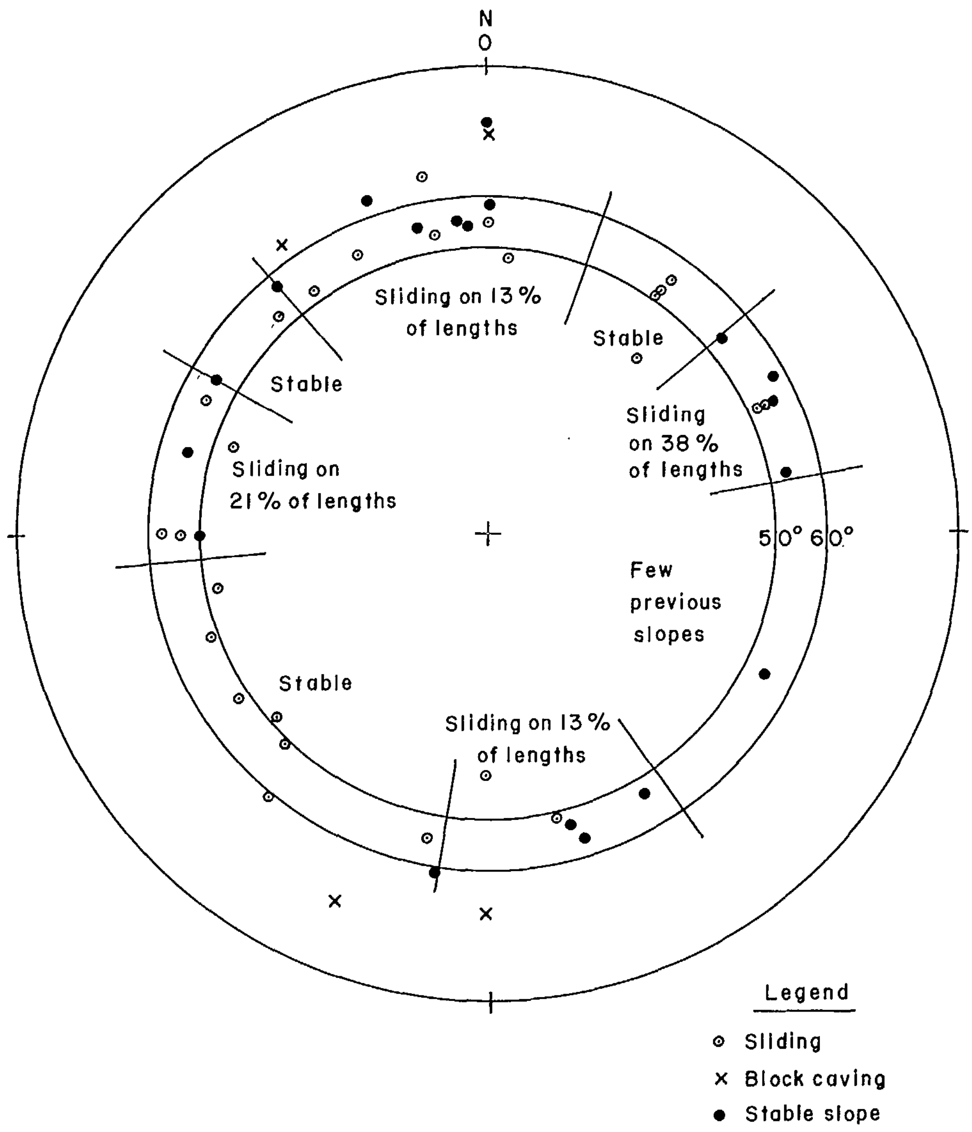


Fig. C-13 - Poles of previous slopes plotted on a lower hemisphere Schmidt diagram. 3489 m of slope lengths (parallel to the face) were surveyed in several previous pits and other slopes; '% of lengths' applies to the summation of lengths of various slopes surveyed (after ref C-6).

63. A similar procedure could be used to account for the effects of horizontal curvature. For example, if the experience curve is for slopes with no horizontal curvature and the pit sector has a radius of curvature, R, of 1000 ft (305 m) at the crest and a height, H, of 500 ft (152 m), the appropriate comparison for the design slope would be with the slope angle from the experience curve plus $I = 3/(2R/H - 3) = 3°$ (see Supplement 5-2 for an explanation of the equation).

64. Where extensive structural information is available on previous slopes, it is useful to plot orientations of discontinuities on a Schmidt diagram. Figure C-13 shows data from several previous pits in the area (C-6). The diagram indicates the feasible wall angles for each quadrant of a new pit.

TRIAL SLOPES

Need

65. It is appropriate in some cases to plan for the excavation of trial slopes. In a sense, all excavated slopes are trial slopes from which operating personnel develop a feel for what is feasible and what is not. However, if it is suspected that assumptions made about rock strength have been very conservative, trial slopes could be planned with the objective of eliminating such conservatism. For example, the specific objective might be to determine if the surveyed sets of joints could combine to form a stepped surface for sliding. The potential benefits of such trials could be great. If it is shown that slopes could be steepened, large reductions in waste excavation could occur.

66. The planning of any such trial slope should include awareness of the stochastic nature of sliding. In other words, behaviour of a wall composed of the same formation can change from section to section because of geologic variations. It is, therefore, not usually possible to plan a trial slope based on the reasoning that if it proves to be stable at 60° up to a height of 150 ft (46 m), then the ultimate wall will be stable at, say, an angle of 55° for a height of 500 ft (152 m); in other words, the probabilities of geologic variation must be considered. Therefore, the objectives, both technical and economic, of any such trials must be planned in some detail. To obtain appropriate statistical data, it might be necessary for a trial slope to encompass a considerable length of wall.

67. If the rate of return of the mine investment is particularly sensitive to wall angles and if operations would not be unduly affected by the implementation of a trial slope in the early years of the mine, then it could be advantageous to conduct a trial. Utilizing information from the trial will essentially be the same as using information from stable and unstable slopes. To clarify these aspects, a report should be written describing the purpose, details and anticipated output from such an experiment.

68. It is not possible to specify when trial slopes should be used nor to establish rigid specifications for their design and excavation. Each mine is different, with different operational requirements and different geologic conditions. The mine planner must consider the usefulness of trial slopes for the particular objectives

Fig. C-14 - A trial slope to determine if it would be feasible to maintain 60° for a depth of 1000 ft (305 m). It turned out that it was difficult to maintain adequate catch benches.

selected. Any trial must then be designed using the basic principles of mine planning and stability to work out a practical scheme.

69. Large properties may ultimately contain several pits throughout the foreseeable future. These pits may be excavated in substantially the same type of rock. Utilizing an appropriate wall in one of the early pits for trial purposes could result, even on a discounted cash flow basis, in considerable benefits in subsequent operations. Such a trial slope might be oriented more towards operating considerations than overall stability research. For example, the question could be asked - how would the surface rock behave and how would operations be affected by trying to mine an ultimate slope at an average angle of 60° for a height of 1000 ft (305 m)? Figure C-14 shows a photo of just such a trial where it was found that adequate catch benches could not be maintained without expending excessive money on perimeter blasting. The experiment cost little, and practical information of both a positive and negative character was obtained.

70. Many open pits are mined through a series of expanding shells or interim walls, which are often in substantially the same rock formations as the ultimate walls. This is often the case in mining a dipping tabular orebody in which the hanging wall is mined progressively down dip in a series of interim wall cuts. These situations favour some conscious experimentation.

71. Production considerations have top priority and usually require that interim slope angles be low enough to provide generous working space to permit maximum efficiency. However, there are pit configurations where interim slopes can be used for experimental purposes without unduly affecting operating efficiency. Areas where instability, should it develop, would not affect haulageways or surface installations and areas where only one working bench is utilized would be favoured.

Weak, Yielding Wall Rock

72. In a wall where the mode of potential sliding is by rotational shear, excavation of trial slopes can be particularly useful. Appropriate laboratory testing of samples of the wall rock will provide data on friction and cohesion. Usually such testing provides reliable values for the angle of internal friction and its variations from point to point in the formation. However, less confidence is placed in the laboratory measurement of cohesion, which may be different in the rock mass and is also likely to vary significantly (C-3).

73. It could be of benefit to design the slope of a section of wall to a depth that will be reached within two or three years. The angle could be selected to give a probability of instability between 25% and 50% according to laboratory tests using procedures described in Supplement 5-2. A monitoring system could be part of the trial slope. Useful information for determining slide surfaces can be obtained from deformation measurements preceding any instability. Groundwater monitoring would also be part of the program so that definite information is available for analyzing any slide.

74. If slides occur, analysis should proceed as described above. Information is obtained in this way on the effective cohesion and its variation. Confirmation or otherwise is also obtained on the expected location of the slide

surface.

75. From the experience of a trial length of wall, strength parameters used in design would be more realistic. Schedules of reliability versus wall height for various slope angles can then be used in the financial programs to determine a more realistic optimum wall angle. For example, in a trial wall 1000 ft long and 200 ft high, there would be five cells, each 200 ft wide. If a slide occurred at only one section, the frequency of instability would be 20%; if slides occurred at two sections, it would be 40%. Such experience could then be compared with the predicted probability of instability based on laboratory tests. Adjustments, if necessary, would be made in the mean value of cohesion, assuming that the laboratory test values are a valid measure of variance of field values in the wall.

Strong, Jointed Wall Rock

76. The feasibility of using trial slopes must be examined carefully where the mode of potential instability is by simple plane shear. It can usually be assumed that mean values and standard deviations of the strike and dip of the discontinuity are known and that laboratory testing provides good values of the mean and standard deviation of the friction angle. The difficult factors to determine are: (a) effective cohesion on the discontinuities, (b) dilatancy or roughness factor, (c) length of discontinuities and (d) proportion of rock bridges that occurs between individual discontinuities. For these four factors, it is necessary to determine mean values and a measure of their dispersions, which means that there are eight factors for which confirmation would be desired. If a trial wall were designed that produced sliding, it would not be easy to select values for all eight unknowns in a post-slide analysis that explained the instability. On the other hand, there may be cases where only one plane is involved and the dilatency factor can be measured; in this case the only factor to be determined would be the average cohesion on that plane. Also, if the geology was consistent throughout the wall then a trial slope could be feasible.

77. In rare circumstances, it is practical to excavate a trial slope in one section at a steep slope angle. In this way it might be possible to determine the combined effect of the unknowns cited above. Having obtained this information, the wall could be mined to deeper levels at a slope angle in which considerable confidence could be placed. Figure 50 shows such a case where it was proved that discontinuities dipping at angles lower than 59° did not provide effective sliding surfaces. Consequently, the wall was mined below the 500 ft (153 m) depth at 59°, rather than the original 45°, to obtain additional ore that paid for the research on the trial slope.

78. Where the potential mode of instability is expected to be by 3-d-wedge sliding, the number of unknowns or factors to be confirmed could be twice that for the two-dimensional case. Under these circumstances, considering there may be 16 unknowns, the decision to design a trial slope would require a careful examination of the purpose and data to be obtained from such an experiment.

79. In any of these cases, good quality supporting information is required from piezometer installations, structural mapping and monitoring of deformation. Measuring the volume of backbreak could also be of practical value in future planning, ie to calculate volumes and to establish actual bench geometry. This can be done by determining the difference in profiles from that assumed in design to that obtained by surveying in the field.

MONITORING

Slides

80. When instability starts to develop in a slope, an intensive monitoring system must be established in the area (see Chapter on Monitoring for details).

81. Frequency of observation varies according to rate of movement. For critical cases one deformation measurement for approximately every 0.5 m (1 cm) of movement is recommended. For those that are not particularly critical, approximately 1 in. (2 cm) is an adequate interval.

82. For example, in typical situations immediately after initial movement, observations five days, thereafter at weekly intervals for four weeks and then for further periods at frequencies of one per month and one per six months. If the slide material has not been removed and still constitutes a hazard, then monitoring is continued until any risk has been eliminated. Deformations, d, and deformation rates, d', immediately preceding major movements can be valuable when monitoring future instability. Dividing by the slope height, H, to make them dimensionless the ratios d/H and d'/H provide guidance on what are likely to be critical movements for other slopes.

83. The deformation behaviour of newly excavated slopes, if recorded, is of some use. Largest movements occur at the surface of the slope. Measurements here can be used to determine the average elastic modulus, E, of the rock mass by using Fig.C-15.

84. These observations must be made over the time the slope is being excavated, which can be a considerable period. Hence, only in special cases will the sustained effort be worth the cost, particularly as the usefulness of E is substantially based on the assumption that the strength of the rock mass is proportional to E. The use that can perhaps be made of such an extrapolation is in analyzing very high slopes for block flow instability. At this stage such analyses are quite speculative although the need can be real.

85. Recordings in these cases should be made at weekly intervals for a period of two months or more. Sufficient time is required to establish

the trend of the deformations. Following this stage, the interval between observations can be increased to monthly and up to semi-annually depending upon the rate of excavation. The interval between observations should be no longer than to permit at least four observations for each slope height, ie before a new cut increases the height.

86. In slopes consisting of more than one material, deformation measurements within each material would be useful. Such observations can be made with either multiple borehole extensometers or inclinometers(see the Monitoring Chapter for details on instruments). The moduli of elasticity for the different materials could be estimated using the deformation measurements and a finite element model. The moduli for the model materials can be selected to produce the surface and borehole measurements actually obtained. With borehole measurements, the basic relation that $E = \Delta\sigma/\Delta\varepsilon$ might be used, where $\Delta\sigma$ is the change in stress between the two points in the model and $\Delta\varepsilon$ is the change in strain between the two measuring points being equal to d/L, d is the difference in displacement of the two points and L is the distance between them. Field measurements of E by a borehole dilatometer might be used to compare with the calculated values. Such studies are only likely to be warranted for slopes well over 1000 ft (305 m) high.

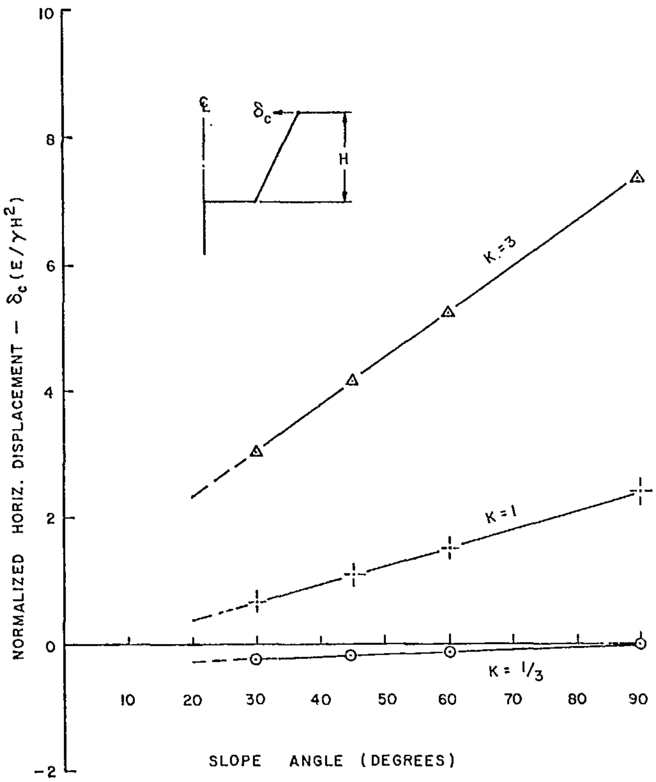


Fig. C-15 - Crest displacement in plane strain finite element models, δ_c, normalized by E, the modulus of deformation of the rock, γ, the rock density, and H, the slope height. The displacement resulting from excavation increases with slope angle and with K, the ratio of horizontal to vertical stress in the undisturbed ground.

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