PIT SLOPE MANUAL

chapter 1

SUMMARY

This chapter has been prepared as part of the

PIT SLOPE PROJECT

of the
Mining Research Laboratories
Canada Centre for Mineral and Energy Technology
Energy, Mines and Resources Canada

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THE PIT SLOPE MANUAL

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

The chapters and supplements can be obtained from the Publications Distribution Office, CANMET, Energy, Mines and Resources Canada, 555 Booth Street, Ottawa, Ontario, K1A 0G1, Canada.

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FOREWORD

Open pit mining accounts for some 70% of Canada's ore production. With the expansion of coal and tar sands operations, open pit mining will 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

PREFACE

The decision to prepare an engineering manual for the design of open pit slopes followed studies of potential benefits and costs of performing research on rock slopes. These studies, done for the Mining Research Laboratories by Don Coates and the late Amil Dubnie, showed that it should be possible, by improving design procedures, to excavate many pit walls closer to their optimum angles. In many cases this would lead to substantial benefits, either from reduced waste excavation or increased ore recovery. It was estimated that, in 1970, these benefits might have been about $40 million.

In 1972, the Pit Slope Project was begun with the goal of developing improved slope design procedures. The importance of assistance from the mining industry was recognized from the beginning. Accordingly, the project has been a cooperative venture between industry and the federal government. The planning and coordination of the project, and the drafting of the chapters, have been the responsibility of the Mining Research Laboratories. Much of the development work has been done by Canadian mining companies, consulting engineers and universities. Occasionally, additional experience has been called upon from outside Canada.

So many individuals from all aspects of mining have participated in the project that for the most part their contributions cannot be specifically acknowledged. Where possible, individuals are cited in the acknowledgements to each chapter of the manual. However, the particular contribution of Don Coates should be recognized here. He was largely responsible for the original concept. In the five years required to bring the manual to fruition, he has been the driving force and inspiration behind the many and varied efforts required.

The Pit Slope Manual is meant to be a practical working tool for mine staff to use on their properties. There will always be problems that may require experience and expertise beyond the guidance supplied by the manual. However, at the very least mine staff must recognize the need for, and be able to implement, rational slope design. The Pit Slope Manual is intended to place this capability into the hands of all Canadian open pit operators.

Ottawa, Canada | R. Sage
July, 1976

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INTRODUCTION

1. Designing pit walls is an important part of open pit mine planning. Wall layout affects both the amount of ore recovered and the amount of waste excavated. It therefore greatly affects the profitability of mining. The object of slope design is to determine the layout that maximizes the economic benefits of mining.

2. When deciding the angle at which to mine pit walls, the slope designer must offset the cost of possible slope instability against the cost of waste excavation. The steeper the wall the more likelihood there is of instability. This may result in costs for cleaning-up, delayed production or loss of ore. Flatter walls, on the other hand, are more stable but result in higher cost for excavating waste. The optimum design minimizes the combined cost of excavating waste and of any instability which may take place.

3. Conventional methods of slope design do not allow the economic influence of pit layout to be included in mine planning. One reason is the implication - inherited from civil engineering - that instability of any kind must be avoided. It is not recognized that, in mining, instability will be economically advantageous if any cost incurred - including the cost of ensuring safe working conditions - is offset by the lower cost of waste removal resulting from steeper slopes.

4. A second disadvantage of conventional design methods is that the variable nature of soil and rock is not recognized. This variability prevents precise determination of slope stability. From time to time instability will occur in slopes that analyses indicate are stable. A better approach is to determine the likelihood that a slope will be unstable. The risk of a slide can then be assessed and included in the risk analysis of the complete mining venture.

5. The Pit Slope Manual describes procedures to determine the risk of slope instability. It also explains how to incorporate the benefits and costs associated with steeper slopes into overall planning. These procedures are based on reliability theory. In this, the variabilities of the many factors influencing stability are measured. These are incorporated in an analysis which determines wall reliability - that is, the probability that a wall or part of a wall will remain stable.

6. The reliability approach recognizes that there may be instability, which has associated costs. The manual gives methods of estimating these costs. Knowing these and the mining cost the planner can determine the costs and benefits of a given wall layout and the associated risks. This information is used in making mine investment decisions and in selecting the optimum layout.

7. Three stages of mine development are recognized in the manual. The feasibility stage covers investigations and analyses needed to make a commitment to mining. Preliminary pit designs must be drawn up in this stage. Much of the field data can also be gathered as part of the orebody evaluation program.

8. The mine design stage follows the commitment to mining. Intensive field data gathering is required, together with complete analyses of the potential pit walls and incorporation of results into mine financial risk analysis.

9. During the operating stage, additional data is collected, taking advantage of the large surface areas exposed by mining. The assumptions made for mine design are verified and, where necessary, redesign is carried out.

10. Pit slope design begins with investigations to determine properties of the material to be mined. Chief among these is Structural Geology because, in rock slopes, instability is usually caused by discontinuities (faults, joints and bedding planes). Next is testing of the slope material to determine Mechanical Properties such as compressive and shear strengths. The third activity concerns Groundwater which is significant to mine operations in general and to slope stability in particular.

11. Data gathering is followed by slope stability analysis and financial analysis to select the optimum pit layout. This is the Design activity and requires consideration of ore values and operating methods as well as of wall stability.

12. When instability does occur, remedial measures may be required. Critical slopes can sometimes be stabilized by increasing rock strength through Mechanical Support. The quality, and therefore stability, of walls can be substantially improved by controlled Perimeter Blasting.

13. Slope design must ensure safety. This can be achieved by careful Monitoring of slope movements during mining. This activity also includes the verification of the assumptions used in mine design.

14. Each of the activities underlined above is the subject of a chapter in the manual. Two additional chapters describe the special requirements for designing Waste Embankments and the role of Environmental Planning in open pit mines. A summary of each of the nine chapters now follows.

STRUCTURAL GEOLOGY

15. Most rock slope instabilities are caused by discontinuities - joints, faults and bedding planes. Discontinuities also influence groundwater flow which in turn affects stability.

16. Stability analyses must account for these effects. The structural geologist must determine the location, orientation and extent of discontinuities and help determine how they will affect slope stability.

DATA GATHERING

17. The first step in a structural geology investigation is to review existing geological information and determine regional geology. From this, stability problems can be assessed and a program of geological investigation developed. Even where previous information is lacking, study of the general characteristics of the rock formations around a proposed mine can identify typical problems. Studies might include climatic effects, weathering, zones of shearing and faulting and rock types associated with unstable conditions - for example, solution cavities in limestone.

18. The initial assessment is followed by detailed data collection. This is best done by on-site mapping of the orientation, length, spacing and appearance of discontinuities and rock type distribution. Subsurface geology can be mapped by logging drill core. The objective is to obtain a fully representative record of all discontinuities that might affect slope stability.

19. Plane table photography and terrestrial photogrammetry are useful to supplement in-pit mapping. Plane table photography (Fig 1) is the simpler. Terrestrial photogrammetry is more accurate but also more expensive - it is economic only when a large number of joints must be mapped. Surface and borehole geophysical measurements provide useful information on the distribution of soil and rock types, and on rock fracturing. Surface geophysics in particular is straightforward and can be carried out with low-cost commercial equipment.

DATA STORAGE

20. Geological investigations produce large amounts of data which must be recorded accurately and clearly and be readily available.

21. The traditional approach to handling data is to plot major discontinuities and rock types on plans and sections, and to analyze data by graphical or numerical methods. This requires experienced staff. Computers provide an alternative approach to handling geological data. This has the particular advantage that an engineering geologist can break down the task of mapping and analyzing into a series of simple steps which can be performed by relatively inexperienced personnel.

22. The use of computers requires a standard approach to geological mapping. A suitable approach and associated field guides and computer program package - called DISCODAT - have been developed for the Pit Slope Manual (Fig 2).

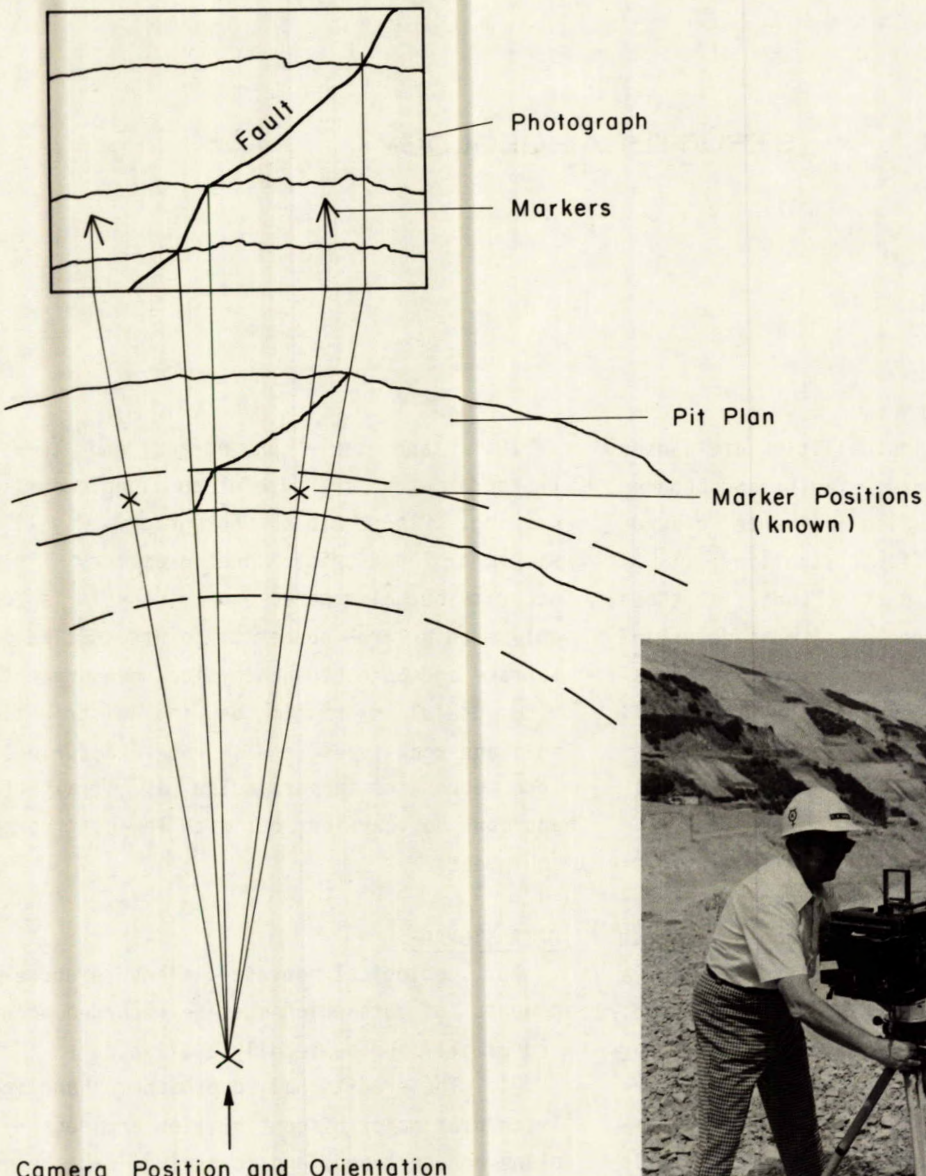
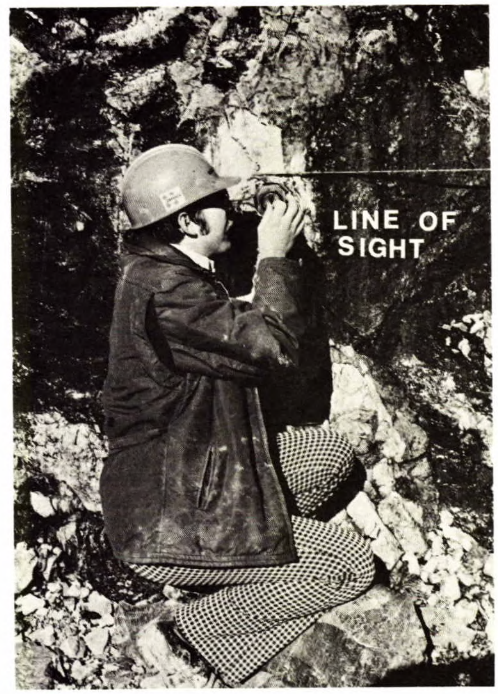


Fig 1 - Plane table photography for mapping pit faces with a long focal length camera and pit survey plan (courtesy of Kennecott Copper Corporation).



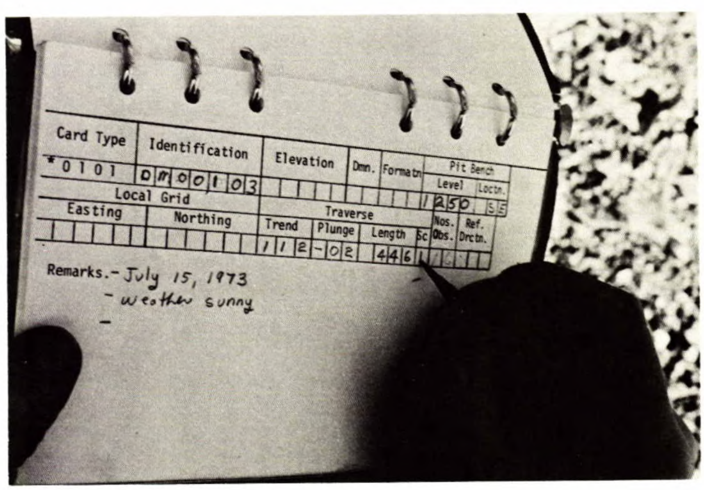


Fig 2 - Field mapping procedures using a Brunton compass. Top left: measuring a traverse trend. Top right: measuring traverse plunge. Bottom: recording information for the computerized line mapping system.

DATA REDUCTION

23. Geological data for slope stability analysis fall primarily into three groups - rock types, major structures and minor structures. Figures 3 and 4 show how such data are presented. The slope designer requires additional information, in particular rock strength and geotechnical properties of the major and minor structures.

24. Geological field observations should include descriptions of characteristics, such as hardness, from which preliminary estimates of rock strength can be made. The geotechnical characteristics of major and minor discontinuities must be recorded. These include location, orientation, length, spacing, waviness and strength of fillings and strength of fracture walls. Major discontinuities are described and mapped individually. Minor features such as joint sets are treated statistically; for example, their spacing is described by the dispersion about a mean value.

25. The most important characteristic of structural discontinuities is their orientation. The statistical treatment of orientation requires the plotting of observations in equal area nets. This reveals the grouping or clustering of discontinuities with similar orientations. Figure 4 shows typical clusters of joint orientations for an open pit. The mean or representative value of these clusters and the dispersion are determined statistically.

26. The length and spacing of fractures is important because these affect how a series of fractures may link together to form a surface of sliding. Determining exact fracture length and spacing is difficult. Present techniques can provide, however, estimates of the trace length and spacing of a joint. These can be used for assessing the extent to which fractures form slide surfaces.

27. Waviness and roughness add to the shear resistance of a rock fracture during sliding. Roughness refers to small irregularities on a rock surface. Waviness refers to the longer wave length undulations which would probably survive shearing. These characteristics, together with estimates of the strength of fracture fillings, are recorded.

28. Groundwater occurrences should be recorded during geological mapping. They should be classified in a range from just moist to actual seepage. These observations guide subsequent groundwater investigations.

DESIGN SECTORS

29. When data collection and interpretation are complete, the engineering geologist should divide the pit into preliminary design sectors. In each sector the types of discontinuities and their orientations to the proposed pit walls should be similar. Sectors can be selected by assuming an arbitrary pit slope angle of 50° and outlining the proposed pit on a map showing major structures, rock types and minor discontinuities. The pit outline can be approximated by straight lines and design sectors delineated (Fig 5). Structures with the same orientation may have different effects on opposite walls of the pit. Straight or curved sections of the pit may also affect slope stability in different ways. These factors must all be considered.

30. At this point the geologist should assess which modes of instability may occur in each of the preliminary design sectors. This provides a focus for structural geology investigations and also, by overlapping the design process, creates continuity between the activities. Three basic instability modes are considered - plane shear, rotational shear and block flow.

31. An example is given for design sector III of Fig 5. For this sector, field observations have provided the following details:

Rock substance - siliceous dolomite
Rock strength - R4 - very hard rock
Joint orientation - Set I 310/40*
Set II 208/81*

Bedding planes(infrequent) - 118/64*
Hanging wall shear - 125/60-65*
(gouge filled)

Assumptions are that the slope face is 320/50*, and the angle of friction is 25°. Orientations (marked *) are dip direction/dip.

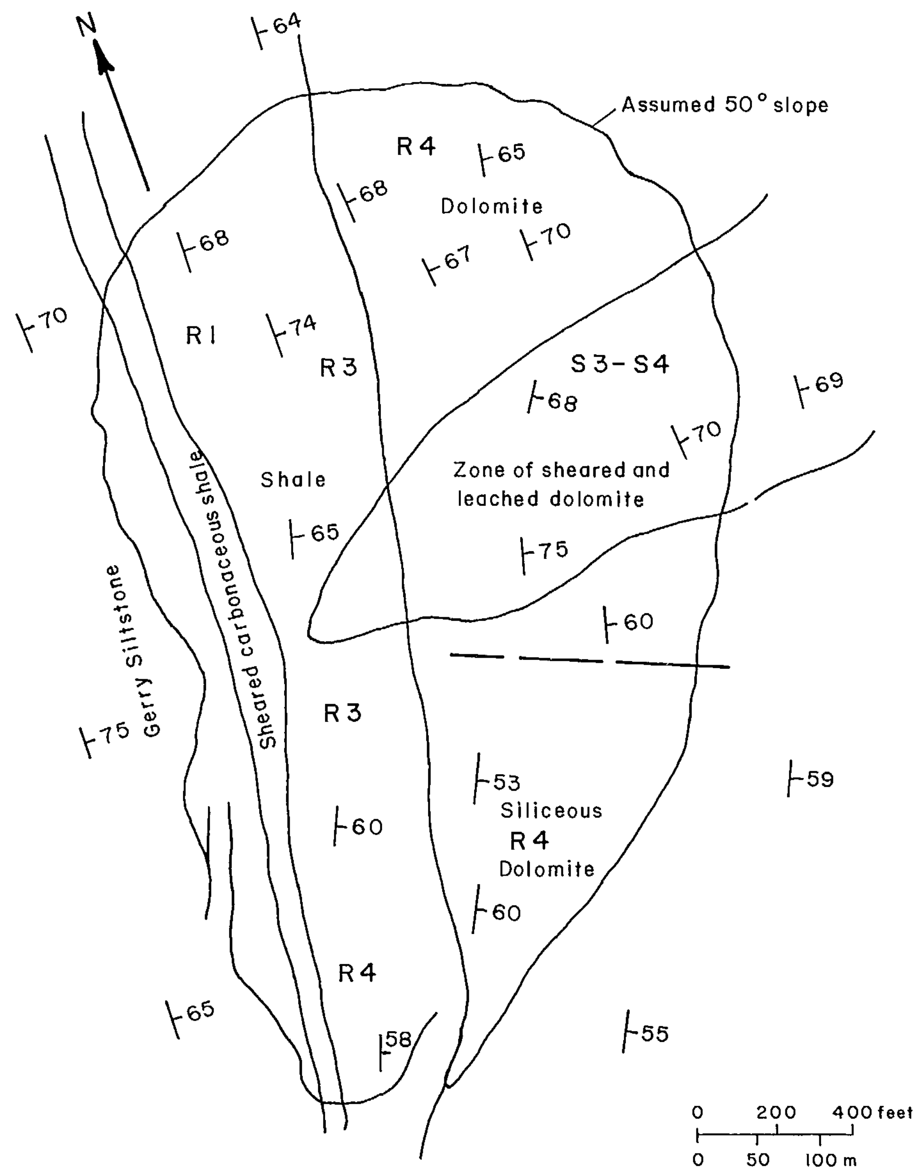


Fig 3 - Sketch map showing rock types and bedding plane orientation for a pit site.

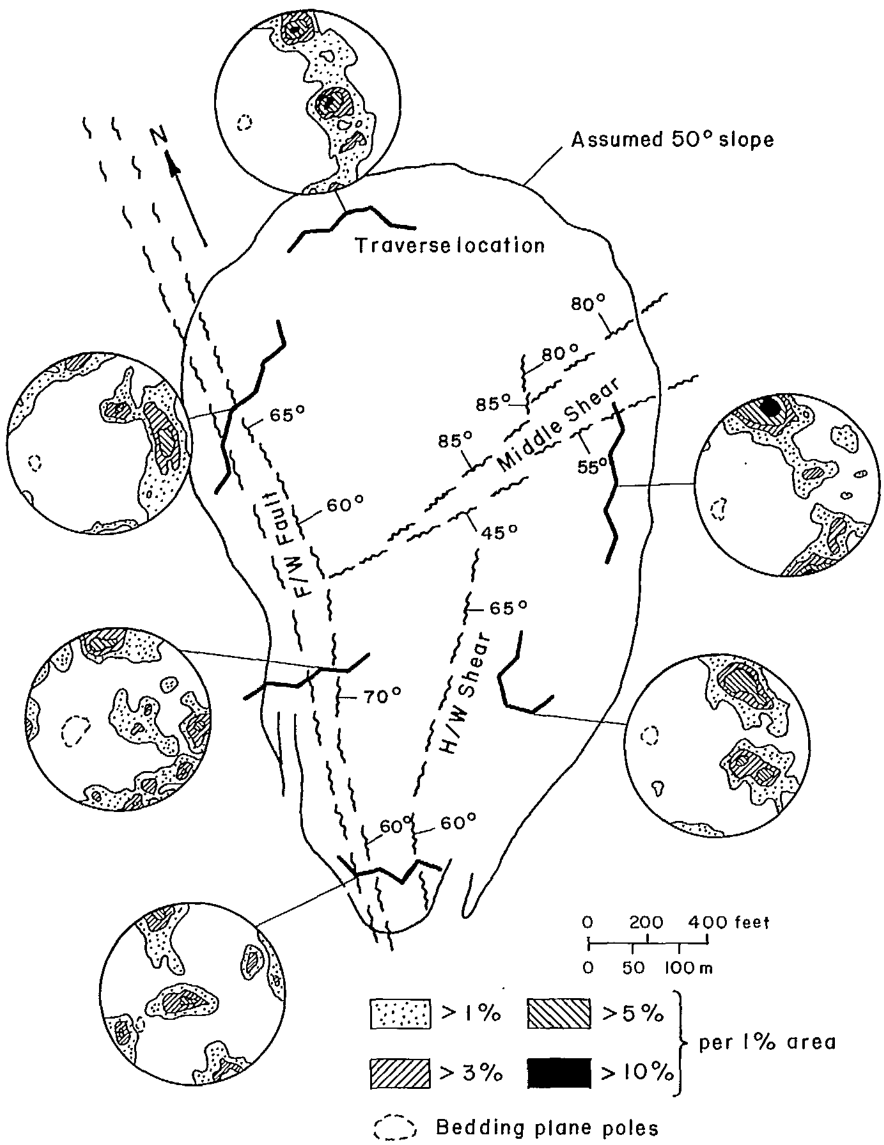


Fig 4 - Sketch map showing major structures and orientation of minor discontinuities.

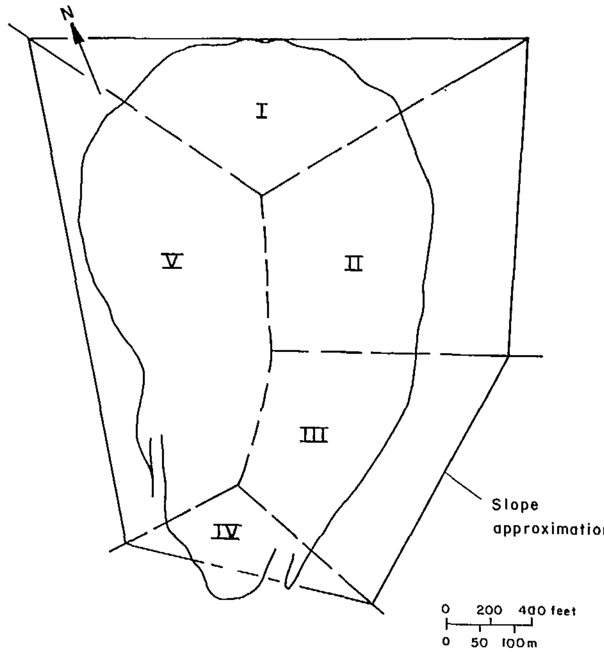


Fig 5 - Selection of design sectors.

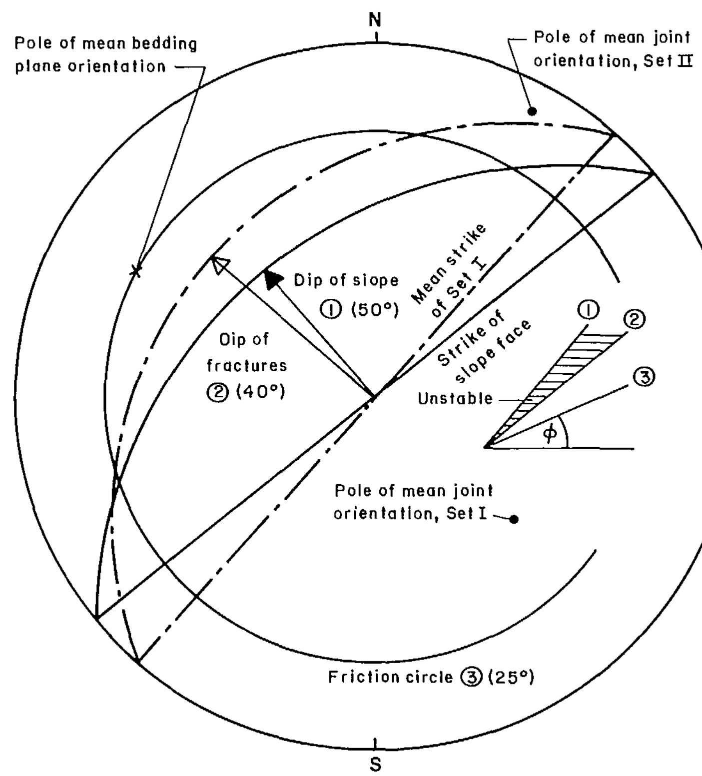


Fig 6 - Preliminary assessment of possible instability modes for design sector III of Fig 5.

32. In Fig 6 the mean orientations of discontinuities and bedding planes are plotted as poles. For discontinuity set I, which has a potentially critical dip into the pit, the trace is plotted and the friction circle for a friction angle of 25° is drawn. With the slope face plotted as 320/50, preliminary conclusions are that plane shear sliding along set I is possible, and that rotational shear through the very strong rock is unlikely.

MINE DEVELOPMENT STAGES

33. Collecting and analyzing structural geology data continue through all stages of mine development.

34. The objective in the feasibility stage is to obtain a reasonable knowledge of the geological structure and possible instability modes. Investigations are limited to regional geology, reconnaissance mapping and logging of core drilled for orebody evaluation. However, even these limited data permit an assessment of feasible slope angles and may make the difference between a profitable and an unprofitable operation.

35. In the mine design stage, all relevant data must be collected to furnish the best possible information to the slope designer. Fault locations must be determined, the mean and dispersion of joint sets and bedding planes evaluated and the distribution of rock types established.

36. During the operating stage, data collection should continue, taking advantage of the slope surfaces exposed by mining. The objectives in this stage are to verify the geological assumptions used in design, and to make fresh data available for slope redesign. Structural mapping of benches should be part of the routine work of a mine geology department.

COSTS

37. The amount of structural information required for mine planning varies with the type of open pit. A small or shallow orebody will need less investigation than a large and particularly a deep pit. The waste/ore ratio and whether the orebody has geological or assay boundaries will influence investigations. Complex geology will require more investigation than a simple geological environment.

38. The costs of structural geology investigations vary accordingly. For guidance, approximate costs in 1974 dollars are shown in Table 1.

Table 1: Cost estimates for structural geology investigation

Stage:	Feasibility		Design		Operating	
Depth of Pit	Man days	Cost	Man days	Cost	Man days	Cost
< 100 ft (30 m)	5	1500	15	4500	35	10500
100 - 1000 ft (30-300 m)	15	4500	40	12000	120	36000
> 1000 ft (300 m)	20	6000	50	15000	160	48000

MECHANICAL PROPERTIES

39. Slope stability analysis requires the measurement of material strengths by appropriate field and laboratory tests. Properties used in calculating forces and displacements - for example, density and elastic modulus - must also be measured.

TESTING PROGRAM

40. The large volume of soil and rock involved in open pit mining means a variety of mechanical properties is encountered. For example, not only does each rock type have different inherent characteristics, but within a given rock type properties may vary because of alteration and the presence of discontinuities. A well planned program of field and laboratory tests is required if all relevant mechanical properties are to be determined.

41. Investigation of mechanical properties should be based on the preliminary pit zoning done during structural geology investigations. This zoning includes estimates of mechanical properties and expected instability modes for each sector. These indicate the mechanical properties which must be measured.

42. A test program should be drawn up for each design sector. The factors to be considered are:

- the volume of material to be mined and the significance of the sector to the mining operation;
- specific locations and methods for sampling to ensure that test specimens are representative;
- the cost of sampling and specimen preparation;
- the type of material to be tested, the availability of testing facilities, the nature and accuracy of the information required and the cost of testing.

The test program is best presented in tabular form for ease of reference and convenience in subsequent reporting. Part of a hypothetical program is shown in Table 2.

Table 2: Hypothetical Test Program

Required mechanical property	Priority	Testing method	Test result to be used for:	Sampling source and method	Extent of testing	Remarks
Shear strength of α_3 joint system Sector B	1	Laboratory direct shear on irregular specimen	joint system α_3 of sector A; joint system α_3 of Sector B	Hand picked on face	complete	
		Laboratory direct shear on core specimen	joint system α_3 of B-23 area, Sector B	NX holes at 21-17, 21-21 and 21-38	limited for checking purposes	
Shear strength of α_2 joint system, Sector A	1	Laboratory direct shear on core specimen	joint system α_2 of Sector A; joint system α_4 of Sector B; all joints of Sector G	core samples of H-51 drill hole, below elev 650	complete	
		Laboratory direct shear on core specimen	Top portion of Sector A; joint system α_4 B-23 area Sector B	core samples of H-51 drill hole, above elev 650	limited for checking purposes	
Compressive strength of dolomite	2	Point load	altered B-23 area of Sector B	core samples H-51, above elev 650	limited for checking purposes	Otherwise use test results of Test Report: 16-73 for dolomite in entire pit
Rock mass density	3	Drill hole test	Sectors A,B, G and K	production blast holes	complete	
etc.			etc.		etc.	

TESTS

Discontinuity Shear Strength

43. The shear strength of discontinuities is the most important strength parameter in rock slope stability analysis. Sliding on discontinuities is the most common form of instability.

44. The best method of determining the shear strength of a discontinuity is to perform an in situ test. The large scale possible in such tests - say on a surface 3-5 ft (1-2 m) square - means the effect of surface irregularities on the shear strength can be measured and results are more likely to be truly representative of field conditions. However, these tests are expensive.

45. The test set-up is shown in Fig 7. A block of rock on the discontinuity to be tested is isolated by cutting away the surrounding rock. Hydraulic jacks are used to provide a normal and a shearing force. In places it may be possible to perform the tests in an exploration adit with the tunnel walls providing a surface to jack against. Above ground, however, rock anchors or bolts are required to provide jack reaction.

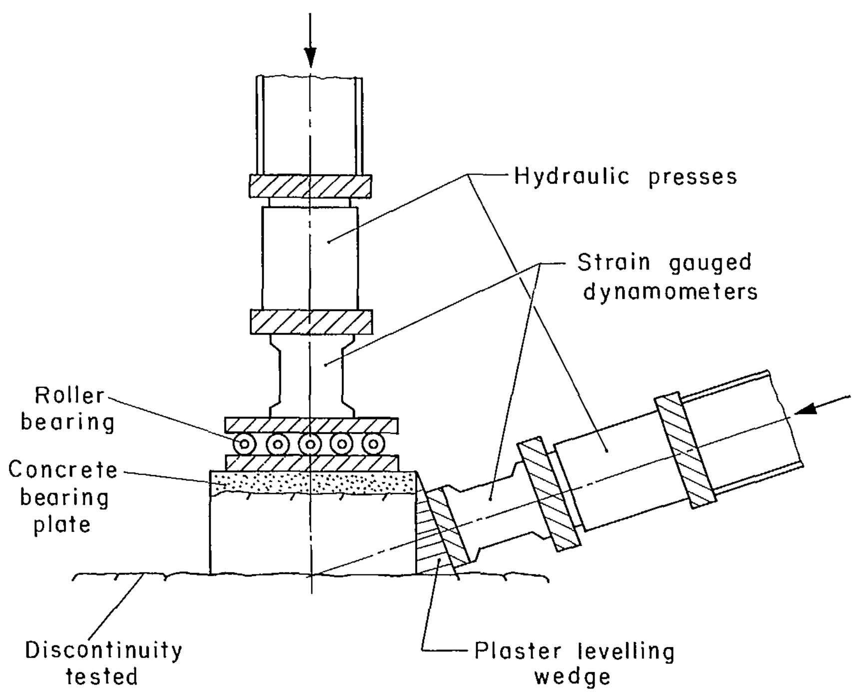


Fig 7 - Field shear test.

46. Where in situ tests are not used, laboratory tests are required. There are two basic tests, triaxial and shear box.

47. Triaxial apparatus is shown in Fig 8. The main feature is a chamber filled with fluid - usually oil or water - under pressure. This exerts a lateral stress on the specimen. An axial stress is applied through sealed bearing plates. The discontinuity to be tested is oriented roughly at 45° to the vertical. In this case, the shear stresses on the discontinuity are close to a maximum and this ensures failure by sliding on the discontinuity. The test pressures should correspond to the likely range of stresses in the slope being investigated.

48. The shear box is shown in Fig 9. A normal and a shearing force are applied by jacks and the discontinuity is oriented so that displacement of the box halves causes shear movement. Usually the specimen must be set in plaster or cement. A portable unit that can be used in the field is commercially available.

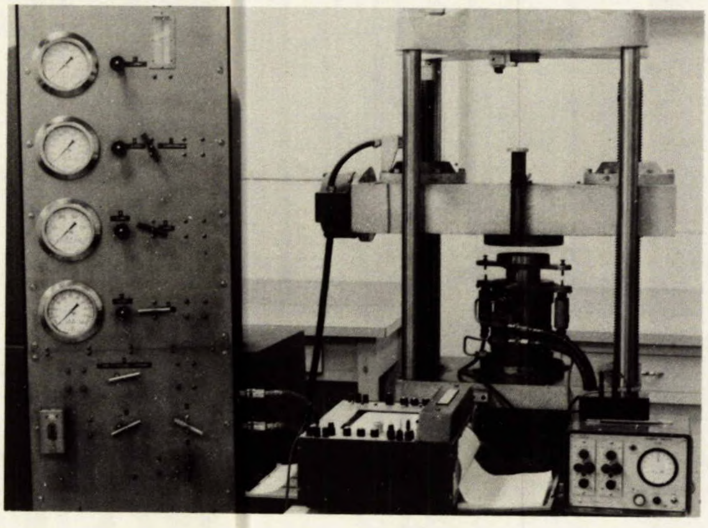


Fig 8 - A triaxial test machine with automatic data recording.

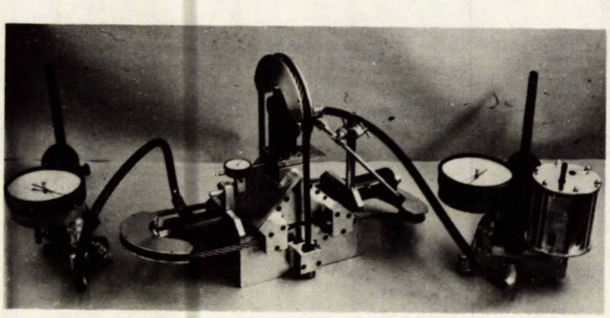


Fig 9 - Shear box for tests on small samples. The equipment shown here is portable.

Substance Shear Strength

49. Substance refers to intact rock or soil material. Substance shearing may occur in homogeneous weak masses such as soil overburden and weak rock. It may also occur in heavily altered and weathered rock with a random geological structure, so that preferential sliding surfaces do not form. Shearing of infill on discontinuities may also occur.

50. The in situ test for discontinuity shear strength can also be used to determine the rock substance strength. The principles are as shown in Fig 7, except there is no discontinuity. In situ testing on a large scale is expensive and laboratory tests are more commonly used to determine rock substance shear strength.

51. Triaxial tests as shown in Fig 8 are the best method for laboratory determination of rock substance shear strength. The procedure is as described for discontinuity testing except that the sample is of intact substance. Failure invariably occurs on a plane inclined to the vertical axis, reflecting the peak shear stress distribution.

Analysis of Shear Tests

52. The triaxial, shear box and in situ tests require analysis to determine the strength parameters for slope design. This is done by plotting shear stress against normal stress; typical curves for rock are shown in Fig 10.

53. For rock, these three curves usually intersect at a single point T, known as the transition pressure point. For practical purposes, the transition pressure equals the uniaxial compressive strength, Q_u, of the rock substance. A diagram such as Fig 10 allows the strength parameters appropriate to the stress range under consideration to be selected for slope design.

54. The statistical variation in mechanical properties should be measured. Mean values and the dispersion about the mean for representative samples should be evaluated for each test, taking into account test accuracy and reproducibility. These values are used to determine the mean and dispersion of the design parameters.

Rock Compressive Strength

55. Compressive strength usually governs stability in homogeneous strong rock with a random geological structure. If large horizontal stresses (tectonic stresses) occur at the toe of a high slope, crushing of blocks of rock at the toe may occur. This may lead to progressive breakdown of the rock and a block flow type of instability.

56. It is not practical to measure the compressive strength of large rock blocks directly. Instead, the uniaxial strength of small representative specimens must be determined and the strength of large blocks estimated from this.

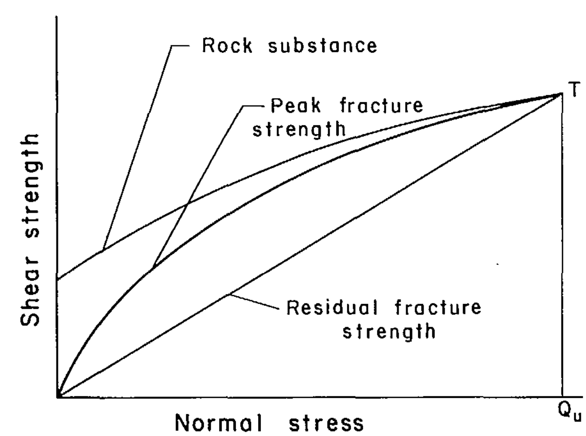


Fig 10 - Rock shear strengths plotted against normal stress. The top line represents rock substance strength. The middle line represents the peak shear strength of rough clean fractures. The bottom line represents the residual shear strength of a discontinuity - that is, after considerable shear movement has taken place.

Fig 11 - Point load test machine to determine the uniaxial compressive strength of rock.

57. Several tests can be used to determine the rock substance uniaxial compressive strength. It can be measured directly in a triaxial test with zero lateral pressure and can be measured by a point load test (Fig 11). In the latter, the load at failure is correlated empirically with the uniaxial strength. Irregular or random shaped specimens can be used.

58. The compressive strength of rock decreases with size because large rocks are more likely to contain flaws. In principle, the relationship between strength and size can be determined in the laboratory by testing progressively larger specimens. In practice, however, it is more convenient to use established empirical relationships.

Physical Properties

59. The three deformation properties - Young's modulus, Poisson's ratio and shear modulus - are used to calculate slope displacement during excavation, and the distribution of stresses. There is also a correlation between these properties and strength.

60. They can be determined in the laboratory by measuring changes in length and width of specimens under load. They can also be measured in situ by loading the surface of a rock stratum and measuring displacement, but these tests are expensive.

61. A more convenient in situ measurement can be made indirectly by measuring the speed of sound waves in the soil or rock mass. Commercial equipment for this purpose is readily available. Measurements can be made on the surface or in boreholes. The relationship between speed of sound and deformation properties is well established. Although it applies strictly to an ideal elastic medium, it does provide a good estimate of the deformation properties.

62. It may be important in some materials to measure the time-dependent deformation properties. These "creep" or "plastic" characteristics will affect slope design if they result in progressive breakdown of the slope face. They can be determined by applying a steady long term load and observing deformation until it effectively ceases. The test is carried out at various loads to determine the characteristics at the stress ranges likely to occur in the actual slope.

63. Index properties such as liquid and plastic limits for soils and grain size distribution in rock can be correlated with the properties required for design. They can also be used to classify material.

64. Index properties are also used to measure the swelling pressures and displacements that occur in some rocks and soils when wet. These phenomena can affect slope stability. Swelling indices are determined by immersing the material under controlled conditions and measuring the resulting pressures and displacements.

65. Density, which is required to determine weights in stability analyses, can be measured in the laboratory or the field. Porosity and water content can affect material behaviour. They can be determined from the normal, dry and saturated

weights of a sample of known volume.

66. Two important procedures are not described in the mechanical properties chapter. One of these is the determination of permeability, which governs groundwater movement and hence groundwater pressure, and is described in the groundwater chapter. The second is rock quality designation (RQD), described in the chapter on structural geology. It consists of logging the proportion of rock core with an intact length greater than 4 in. (10 cm). There is a correlation between RQD and rock mass strength.

MINE DEVELOPMENT STAGES

67. In the feasibility stage, testing must provide preliminary information for slope stability analysis at the least cost. Maximum use should be made of indirect tests that are relatively straightforward, such as measuring sound velocity, and index tests such as RQD. Direct testing is usually limited to drill core samples. Where possible, the best examples of discontinuities in core should be reserved for testing.

68. The major testing effort takes place in the mine design stage. Detailed information on material properties is then required, and efforts are made to plan an adequate program that will provide representative samples at reasonable cost. The test program is guided by investigations of structural geology, but considerable interaction is necessary between the testing, design and structural geology tasks. Preliminary design work may indicate drilling for samples is required. This would be integrated with the structural geology and groundwater tasks.

69. In the operating stage, testing is facilitated by the availability of samples taken while mining. The main requirement in this stage is to provide data for redesign, and also to amplify and verify the properties established previously, particularly as new areas are exposed.

GROUNDWATER

70. Groundwater concerns the slope designer because groundwater pressure reduces the shear strength of discontinuities. Resistance to sliding is proportional to the normal force acting through points of contact on the discontinuity. If water pressure acts on the surface, part of the normal force is transmitted through the water. Less force acts through the points of contact, and the frictional sliding resistance is therefore reduced (Fig 12).

OBJECTIVES

71. Groundwater investigations have two objectives:

a. to determine the groundwater pressures for use in slope design;

b. to determine ways of reducing groundwater pressure, through drainage or other controls, if necessary.

72. Slope stability analyses must evaluate the influence of groundwater pressure. If this is critical, methods of reducing pressure must be examined and their costs and benefits assessed.

PRESSURE DETERMINATION

73. Groundwater pressure can be measured directly by piezometers. The simplest is a tube sealed in a borehole. The lower end is open so that water can enter or leave. The level of water in the tube, or standpipe, is a measure of the

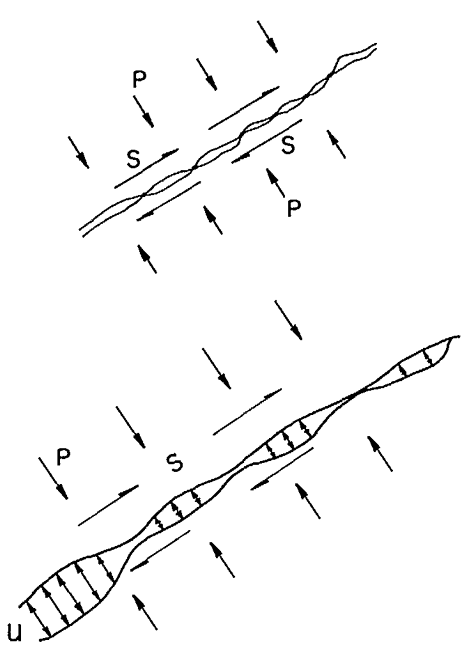


Fig 12 - Effective stress principle. In dry discontinuity shown in upper figure, stress P is transmitted through contact points. Shear strength S is proportional to force through contact points: $S = C + P \tan\phi$. In saturated discontinuity shown at bottom, stress P is transmitted partly through water stress U and partly through contact points. Strength S is now proportional to reduced force through contact points: $S = C + (P - U)\tan\phi$. P-U is the effective stress.

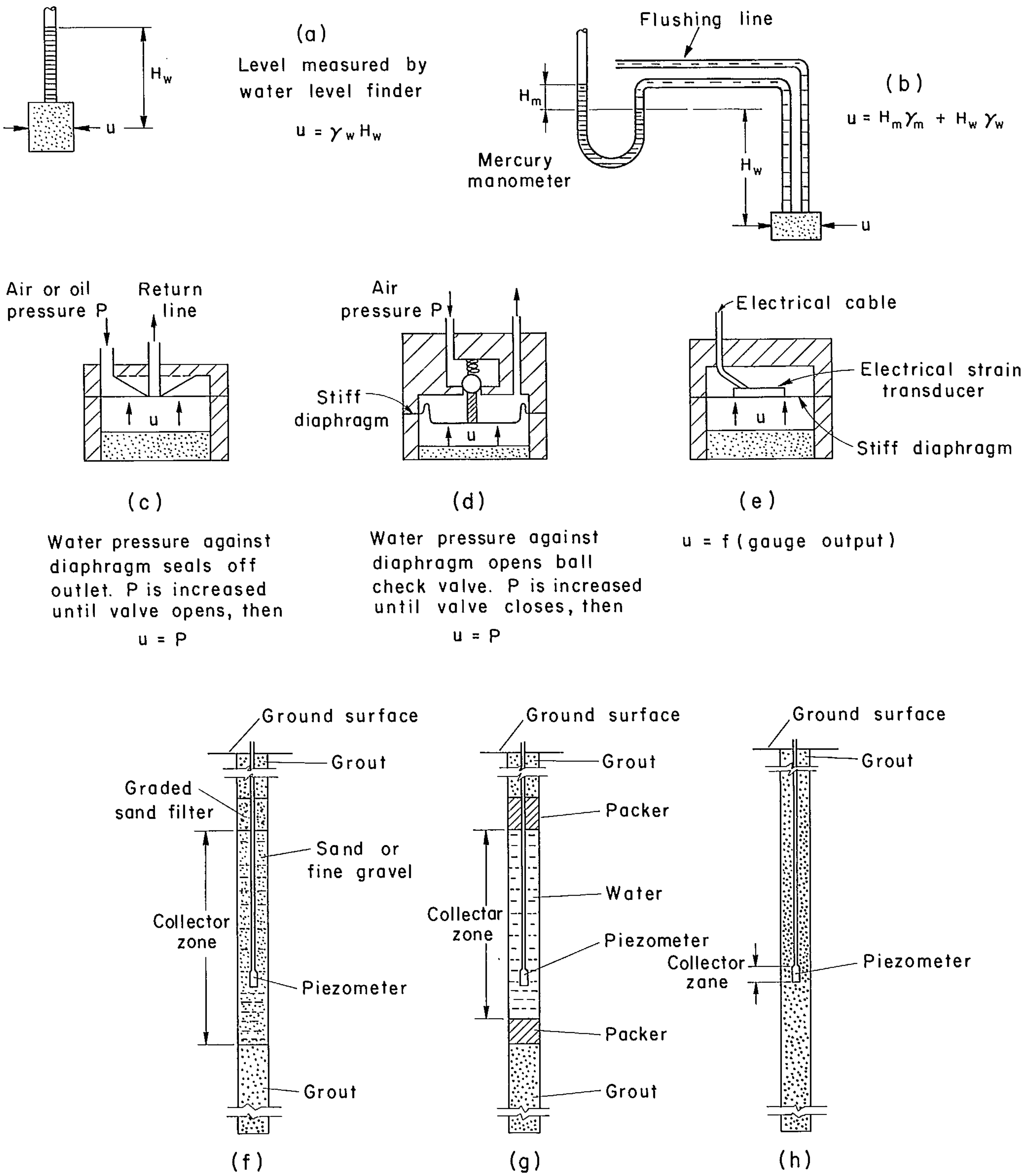


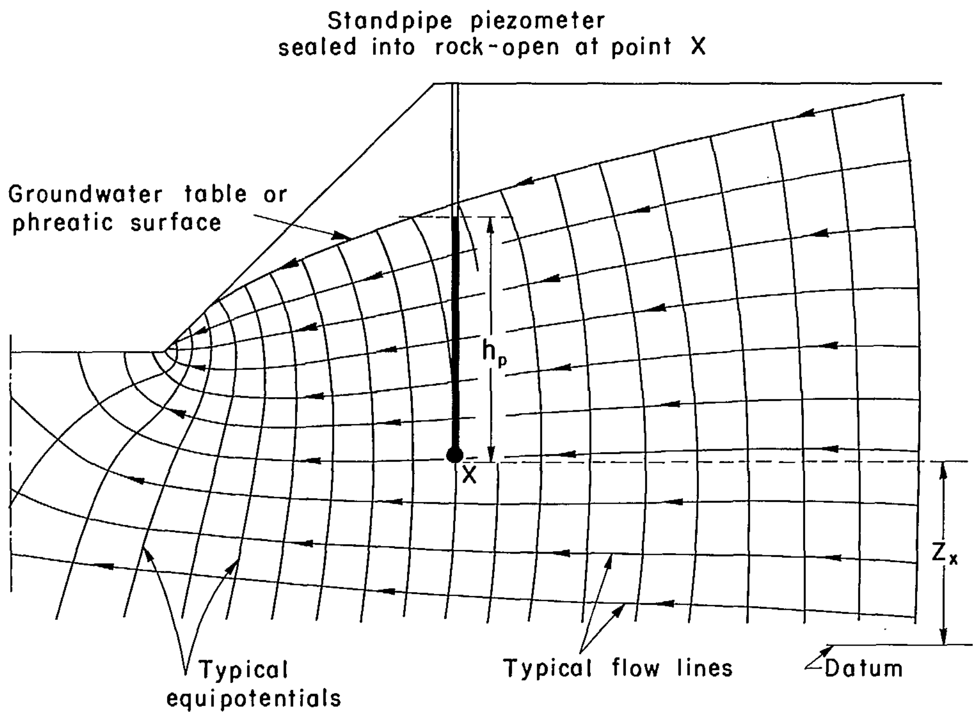
Fig 13 - Five principal types of piezometer are: (a) standpipe; (b) manometer; (c) mechanical diaphragm; (d) ball valve; (e) electrical strain gauge. Installation configurations include: (f) sand or gravel collector zone; (g) pressure packers; (h) fully grouted.

pressure at the bottom of the tube. More sophisticated piezometers use direct pressure reading instruments sealed into the hole.

74. Piezometers must be carefully installed if they are to give reliable results. It is important to choose the correct type. For example, the standpipe, which requires an inflow of water to register pressure change, will respond very slowly to pressure changes in ground of low permeability. A more sophisticated piezometer that requires no water flow may be necessary if this delay is unacceptable (Fig 13).

75. Piezometers alone are not sufficient to determine groundwater pressures for design. First, it is not feasible to install the large number of piezometers required. Second, an important part of groundwater investigation is predicting pressure distribution in future slopes, when not only will mine geometry have changed but sources of groundwater such as streams may also have changed.

76. In practice, the groundwater pressure distribution throughout a slope is determined by combining field measurements and theoretical studies. The field measurements determine the slope material properties that affect groundwater flow and ascertain sources of groundwater. Theoretical studies are then used to predict the groundwater pressures throughout the slope. Piezometers provide input data for theoretical studies, and - of most importance - monitor changes in pressure and verify the accuracy of predictions.



Hydraulic potential at point X = elevation of X + pressure head

$$Z_x + h_p$$

h_p is determined from intersection of equipotential through point X and groundwater table.

Fig 14 - Flow net for seepage through a slope.

ANALYSIS

77. The key to theoretical study of groundwater pressure distribution lies in groundwater flow. A region of groundwater flow can be represented by flow lines and by lines of equal potential. Potential is an important parameter in groundwater flow. It is defined as the elevation at a given point plus the pressure expressed in head of water. Groundwater moves from high to low potential; there is no flow along equipotentials.

Flow Net

78. Groundwater flow can be represented by a pattern of flow lines and equipotentials called a flow net (Fig 14). The upper boundary of flow is the water table, or phreatic surface. The water level in a standpipe installed in a slope rises to the level where the equipotential through the standpipe tip meets the phreatic surface.

79. Thus groundwater pressure can be determined from the equipotentials in a flow net. Strictly, the flow net represents water flow through a uniform porous medium. Flow through rock slopes occurs mainly along discontinuities rather than through the intact rock. However, a flow net can be used to represent average flow through rock slopes.

Permeability

80. The chief characteristic of the slope material that must be known before a flow net can be constructed is permeability. This is a measure of how much water will flow under a given potential difference. Various techniques are used to measure rock mass permeability in the field.

81. Surface and borehole mapping and associated core logging and drill records are used to locate fractures which affect permeability.

82. Falling head tests raise the pressure in a borehole and then measure the drop in this pressure with time. The relationship between flow and pressure is then used to calculate permeability.

83. Constant head tests measure the flow of water required to maintain a pressure above (or below) the equilibrium pressure. These tests are usually carried out on borehole sections at depth, isolated by inflatable seals. The pressure/flow ratio can be used to calculate permeability.

84. Well or drawdown tests measure the change in groundwater pressure as water is pumped from a borehole. At steady state conditions, the rate of withdrawal can be used to calculate the permeability of the surrounding ground.

Flow Net Construction

85. Once permeability and other parameters such as sources of groundwater have been determined construction of the flow net can begin. There are several possible methods.

86. Graphical sketching is the simplest and has the advantages of being cheap and straightforward; for simple flow patterns it gives an insight into actual flow conditions. The principle is that flow lines and equipotential lines meet at 90°. Trial and error is used in sketching a set of lines to achieve this.

87. Electrical resistance analogues use the similarity between Darcy's law for the flow of water and Ohm's law for the flow of electricity. A network of resistors or a sheet of conducting paper is used to model the slope. The flow of current through the model represents water flow and voltage represents potential.

88. Numerical analyses using digital computers are the most powerful techniques available. In these analyses, the equations of flow for the slope are solved approximately and the rate of flow and distribution of pressure predicted. The advantage of numerical methods is that varying permeability can be analyzed with relative ease.

89. All these methods allow various slope geometries and other factors controlling groundwater to be considered. The best method for most purposes is numerical analysis by computer, though graphical sketching is a useful tool for simple cases.

Results of Analysis

90. Analyses result in a series of flow nets for existing and planned slope geometries, taking into account such factors as stream diversion and

variation in rainfall. Flow nets allow groundwater pressures throughout the slope to be predicted; the effect of these pressures on shear strength is included in the stability analyses for design.

91. The actual pressure distribution in any but the most simple slope is complex. Current stability analysis techniques require a simplified distribution. This can be done by estimating pressure as the vertical depth below the water table. This is usually conservative because pressure is overestimated (Fig 15).

92. If variation of the groundwater pressure distribution is to be considered - for example, because of seasonal variations - this can be done by specifying an upper and lower boundary to the water table (Fig 16).

DRAINAGE

93. If groundwater pressure is contributing to instability, drainage may be a satisfactory remedial measure.

94. Before deciding on drainage, careful study of the undrained stability and of drainage methods and influence is required. This means appraising previous stability analyses, performing field tests to determine the potential of drainage and making theoretical analyses of the effects of drainage on groundwater pressure.

95. The theoretical evaluation of drainage uses the same techniques required to determine groundwater pressure distribution. For example, computer analyses can be used to obtain a flow net for the drained slope. Groundwater pressures can then be determined from this flow net.

96. It is essential to carry out field trials to verify that a proposed drainage scheme will achieve the desired results. A typical procedure is to install piezometers in a critical zone and then drill several drain holes or sink a well. Pressures before and after drainage can then be compared. It is important both in trials and in actual installations that the water-bearing formations or water-carrying discontinuities be tapped. The volume of water flowing from a well or drain-hole is not a measure of drainage effectiveness; the objective is to reduce pressure.

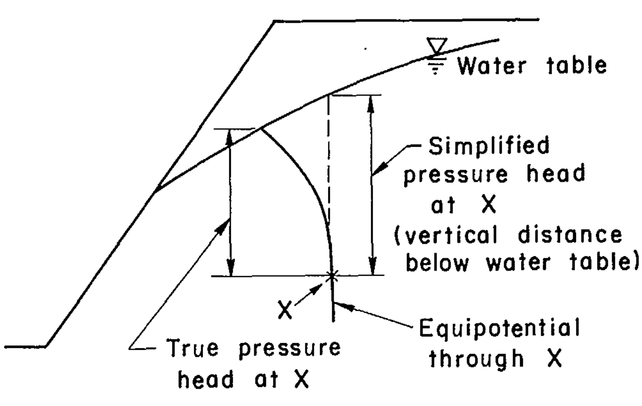


Fig 15 - Simplified pressure distribution. Approximating pressure distribution by vertical depth below water table usually overestimates pressure.

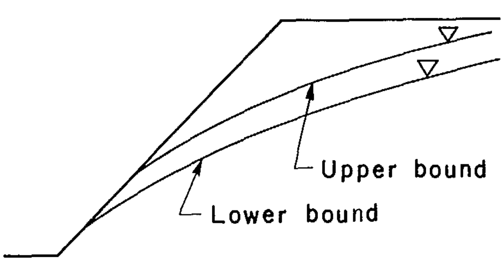


Fig 16 - Variation in groundwater pressure distribution due to seasonal changes or uncertainty in measured parameters can be accounted for by bounds to the water table.

Drainage Methods

97. The choice of drainage method depends on many factors including slope height, permeability and economic and operational constraints. Four methods are widely used (Fig 17).

a. Horizontal or near-horizontal holes in the slope face are simple and relatively easy to drill; they require little maintenance and drain by gravity. Holes are usually lined with perforated pipe.
b. Vertical wells drilled behind the slope crest or on the slope face have the advantage of being away from the workings, and if behind the crest can be used for dewatering before excavation begins. Pumps are required, however, with corresponding maintenance costs.
c. Trenches down or along the slope face are necessarily shallow and can only drain surface regions. However, where shallow instabilities are critical, trench drainage can be satisfactory.
d. Galleries excavated in the rock mass behind the slope are expensive, but, where large-scale drainage is required, are often the most effective method. They do not hinder operations and can be used for other purposes such as ore evaluation and structural mapping. Supplementary drain holes can be drilled from the gallery.

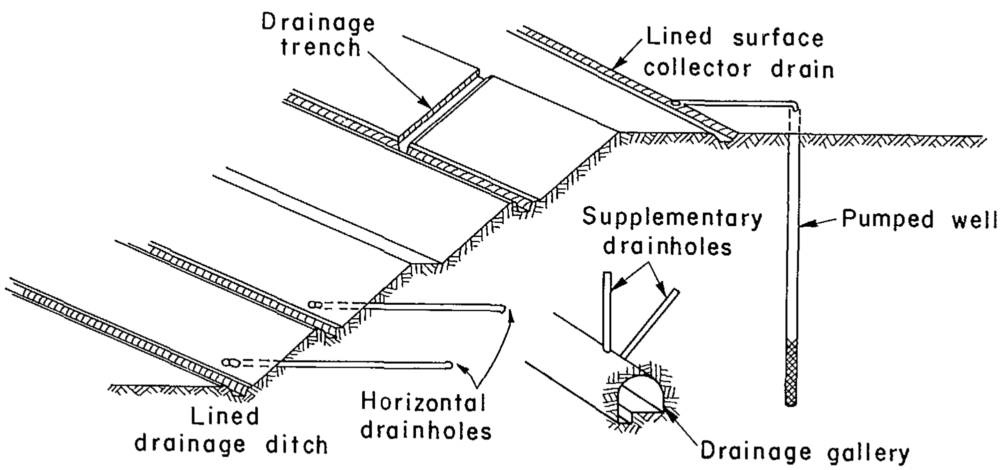


Fig 17 - Slope drainage systems.

Cut-offs

98. An effective method of groundwater control in some circumstances is to use a cut-off to prevent water entering the slope. Cut-offs can be used only where there is a well-defined stratum of water bearing material, such as an old river channel. They can be constructed from sheet piling or by backfilling trenches with impermeable material, but the most common method is by injecting grout.

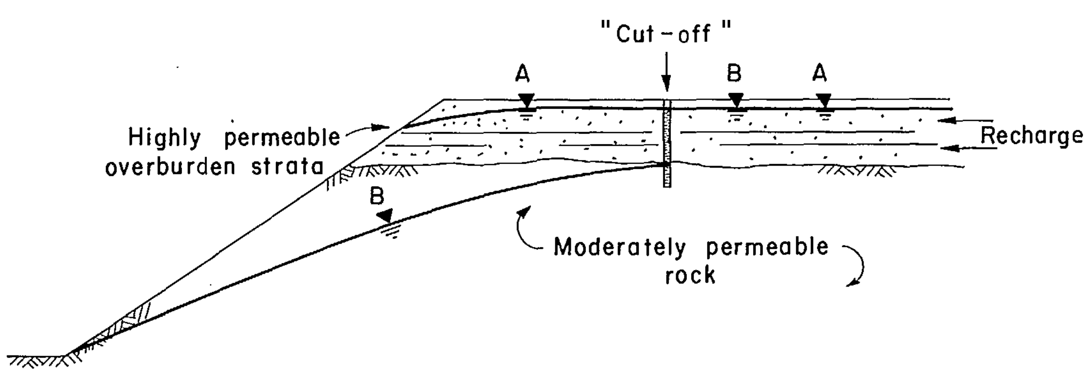
99. For the grouted cut-off, holes are drilled and grout is pumped to refusal. Piezometers on either side of the cut-off monitor the pressure reduction and additional holes are drilled and grouted until the desired control is achieved (Fig 18).

MONITORING

100. An essential aspect of groundwater investigation and control is regular monitoring of groundwater pressures by piezometers. Drain discharge, flow in streams and in adits and observation of face seepage are also useful monitoring techniques.

MINE DEVELOPMENT STAGES

101. Appropriate action in the groundwater task at various stages is shown in Fig 19. A preliminary assessment of groundwater is made in the feasibility stage. Regional characteristics are surveyed and existing information obtained from aerial photographs and maps and from well, stream flow and precipitation records.



A Groundwater table without cut-off (high recharge into slope)
B Groundwater table with cut-off (natural drawdown occurring)

Fig 18 - Cut-off to control seepage into a slope.

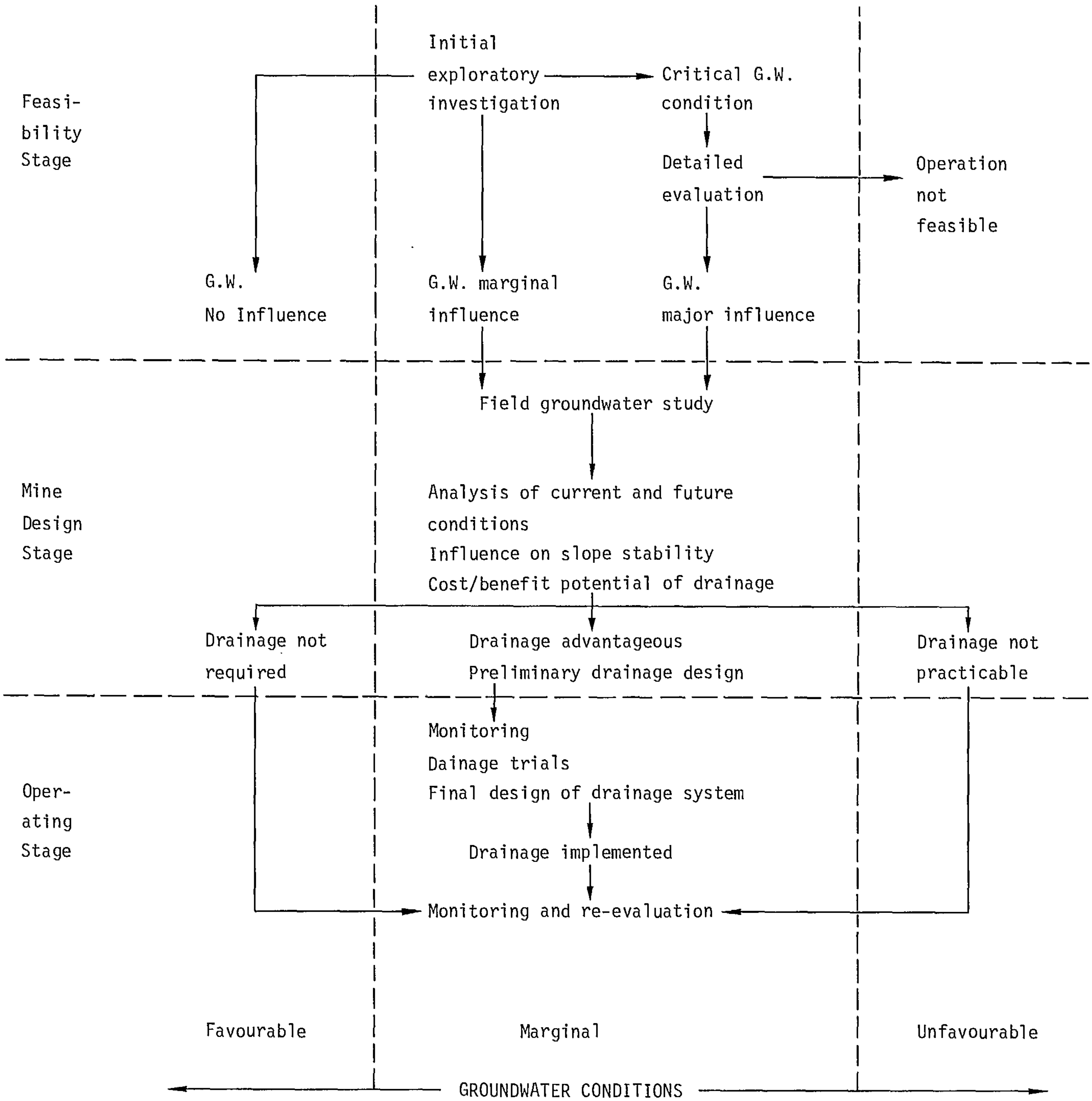


Fig 19 - Groundwater studies for open pit mining.

102. Exploration drilling can be used at little extra cost to measure permeability, log core, identify discontinuities that form flow channels and for piezometric measurements.

103. Detailed groundwater investigations are required at the mine design stage. All exploration holes should also be used for permeability tests and piezometric measurements. Additional drilling specifically for groundwater investigation is usually required.

104. Data gathering is followed by theoretical analyses required to produce flow nets. These in turn indicate where drainage may be necessary and thus guide further investigations.

105. During the operating stage, groundwater conditions should be monitored regularly by a network of piezometers and by observing seepage, flow from drains, etc. The information obtained guides the re-investigation and analysis which may be required because actual ground conditions are not as anticipated or because the pit layout has changed.

COSTS

106. Costs for groundwater investigation vary greatly from mine to mine. The following estimates summarize actual experience for various depths of conical pits. Drilling and drainage costs are not included; 1975 dollars are used.

	Final Pit Depth - ft (m)		
Stage	100(30)	500(150)	1000(300)
Feasibility	$ 8000	16000	24000
Mine Design	$10000	20000	39000
Operating	$ 6000	11000	16000

107. The approximate time requirements for the various stages are as follows:

	Final pit depth - ft (m)		
Stage	200 (60)	200 - 500 (60 - 150)	500 - 1000 (150 - 300)
Feasibility	4 months	5 months	6 months
Mine Design	6 months	8 months	10 months
Operating	3 months	4 months	5 months

DESIGN

108. The objective of pit slope design is to determine the pit layout that maximizes financial returns and mineral recovery. The designer must consider both ultimate and interim slope angles, bench angles, location of ramps and sequence of mining (Fig 20).

109. The volume of waste excavation usually decreases as the walls are steepened. Steep slopes are therefore often desirable. However, the risk of slope instability becomes greater as slopes are made steeper, and the effect on safety must be a paramount consideration. Open pit mining has an excellent safety record, accidents due to rock falls being particularly rare (Table 3). It follows that, with proper care, steeper walls can be considered without jeopardizing safety.

110. The main benefits of steeper walls are reduced waste excavation and deferred stripping. However, steeper walls may also result in increased costs. Wall instability may mean cleaning up slides, postponing mining or loss of ore. Fewer and narrower working benches may cause decreased operating efficiency. The designer must offset the costs of steeper walls against the savings in waste excavation.

RELIABILITY

111. The pit slope designer must deal with inherently variable parameters such as ore values, grades and rock strengths. The best approach in dealing with variable materials is to use re-

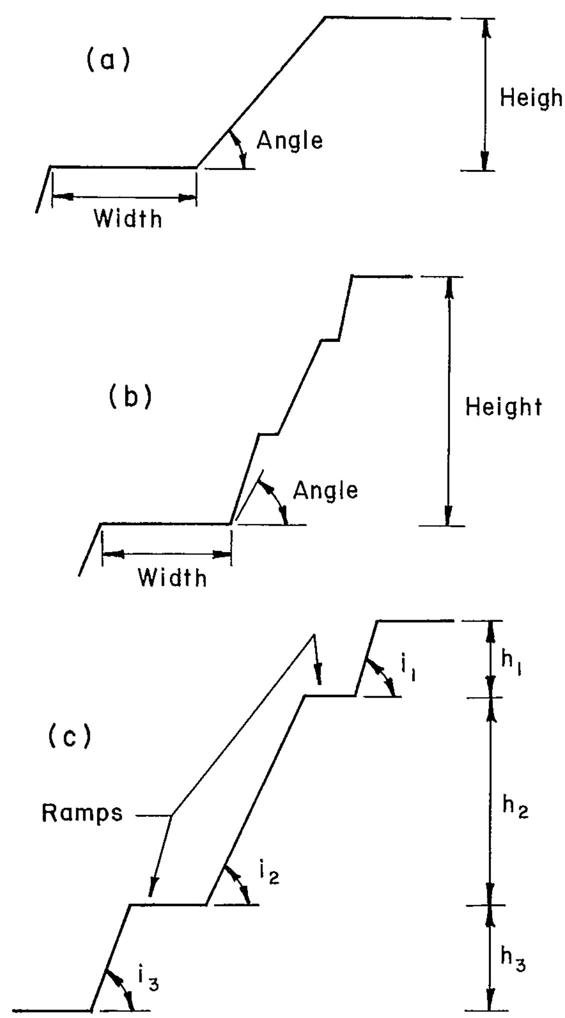


Fig 20 - Various aspects of pit walls that concern the designer: (a) bench geometry; (b) berm geometry; (c) inter-ramp geometry.

Table 3: 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

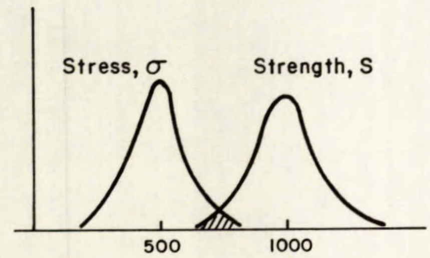


Fig 21 - Reliability theory recognises that, because of variations in geologic materials, wall profiles and groundwater conditions, the strength resisting sliding and the stress promoting sliding both vary from section to section. In this figure, although the mean strength is twice the mean stress, variations in both can result in sections where the stress will exceed strength, as shown by the hatched area, and sliding will occur.

liability theory (Fig 21, 22). In slope stability analysis, reliability is defined as the probability that a slope will be stable.

112. Mine investment decisions are to a large extent based on prospective rates of return and associated risks. Mine planners have already recognised the advantages of reliability theory in evaluating investment risks. With it they can include the variability of commodity prices, labour rates and ore reserves in their analyses. The integration of wall design into mine design and risk analysis similarily requires the use of reliability theory.

ANALYSIS

113. Pit slope design is considered to have two parts. The first is to evaluate pit slope stability for all the potential pit layouts. The second is to incorporate these data into financial analysis.

Fig 22 - Manifestions of variability in a pit wall: a slide on a wall at a section that seemed no different from any other section where sliding did not occur. In other words, the slide was impossible to predict with certainty.

Stability Analysis

114. The first step in stability analysis is to bring together the results of investigations into structural geology, mechanical properties and groundwater and establish design sectors for the pit. Within each sector, rock strength, structural features and other factors affecting stability are roughly uniform. The choice of sectors may also be affected by the required wall reliability. For example, the designer may have to plan for greater reliability in the area of surface plant or a haulage system. Fig 23 shows typical layouts of sectors.

115. Possible modes of instability are noted for each sector. Plane shear modes (Fig 24) are

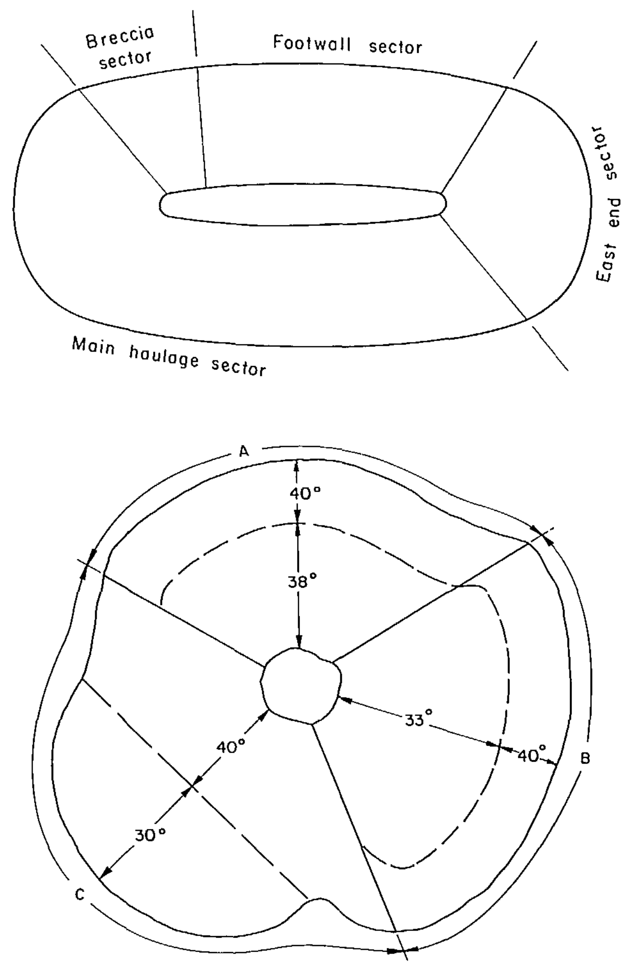


Fig 23 - Top: possible factors governing slope design sectors. Bottom: a large open pit divided into slope design sectors.

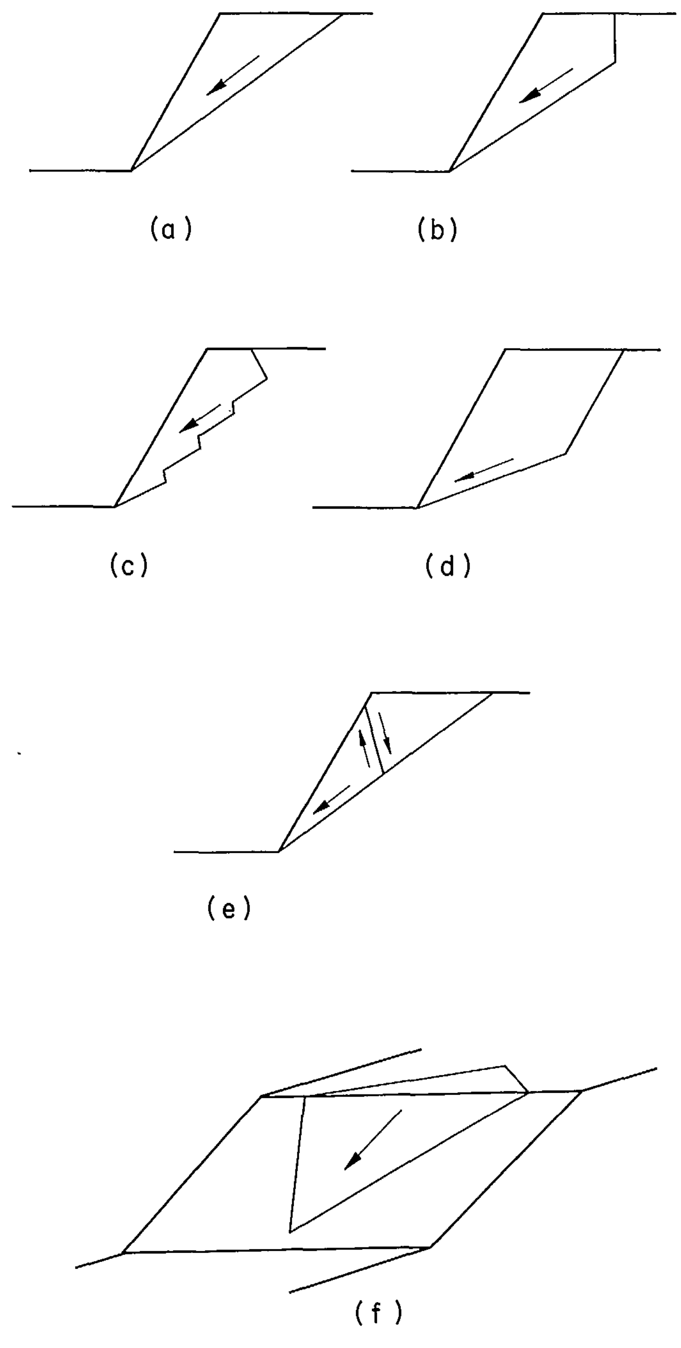


Fig 24 - Plane shear instability modes: (a) one sliding plane and one block; (b) one sliding plane and a tension crack; (c) a series of short sliding planes with connecting cross joints; (d) two sliding planes; (e) a single sliding plane with the sliding block shearing into several blocks; and (f) two oblique sliding planes and a 3-d wedge.

most common; they occur in rock slopes where discontinuities are unfavourably oriented. In overburden or ductile rock, rotational shear (Fig 25) may occur. Both plane and rotational shear are characterized by surfaces of sliding. Stability analysis requires that shear strength on these surfaces be determined, as well as forces tending to cause sliding.

116. Computer programs for reliability analysis of plane and rotational shear are included in the manual. In addition, hand procedures suitable for initial estimates of stability are given.

117. Figure 26 shows the block flow and toppling modes of instability possible in brittle rock. These are less common than plane or rotational shear and are more difficult to analyze. The critical factor is the stress concentration which occurs at the toe of a slope. If this stress - which can be severely aggravated by tectonic action - exceeds the compressive strength of the rock, progressive breakdown from the toe may occur.

118. The recommended analysis is to investigate stress distribution using computer programs or approximate methods including the best estimate of tectonic effects. Instability is assumed to occur if the compressive strength is exceeded.

119. For each design sector, stability analyses are used to prepare reliability schedules for different wall heights and angles. These schedules (Fig 27) are used as input to the financial risk analyses.

Financial Analysis

120. Financial evaluation requires considerable information in addition to reliability schedules for the possible walls. Economic geology data and mining costs are required. The cost of possible instabilities must be estimated (Fig 28, 29).

121. Handling these data is best done by computer. A program for benefit/cost analysis is included in the manual. The program requires as input a complete set of mine plans as a base case. These plans must include mining sequence and the distribution of ore and waste as well as ore values and mining costs.

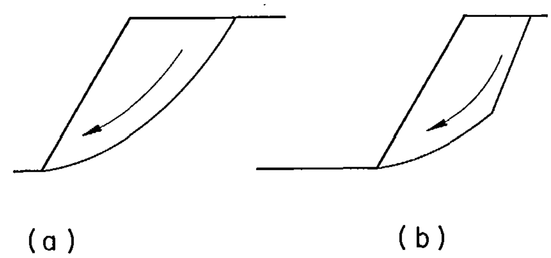


Fig 25 - Rotational shear. In ductile rock without critically oriented planes of weakness, the potential mode of instability would be by rotational shear as shown in (a), which sometimes occurs in combination with a sliding plane as shown in (b).

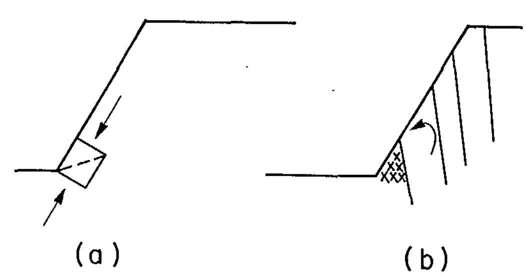


Fig 26 - Block flow and toppling. In brittle rock without a critically oriented plane of weakness, instability would be inititated by crushing or shearing at the toe of the slope as indicated in (a), possibly leading to block flow and toppling of slabs represented by (b).

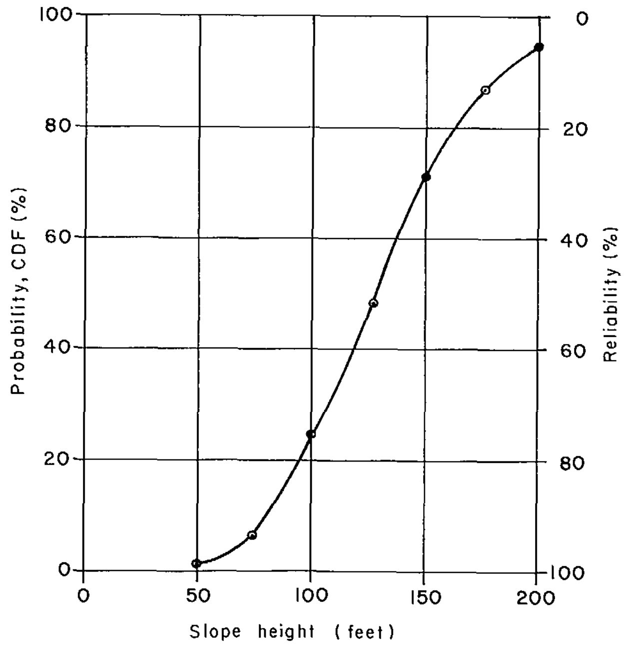
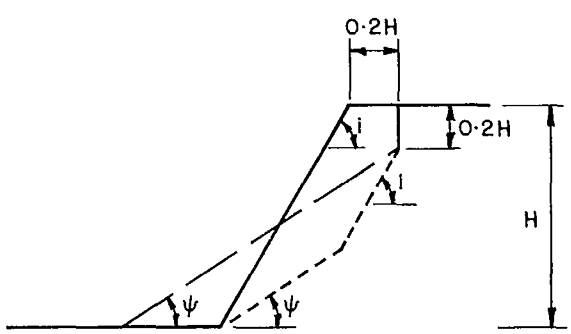


Fig 27 - Reliability schedule. Variation of probability of sliding with slope height in a 65° slope.



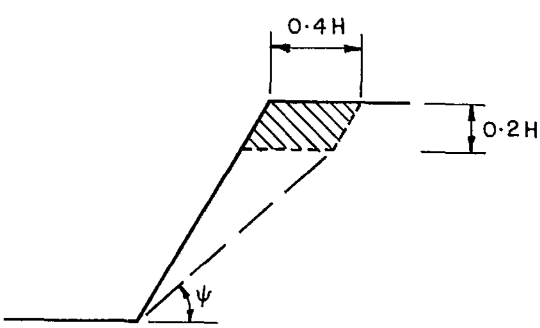


Fig 28 - Model for determining the prospective cost of instability. The volume of slide debris to be mucked out is assumed to lie between the dashed and dotted lines. This is only one of the options that might be appropriate.

Fig 29 - When instability is developing, a course of action that could enable operations to continue is to unload the potential slide zone by excavating rock from the crest. The figure provides guidance on the amount of ground that might be moved, which then is the basis for estimating the prospective cost of instability.

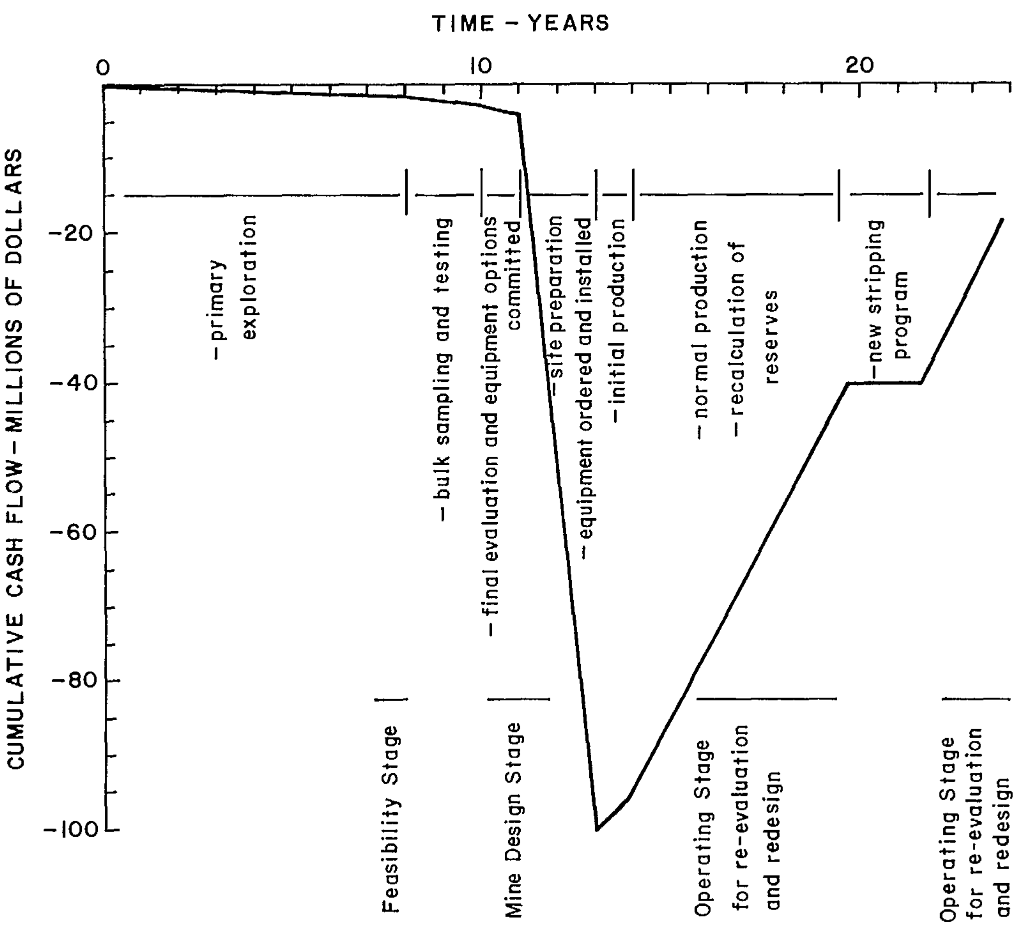


Fig 30 - Cash flow diagram. The three stages in the development of a mine in which wall design is required are feasibility, mine design and operating.

122. The benefit/cost program calculates the effect relative to the base case of steepening pit walls, including both savings in waste excavation and costs of instability. The benefits and costs can be determined for each year of the prospective mine life, and discounted to the present. The optimum layout is selected by comparing the net present values of the various layouts.

123. The risk of instability is included by Monte Carlo sampling. In this technique, the analysis of each sector is repeated a large number of times. In any given analysis, inclusion of instability is made at random. However, the computer is programmed so that the overall occurrence of instability corresponds to the reliability schedules. This simulates the likelihood of instability in actual mining.

124. A good estimate of the expected benefits and costs requires between 30 and 100 analyses. The program is efficient; the time on a large computer for a typical 15-year mining plan is at most a few minutes. The optimum sector geometry determined by machine, however, must be smoothed by the engineer to eliminate abrupt changes at sector boundaries.

125. The benefit/cost appraisal of pit layout is only part of the information considered in a full mine risk analysis. Other factors - all of which are variable - include tax rates, royalties and capital expenditures. A computer program for complete mine economic risk analysis is also included in the Pit Slope Manual.

126. The risk program uses the results of the benefit/cost analysis as input data. To this are added data on expected royalties, tax rates, etc, including their probable variation. A number of Monte Carlo simulations of mining are made to determine the effect of such variation on mining returns, and so evaluate financial risk.

MINE DEVELOPMENT STAGES

127. Design activities at the various stages of mine development are shown in the cash flow diagram (Fig 30). Investigations at the feasibility stage concentrate on data gathering. Emphasis is given to structural geology, groundwater and mechanical properties of the wall materials, but previous slopes and past experience also give useful data. With good coordination, much of this work can be combined with orebody evaluation and the costs, particularly of drilling, minimized. It is important that all information be recorded in reports because there is often a time lag between the feasibility and mine design stages.

128. Data collection is followed by definition of sectors and preparation of schedules of reliability using approximate analyses. Detailed financial analysis to determine optimum slope angles may not be required. The design steps at the feasibility stage are shown in Fig 31.

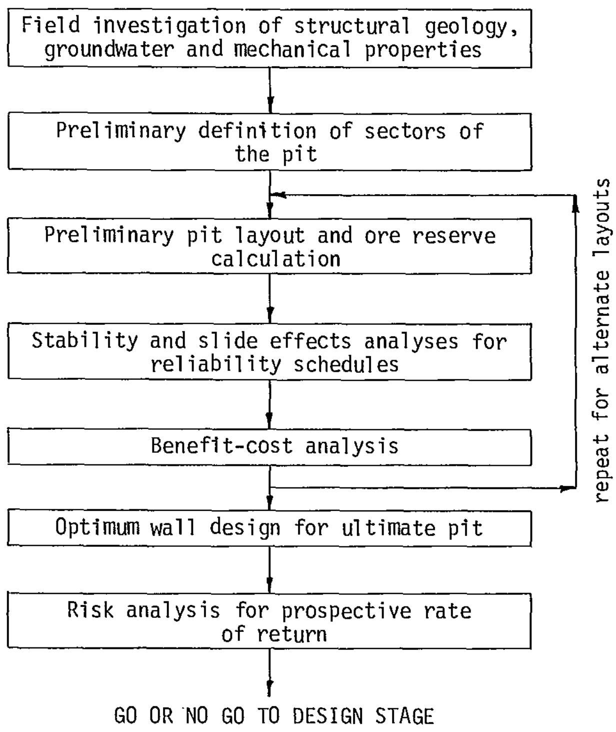


Fig 31 - Feasibility stage: activity flow chart for wall design.

129. Detailed stability analyses and a full financial analysis are required at the mine design stage. At least two complete sets of annual mine plans must be prepared for walls at different angles. Detailed layouts of ramps and interramp and bench angles must be considered. The provision of safety berms must be examined; special catch benches might be needed for particularly steep wall angles. Complete reports of all investigations, stability analyses and financial analyses must be prepared; they are an essential legacy to future planners. Figure 32 shows activity at this stage.

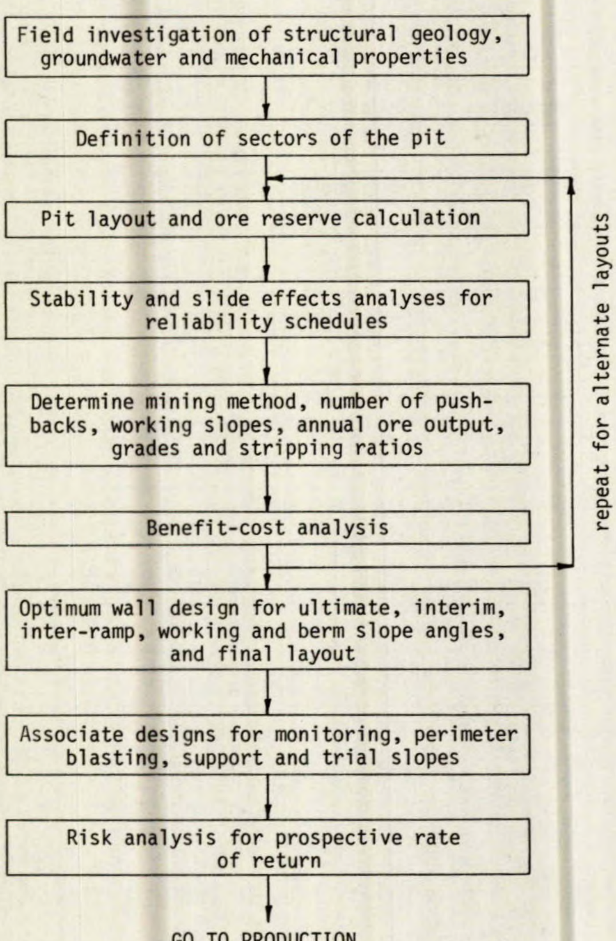


Fig 32 - Mine design stage: activity flow chart for wall design.

130. A large amount of additional design data becomes available during the operating stage, particularly through structural mapping of exposed benches. The performance of existing slopes is a valuable guide to the behaviour of future walls. It may be possible to mine an interim slope as a trial for the ultimate wall.

131. This information will reveal where the original design assumptions are invalid. If necessary, stability analyses can be revised and the mine redesigned. Redesign may also be required because of changing ore values and mining costs, or because of expansion of mining.

ASSOCIATED DESIGNS

132. If the stability and financial analyses indicate rather steep optimum slope angles, it may be necessary to design a comprehensive monitoring system to ensure safety. The cost of such a system should be offset against the benefits of mining at steeper angles.

133. The feasibility of using steep slopes may depend on the degree to which the walls are damaged by blasting. Controlled perimeter blasting can minimize wall damage. In conjunction with good scaling, it may also eliminate loose face rock which might otherwise limit steepness.

134. Mechanical support may be economically advantageous under certain conditions (Fig 33). For

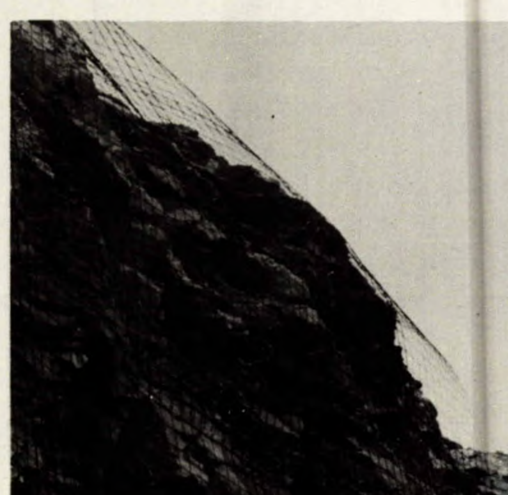


Fig 33 - A field experiment using rock anchors and mesh for mechanical support of part of a wall.

example, it might permit mining ore whose removal would otherwise jeopardize surface plant. In addition, it may at times be cheaper to use mechanical support and a steep slope angle than to excavate waste rock back to an unsupported wall of equivalent stability.

COSTS

135. The cost of mine design varies greatly. However, to give some guidance to planners, Table 4 gives estimated costs in 1973 dollars for the various mine development stages. The figures are based on the experience of a number of companies.

Table 4: Design budget guidelines* (1973 $)
(typical ranges in brackets)

Depth	<100 ft <(30m)	100-1000 ft (30-300 m)	>1000 ft >(300m)
Feasibility Stage	$ 10000 (1000-50000)	$ 20000 (5000-200000)	$100000 (10000-500000)
Mine Design Stage	$ 15000 (1000-50000)	$ 50000 (5000-200000)	$100000 (10000-500000)
Operating Stage	$ 10000 (1000-50000)	$ 50000 (5000-200000)	$100000 (10000-500000)

*Based on 2 x salaries plus cost of special equipment.

MECHANICAL SUPPORT

136. Mechanical support stabilizes pit slopes by increasing rock strength. However, the increase achieved is relatively small. Support therefore is usually used only if a slope shows signs of instability during mining. Support in this case may increase strength enough to ensure long-term stability.

137. It may occasionally be cheaper to design a slope with support rather than excavate to a naturally stable slope. An example would be a clearly defined ore contact which might be a natural mining boundary except for potential sliding on bedding planes (Fig 34). However, in such a case the stability investigation and analysis must be very accurate. The degree of instability must be clearly known before support can be justified.

METHODS OF SUPPORT

Rock Anchors

138. Most open pit slides are caused by slip on planes of weakness such as joint sets and bedding planes. The resistance to sliding can be increased by compressive forces acting across these planes. This can be done by installing rock anchors through the planes of weakness (Fig 35).

139. A typical rock anchor (Fig 36) consists of a steel cable inside a borehole. The cable is anchored by cement grout in the bottom of the hole. At the surface an anchorage is formed by a concrete bearing block and a metal bearing plate. A steel anchor block with wedges secures the cable after it is tensioned. The borehole is subsequently filled with grout to bond the cable

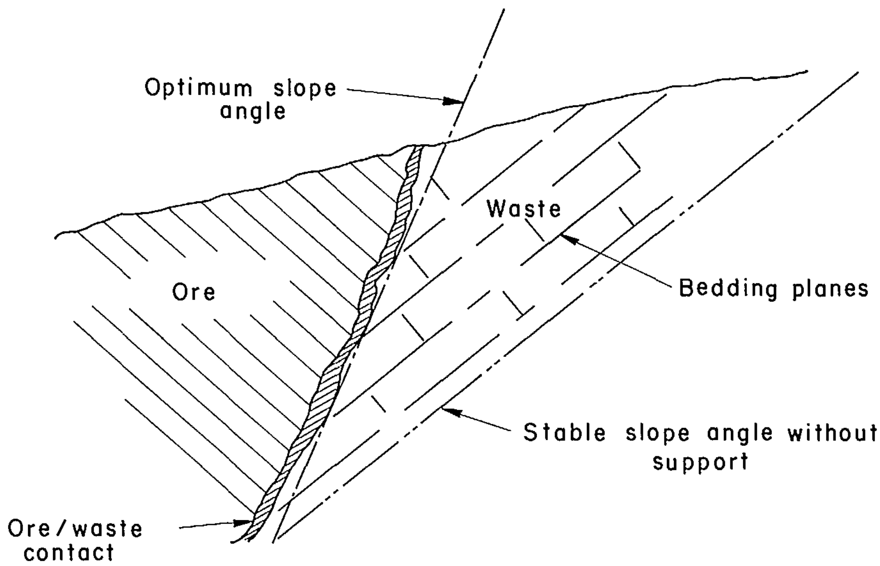


Fig 34 - Support may be appropriate if the ore contact is clearly defined but bedding planes are unfavourable.

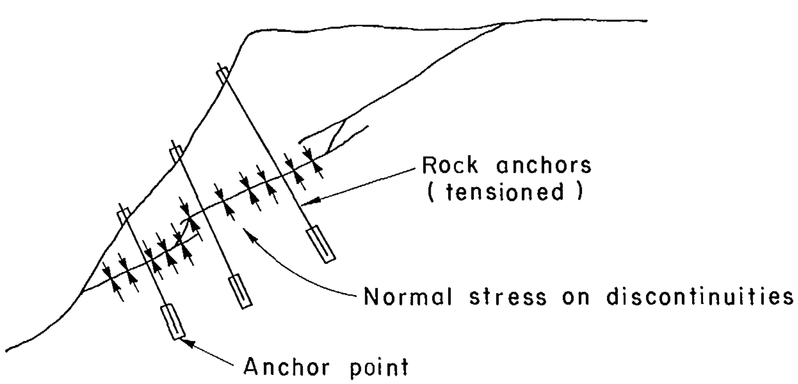


Fig 35 - Rock anchors increase compressive force on discontinuties and so increase resistance to sliding.

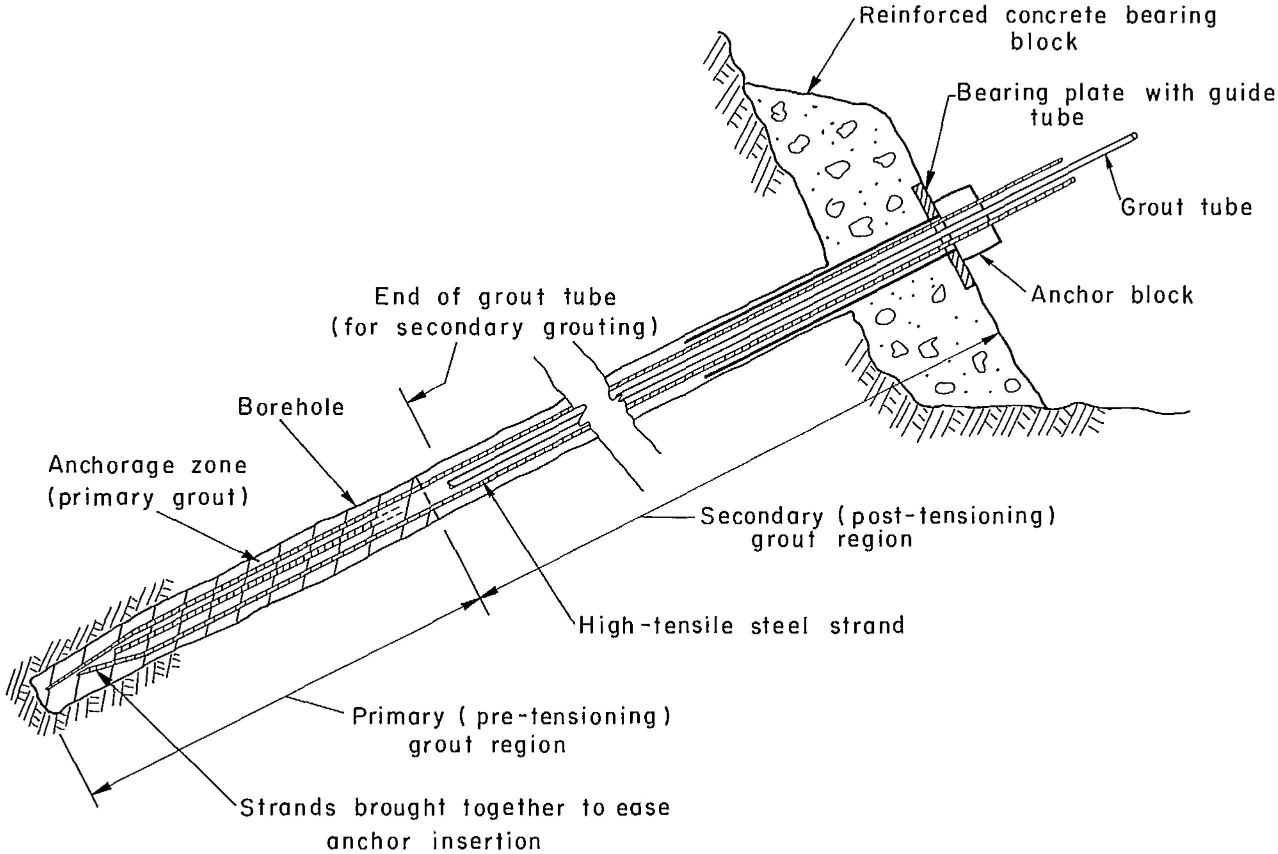


Fig 36 - Typical rock anchor installation.

permanently. Solid bars can also be used.

140. The tension in a rock anchor is usually from 30-300 tons (300-3000 kN), though anchors up to 2000 tons (20000 kN) have been used.

141. The techniques and materials of rock anchors were developed by the prestressed concrete industry. Steel must be of very high strength and requires care in handling. It should not be kinked or permanently bent, nor pitted by rust - the pits can become points of stress concentration leading to failure when tensioned. Cutting must be by carborundum wheel; the heat from flame cutting may cause loss of strength.

142. The installation of a rock anchor requires care. Large forces are used in jacking and broken cables may fly. Precautions to protect personnel are necessary.

Rock Anchor Corrosion

143. The high tensile stresses involved in rock anchors both aggravate the effects of rusting and introduce the brittle failure problems of stress corrosion. This is a poorly understood phenomenon that occurs only at relatively high stresses and results in internal cracking and sudden failure.

144. The best form of corrosion protection is to fully grout the anchor after tensioning. However, if the anchor load must be monitored grouting cannot be used and greasing or sheathing the anchor is necessary.

Fig 37 - Applying shotcrete to stabilize a pit wall.

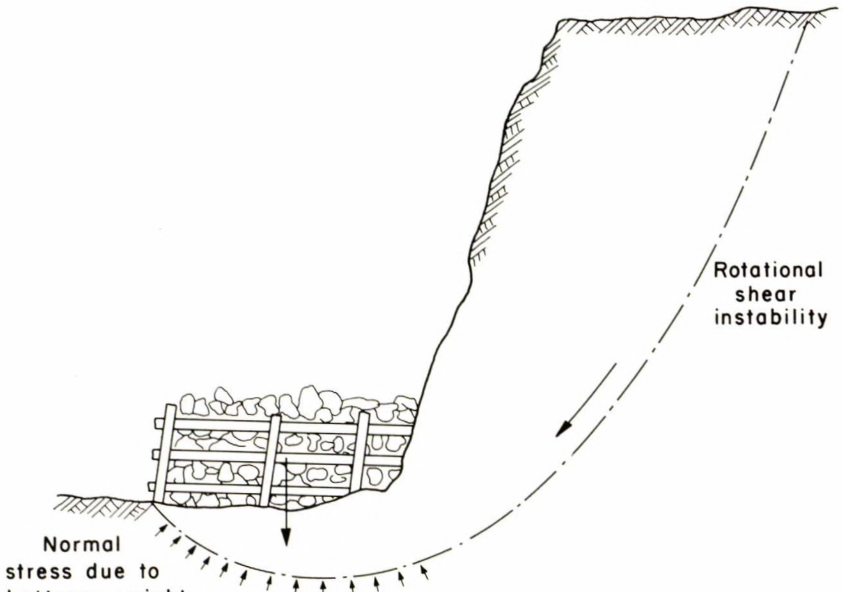


Fig 38 - A buttress or crib with dead load of rock fill provides counterweight to resist rotational shear and increases normal stress and therefore sliding resistance in toe region.

Shotcrete

145. Shotcrete is a surface layer of concrete applied by spraying a mixture of aggregate, sand, cement, water and a flash-set additive (Fig 37). The shotcrete effectively sets on impact. There is some loss of material through rebound from the face.

146. Shotcrete is used to retain loose surface rock and prevent progressive deterioration. It can provide shear resistance across joint boundaries.

147. Reinforced shotcrete has recently been developed. Chopped steel wire is added to the mix, increasing the tensile strength. Drawbacks are cost, and difficulty in handling.

148. A disadvantage in using shotcrete is the risk of sealing the slope face and allowing build-up of groundwater with a possible threat to stability. Where necessary, drain outlets must be installed. Another disadvantage is that shotcrete can only be applied at temperatures above freezing.

Buttresses

149. Buttresses stabilize by adding dead weight to counteract a tendency to slide (Fig 38). They have been used in Canada to stabilize pit slopes but are not usually a suitable support method.

DESIGN

150. Support design has three steps:

a. identifying instabilities and collecting relevant data on geology, mechanical properties and groundwater,
b. engineering analysis, and
c. economic analysis.

151. The principles of engineering analysis are illustrated in Fig 39. The disturbing force is the component of weight W acting parallel to the discontinuity. The resisting forces are the component of support force T and shear resistance on the discontinuity. The shear resistance is proportional to the forces W, T and groundwater force U acting across the discontinuity.

152. The analyses of the design chapter are used to determine the support force required to achieve the desired wall reliability. Once the required support force is known, economic appraisals are made. Both wall reliability and the cost of support depend on the layout of rock anchors, particularly on the angle of inclination. However, reliability is relatively insensitive to anchor inclination and the least cost layout can usually be chosen.

MONITORING

153. Load cells can be used to monitor anchor behaviour directly. A typical load cell is shown in Fig 40. The cell is placed between the anchor block and bearing plate and thus transmits the full anchor load to the rock surface.

154. Load cells can be hydraulic or mechanical. In the former, fluid pressure within the cell is measured to calculate the load. The latter has gauges to measure the strain in a steel cylinder previously calibrated to correlate strain and load.

155. A monitored rock anchor cannot be grouted. Corrosion protection must be provided by greasing and sheathing the cable.

156. A monitoring technique which is not recommended is the "lift-off" test, in which the anchor is rejacked to check the tension. The disturbance to the anchor is most undesirable.

COSTS

157. Estimating support cost is difficult because of lack of large-scale experience in mining. Experience in civil engineering is usually not

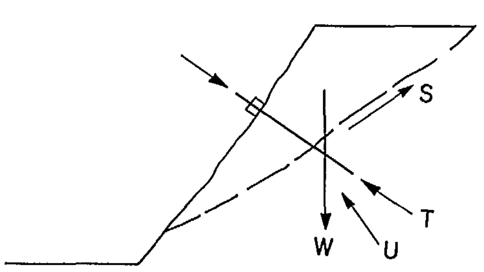


Fig 39 - Support of plane shear when a discontinuity strikes roughly parallel to the slope but at a flatter dip.

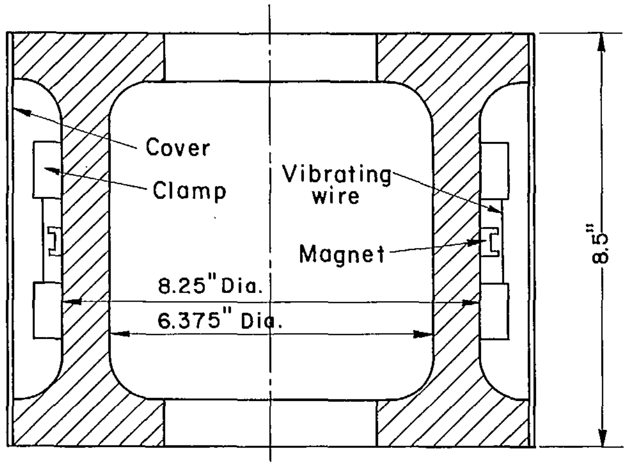


Fig 40 - Layout of 250-ton (2500 kN) load cell: Strain in the steel cylinder through which the rock anchor passes is measured by vibrating wire gauges. Electrical readout connections are not shown.

applicable because of differing purposes and standards.

158. The following examples provide a guide to the cost of those additional activities beyond normal mining investigation and monitoring required for support. Three cases are considered: bench scale, moderate slope and large slope; cost details are given in Table 5.

Bench Support

159. A bench 40 ft (12 m) high by 100 ft (30 m) long is to be supported with 20 solid bar rock anchors of 40 kips (180 kN) capacity, averaging 40 ft (12 m) long. Four anchors are monitored. Two diamond drill holes 50 ft (15 m) long are required for investigation and to provide test samples. Support cost is $1.75/ft^2$ ($19/m^2$).

Moderate Slope

160. A slope 100 ft (30 m) high by 320 ft (100 m) long is supported by 64 cable rock anchors, each of 300 kips (1300 kN) working load and an average length of 100 ft (30 m). Ten anchors are monitored. Six diamond drill holes 150 ft (50 m) long are required for investigation. Five man days are spent in structural mapping. Shear box tests are carried out on core from the drilling program and on samples collected from the slope. Piezometers are installed in two of the diamond drill holes behind the slope crest. Support cost is $3.25/ft^2$ ($35/m^2$).

Large Slope

161. A slope 500 ft (150 m) high by 1000 ft (300 m) long is supported by 300 cable rock anchors, each of 450 kips (2000 kN) capacity and average length 150 ft (50 m). Thirty anchors are monitored. Ten diamond drill holes 300 ft (100 m) long are required for investigation. Twenty man days are spent on structural mapping and 20 man days in logging core, collecting samples for testing and supervising field work. Four of the drill holes are used for piezometers. A laser distance measuring instrument is used for monitoring. Support cost is $1.60/ft^2$ ($17/m^2$).

162. No allowance is included in these costs for ongoing monitoring effort or report writing. These activities are essential but should add only moderately to the total outlay. These rough estimates indicate that rock anchor support costs are in the order of $1.50 to $2.50 per square foot of slope face ($16-$27/m^2$), and that 10% to 20% of the total cost is required for analysis and design.

Table 5: Support costs (1974)

		Bench	Moderate slope	Large slope
Investigation and analysis				
Supervision at $150/day		150	750	2250
Diamond drilling at $10/ft ($33/m)		1000	9000	30000
Structural mapping at $150/day		300	750	3000
Sampling and testing		200	2000	7750
Piezometers		-	1000	3000
Analysis at $150/day		150	2250	6000
Installation and monitoring				
Percussion holes	at $2/ft ($7/m)	1600	-	-
	at $3/ft ($10/m)	-	19200	-
	at $4/ft ($13/m)	-	-	180000
Rock anchors	at $3/ft ($10/m)	2400	-	-
(materials	at $10/ft ($33/m)	-	64000	-
and labour)	at $12/ft ($40/m)	-	-	540000
Load cells and readout		1200	5000	15000
Laser unit and targets		-	-	14000
TOTAL		7000	103950	801000

Shotcrete Costs

163. The costs given below for shotcreting are based on actual material cost plus an hourly rate for equipment. Coverage per cubic yard with a 4 in. nominal thickness and 10% rebound loss is 70 ft^2/yd^3 ($10\ m^2/m^3$). At an application rate of 15 cubic yards per hour ($12\ m^3/h$) cost is $\$0.60/ft^2$ ($\$6.50/m^2$).

164. The cost of wire reinforced shotcrete is about $100 per cubic yard ($\$130/m^3$) for materials; equipment costs are similar but production per hour is lower, at about 12 cubic yards per hour ($10\ m^3/h$). The appropriate support cost is therefore approximately $\$1.65/ft^2$ ($\$17/m^2$).

	1974 dollars
Shotcrete material - aggregate and additive per cu yard (m^3)	$30 ($39)
Equipment, per hour	
Shotcrete machine	10
Crane	30
Concrete trucks, 2	50
Loader	20
Compressor	20
Labour per hour (6 men)	60
TOTAL	$190

PERIMETER BLASTING

165. Perimeter blasting techniques limit the damage to final pit walls and benches. This is done by lowering the explosive energy concentration at the pit perimeter, and by controlling the energy concentration at the pit wall due to the main production blast. The more common forms of blast damage are backbreak, crest fracture or loose face rock on the immediately adjacent pit wall. However, blast vibration may damage pit walls and buildings some distance from the blast.

FACTORS AFFECTING BLASTING

Explosives

166. Explosives for open pit mining have a wide range of borehole pressures. For a given loading density, backbreak and vibration will increase with borehole pressure of the explosive used. Thus wall damage can be reduced by using a lower pressure explosive.

167. For a given explosive, borehole pressure increases with charge diameter. Figure 41 shows the relationship between fracture radius and blasthole diameter. It can be seen that AN/FO and permissible explosives produce a fracture radius two to four times smaller than dynamite.

Decoupling and Decking

168. Borehole pressure can be reduced by decoupling and/or decking charges. In decoupling, a space is left between the explosive charge and the borehole. In decking, short charges are taped to a primacord line or spacers are alternated with explosives to produce a discontinuous explosive column. However, if the blastholes are water filled, the effectiveness of decoupling is greatly reduced.

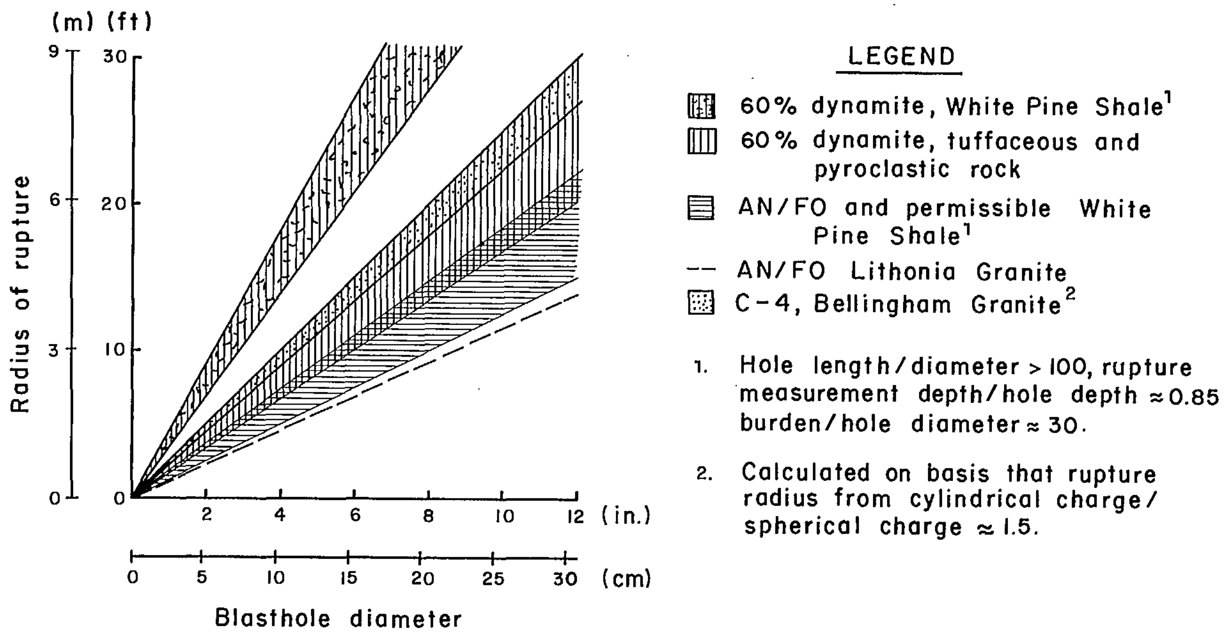


Fig 41 - Radius of rupture vs hole diameter for cylindrical charges of various explosives.

Delays and Spacing

169. To minimize backbreak and vibration, blasts should be sequenced so that each row or hole can break to a free face. The vibration level depends on the charge weight per delay. In general, all charges not separated by at least a 15 millisecond delay act together in causing vibration. Some additive effects do occur between delayed charges, as can be seen from Fig 42. However, the maximum increase is twice the single charge effect.

Collar and Subgrade Drilling

170. The depth of subgrade drilling and blasthole collar affect crest fracturing. In competent rock - static compressive strength > 30000 psi (20MPa) - a collar of 12 charge diameters should limit wall damage. In medium strength rock - static compressive strength about 15000 psi (10MPa) - the collar should be about 22 times the charge diameter. In soft or incompetent rock - static compressive strength about 5000 psi (3.5MPa) - a collar of 30 charge diameters should be used.

171. Subgrade drilling should be seven to ten times the charge diameter. It is not required in horizontally bedded or jointed rock and should be avoided over future haul roads or final berms.

Site Conditions

172. Rock properties must be taken into account if successful perimeter blasting procedures are to be developed. Most significant are the frequency and orientation of structural features, and in situ dynamic rock strength.

173. Backbreak occurs when the in situ dynamic compressive strength of the rock mass is exceeded. This strength can be determined by test blasts. The dynamic compressive strength is equal to the maximum explosive pressure which does not produce crushing in the borehole wall.

174. Dynamic tensile strength is determined from blasts in paired holes. Using an explosive loading density that will not crush the borehole wall, the maximum spacing is established at which a good connecting crack is produced between the paired holes. The dynamic tensile strength of the

rock mass is then determined from the explosive pressure on the hole and the area of rock fractured.

175. Figures 43 and 44 compare presplit faces in competent and in heavily jointed and fractured rock. The latter shows considerably more backbreak.

176. Areas where joints are tight or in-filled have less backbreak than areas with open joint systems. Figure 45 shows a good face in a highly jointed rock formation. Smooth faces are easily achieved when a weak structural feature parallels the desired face (Fig 46). Undercutting of joints or faults almost parallel to the final pit wall can produce loose face rock and overbreak (Fig 47 and 48). Good wall conditions should be attainable where steeply dipping joints are parallel to the final pit face (Fig 49). With steeply dipping joints normal to the pit face, as shown in Fig 50, some backbreak involving cross bedding fractures can be expected. Backbreak problems rarely occur in flat-lying sedimentary deposits if cross bedding fractures are not excessive.

177. The frequency of discontinuities has a major influence on backbreak and crest fracture. Discontinuities interfere with control blasting when their spacing is less than hole spacing. This can be seen by comparing the face shown in Fig 43, which has large joint spacing, with that in Fig 51 which has close joint spacing. Figure 52 illustrates how closely spaced joints can cause problems of crest fracture.

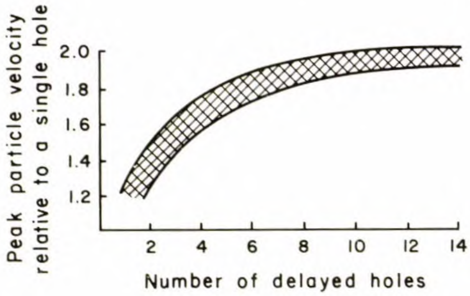


Fig 42 - Effect of successive delays on peak particle velocity.

Fig 43 - A smooth clean pre-split surface in competent rock.

Fig 44 - Backbreak and loose face rock on pre-split surface in intensely fractured rock.

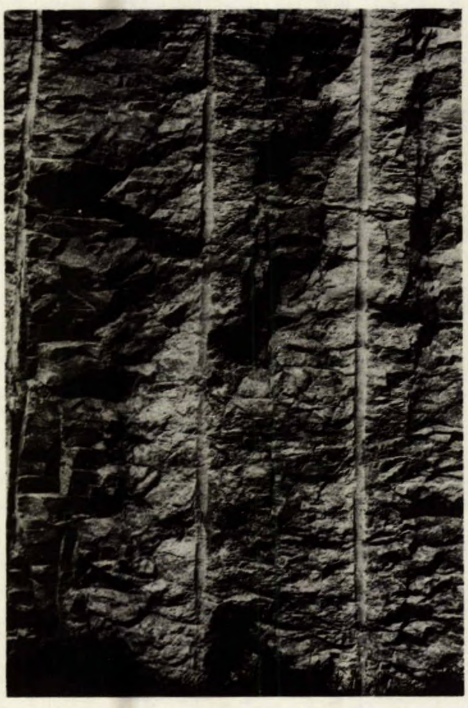


Fig 45 - A good clean pre-split in jointed rock.

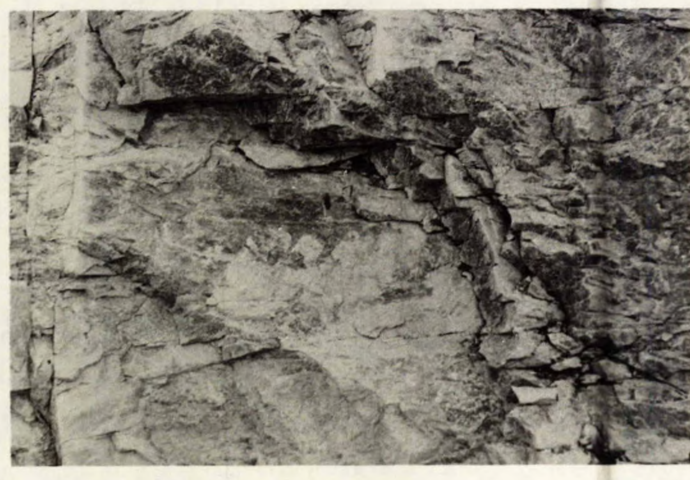


Fig 47 - Backbreak and loose face rock due to undercut jointing almost parallel to pre-split line.

Fig 46 - Pre-split surface corresponds to major jointing parallel to rock face.

Fig 48 - Backbreak and loose face rock due to undercut fault almost parallel to pre-split line.

Fig 49 - Steeply dipping joints striking parallel to rock face but not undercut.

Fig 51 - Frequent vertical jointing at 90° to rock face.

Fig 50 - Open joints at 90° to rock face.

Fig 52 - Crest fracture.

PERIMETER BLASTING TECHNIQUES

Buffer Blasting

178. Buffer blasting is the cheapest form of perimeter blasting. Drilling costs for buffer blasting are slightly higher than for production blasting because of reduced hole spacing.

179. The burden and spacing for the buffer row should be 0.5 to 0.8 times that for the adjacent production row. Hole spacing for the buffer row should be 1.25 times the burden. The charge per hole should result in an effective powder factor about 0.6 times that used for production blasts.

Cushion Blasting

180. In cushion blasting, holes are detonated after the production blast to trim the slope to the planned excavation limit. The charge is designed to create a low borehole pressure and limit backbreak. For best results all cushion blastholes should be fired together.

181. Some average powder factors for cushion blasting in various rock materials are given in Table 6. Add or subtract approximately 20% to these values if the rock is highly competent or fractured.

Table 6: Powder factors for use as first approximation in cushion blasting

	Powder factor lbs/ft^3 (kg/m^3)	Powder factor lbs/ton* (kg/tonne)
taconite	0.043 (0.69)	0.39 (0.20)
copper ore	0.025 - 0.035 (0.40 - 0.56)	0.29 - 0.40 (0.14 - 0.20)
asbestos ore	0.010 (0.16)	0.11 (0.06)

* ton of 2000 lb

182. The burden/spacing ratio in competent rock should be 0.8 to 1.25. In highly fractured or soft rock it should be 0.5 to 0.8. In the case of cushion rows adjacent to a final production row, the burden is measured from the backbreak line of the production row. Guide holes may be helpful if it is difficult to obtain a good wall with cushion blasting.

Pre-splitting

183. In pre-splitting, a line of lightly charged holes is fired prior to the production blast to produce a continuous fracture along the planned excavation line.

184. For best results, accurate drilling is required as the holes should be in the same plane. Bad toe from preceding blasts should be removed so that the pre-split holes will be easier to drill.

185. The explosive charge must not crush the surrounding rock. A buffer row of blastholes is used to shield the pre-split line from the effects of the production blast.

186. Pre-split holes should if possible be fired 50 milliseconds before the main blast. However, if hole caving is likely, loading and firing in groups of not more than six holes as they become available is recommended. Operational constraints may also require the firing of groups of pre-split holes as they become available.

Line Drilling

187. In line drilling, a row of closely spaced non-blast holes is drilled at the planned boundary. This produces a plane of weakness to which the final production row breaks. A buffer row is required to give protection from the production blast.

188. The most common hole sizes are 1.5 and 3 in. (4 and 8 cm), but large diameter holes can be used. Table 7 provides hole spacing factors for selected rock types. To calculate hole spacing, the hole diameter is multiplied by the hole spacing factor.

Table 7: First approximation to hole spacing in line drilling

Rock type	Hole spacing factor
Taconite	24
Copper ore	30
Asbestos ore	48

189. Where control blasting is not successful, the procedures used should be carefully appraised. Table 8 suggests possible corrective action.

COSTS

190. The cost of control blasting is conveniently measured by the increase over normal

Table 8: Solutions to perimeter blasting problems

Problem	Probable cause	Solutions
backbreak throughout wall (no boreholes showing)	(a) buffer row overloaded or too close (b) control blast may be overloaded	(a) move buffer row further from pit limit, reduce borehole pressure of buffer charge, use 15 msec delay between buffer charges (if not already being done) (b) increase hole spacing or decrease powder load (by decoupling or decking) of cushion or pre-split holes
backbreak around boreholes	borehole pressure greater than in situ dynamic compressive rock strength	decouple or deck charges in cushion or pre-split holes, decrease burden (for cushion blasting)
backbreak between boreholes	buffer holes too close	increase spacing, decouple or deck charges
jointing interferes between blastholes	(a) spacing too great (b) burden insufficient (c) delays between perimeter holes too large	(a) reduce spacing and powder load (b) make burden larger than spacing (c) detonate holes on perimeter row simultaneously
very poor fragmentation at pit limit or blast fails to break to pre-split or line	buffer row too far from pit limit	decrease distance from buffer row to pre-split or line drilled holes
crest fracture	collar insufficient or rock exceptionally weak (fractured or weathered) at crest	increase the height of collar, eliminate subgrade in drill holes overlying the crest of a berm, use spacers in the upper portion of the explosive column, drill small diameter guide holes

production blasting. Line drilling is the most expensive, followed in decreasing order by pre-splitting, cushion blasting and buffer blasting.

191. Typical costs for buffer blasting are shown in Fig 53 to 56. Note that cushion blasting for asbestos ore seems cheaper than production blasting (Fig 54). The reason is that the powder factor for cushion blasting is less than half that for production blasting. The costs of pre-splitting and line drilling both include the necessary buffer blasting. Line drilling costs are an order of magnitude greater than costs of the other perimeter blasting techniques.

BLASTING AND GROUND VIBRATIONS

192. Production blasts must be designed to minimize ground vibration to protect nearby structures such as buildings or underground openings or to prevent minor falls of loose rock. Control of vibration can be exercised through the delays, firing sequence, blast geometry, explosives and stemming used in the production blast.

193. Delays between rows of blastholes should be greater than 15 milliseconds, particularly for the final two production rows before the pit limit.

194. V-cuts allow individual charges to break to a free face; this reduces vibration. Square patterns produce higher vibration than rectangular patterns. The explosive with the lowest pressure consistent with the necessary rock breaking power should be used in production blasting.

195. Vibration from blasting will be slightly reduced if there is no stemming. However, in most cases this decrease is not enough to warrant the increased airblast and flyrock.

Damage Levels and Measurement

196. Ground particle velocity gives the best indication of probable blast damage. Table 9 lists particle velocity levels and associated damage.

197. Damage should not occur at a particle velocity below 2 in./sec (5 cm/sec). An empirical relationship between particle velocity, charge weight per delay and distance from blast can be used in blast design to ensure this velocity is not exceeded. Alternatively, ground vibrations can be measured using a seismograph to establish the time relationship between charge weight per delay, distance and particle velocity.

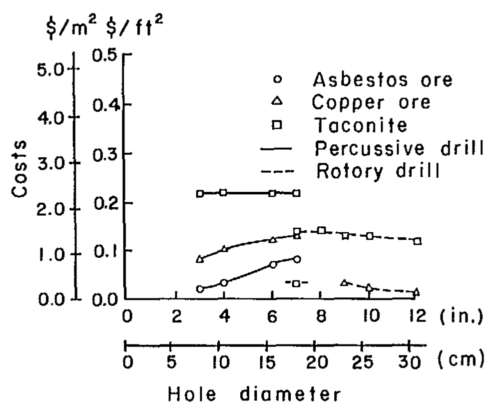


Fig 53 - Cost of buffer blasting minus cost of production blasting using same hole size vs hole diameter for various rock types.

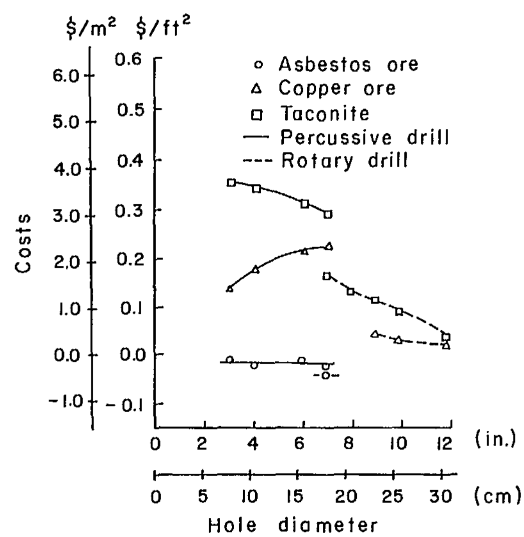


Fig 54 - Cost of cushion blasting minus cost of production blasting necessary to break same volume of ground vs hole diameter for various rock types assuming an explosives cost of $0.25/lb ($0.56/kg)

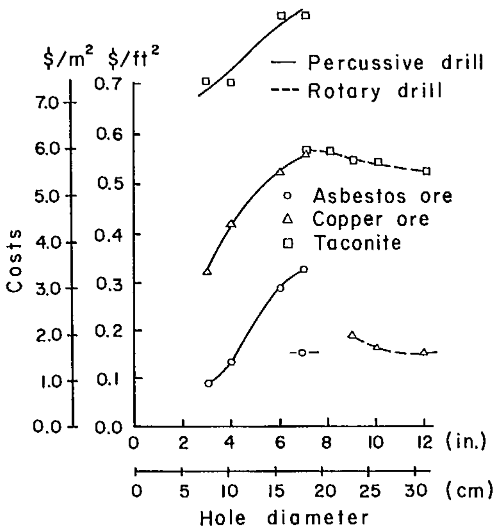


Fig 55 - Cost of pre-splitting and buffer blasting minus cost of production blasting necessary to break same volume of ground vs hole diameter for various rock types assuming an explosives cost of $0.25/lb ($0.56/kg).

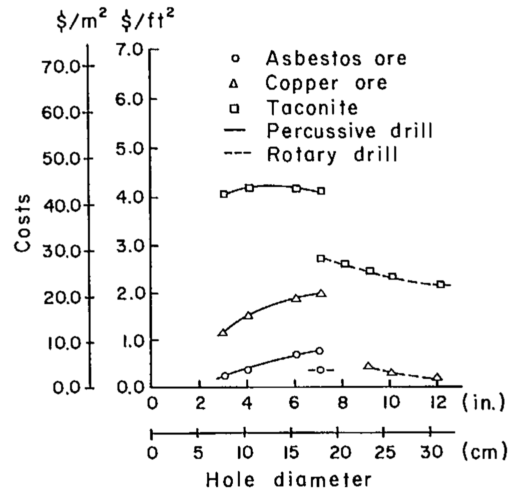


Fig 56 - Cost of line drilling and buffer blasting minus cost of production blasting necessary to break same volume of ground vs hole diameter for various rock types assuming an explosives cost of $0.25/lb ($0.56/kg).

Table 9: Type of damage related to peak particle velocity

Type of structure	Type of damage	Peak particle velocity threshold at which damage starts in./sec	cm/sec
Rigidly mounted mercury switches	trip out	0.5	1.5
Houses	plaster cracking	2	5
Concrete block as in a new house	cracks in blocks	9	25
Cased drill holes	horizontal offset	15	40
Mechanical equipment - pumps, compressors	shafts misaligned	40	100
Prefabricated metal building on concrete pads	cracked pads, building twisted and distorted	60	150

MONITORING

198. The objective of monitoring is to detect possible pit wall instability so that appropriate remedial measures can be taken. The main concern is the protection of men and equipment.

199. The principal monitoring activity is measuring movement. It is also important to monitor groundwater levels and blast vibration, which may affect stability. If rock anchors are used to support a slope, their load must be measured regularly. Regular visual inspection can detect early signs of instability, such as cracks and loose rock.

200. Recent improvements in monitoring instruments include the development of electro-optical distance measuring units (EDM) for monitoring surface displacement. Improvements in telemetry - the remote reading of instruments by means of radio or cable - now permit its use in open pit mines.

201. Monitoring takes place during the operating stage of mining. However, advance planning is essential, and the nature and location of monitors should be decided during the mine design stage.

MONITORING LEVELS

202. In the manual, monitoring is divided into three levels. Level I is the overall monitoring of the walls, designed to locate areas of potential instability. Activity at this level is planned during the mine design stage and commences with mining. Level II is the detailed monitoring of these potential instabilities. Level III is the monitoring of actual instabilities, so that mining can continue with safety.

Level I

203. Monitoring of crests and bench areas using survey equipment is recommended at Level I. Measurements should be the minimum necessary to detect movement. The instruments used should be a theodolite, a theodolite and an EDM, or an EDM

with angular measuring capability. Survey monitoring is shown schematically in Fig 57. T represents a crest or slope target to be monitored from observation stations S_1 and S_2. The survey monuments must be durable.

204. If an EDM-theodolite is available, the displacement of crest zones can be monitored by traversing. In this procedure, the instrument is positioned over the target to be monitored and distance and angle measurements are made to adjacent targets. Simple but well anchored survey plugs should be used as targets.

205. Precise levelling complements surface displacement monitoring in areas where there is little overburden and the surface is reasonably flat. Figure 58 shows the recommended location of level stations in relation to EDM-theodolite targets.

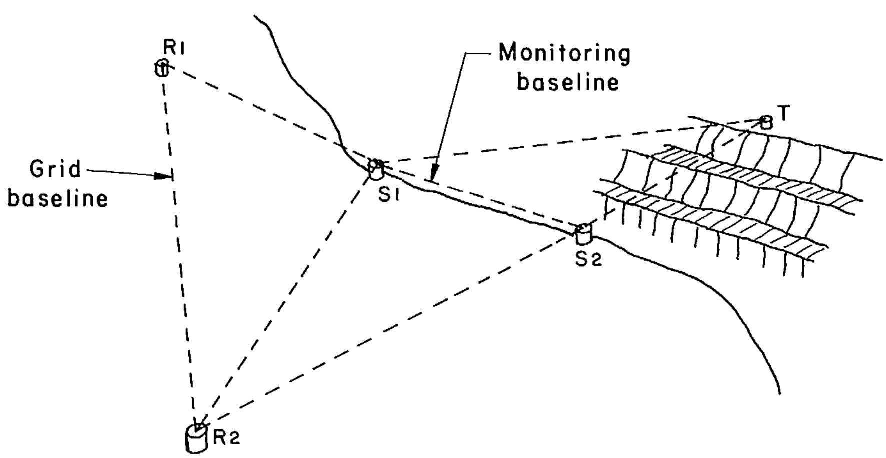


Fig 57 - Relative location of mine, observation and target monuments for monitoring pit movement.

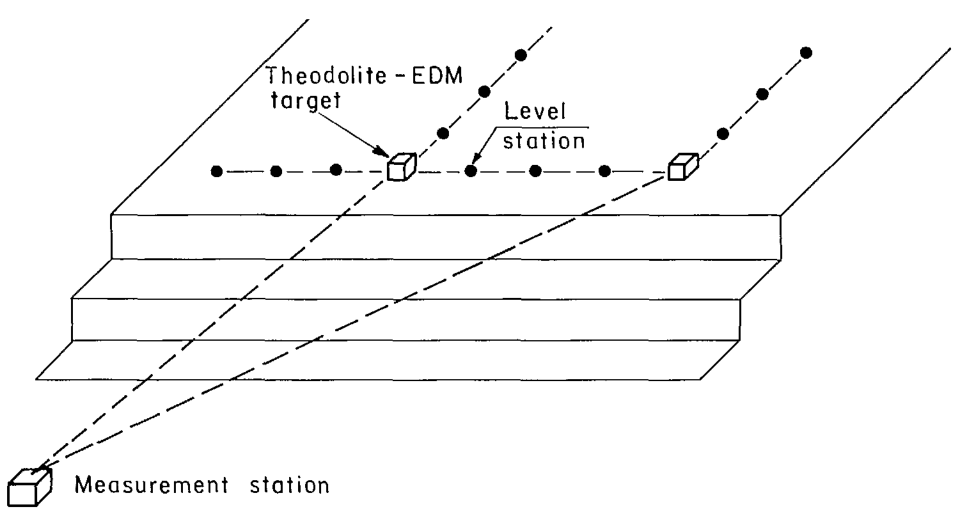


Fig 58 - Recommended location of level station relative to EDM-theodolite targets.

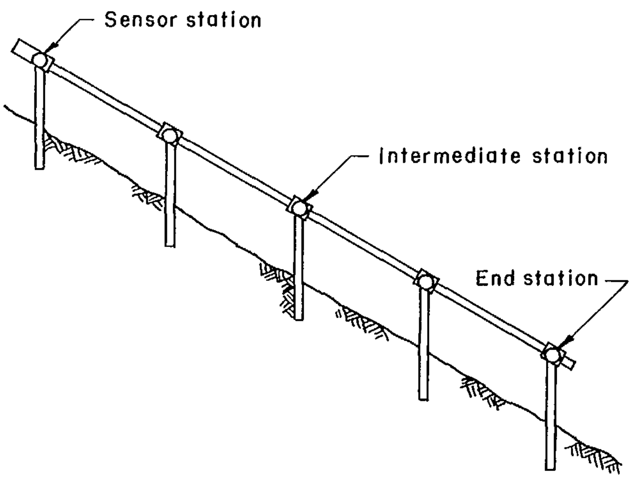


Fig 59 - Strain extensometer installation.

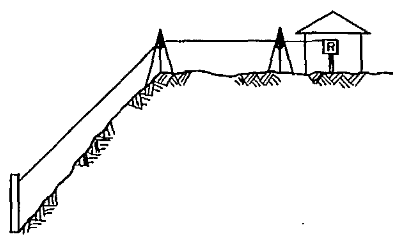


Fig 60 - Wire extensometer in a dump crest area.

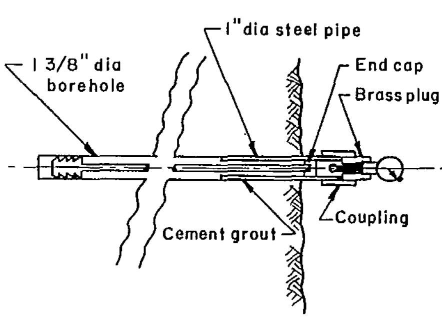


Fig 61 - Rock bolt extensometer.

206. Groundwater measurements are important if adverse groundwater conditions are anticipated. Piezometers should be installed to monitor groundwater pressure.

207. Vibration should be measured and the extent of blast damage noted as production blasting procedures are developed. This information helps ensure acceptable vibration levels are not exceeded and guides the design of perimeter blasting.

208. Automatic monitoring with telemetry is unlikely to be needed in the early stages of mine development. However, when purchasing mine radio equipment, consideration can be given to future telemetry capability.

Level II

209. When a potential instability is detected, the monitoring effort must be intensified. It is important to determine:

a. the area and boundaries of the unstable zone,
b. the amount and rate of movement,
c. the general direction of motion.

This can be done by expanding the EDM-theodolite and precise levelling measurements.

210. Tension cracks often appear on crests and berms at an early stage in pit slope movement. Tapes can measure movement between stakes anchored on either side of a crack.

211. If safety is of particular concern, more sophisticated monitoring techniques are recommended. A telescoping pipe protecting a precision measuring chain anchored between stakes, as shown in Fig 59, will measure movements from 0.02 in. to 20 in. (0.5 mm to 0.5 m). For larger movements, a wire system (Fig 60) is recommended. Movement can be continuously recorded or a predetermined movement can trigger a warning device.

212. Sub-surface displacements can be measured with borehole inclinometers, borehole extensometers (Fig 61) and inverted pendula (Fig 62). The inverted pendulum is the most sensitive and accurate instrument for monitoring sub-surface movement. However, it can be used only at depths less than about 100 ft (30 m).

213. If rock anchors are installed, their loads

should be monitored by a dynamometer or load cell. The dynamometer recommended is a steel cylinder or ring whose deformation under load is sensed by calibrated strain gauges.

214. Ground vibration due to blasting can contribute to instability. Vibration should be measured in movement zones. If a relationship between movement and blasting is established, production blast patterns should be redesigned.

Level III

215. If a pit wall cannot be stabilized, allowance must be made for eventual sliding. The primary function of monitoring in this case is to permit continued safe mining.

216. When monitoring for safety, surveying instruments are not usually adequate. Primary dependence must be placed on rod or wire surface extensometers either equipped with limit switches and warning devices or read regularly by telemetry. Devices such as slide fences used by railways to detect rock fall zones can be used.

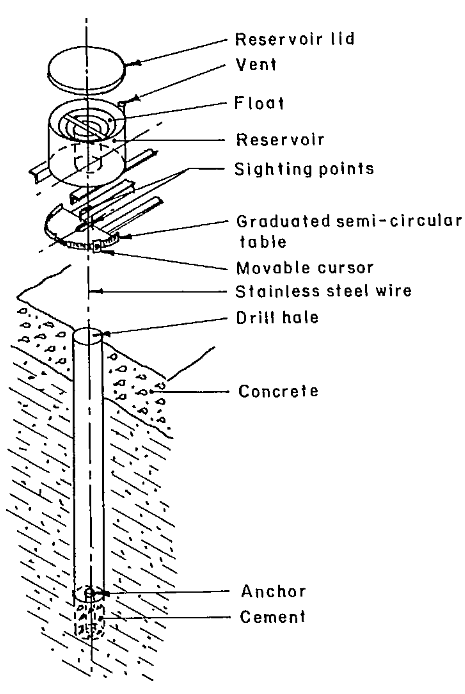


Fig 62 - Inverted pendulum.

217. Extensometers should be considered. Rock bolt extensometers are more resistant to damage than multi-wire extensometers but differential displacement of the rock normal to the axis of the hole will quickly make them inoperative. Extensometers should be read remotely by means of telemetry, or should be equipped with limit switches which provide a suitable warning when movement occurs.

TELEMETRY

218. An automatic telemetry monitoring system can read instruments and transmit, process and store data. All the monitoring instruments must include a sensor to convert readings into an electrical signal. Automatic monitoring can be set up so that each instrument will signal when a preset limit is exceeded. However, instrument failure would result in no warning - ie, the system is not fail-safe. A better method is for a master station to "interrogate" the instruments periodically (Fig 63). Erroneous readings or no response can then be investigated.

Computer Control

219. The use of telemetry means considerable data can be collected rapidly. As a result, data logging and processing can be difficult, particularly if safety monitoring requires fast data processing to warn of possible instability. Computerized data processing can overcome these problems.

220. The use of a mini - computer for as few as 20 sensors could be justified if the following capabilities are considered essential:

a. safety monitoring,
b. automatic scanning of remote stations,
c. compensation for sensor variation (eg, non-linearity of response),
d. readout in engineering units,
e. quick processing for trend analysis,
f. data logging of variables such as displacement,
g. rapid adjustment of scanning cycles.

221. Most of these functions could be performed by specially designed instruments. A mini computer however, gives a flexibility that instru-

ments do not provide. With the use of a computer, changes in the monitoring system or in the functions to be performed involve only program modifications.

222. Computer output can usually be printed by teletype. Monitoring should not require continuous data logging; in normal conditions a daily report is sufficient. Abnormal conditions can be reported at once by means of the teletype. The computer prints the time when the anomaly occurred and the identity and value of the relevant variables.

223. The computer can be programmed to modify scanning procedures when unusual conditions arise so that the appropriate sensors are read more often. The computer can also set off alarms such as horns or blinking lights if a dangerous condition develops.

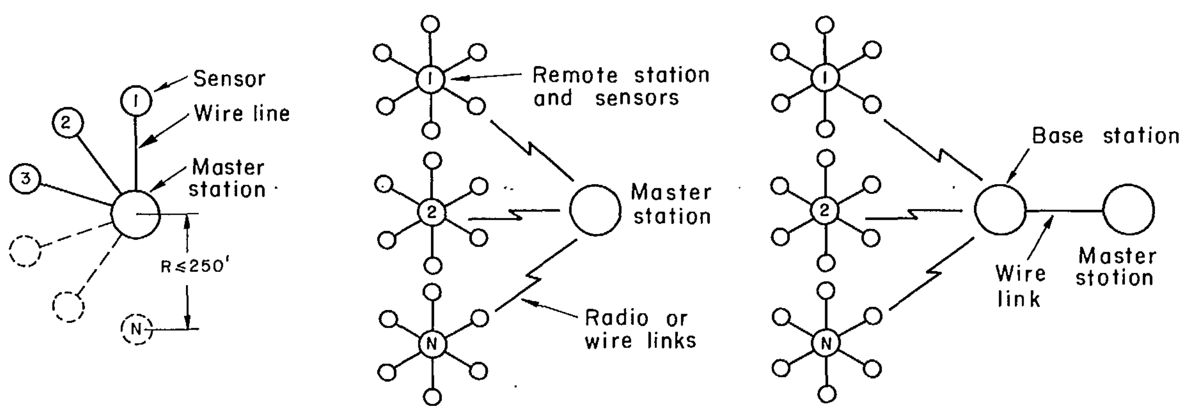


Fig 63 - Three possible telemetry configurations.

WASTE EMBANKMENTS

224. The term "waste embankment" includes tailings dams, rock dumps and overburden or soil dumps. The design specifications in the manual are primarily for purposes of stability with some consideration given to the effects of embankments on natural water courses and groundwater. Reclamation aspects are described in the chapter on environmental planning.

INVESTIGATION

225. Locating a waste embankment requires the following investigations:

a. engineering and geological surveys,
b. foundation testing,
c. borrow pit testing,
d. waste material testing,
e. topographic mapping.

226. Besides the mechanical properties needed for stability analysis, other properties are of interest. Sedimentation properties of tailings influence the retention time and depth of water required in the tailings pond. Oxidation potential, particularly of sulphide minerals, is important when considering possible effects of seepage on natural water courses. Spontaneous combustion of coal in waste can be tested to determine special design requirements. The nature of the processing reagents discharged with tailings effluents is also important because of their potential effects on natural water courses.

227. Climatic and ecologic data are required for tailings dams and revegetation planning. These are published by the Atmospheric Environment Service of Environment Canada, Ottawa. Maps are

available showing contours of temperature, humidity, wind, rainfall and snowfall. Streamflow records are published by the Water Survey of Canada, Environment Canada, Ottawa.

228. Subsurface investigations determine the nature of any soil deposits overlying bedrock. The thickness and composition of each stratum are required. The groundwater regime and its variation over the years are desirable information. Details of existing or planned mine workings are important. It is also valuable to know if there are solution caverns.

DESIGN

229. Rotational sliding is the typical mode of instability for waste embankments (Fig 64). The cause might be too steep slopes, overloading weak foundations or a rise in water level in the embankment.

230. Sliding sometimes occurs if a sloping foundation is composed of weak clay or silt or has a thin layer of altered material on top of bedrock (Fig 65).

231. Appropriate analysis techniques to evaluate stability are described in detail in the design chapter. Except for foundation problems, embankment design has the advantage that the nature of construction material and procedure are known to the engineer. Stability calculations therefore have more certainty than do analyses for mine slopes.

232. Liquefaction occurs when tailings become temporarily suspended in water and flow like a viscous liquid. This often happens when tailings dams are breached. If tailings in an embankment are loose and saturated, vibration from earthquakes or trucks might cause liquefaction. If necessary, the designer must specify deposition procedures or incorporate drainage to avoid this condition.

233. Overtopping of a tailings dam can lead to breaching and loss of control of the pond; this is probably the most common cause of dam instability. The designer must determine the appropriate freeboard and width of crest. It may be necessary to use material that is less subject to erosion than normal tailings.

234. Erosion of embankment slopes can lead to instability. In tailings dams, the concentration of water flow at re-entrant corners and other changes of contour can lead to the transporting of solid particles by seepage water passing through or under the dam. When sufficient material has been carried out by the water, a pipe may be formed that progressively extends under the embankment until water flow becomes excessive and the dam is breached.

235. Settlement beneath an embankment may lead to fracturing of nearby pipes. In tailings dams, it may crack the impervious seal. Design must anticipate such movement.

236. Coal waste piles can be ignited by spontaneous combustion. Poisonous gases can be generated, and the fire may become very difficult to control. In attempting to extinguish it, the generation of coal dust could lead to an explosion. Design and construction of these dumps should take into account this hazard.

237. Seepage through tailings embankments should be inhibited and drawn down into drains. In this way stability is maintained and environmental impact can be controlled.

238. The catchment area behind a tailings dam can be extensive. Relocating streams, ditches and dykes should be considered to minimize the run-off into the tailings pond. This can reduce possible storm flooding considerably and minimize the quantity of seepage effluent to be treated.

CONSTRUCTION AND OPERATION

239. Tailings dams are commonly constructed in one of three ways: the downstream method, the upstream method and the fixed centreline method (Fig 66). Stripping of organic matter and preventing seepage contribute greatly to embankment stability.

240. Maintenance, particularly of tailings dams, is important after an embankment has been constructed. The conditions under which they exist change continuously. Problems can develop very quickly particularly during periods of high precipitation. If severe erosion is anticipated,

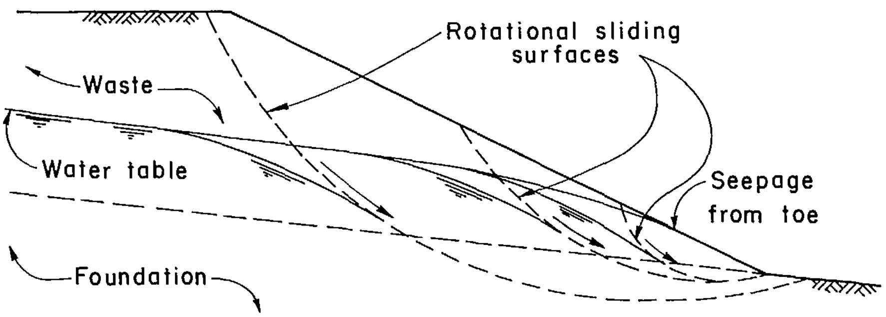


Fig 64 - Rotational sliding in a waste embankment.

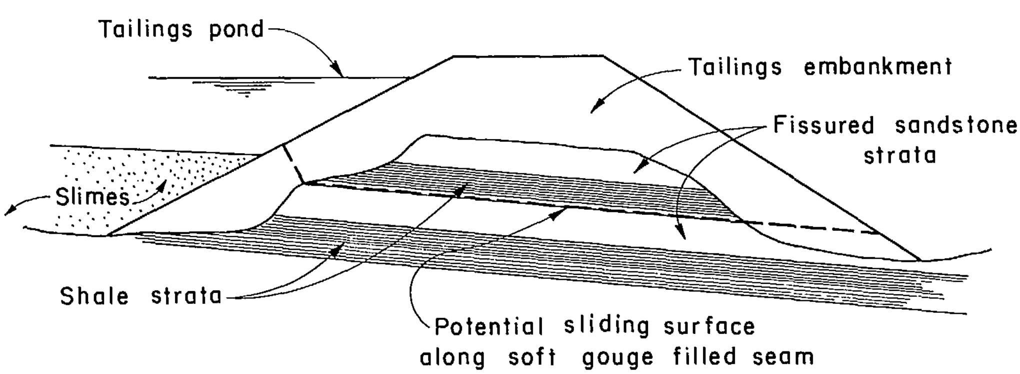
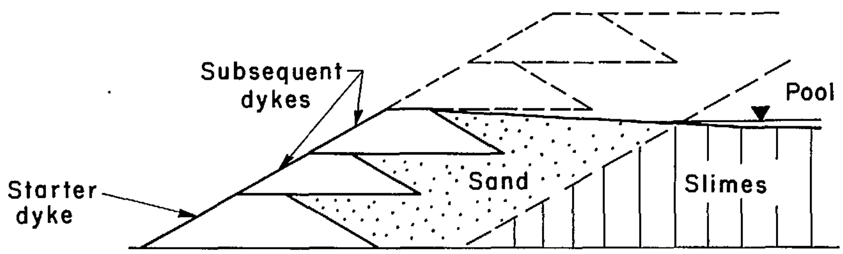
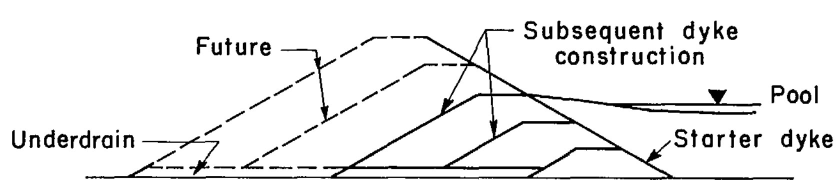


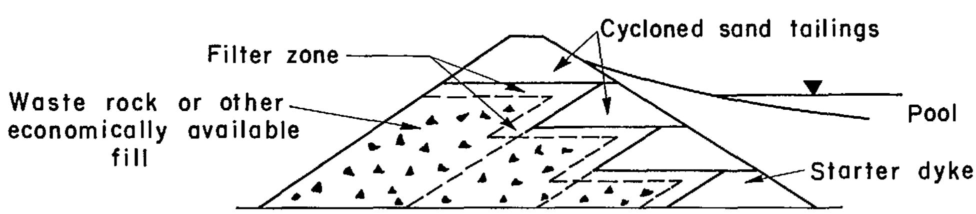
Fig 65 - Sliding on a weak foundation.



UPSTREAM METHOD

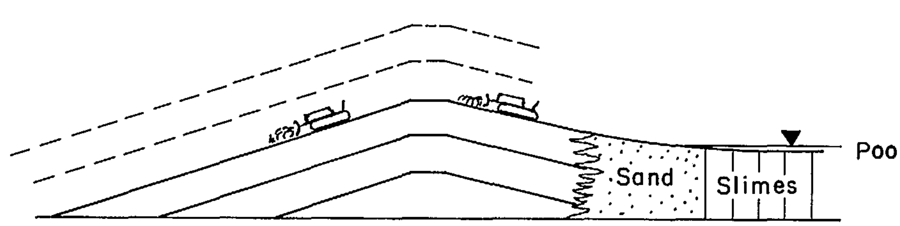


DOWNSTREAM METHOD



DOWNSTREAM METHOD

(Supplementary fill incorporated within embankment section because rate of production of suitable cycloned tailings is limited).



CENTRE LINE METHOD

Fig 66 - Tailings embankment construction methods.

riprap or vegetation should be used for protection.

241. Monitoring crest movement and groundwater, particularly in tailings dams, provides early warning of problems. Such measurements should be an integral part of the mine monitoring program.

PERMAFROST

242. Locating waste embankments on permafrost can produce unusual results. For instance, permafrost may rise into the embankment. This may result in slope instability if ice lenses are created that thaw in summer. On the other hand, permafrost may recede into the foundation and this could result in excessive settlement if the thawed material is clay or silt. Embankments on permafrost slopes can encounter unusual conditions for sliding, groundwater flow and ice formation. Correcting these problems can be very difficult. It is better to anticipate the difficulties and take measures to avoid them.

ENVIRONMENTAL PLANNING

243. Mine designers must consider the effects of a mine on its surroundings. Engineering alternatives at both new and existing mines have to be judged according to their environmental influence. After a plan is adopted, the effects on the environment should be monitored. Chief among the many environmental factors of concern in mine planning are water pollution control and revegetation.

EXPLORATION

244. In the early stages of a mining venture, maintaining environmental quality is usually inexpensive and fosters good public relations. However, staking, line cutting, geophysical surveying and diamond drilling are frequently done by contractors who may not be concerned with protecting the developer's image. Consequently, contract supervision is particularly important. Some aspects to note are:

a. Fuel supplies should be stored above high water level.
b. If oil is spilled on land, it should not be removed or buried. Attempts to do so can create more problems; the area affected can be fenced to protect wild life.
c. Stream crossings should be designed so that flow is not affected, causing deposition of sediment.
d. Oil, drilling fluids, garbage and other contaminants should not be dumped in lakes and streams.
e. Garbage may be burned with caution but should preferably be buried.
f. Sanitation facilities should be located away from fresh water.
g. Cutting of lines and stripping of vegetation should be minimized.
h. Test pits, trenches and other excavations should be filled in after use.

245. It may be advisable to take samples of fresh water sources to establish natural conditions before exploration starts. If there is a possibility of contaminating drinking water wells, it may be advisable to grout all exploration holes.

246. When close to residential areas, potential noise problems must be considered. Prior consultation with local government and other groups can often forestall complaints.

ECOLOGICAL INVESTIGATION

247. When a decision is made to bring an orebody into production, the local ecology along with social and economic conditions should be studied. This provides baseline data against which to measure the effects of mining. Aerial photographs provide information on the surface water regime and types of vegetation. A control area similar to the mine site but far enough away to be unaffected by operations should be selected as a reference.

248. Water courses are usually more vulnerable to change than land ecology. Considerations in pre-mining surveys and subsequent monitoring include:

a. physical aspects such as depth, temperature and sediment texture;
b. water characteristics such as colour, turbidity, dissolved solids and organic content;
c. toxicants such as heavy metals and detergents;
d. biota such as phytoplankton concentrations, plants, fish and bacteria.

249. Terrestrial studies usually include soil types, vegetation and animal life.

SOCIO-ECONOMIC STUDIES

250. Plant operation will affect the local economy and population distribution and will constitute a new land use. Local road and rail traffic and the community water supply may be significantly affected.

251. Harmonious relations with local residents are obviously desirable. Public presentations to discuss such aspects as parking facilities, location of the construction campsite, hours of work and reclamation plans can be very useful.

METEOROLOGICAL STUDIES

252. Knowledge of the local climate may be important. Air blast effects can be influenced by wind, as can plume dispersion from mills and smelters. It may be advisable to establish a local weather station to develop a data bank of pertinent information.

NORTHLAND CONSIDERATIONS

253. Although all of Canada is in the North, operating in the Arctic and sub-Arctic poses special problems. In permafrost regions, excavated slopes and stripped frozen soil may thaw, resulting in slumping. The disposal of garbage and sanitary waste is more difficult. Oil spills are more troublesome because of slower breakdown of hydrocarbons. Once initiated, stream erosion tends to continue with possible unsatisfactory consequences.

EFFECTS OF CONSTRUCTION

254. Vegetation extracts water from the ground, and its removal during construction can affect the groundwater regime. Drainage of operations will affect groundwater.

255. Removal of vegetation near water courses can lead to increased nitrogen levels in the water. Preproduction site drainage and drainage water from operations can also affect receiving waters. Fish and plant life can thus be affected.

WATER POLLUTION

256. The impact of operations on existing water courses is not easy to predict. Changes in one physical property can produce changes in others. For example, sediment may have no direct toxic effects on fish but may, however, destroy plant life and deprive the fish of food. Possible effects are best predicted by observing similar mining operations in similar environments.

257. The problem of acid contamination from mines and waste dumps is common throughout the country. It usually arises from the presence of pyrite, pyrrhotite and chalcocite. The production

of acid can be prevented by excluding any one of oxygen, water or sulphide. The most economic approach must be determined for each property.

258. Groundwater levels as well as quality can be affected by operations. This may cause local water users considerable distress. Lowering of local wells is not uncommon but raising of groundwater levels because of an extensive tailings pond can also occur.

259. The cost of an antipollution program may be minimized by determining water quality before operations start. This will prevent having to fulfill subsequent quality requirements that exceed the natural conditions. By rerouting streams and by ditching and dyking it is often possible to reduce the quantity of water that must be controlled.

260. Recycling of water may be possible, thereby reducing freshwater requirements. This could appreciably reduce the volume of water to be treated and consequently the extent of facilities. In some cases, reagent costs may be reduced.

261. The cost of controlling the impact of tailings ponds on water resources can be minimized by appropriate procedures. A seal on the upstream side of a dam, for instance, can be effective in reducing contamination. Vegetation of tailings will reduce mineral leaching by rainwater.

RECLAMATION

262. Reclamation requires establishing objectives for ultimate use of the site. In many cases there will be no ultimate use; consequently, ensuring safety of the casual visitor plus a minimum of revegetation will be adequate. In other areas, recreational, commercial or industrial developments may require land shaping and revegetation.

263. Revegetation requirements vary between and within mining properties. Tailings ponds and dykes, waste rock dumps and overburden soil dumps all require different treatment. Initial neutralization is often required together with the addition of fertilizer. A judicious selection of seeds or seedlings can produce a maintenance-free, self-sustaining vegetative cover. This may require an appropriate succession of plants and a maintenance period in which additional fertilizer and neutralization may be necessary. The period when maintenance is required can be as brief as three years. However, it may be much longer under severe conditions either of the environment or of the material on which growth is being promoted.

264. Many examples of successful revegetation programs now exist, and an inventory of selected properties across the country is included in the manual. This can provide guidance to new programs.

OTHER OPERATIONS

265. Haulage roads can create unsatisfactory environmental effects. Culverts must be planned to avoid the effects of flooding and silting. The effects of chemicals applied to road surfaces should be examined before use. Truck noise might be of concern to local residents, especially at night.

266. Pipelines - tailings, slurry, water and oil - must be designed to cope with blockage, power failure or breakage. Easy access is important and so is the provision of emergency dump points at suitable low points.

267. Mine dewatering plans must consider possible effects on groundwater and surface waters. Treatment may be required before disposal.

268. Blasting may require special consideration if the property is close to residential or urban areas. Blasts should be designed to avoid damaging vibration. Individuals are much more sensitive than buildings to ground shock. They may believe that blasting has caused minor structural damage that in fact existed previously. Consequently, photos and measurements before blasting begins can avoid later expense. An appropriate public relations educational program could be a good investment.

LEGISLATION

269. Environmental protection is an area of joint jurisdiction between the federal and provincial governments. In most cases provincial laws apply. However, federal laws set minimum standards.

270. Three federal acts are of particular importance. The Fisheries Act sets national standards for water quality, and provides powers to prevent industrial pollution. The Canada Water Act provides for water resource management and prescribes limits of waste discharge. The Clean Air Act sets standards for air quality.

271. The manual includes a list of the relevant legislation affecting environmental planning for mining. In addition, a review is available of the regulations issued under authority of the various laws.

COSTS

272. A preliminary ecological reconnaissance of the proposed mine site should require about five man-days, costing about $2000 (1974).

273. A complete ecological investigation with baseline studies might require 30 to 250 man days, and cost $10000 to $60000 (1974).

274. Most operating mines incur costs for water or air sampling. There may be a major requirement for waste water treatment; one mine in Eastern Canada spends $300000 (1972) per year for operating and amortizing its treatment plant; an Ontario mine treats 3.2 billion gallons (14.5 million m^3) of waste water per year at an annual cost of $31000 (1970).

275. Revegetation of waste was estimated to cost $150 to $500/acre ($60 to $200/ha) in 1973, plus $200/acre ($80/ha) annually for maintenance. However, if soil cover is necessary, costs rise rapidly; revegetation of coal mines has been estimated at $300 to $3000/acre (1972) ($120 to $12000/ha). A Manitoba mine estimated initial and maintenance costs for sulphide tailings reclamation to be $750/acre ($300/ha) and $425/acre ($170/ha) respectively in 1972.