

PIT SLOPE MANUAL

chapter 3

MECHANICAL PROPERTIES

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PIT SLOPE PROJECT

of the

Mining Research Laboratories

Canada Centre for Mineral and Energy Technology

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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

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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.

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The Pit Slope Manual is the result of five years of research and development, cooperatively funded by the participating Canadian mining companies, consultants, universities and the Government of Canada.

The Pit Slope Group consisted of D.F. Coates*, M. Gyenge*, G. Herget, B. Hoare, G. Larocque, D.R. Murray, R. Sage* and M. Service.

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SUMMARY

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.

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.

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.

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 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.

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.

Where in situ tests are not used, labora-

tory tests are required. There are two basic tests, triaxial and shear box.

The main feature of a triaxial apparatus 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.

In the shear box, 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.

Substance ie, intact rock or soil material, 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.

The in situ test for discontinuity shear strength can also be used to determine the rock substance strength. In situ testing on a large scale is expensive and laboratory tests are more commonly used to determine rock substance shear strength.

Triaxial tests 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.

The result of 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. From the obtained diagrams the strength

parameters, appropriate to the stress range under consideration, are selected for slope design.

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.

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.

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.

Two methods 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. In the latter, the load at failure is correlated empirically with the uniaxial strength. Irregular or random shaped specimens can be used.

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.

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.

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.

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.

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.

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.

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.

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.

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.

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.

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.

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INTRODUCTION

PURPOSE AND SCOPE

1. The purpose of the open pit design is to determine the geometry of the mine boundaries, or the optimum slope angle for a desired pit height.

2. Adequate information is needed about the geological structure of the wall, the mechanical properties of the wall material, and the groundwater conditions. Investigation and analysis procedures for the structural features and for the groundwater conditions in the wall material are detailed in the Structural Geology and Groundwater chapters of the Pit Slope Manual.

3. In this chapter and its supplements guidance is given on the procedures required to obtain adequate knowledge of the mechanical properties of the soils and rocks which constitute the walls of the pit.

4. This chapter explains why it is necessary to seek certain data and information, and tests that are required to obtain these data. In the attached supplements these tests are described.

5. Techniques for determining the mechanical properties of a rock mass have been the object of much research during the past decade; they are still under development, subject to continuous

improvement and modification to suit specific requirements. Consequently, the suggested procedures must be viewed only as the best presently available in the opinion of the specialists who worked on this chapter.

INTERACTION BETWEEN ACTIVITIES

6. Large volumes of rock masses usually display considerable variations in their mechanical properties, due to the varying degrees of alteration and weathering, as well as to the presence of minor and major discontinuities.

7. If the discontinuity planes are of sufficient extent and are critically oriented, they represent weakness planes within the rock mass; in this case, the shear resistance along them will govern stability of the slope. On the other hand, if planes of weakness are absent or are not critically oriented but the rock substance is weak, the strength of the rock substance may be the dominant feature in the design.

8. Due to variations in the wall materials and their conditions, and in structural features, the rock mass is usually divided into zones with varying degrees of engineering significance. The

division of an open pit site into a limited number of clearly defined homogeneous zones or design sectors, is primarily a problem in structural geology.

9. By taking into consideration the excavation depth, the orientations of the discontinuity planes, the rock types, and the rock mass qualities, an initial appraisal of possible instability modes can be made for each design sector. This initial stability assessment is primarily a design task.

10. A testing program, including both field and laboratory tests on representative samples or areas, must be established for each design sector.

11. The results of the test program are then analyzed and presented in the form of mean values and their standard deviations, and used in the slope design in terms of statistical probability of stability.

12. The principal use of mechanical rock properties is in wall design; as shown in Fig 1, these test results also provide essential information for associated designs.

13. Situations can arise in which it is possible to use mechanical support. The strength and deformation properties of the rock mass, or the plane of weakness of a sector which needs support, represent the basic data in designing appropriate mechanical support.

14. To eliminate loose rock falls from the bench faces, and consequently to improve the overall stability of the rock slope, perimeter blasting may be used. Design of such a system requires both static and dynamic elastic properties, as well as the strength of the rock substance.

15. Waste dump planning is an integral part of any pit design. The depth, strength and index properties of the overburden are required to avoid base failure under the dump pile.

DESIGN STAGES

16. In the life of a mine there are three principal development stages when design requirements arise, namely:

- a. the feasibility stage, when results from exploration are analyzed;
- b. the mine design stage, when the existence of an

orebody has been established, and financing for production has been approved;

- c. the operating stage, when the assumptions contained in the original mine design have been superseded, eg, commodity prices have changed, a new formation has come to light, ground conditions have drastically changed, etc.

17. Programs for investigating the rock mass properties will vary with the stage of the mine development. The parameters required during the three phases are essentially the same; the methods for obtaining them may differ however due to the difference in the accessibility of the test site, or to the changed availability of appropriate test samples, or to previous experience in each of the three stages.

18. The quality of available data, and consequently the statistical reliability of the mechanical rock properties, will also vary accordingly. This does not represent a serious problem for the quality requirements are usually in harmony with the availability of test samples.

METHODS OF ASSESSING IN SITU ROCK MASS PROPERTIES

19. When determining mechanical rock properties, the emphasis must be on providing data on the strength of the rock mass which is resisting failure.

20. Among strength parameters, the most important is shear strength; in case of hard rocks this applies to the shear strength of a discontinuity, which for weak materials it refers to the shear strength of the substance.

21. Determining the representative shear strength of a rock mass is often a time-consuming, costly, and difficult process. Large-scale in situ shear tests are desirable, but are expensive.

22. Three lines of approach that can be used to obtain information on in situ behaviour of the rock mass for slope design, are:

- a. direct measurements by means of field and laboratory tests
- b. evaluation by means of mathematical simulation
- c. evaluation of stable and unstable natural and excavated slopes.

23. Direct testing would be satisfactory, if an adequate number of large-scale tests could be

performed on the site. In practice, because of the non-homogenous nature of the rock mass and of the large scale and complexity of the geological features, an exclusive use of this direct method is however impractical. The economic use of test data, either on a large or on a laboratory scale, or both, is enhanced if coupled with the two other approaches.

24. Mathematical simulation of the rock mass may furnish a useful framework for estimating the large-scale behaviour of the mass from the properties and disposition of its principal components. Consequently, the requirements for large-scale testing can be reduced. In addition,

such a mathematical simulation ie, theoretical and empirical shear equations, provides a means for assessing the effect on the strength of the rock mass by the variation of several basic parameters. Among these are: strength of the rock substance, degree of interlocking in the failure surface, and normal stress on the failure plane.

25. Experience with previous slopes is one of the best methods for estimating the full-scale strength of a rock mass, at least for the conditions and time period of past failures. To undertake such analyses it is necessary to have certain data as described in Appendix D of the Design Chapter.

PLANNING OF TEST PROGRAM

26. The required excavation depth, orientation of discontinuity planes, rock types, and rock mass qualities, etc are all considered, and an initial stability assessment is made for each domain. The assessments are based on past experience gained through stability analyses of walls of the same or similar ground conditions and wall geometry.

27. From the initial stability assessment the design demands of the various parts of the mine are recognized, and the pit is delineated into design sectors, as schematically shown in Fig 2(a). This structural geology plan is the point of departure for planning a test program.

28. For a test program to be effective, it must recognize the needs of the different design sectors. It must also take into account the variability of the wall material. With limited tests, the mining engineer must be able at least to anticipate the range of values likely to be encountered. Figure 3 illustrates the variability of the different wall materials the mining engineer often deals with. The material on the right side of the photograph is competent enough and the benches stand quite well. Next to that section, in the centre of the picture, weak

material has resulted in much sloughing, and the benches have disappeared. On the left side the material is again somewhat competent, and parts of the benches are still in place, despite sloughing.

29. On information provided by the structural geology plan, eg, Fig 2(a), and on other available data, the test requirements are established for each design sector. Cross-sections or three-dimensional views, and brief narrative descriptions of the sectors, are helpful aids in interpreting the prevailing structural features, and in deciding details of the test requirements. For example:

30. Sector A, whose simplified cross-section is shown in Fig 2(b), is located in bedded blocky dolomite, striking $N\delta_1^\circ W$ and dipping $\beta^\circ SW$. The sector contains two joint systems; both strike at $N\delta_2^\circ E$, or nearly parallel to the proposed slope, with dip angles of $\alpha_1^\circ SE$ and $\alpha_2^\circ SE$. Instability can be anticipated along the continuous joint plane of the α_2 system, which will probably surface in the vicinity of the proposed toe. Although $\alpha_1^\circ < \alpha_2^\circ$, due to calcite coating of the α_1 joint system, instability could also occur along the continuous weakness plane which has

emerged at height h . The uniaxial compressive strength of dolomite is available in Test Report No. 16 - 73. The wall sector contains groundwater all year.

31. Sector B, of which a simplified three-dimensional view is shown in Fig 2(c), is in the same bedded blocky dolomite as Sector A, striking $N\delta_3^\circ E$ and dipping $\beta_1^\circ NW$. The two joint systems strike $N\delta_4^\circ E$ and dip $\alpha_3^\circ SE$, and strike $N\delta_5^\circ W$ and dip $\alpha_4^\circ NE$, form numerous 3-d-wedges. Bench slides in the past justify concern about stability of the wedge which will probably surface at height h_1 above the proposed floor level. Fracture planes of the α_3 and α_4 systems can be typified as: (a) coated, rough, partially open, and (b) clean, rough, interlocking joints, respectively. The wall material within Sector B is dry. This wall sector contains the haulageway and any instability here could be critical.

32. A planning decision must now be made regarding:

- the required mechanical rock properties
- the order of priority for obtaining these properties
- the general approach to be followed, ie, in situ or laboratory testing
- the adequacy of any available test data, and whether additional tests are required
- any expected variation in the mechanical rock properties.

33. At this point all the test requirements must be listed for each sector irrespective of repetition. Though seemingly redundant, this action will ensure that all important rock properties will have been considered.

34. It is good practice to summarize all decisions as shown in Table 1.

35. The final step in planning a test program

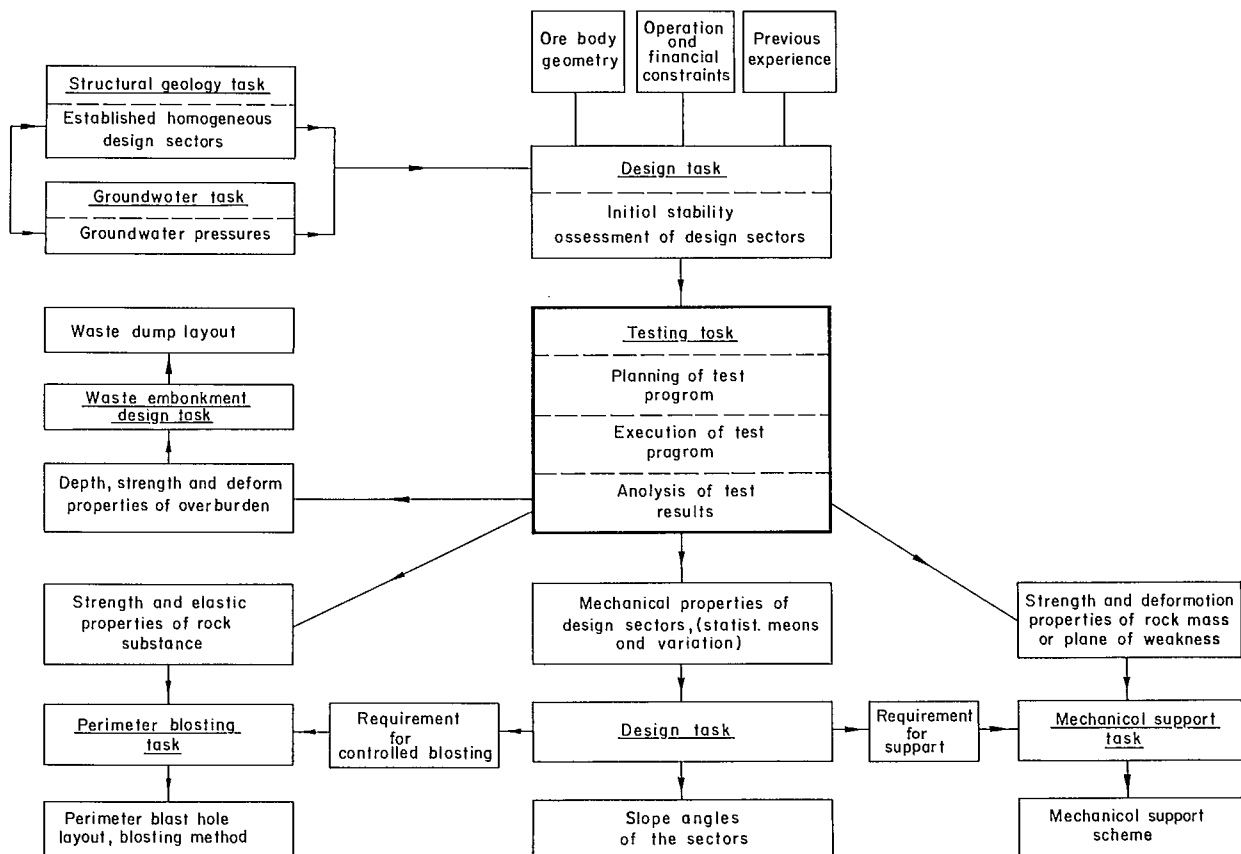


Fig 1 - Interaction between the activities involved in slope design.

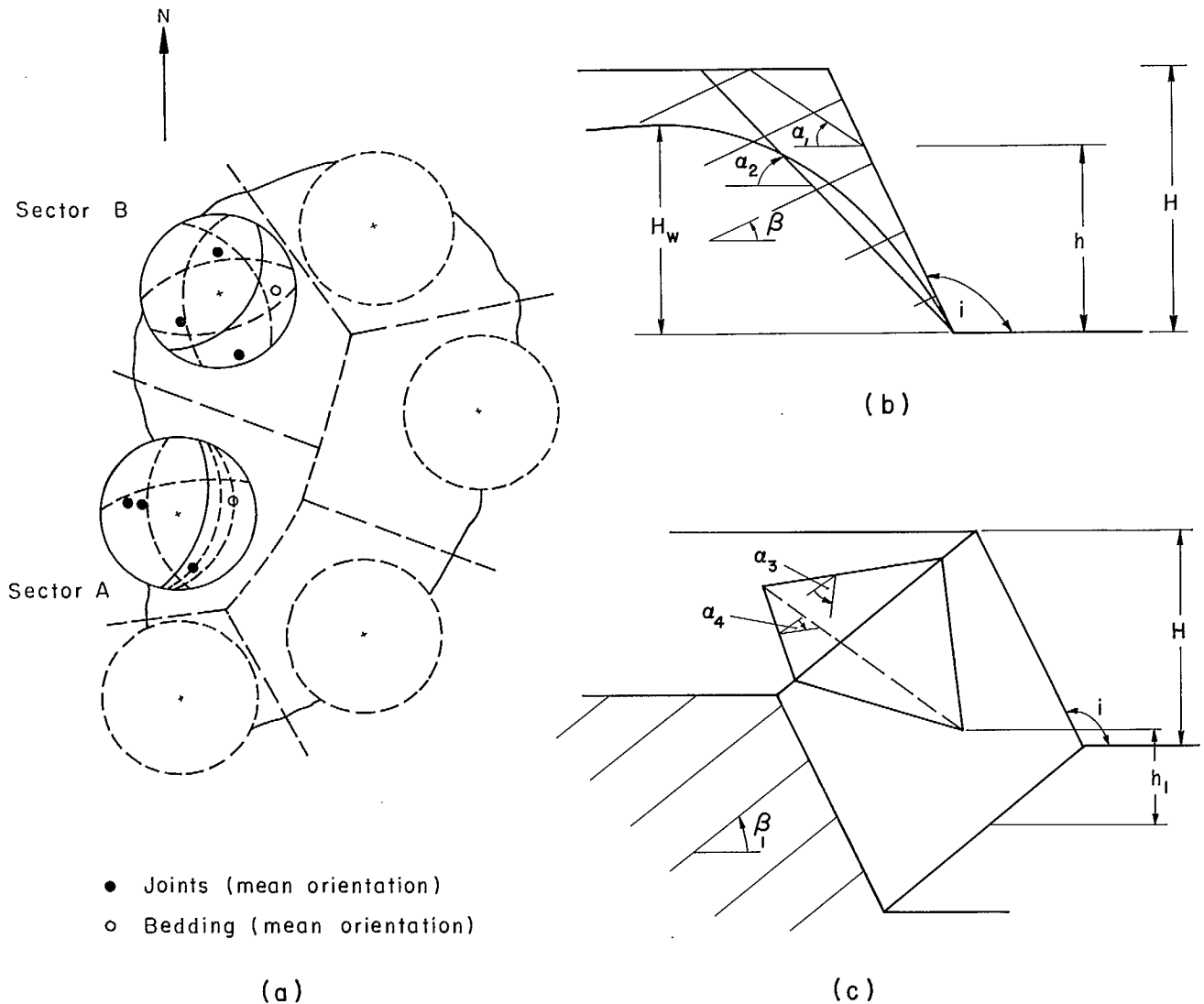


Fig 2 - Structural geology plan; represents the starting point for planning a test program: (a) schematic outline of a structural geology plan showing the design sectors, (b) cross section for Sector A, (c) three-dimensional view for Sector B.

includes decisions regarding:

- the locations and methods for taking samples so that the tested specimens adequately represent the in situ conditions and their variations
- the availability of suitable samples so that the sampling and specimen preparation expenses can be minimized
- the required extent and depth of testing, by considering the known geological variations, by

taking into account available test data originating either from the mine site or from other sites, and by considering the relative importance of the specific mechanical properties in the stability analyses to be performed.

36. The decisions reached are listed in tabular form, such as in Table 2, which contains the program as it is to be executed.

Table 1: Examples of the required mechanical rock properties of design sectors

Design sector	Required mechanical properties in order of priority	Testing approach	Available test data	Remarks
A	1. Shear strength of α_2 joint system	laboratory shear		some weathering above 650 ft elevation
	1. Shear strength of α_1 joint system			calcite coating
	2. Compressive strength of dolomite		uniaxial compressive strength, Test Report #16-73	adequate test data
	3. Rock mass density	field test		
B	1. Shear strength of α_3 joint system	laboratory shear		calcite coating, alteration in B-23 area
	1. Shear strength of α_4 joint system			alteration in B-23 area
	2. Compressive strength of dolomite	laboratory uniaxial	uniaxial compressive strength Test Report #16-73	some additional test required for B-23 area, due to alteration
	3. Rock mass density	field test		
C	etc			

EXECUTION OF TEST PROGRAM

37. Procedures for determining the mechanical rock properties are specified in the Supplements to this chapter. Each detailed specification or standard, with the practical examples included essentially tells how to proceed. Prior to that it is however necessary to discuss the need of certain data or information and how it is to be obtained.

38. Instability occurs within the wall material whenever its strength is exceeded; consequently, most important are the strength properties, and in particular the following three:

- a. shear strength of discontinuities
- b. shear strength of intact material or substance
- c. compressive strength of intact material or substance, required for a measure of compressive strength of the rock mass.

SHEAR STRENGTH OF DISCONTINUITIES

39. If the discontinuity planes are large and are critically oriented, they represent weakness planes within the rock mass; the shear resistance along such planes governs the stability of the slope.

40. Depending on the structural features of

the wall, sliding can occur in the following forms:

- a. plane sliding, if sliding involves a single plane of weakness and a single block
- b. multi-plane or multi-block sliding, if sliding involves several weakness planes, and formation of several blocks
- c. 3-d-wedge sliding, if sliding occurs along two or more planes of discontinuity which strike at angles oblique to the slope face.

41. An example of a single sliding plane is shown in Fig 4(a) after sliding of the single block, shown schematically in Fig 4(b).

42. Another example of a single sliding plane is shown in Fig 5(a). In the wall sector located at the centre of the photograph, sliding occurred along a series of short planes with connecting cross joints, as shown in Fig 5(b).

43. The photograph in Fig 6(a) shows a typical wall section of a potential multi-block instability as schematically illustrated in Fig 6(b).

44. The slide shown in Fig 7 involved a block of 3-d-wedge-shaped material. Shear strength was exceeded along the two planes of discontinuity



Fig 3 - Material variation within a pit wall; on the right, material is competent enough for the benches to stand quite well, next to that, weak material has resulted in much sloughing and causing the benches to disappear.

which form oblique angles with the slope face. The third plane, which faces the reader, was not directly involved in the slide. It contributed to it however, because as an open joint it channelled surface water to the sliding planes, which in their turn reduced the stability of the 3-d-wedge.

45. Although the geometry of these slides differ, the stability of the wall sectors in each case depends entirely on shear resistances of the critically oriented geological discontinuities.

46. The purpose of the mechanical properties test is to determine the shear strengths along the planes of discontinuity. Measurement of the mechanical properties is not easy mainly because of their variations. The number of tests that can be conducted is restricted by expense. It is imperative however that an adequate number of tests be conducted so that the range of values may be assessed. Engineering judgement regarding the adequacy and expense of the tests must be exercised in each situation.

47. Testing procedures designed to establish shear strength parameters of discontinuity planes must consider the following qualitative variables of the planes:

a. rough clean discontinuity

b. coated discontinuity

c. discontinuity with gouge.

48. The testing procedures used to establish the strength parameters of a discontinuity are detailed in Supplements 3-2 and 3-3. The following approaches can be followed:

a. in situ tests

b. laboratory tests.

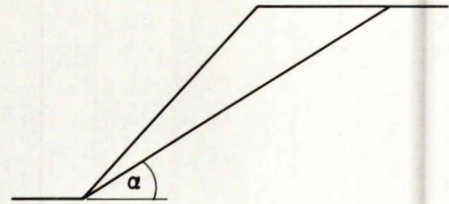
In Situ Tests

49. As the effects of surface irregularities are best represented by large specimens under in situ conditions, the large-scale in situ direct shear tests described in Supplement 3-3 para 11 to 23 might be preferred. Because of the high cost of in situ tests, these cannot be considered on a regular basis, but only in cases of special circumstances such as where extremely stringent stability requirements may attain.

50. In practice, the principal components of the strength behaviour of a rock mass are therefore determined by means of laboratory-scale specimens; then strength parameters of the in situ discontinuity are predicted by using semi-empirical equations.



(a)

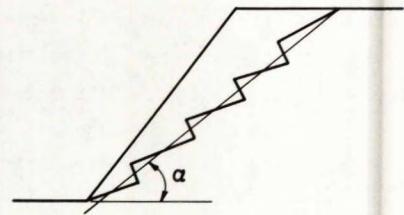


(b)

Fig 4 - Single sliding plane: (a) surface of instability after sliding, (b) simplified cross section.



(a)



(b)

Fig 5 - Series of short sliding planes with connecting cross joints: (a) within the wall section shown in the centre, movement occurred along the short sliding planes when the material bridges failed, (b) schematic cross section of the instability mode.

Table 2: Example of a test program

Required mechanical property	Priority	Testing method	Test result to be used for:	Sampling source and method	Remarks
Shear strength of α_3 joint system, Sector B		laboratory direct shear on irregular specimen	joint system α_1 of Sector A; joint system α_3 of Sector B	block samples from face	complete testing
	1	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 tests 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	
		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 tests 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 tests 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 testing
etc			etc		etc

Laboratory Tests

51. Two kinds of tests are available to establish the shear strength parameters.

52. The triaxial test described in Supplement 3-2 para 64 to 72, can be used to establish the shear strengths at, and in the close vicinity of, their peak values. The limited displacement available to this method is not usually enough however to establish the residual values of the shear strengths, which are important in stability analyses. This problem can be overcome if the triaxial tests are supplemented by the direct shear test described in Supplement 3-2 para 46 to 54, as conducted on saw-cut planes. To use the triaxial test method, accurately prepared cylindrical specimens are necessary, which might be too difficult to obtain in some cases.

53. It is more common to use the direct shear test detailed in Supplement 3-2 para 55 to 63. This method is very flexible in its application.

54. In every case, basically the same direct shear test method is used, despite which there are some specific test requirements or conditions that need to be considered and followed in particular situations. These will be discussed later, but the underlying mechanism of the direct shear test is briefly presented first.

55. During the test the specimen is first

subjected to a normal force N , equivalent to the in situ normal force; the upper half of the specimen is then displaced along the plane of discontinuity at a constant rate by the application of shear force S (Fig 8). Figure 9(a) shows the resultant shear displacement versus shear force curve typical for an interlocking rough shear surface of a hard rock, tested with a substantial normal force.

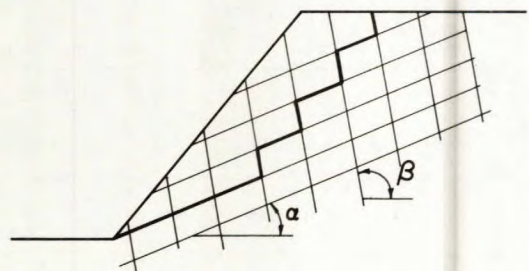
56. The shapes of these curves will be varied, since they are much influenced by such factors as surface roughness, applied normal force, rock quality, strength of rock, type of discontinuity, etc. The test result is reproducible however from one specimen to the next within a sample. Once experience has been obtained on the behaviour, the effects of these factors can be accounted for during the data analysis.

57. As the objective is to obtain the shear force versus displacement curve for the case being investigated, it would be redundant to show here all the possible curves. To illustrate the effects of surface roughness and normal force a few examples are shown in Fig 10.

58. As well as the shear force, the displacement normal to the shear plane should also be recorded as a function of the shear displacement. The shear displacement versus normal displacement,



(a)



(b)

Fig 6 - Multi-block instability: (a) shows a typical example of potential multi-block instability, (b) simplified cross section.



Fig 7 - Wedge sliding; 3-d-wedge instability developed when shear strengths along the weakness planes, oblique to the face, were exceeded.

shown in Fig 9(b), is important for evaluation.

59. The initial part of the shear force versus shear displacement curve of Fig 9(a) indicates an approximately linear rise to S_p , the peak value of shear resistance. At the same time, the normal displacement curve shows an initial dilatancy. As shear displacement continues, the shear resistance passes its peak value and drops rapidly. The corresponding normal displacement curve still shows dilatancy, but at a lower rate. The shear resistance corresponding to further shear displacement continues to drop, but at a lower rate until it reaches a plateau with an average value of S_r , referred to as the residual shear resistance. Meanwhile, the normal displacement curve also reaches a plateau with essentially a zero rate of dilatancy.

60. The shear displacement versus shear force curve, and the corresponding normal displacement curve obtained by testing clean hard rock fracture surfaces with interlocking asperities at a substantial normal force, can usually be separated into the phases A to D shown in Fig 9(a). These are characterized by the following underlying shear mechanism:

Phase A = elastic deformation phase with closure of fractures for full interlock

- Phase B = upward movement along the inclined planes of undulation and asperities, until the top part of the surface irregularities is crushed
- Phase C = further crushing and grinding of irregularities and of loose broken chips, with some dilatancy caused by the stable and more gentle undulations
- Phase D = essentially flat, debris-filled sliding surface condition corresponding to the residual shear resistance. Should the surface be subjected to renewed shearing in the same direction as in the original test, a shear resistance curve of the type shown in the lower graph of Fig 9(a) would result.

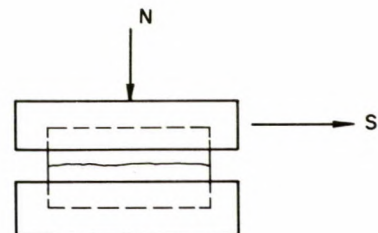


Fig 8 - Principle of direct shear test.

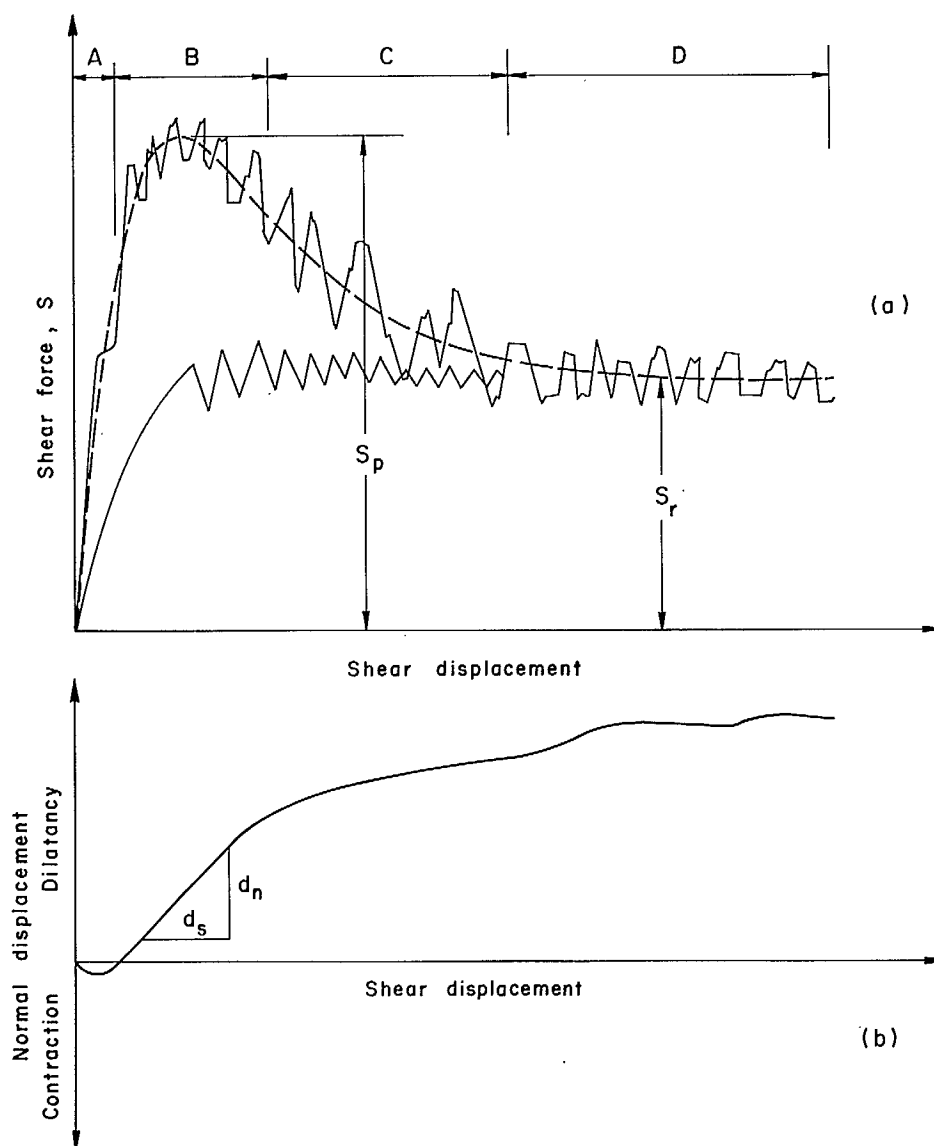


Fig 9 - Recorded graphs of a direct shear test: (a) shear force vs shear displacement, (b) normal displacement vs shear displacement.

61. As for practical purposes phases D and B corresponding to the residual and peak shear resistance phases, respectively, are very important in establishing the shear strength parameters, they will be discussed in more detail.

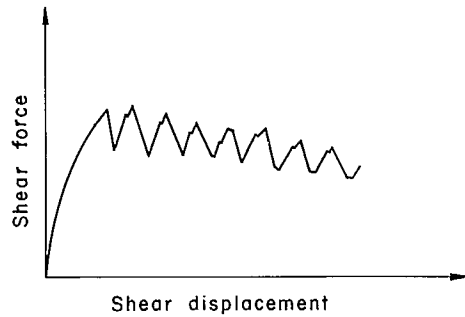
Residual Shear Resistance

62. For every type of rock surface residual shear resistance can be reached when the displacement is sufficiently large, and the gouging and debris developed by shearing has produced a

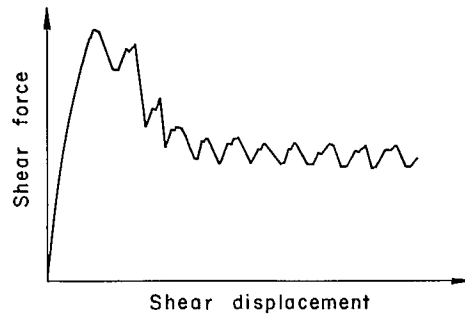
flat sliding surface. The value of the residual shear resistance varies somewhat with the original surface roughness and with the applied normal stresses. The variation for a similar type of rock is however rather small.

63. Using the method described in Supplement 3-2 para 55 to 63, a specified number of specimens are tested at normal forces within the range of the existing field forces; from these tests normal stresses σ_n , and the corresponding residual shear stresses τ_r , are calculated.

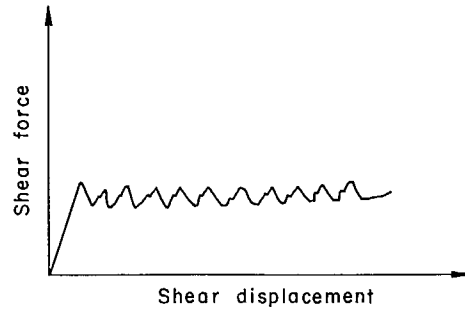
64. A plot of the normal versus residual



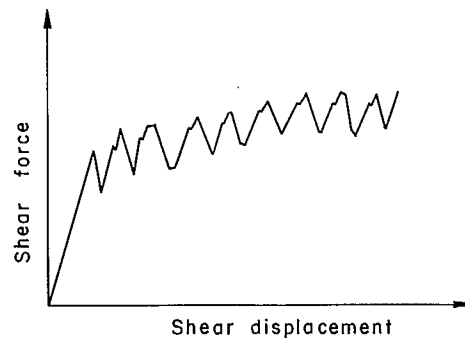
(a) Hard rock surfaces with interlocking asperities, low normal force, considerable overriding during displacement.



(b) Hard rock surfaces with interlocking asperities, high normal force; asperities are sheared during displacement.



(c) Smooth hard rock surfaces, low normal force, no surface damage during displacement.



(d) Smooth hard rock surfaces, high normal force, surface damage during displacement.

Fig 10 - Shear displacement vs shear force graphs; showing the effect of surface roughness and normal force.

shear stresses shows a straight line function with a slope angle of ϕ_r , known as the residual friction angle (Fig 11). The residual shear resistance of the discontinuity plane can then be calculated by the equation:

$$\tau_r = \sigma_n \tan \phi_r \quad \text{eq 1}$$

65. To obtain a representative residual friction angle, it is necessary that gouging occur, and debris be developed on the rock surface, even if flat. Due to unusual material properties and surface roughness or to limitations of the equipment used, it may occasionally happen even with considerable wear on the rock surface, that

Table 3: Residual angles of friction, ϕ_r , for various rocks

Rock	Normal stress, σ_n		ϕ_r dry	deg wet	Reference
	psi	MPa			
Andesite			28-30		
Amphibolite	14-600	0.1-4.2	32		Wallace et al (1)
Basalt	14-1200	0.1-8.3	35-38	31-36	Coulson (2)
Basalt			24		
Clay shale			10-13		
Chalk	0-60	0-0.4		30	Hutchinson (3)
Conglomerate	40-500	0.3-3.4	35		Krsmanovic (4)
Dolomite	14-1050	0.1-7.2	31-37	27-35	Coulson (2)
Gabbro			33		
Gneiss (schistose)	14-1150	0.1-8.0	26-29	23-26	Coulson (2)
Granite (fine grained)	14-1080	0.1-7.4	31-35	29-31	Coulson (2)
Granite (coarse grained)	14-1080	0.1-7.4	31-35	31-33	Coulson (2)
Limestone	0-72	0-0.5	33-39	33-36	Patton (5)
Limestone	14-1030	0.1-7.1	37-40	35-38	Coulson (2)
Limestone	14-1200	0.1-8.3	37-39	35	Coulson (2)
Monzonite			28-32		
Porphyry	0-145	0-1.1	31		Barton (6)
Porphyry	600-1930	4.1-13.3	31		Barton (6)
Quartzite			26-35		
Sandstone	0-72	0-0.5	26-35	25-33	Patton (5)
Sandstone	0-40	0-0.3		29	Ripley and Lee (7)
Sandstone	40-435	0.3-3.0	31-33		Krsmanovic (4)
Sandstone	14-1060	0.1-7.3	32-34	31-34	Coulson (2)
Shale			27-32		
Shale	0-40	0-0.3		27	Ripley and Lee (7)
Siderite	500-2000	3.4-13.8	34		Herget et al (8)
Siltstone	0-40	0-0.3		31	Ripley and Lee (7)
Siltstone	14-1060	0.1-7.3	31-33	27-31	Coulson (2)
Slate	0-160	0-1.1	25-30		Barton (6)

stable characteristics for residual shear resistance cannot be achieved by testing the natural discontinuity planes. In these cases the residual shear strength parameters can be established by using saw-cut planes, as described in para 46 to 54 of Supplement 3-2. Some typical values of residual friction angles for various rocks are given in Table 3.

Peak Shear Resistance

66. For dry surfaces the residual angle of

friction provides the minimum shear resistance available for a particular fracture. Practical experience gained from analyses of slides and of stable slopes suggests however that the safe friction angle for design analyses could be higher, possibly approaching the peak angle. Testing rough fractures at low normal stresses results considerably higher friction angle values than the residual angle of friction of the same materials. Some of the measured friction are given in Table 4.

Table 4: Friction angles of rough fractures at low normal stresses

Description of shear surface	τ/σ_n		arctan (τ/σ_n)	Reference
	psi	MPa		
Basalt; dense, breccia with clay-filled voids	64/25	0.44/0.17	69°	Ruiz et al (9)
Granite; rough, undulating, artificial extension fractures	65/22	0.45/0.15	72°	Rengers (10)
	133/50	0.92/0.35	69°	
Limestone; slightly rough bedding surface	99/23	0.68/0.16	77°	Goodman (11)
	96/30	0.66/0.21	72°	
	244/87	1.68/0.60	71°	
Limestone; rough bedding surface	99/45	0.68/0.31	66°	Goodman (11)
	300/100	2.07/0.68	72°	
Shale; closely jointed seam in limestone	7.2/3	0.05/0.02	71°	Goodman (11)
	8.7/3	0.06/0.02	70°	
Quartzite; gneiss and amphibolite, natural slopes excavated slopes			80°	Paulding (12)
			75°	

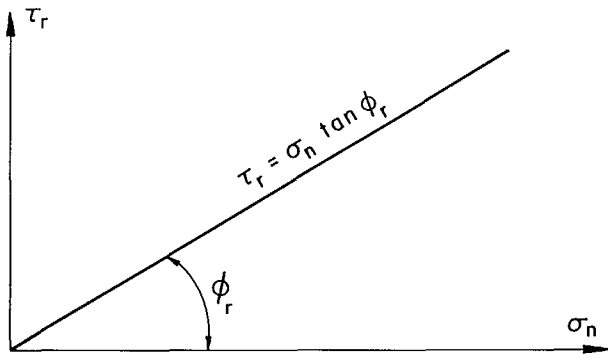


Fig 11 - Normal stress vs residual shear stress.

67. The reason for the higher friction angle values is obvious. The rock mass of the wall material can slide along a discontinuity plane only after overriding the stable undulations, crushing the sharp and interlocking asperities, and gouging the flat portions of the discontinuity surface. The in situ scale of the undulations and interlocking asperities is larger than that exhibited by the laboratory test specimen. Test results can however be used with dilatancy angles determined in the field.

68. To slide a rock mass along a rough or wavy surface, it is necessary to overcome the frictional resistance and to push the material

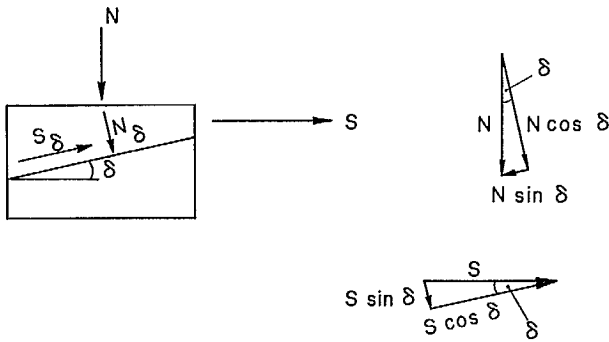


Fig 12 - Shearing along an inclined plane.

uphill, either against gravity alone for the in situ case, or against a normal force for the test case.

69. The shear force necessary to move the material along a flat surface is:

$$S = N \tan \phi$$

where, N = normal force

$\tan \phi$ = coefficient of friction.

70. If the surface is inclined at an angle δ , the shear force component S_δ parallel to, and the normal force component N_δ at a right angle to the inclined surface, from Fig 12 are:

$$S_\delta = S \cos \delta - N \sin \delta$$

$$N_\delta = N \cos \delta + S \sin \delta$$

Since the coefficient of friction on the sliding surface is $\tan \phi$, it follows that:

$$S_\delta = N_\delta \tan \phi$$

and on substitution the following is obtained:

$$S = N \tan (\phi + \delta)$$

Written in terms of stresses the expression becomes:

$$\tau = \sigma_n \tan (\phi + \delta) \quad \text{eq 2}$$

where τ and σ_n are the shear and the normal stress, respectively, acting on the rough or wavy surface.

71. The dilatancy due to the inclination can only be fully realized if the prevailing stress conditions allow for sliding along the surfaces, and if the material strength in the asperities is high enough to transmit the normal force acting on the block of rock without failure.

72. If this is not the case, then shearing of asperities occurs, and in addition to surface friction, cohesion appears. In this case the shear stress can be defined by the following equation:

$$\tau = c_0 + \sigma_n \tan \phi_0 \quad \text{eq 2a}$$

where c_0 is the cohesion as given by the intercept of the peak strength envelope which has a slope angle ϕ_0 .

73. The combined mechanism, demonstrated by testing model specimens with regular undulations at a constant friction angle results in a bilinear peak strength envelope as shown in Fig 13 (5).

74. A similar failure mechanism takes place when natural discontinuities are tested. The resulting curved peak failure envelope shown in Fig 14 was obtained by testing natural discontinuities at various normal stresses within the range of in situ values. It reflects the complexity of the actual surface profile. There is no sharp break in the failure envelope as in the case of the regular undulations of the model test of Fig 13. The curvature of the failure envelope is a clear indication of general decrease in the τ/σ_n ratio with an increase in normal stress.

75. Using the normal displacement versus shear displacement curve of Fig 9(b), the dilatancy angle d and basic friction angle ϕ_b , are calculated by the method given in Supplement 3-2 para 59. These are considered to be the true controlling parameters of the peak shear strength of the discontinuity under laboratory conditions.

76. The value of the basic friction angle, ϕ_b , is usually somewhat larger than the residual friction angle ϕ_r . The difference is negligible however and for practical purposes ϕ_b and ϕ_r are

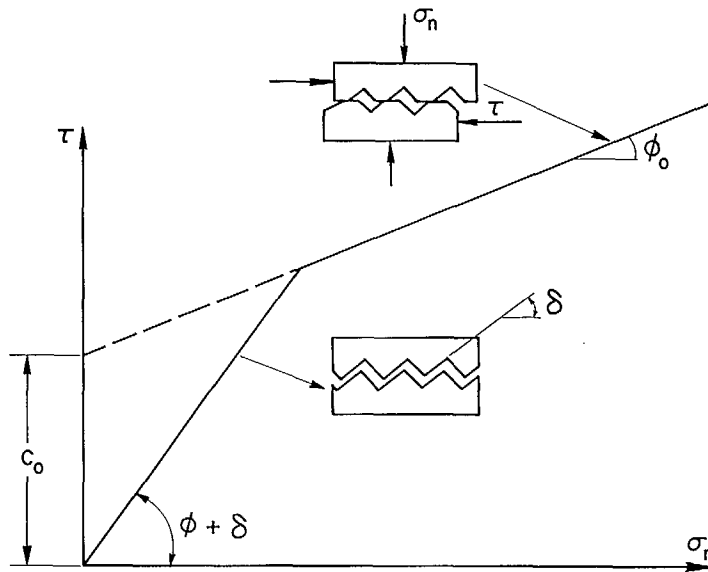


Fig 13 - Bilinear peak strength envelope.

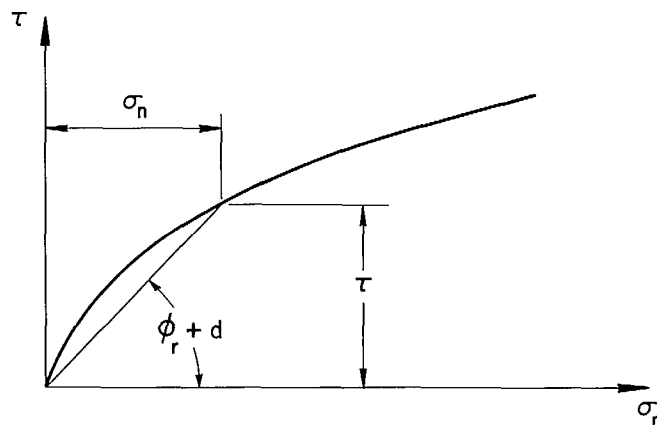


Fig 14 - Peak failure envelope of natural fracture planes.

interchangeable. Further, the basic friction angle can be determined only if the normal displacement is accurately recorded, and this adds some complexity to the testing procedure.

77. On theoretical considerations on controlled model testing and on field measurements, the mathematical representation of the curved line envelope Fig 14 has been worked out by several investigators. These empirical, but sufficiently accurate for practical purposes, expressions clearly demonstrate that a connection exists between the peak shear strength, the residual friction angle, the effect of dilatancy

and the compressive strength of the substance (6)(13)(14).

78. If in eq 2 the geometrical dilatancy δ is replaced by the more representative dilatancy angle d , the value of which depends on the normal stress σ_n , and if the friction angle ϕ is replaced by the basic friction angle ϕ_b , the following shear strength equation describes the curved line envelope:

$$\tau = \sigma_n \tan (\phi_b + d) \quad \text{eq 3}$$

If the basic friction angle ϕ_b is replaced by the

residual friction angle ϕ_r the equation becomes:

$$\tau = \sigma_n \tan (\phi_r + d) \quad \text{eq 4}$$

79. The value of d varies with the normal stress σ_n . One equation which can be used to express the relationship between the normal stress and the dilatancy angle is:

$$\tan d = \tan d_0 [1 - (\sigma_n / Q_u)^{\frac{1}{2}}] \quad \text{eq 5}$$

where d_0 is the average dilatancy angle before shear ie, at $\sigma_n = 0$, and Q_u is the uniaxial compressive strength of the rock substance (14).

80. To evaluate this empirical equation the compressive strength of the rock substance is needed. The testing procedure detailed in Supplement 3-2 para 38 to 45 is used, which results a shear strength envelope of the rock substance.

81. The relationship between the residual shear and peak shear resistances of the discontinuity, and the shear strength of the rock substance is shown in Fig 15. The top curved line represents the shear strength of the rock substance the middle curved line represents the peak shear strength in case of rough fractures; and the lower straight line represents the residual shear strength of the discontinuity. The three strength envelopes may intersect at a single point T, the transition pressure point. For

practical purposes, the transition pressure can be taken as equal to the uniaxial compressive strength of the rock substance Q_u established by the method detailed in Supplement 3-1, para 47 to 55.

82. The purpose of the foregoing discussion was to provide a brief description of the mechanism and principles concerned in the testing and interpretation procedures. The immediate goal is to furnish the background necessary to understand the relationships between the various strength parameters and their effects on the strength of the rock mass.

83. In practice, the application of this knowledge is rather simple. For example, the following practical conclusions can be drawn from the relationships presented in Fig 15:

- a. If the normal stress σ_n equals Q_u , the uniaxial compressive strength of the rock, then the failure mechanism becomes independent of any existing fracture surfaces, and will depend only on the compressive strength of the rock mass. Considering the dimensions of open pits in hard rocks and the uniaxial strengths of hard rocks, the chance of reaching this condition is however remote.
- b. Most often the normal stresses do not exceed $0.1 Q_u$ for pits with the usual dimensions. At this low normal stress value, $\sigma_n \leq 0.1 Q_u$, the shear strength of the rock mass is almost independent of that of the rock substance;

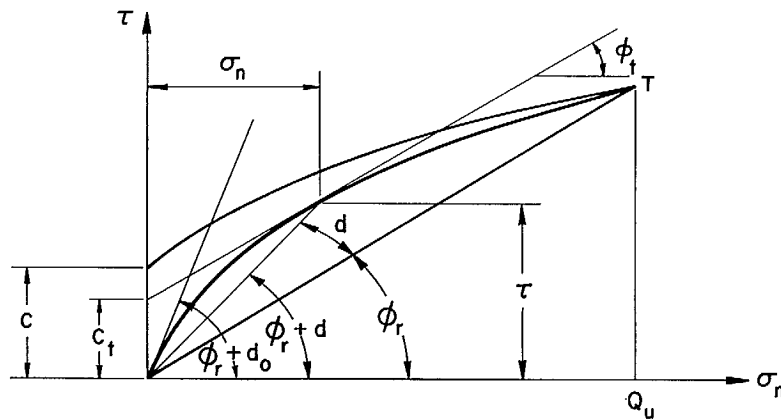


Fig 15 - Relationship between residual shear resistance of discontinuity, peak shear resistance of discontinuity, and shear strength of rock substance.



Fig 16 - Rotational shear; within the altered weak rock mass shown in the foreground, instability developed along a circular slip plane.



Fig 17 - Combination of rotational and plane slides; shear strength of substance and the weakness plane are the test requirements.

essentially, it depends only on the dilatancy angle at zero normal stress d_0 , and on the residual friction angle ϕ_r . Parameter d_0 was defined previously; it is the average angle of the in situ undulations and waviness, and is established by methods given in the Structural Geology chapter. The apparent friction angle to be used in slope analyses in this case is: $\phi = \phi_r + d_0$.

- c. For the range of normal stresses, where $0.1 \leq \sigma_n/Q_u < 1.0$, the strength of the rock mass depends both on the strength of the rock substance and on that of the discontinuity. Consequently, in this case the apparent friction angle to be used in slope analyses is: $\phi = \phi_r + d$. For this range of normal stresses it may be convenient to define the shear resistance for a given normal load by an apparent cohesion value of c_t and tangent friction value of ϕ_t , instead of using a curved failure envelope. This can be obtained by drawing a tangent, to the curved peak resistance envelope of Fig 15, at the selected effective normal stress σ_n .

84. The above description of peak shear resistance concerns rough discontinuities which have not been subjected to previous shear movement. If this is not the case, a reduction of the friction angle can be expected due to a decrease in d caused by the previous shear movement. The shear resistance then varies between the peak value and the residual value ie, $d = 0$.

85. Discontinuities with coated surface require very special attention, especially in that the in situ normal stresses are usually less than $0.1 Q_u$. The glossy surfaces of the majority of coatings are little damaged by shearing, because of the low normal stress. Friction angles obtained by testing specimens with coated discontinuities at low normal stresses are usually considerably lower than the residual friction angle ϕ_r . In the expression $\phi = \phi_r + d$ the residual friction angle ϕ_r must consequently be replaced by ϕ_c , the friction angle of the coating. The same precaution is taken in case of relatively smooth joints, $d_0 < 15^\circ$, coated with clayish

material generally called clay-greased joints.

86. Discontinuities with gouge are divided into two major groups:

- thick gouge, when the ratio between the thickness of the gouge t and the amplitude of irregularity a is equal to or greater than 1.5 ie, $t/a \geq 1.5$,
- thin gouge, when $t/a < 1.5$.

87. Testing procedures for the case of a critically oriented discontinuity with thick gouge will be defined in para 116 and 117.

88. The problem is more complex in case of thin gouge. The strength behaviour of such joints depends on several interacting factors such as the value of angle d_0 , the normal stress σ_n , the uniaxial compression strength Q_u of the rock substance, and the nature of the gouge material.

89. On the basis of test results obtained with models, two methods have been developed to predict the shear strength of joints with thin gouge. It can be said that:

- use Method 1 for any type of gouge material if $d_0 < 15^\circ$, or if $\sigma_n/Q_u < 0.1$;
- use Method 1 for a clay infilling material, if $15^\circ \leq d_0 \leq 30^\circ$, or if $\sigma_n/Q_u < 0.5$;
- use Method 2 for a clay infilling material, if $d_0 > 30^\circ$, or if $0.5 \leq \sigma_n/Q_u < 1.0$;
- use Method 2 for a cohesionless infilling material, such as sand or broken host-rock, if $d_0 > 15^\circ$, or if $0.1 \leq \sigma_n/Q_u < 1.0$.

90. In Method 1 the shear strength of filled joint is calculated as:

$$\tau = C + \sigma_n \tan \phi \quad \text{eq 6}$$

where C is the apparent cohesion and ϕ is the apparent friction angle. The value of C is defined as:

$$C = \frac{c_g}{1 - \tan d \tan \phi_g} \quad \text{eq 7}$$

where $\tan d = m \tan d_0$, c_g is the cohesion of the gouge material, ϕ_g is the friction angle of the gouge material, and d_0 is the dilatancy angle at $\sigma_n = 0$. The reduction factor m is calculated from the following empirical expression:

Table 5: Parameters and testing procedures

Symbol	Parameters Definitions	Testing procedures to obtain parameters
ϕ_r	residual friction angle	Paragraphs 46 to 54, Supplement 3-2 or Paragraphs 55 to 63, Supplement 3-2
ϕ_c	friction angle of coated joint	Paragraphs 55 to 63, Supplement 3-2
ϕ_g	friction angle of gouge material	<u>Soil-like material</u> Paragraphs 24 to 34, Supplement 3-4 <u>Crushed rock material:</u> Paragraphs 73 to 90, Supplement 3-2
d_0	dilatancy angle at zero normal stress	Structural Geology Chapter
d	dilatancy angle	Paragraphs 55 to 63, Supplement 3-2
c_g	cohesion of gouge material	<u>Soil-like material:</u> Paragraphs 24 to 34, Supplement 3-4 <u>Crushed rock material:</u> Paragraphs 73 to 90, Supplement 3-2
Q_u	uniaxial compressive strength	Paragraphs 47 to 55, Supplement 3-1
a	amplitude of in situ surface roughness	Structural Geology Chapter
t	thickness of gouge	Structural Geology Chapter

$$m = (1 - 2/3 t/a)^2 \quad \text{eq 8}$$

where t is the thickness of infilling, and a is the amplitude of the irregularity. The value of ϕ is defined as follows:

$$\phi = \phi_g + d \quad \text{eq 9}$$

where ϕ_g is the friction angle of the gouge material, and $d = \tan^{-1} (m \tan d_0)$.

91. In Method 2 the shear strength of a filled joint is again expressed by eq 6, but the values of the apparent cohesion C , and that of the apparent friction angle ϕ , are now defined as:

$$C = (1 - m) c_g \quad \text{eq 10}$$

$$\phi = \tan^{-1} [m \tan (\phi_r + d) + (1-m) \tan \phi_g] \quad \text{eq 11}$$

where m is the reduction factor of eq 8, c_g is the

cohesion of the gouge material, ϕ_r is the residual friction angle of the clean empty joint, and ϕ_g is the friction angle of the gouge material.

92. The parameters used for calculating the shear resistance, and their appropriate test procedures, are listed in Table 5.

93. The shear strength equations to be used in calculating the shear resistance of various types of discontinuities are summarized in Table 6.

94. Some approximate friction angle values for various infilling materials and coatings are given in Table 7. Representative friction angle values for sands and silts are given in Table 8 (19).

SHEAR STRENGTH OF SUBSTANCE

95. If the discontinuities are not critically oriented but the rock substance is weak, instability may occur through shearing of the intact material.

96. For example, within the altered weak rock mass shown in the foreground of Fig 16, instability developed through rotational shear along a circular slip plane.

97. Similarly, rotational movement together with some plane shear took place within the wall sector shown in Fig 17. In this case the shear strength of substance, as well as the shear strength along the plane of weakness, represent the test requirements.

98. Soil overburden, highly fractured rock, or a critically oriented discontinuity with thick gouge are other cases in which the shear strengths of such substances are dominant in the stability analysis.

99. The relevant testing procedures to determine the strength parameters are detailed in Supplements 3-2, 3-3 and 3-4. In general, two basic approaches can be followed:

- a. in situ tests;
- b. laboratory tests.

In Situ Tests

100. The in situ shear strength test of intact large-scale rock samples is detailed in Supplement 3-3, para 11 to 23. Due to the size of the samples this testing method can provide the most reliable strength parameters if an adequate number

of tests are performed. Because of the high cost of such tests, the in situ direct shear tests are used only in special cases, where the stability requirements justify them. It is therefore more common to determine the shear strength of the rock substance in the laboratory by testing an adequate number of representative specimens.

101. The method to be used for in situ shear strength determination of the overburden is described in Supplement 3-4, para 35 to 38.

102. This commonly used method has the advantages of being inexpensive and simple. It provides only a partial answer to the problem however and does not make allowance for investigating the effects of various factors on the shear strength. If this information is essential for stability analyses, laboratory tests will be required instead.

Laboratory Tests

103. To establish shear strength parameters, triaxial compressive tests are performed on representative specimens. To suit prevailing in situ conditions, specific requirements, or for reasons of inherent sampling difficulties the test set-up and procedures are varied from case to case. The procedure most appropriate for each typical case is given in para 108 to 117. The basic principle of triaxial compression testing is discussed briefly here.

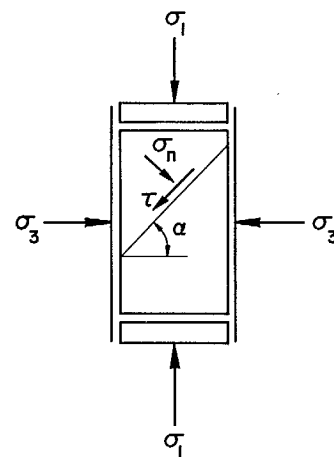


Fig 18 - Principle of triaxial testing.

Table 6: Shear strength parameters and equations used for various types of discontinuities

Weakness plane		In-filling material	Range of application	Required parameters	Shear strength equation used:
Empty	Clean, rough	no filling	$\sigma_n / Q_u \leq 0.1$	ϕ_r, d_o, Q_u	$\tau = \sigma_n \tan \Phi$, where $\Phi = \phi_r + d_o$
			$0.1 < \sigma_n / Q_u < 1.0$	ϕ_r, d, Q_u	$\tau = \sigma_n \tan \Phi$, where $\Phi = \phi_r + d$
	Coated	no filling	$\sigma_n / Q_u \leq 0.1$	ϕ_r, ϕ_c, d_o, Q_u	$\tau = \sigma_n \tan \Phi$ where, if $\phi_r > \phi_c$ $\Phi = \phi_c + d_o$ if $\phi_c > \phi_r$ $\Phi = \phi_r + d_o$
			$0.1 < \sigma_n / Q_u < 1.0$	ϕ_r, ϕ_c, d, Q_u	$\tau = \sigma_n \tan \Phi$ where, if $\phi_r > \phi_c$ $\Phi = \phi_c + d$ if $\phi_c > \phi_r$ $\Phi = \phi_r + d$
Filled	Thick filling $t/o \geq 1.5$	To be treated as a layer of weak rock, see Paragraphs 98, 116 and 117			
	Thin filling $t/o < 1.5$	ony in-filling	$d_o < 15^\circ$ and/or $\sigma_n / Q_u \leq 0.1$	$c_g, \phi_g, d_o, t, o, Q_u$	$\tau = C + \sigma_n \tan \Phi$ where, $C = c_g / (1 - \tan d \tan \phi_g)$; $\Phi = \phi_g + d$ $\tan d = m \tan d_o$; $m = (1 - 2/3 t/o)^2$
		clay	$15^\circ \leq d_o \leq 30^\circ$ and/or $\sigma_n / Q_u < 0.5$	$c_g, \phi_g, d_o, t, o, Q_u$	$\tau = C + \sigma_n \tan \Phi$ where, $C = c_g / (1 - \tan d \tan \phi_g)$; $\Phi = \phi_g + d$ $\tan d = m \tan d_o$; $m = (1 - 2/3 t/o)^2$
		clay	$d_o > 30^\circ$ and/or $0.5 \leq \sigma_n / Q_u < 1.0$	$c_g, \phi_g, \phi_r, d, t, o, Q_u$	$\tau = C + \sigma_n \tan \Phi$ where, $C = (1 - m) c_g$; $m = (1 - 2/3 t/o)^2$; $\Phi = \tan^{-1} [m \tan (\phi_r + d) + (1 - m) \tan \phi_g]$
		frictional (cohesionless) e.g. sand; broken host-rock	$d_o > 15^\circ$ and/or $0.1 < \sigma_n / Q_u < 1.0$	$c_g, \phi_g, \phi_r, d, t, o, Q_u$	$\tau = C + \sigma_n \tan \Phi$ where, $C = (1 - m) c_g$; $m = (1 - 2/3 t/o)^2$; $\Phi = \tan^{-1} [m \tan (\phi_r + d) + (1 - m) \tan \phi_g]$

104. As suggested in Fig 18 the specimen is enclosed in an airtight flexible membrane; σ_3 lateral pressure, of a value within the range of calculated in situ horizontal pressure, is applied and held constant during the test by means of a cell fluid. Axial stress, σ_1 , is then applied and continuously increased until failure occurs. The test is repeated at various lateral pressures to cover the entire range of horizontal stresses expected within the slope being investigated.

105. The resultant Mohr's shear strength envelope, drawn tangent to the Mohr's circles, is either a straight line as shown in Fig 19, or is more often a curved line as shown in Fig 20.

106. In the case of a straight-line envelope,

evaluation of the test results is simple because the same shear strength parameters ie, apparent cohesion C and apparent friction angle ϕ , apply over the total range of the normal stress ϕ_n . The shear stress is then calculated from the following equation:

$$\tau = C + \sigma_n \tan \phi \quad \text{eq 12}$$

107. It is somewhat more difficult to evaluate the test results if the strength envelope is a curved line, because in this case each pair of strength parameters, such as C_1 and ϕ_1 , C_2 and ϕ_2 , etc, applies only to its particular portion of the curved envelope that can be safely replaced by a

Table 7: Approximate friction angles for infilling materials and coatings

Material	Normal stress, σ_n		Angle of friction		References
	psi	MPa	dry	wet	
Clay gouge (remolded)			10 - 20	0 - 10	Hoek (15)
Calcite shear zone material			20 - 27		Hoek (15)
Shale fault material			14 - 22		Hoek (15)
Hard rock breccia			22 - 30		Hoek (15)
Hard rock fill			38		Hoek (15)
Molybdenum-coated hard rock shear surface			15 - 25		
Glossy extremely smooth graphite-coated shear surface	1.5	0.01	13		Herget (16)
Chlorite-coated hard rock shear surface	1000 - 20,000	7 - 140	24		Herget (17)
Smooth graphite-coated shear surface	500 - 5800	3.5 - 40	18 - 22		Jaeger and Rosengren (18)

tangent without excessive error.

Testing Requirements Suited to Specific Cases

108. For designing soil slopes, the strength parameters must be in terms of the effective or intergranular stresses. These can be obtained by the so-called drained triaxial test described in Supplement 3-4, para 24 to 34. The duration of the drained triaxial test depends on the drainage characteristics of the soil; in some cases it takes days. Alternatively, the faster but more complex undrained test with pore pressure measurement can be used, as described in Supplement 3-4, para 24 to 34.

109. Obtaining drill core specimens from highly porous weak rocks usually presents no problems; consequently, the triaxial method specified in Supplement 3-2, para 38 to 45 can be used. Strictly speaking, the results obtained by this method are in terms of total stresses, and not in those of effective stresses as required by the design analyses. If the rock material has a high porosity and low water content, the results obtained can be taken however as intergranular or effective parameters. In case of doubt, as when the water content is at or near saturation, testing methods specified in Supplement 3-2, para

Table 8: Representative friction angle value for sands and silts

Material	ϕ , deg.	
	loose	dense
Sand, round grains, uniform	27.5	34
Sand, angular grains, well graded	33	45
Sandy gravels	35	50
Silty sand	27-33	30-34
Inorganic silt	27-30	30-35

73 to 90, can be adopted.

110. Heavily altered or weathered rock formations present specimen preparation problems. In these cases it is useless to test core specimens obtained from the more competent parts of the mass, since stability of the mass is controlled by that portion which is usually impossible to sample by core drilling. A suitable testing method is described in Supplement 3-2, para 73 to 90, that uses recompacted specimens.

111. A problem similar to the foregoing is represented by thick sheared contact zones; in this case the method of testing is also similar.

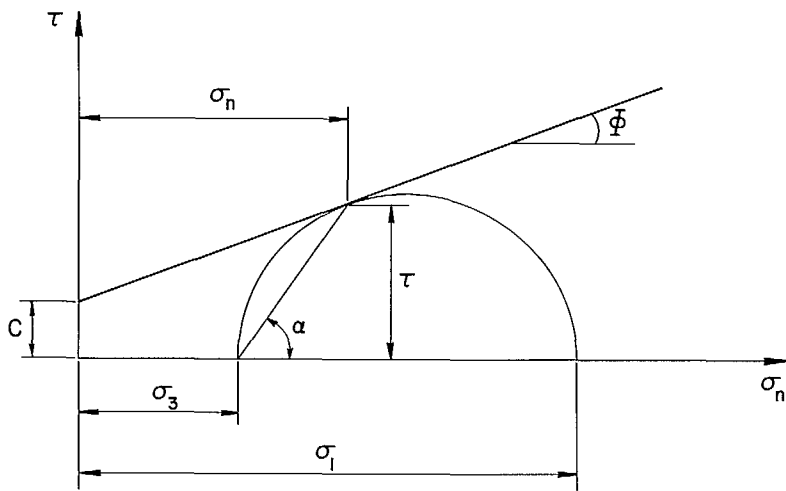


Fig 19 - Mohr's circle with straight line envelope.

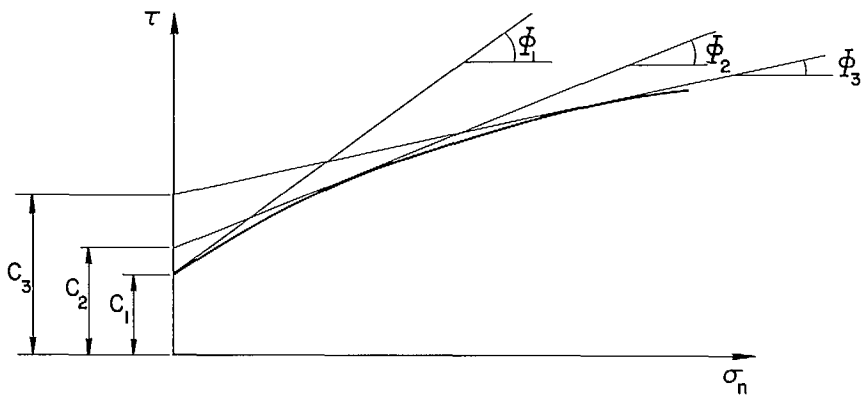


Fig 20 - Curved line Mohr's envelope; showing the variation of the strength parameters.

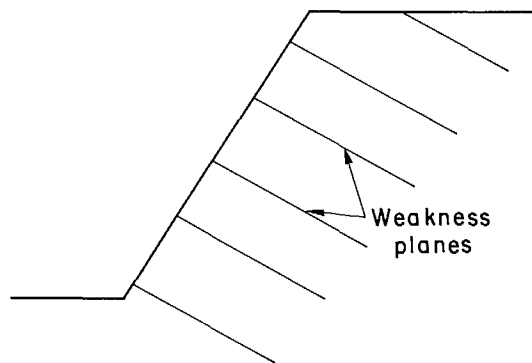


Fig 21 - Weak rock formation with non-critically oriented weakness planes.

112. A typical case of a weak rock formation with non-critically oriented weakness planes is shown in Fig 21. The entire slope is within a weak rock formation which contains discontinuity planes of low shear strength dipping away from the pit. The stability of the slope is therefore controlled only by the strength of the rock itself; the planes of weakness play but a minor role.

113. The appropriate triaxial testing method depends on the in situ condition of the rock involved. If it is highly fractured or weathered, the method described in Supplement 3-2, para 73 to 90, should be used; if it is competent but inherently weak that of para 38 to 45 is appropriate.

114. In Fig 22 the slope is situated within a weak anisotropic rock formation, with cemented layers dipping toward the pit. Depending on the relative strength of the material in a direction across or along the bedding planes, the mode of instability is either circular slip, or circular slip modified by the bedding planes.

115. Shear strength parameters are established separately for the above directions by the triaxial methods described in Supplement 3-2, para 38 to 45, using two sets of specimens. One set is

prepared with the specimen axis coincident with the bedding planes, and the other so that the axis forms an angle between 25° and 40° with the bedding.

116. If the ratio of the thickness of gouge to the amplitude of irregularity of a critically oriented plane of weakness is greater than about 1.5 ie, if $t/a \geq 1.5$, the stability of the slope depends entirely on the shear strength of the gouge material. The strength parameters of the infilling material are thus required for stability. The testing requirements for those cases where $t/a < 1.5$ were discussed in para 88 to 93.

117. The appropriate testing method for a thick gouge depends on its type. If it consists of crushed host rock, recompacted specimens are tested by the method described in Supplement 3-2, para 73 to 90. If the gouge material is soil-like however, the testing methods described in Supplement 3-4, para 24 to 34 are appropriate. Since it is usually difficult to obtain an undisturbed specimen from gouge material, recompacted specimens are often used. The compaction procedure used for fine grained material is described in Supplement 3-4, para 17 to 23.

Table 9: Porosity and unit weight of typical soils in natural state

Material	Porosity n%	Water content when satu- rated, w%	Unit weight			
			lb/ft ³		g/cm ³	
			γ_d	γ_{sat}	γ_d	γ_{sat}
Uniform sand, loose	46	32	90	118	1.43	1.89
Uniform sand, dense	34	19	109	130	1.75	2.09
Mixed-grained sand, loose	40	25	99	124	1.59	1.99
Mixed-grained sand, dense	30	16	116	135	1.86	2.16
Windblown silt (loose)	50	21	85	116	1.36	1.86
Glacial till, very mixed-grained	20	9	132	145	2.12	2.32
Soft glacial clay	55	45	76	110	1.22	1.77
Stiff glacial clay	37	22	106	129	1.70	2.07
Soft slightly organic clay	66	70	58	98	0.93	1.58
Soft very organic clay	75	110	43	89	0.68	1.43

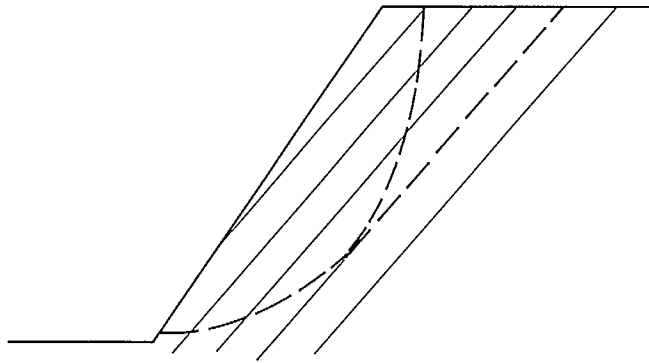


Fig 22 - Slope is within a weak anisotropic rock formation; instability mode is either circular slip or circular slip modified by the bedding planes.

COMPRESSIVE STRENGTH OF ROCK MASS

118. In a homogeneous rock formation the stress trajectories are deflected around the toe of the slope, and this results in a concentration of stresses at the toe. With large horizontal tectonic forces, the concentration can be significant.

119. Due to the stress concentration, blocks situated at the toe may be crushed. After such local crushing, the stresses are transferred to adjacent blocks which in turn are subjected to excessive loads, leading to further crushing. The progressive breakdown could continue until the entire rock mass becomes unstable.

120. Crushing and continuous breakdown of a weak layer within the toe area as shown for example in Fig 23, could lead to stability problems within the entire wall, even if the overlying formation consists of competent hard rock.

121. The stability of wall sectors included in this group of problems depends on the compressive strength of the rock mass. The appropriate testing procedures required to determine the compressive strength of a rock substance, and the methods for estimating the in situ compressive strength, are discussed in this section.

Testing Procedures

122. The compressive strength of a rock substance is established in the laboratory by the testing methods described in Supplement 3-1. The following two can be used:

- a. uniaxial compressive test
- b. point load test.

123. The uniaxial compressive test is detailed in Supplement 3-1, paras 47 to 55. Representative cylindrical specimens are subjected to continuously increasing axial loads until the specimen fails (Fig 24). From the maximum load carried by the specimen the uniaxial compressive strength, Q_u , is calculated. A disadvantage of this is that it requires accurately prepared cylindrical core specimens, with relatively high preparation costs.

124. Less expensive is the point load test, detailed in Supplement 3-1, para 56 to 66. This is not as accurate as the uniaxial compressive test; due to its simplicity and low cost, however, the number of specimens tested can be increased, and the coverage improved accordingly. Very little specimen preparation is required and either drill cores or irregular lump specimens can be used.

125. The result of a point load test, I_{sc} , is defined in terms of either psi or Pa, as the corrected point load strength index. The relationship between this index and the uniaxial compressive strength, Q_u , can be expressed for Canadian rocks by the following experimental equation:

$$Q_u = 18 I_{sc} \quad \text{eq 13}$$

The relationship is shown in Fig 25 in graphic

form (20).

Estimation of the Compressive Strength of a Rock Mass

126. It is known that the strength of a rock specimen tends to decrease as its size increases. This can be due to the greater number of large flaws present in the larger specimen. The estimation of the compressive strength of an in situ block bounded by discontinuities therefore needs to include an appropriate reduction factor.

127. The relationship between size and strength is given by the following experimental equation:

$$Q_B = Q_U (B_B/B_O)^{-b} \quad \text{eq 14}$$

where Q_B is the uniaxial compressive strength of a rock block having a base length of B_B , Q_U is the uniaxial compressive strength of the representative specimens as established in the laboratory, B_O is the base length of the specimens, or diameter in cases of cylindrical specimens, and b is an exponent whose value depends on the particu-

lar rock.

128. To determine the value of this exponent for a particular rock, tests are required on specimens of varying sizes. When an extensive testing program is not warranted, it is recommended that the value of $0.17 < b < 0.07$ be used. If the rock is very brittle, such as quartzite, the appropriate value is 0.17; if the rock is more plastic, such as limestone, the value 0.07 is used.

129. The relationship expressed by eq 14 is shown in graphic form in Fig 26 for both limiting values of exponent b .

MECHANICAL PROPERTIES OTHER THAN SHEAR AND COMPRESSIVE STRENGTH

130. As mentioned before, other mechanical properties also enter slope stability analyses, either directly or indirectly. Some are always required for every slope design problem, while others are necessary only under special conditions.

131. The testing procedures used for determin-

Table 10: Relationship between the consistency of cohesive soils and the approximate undrained shear strength

Consistency	Field identification	Approximate undrained shear strength	
		lb/ft ²	kPa
Very soft	Easily penetrated several inches by fist	< 250	< 12
Soft	Easily penetrated several inches by thumb	250-500	12-24
Firm	Can be penetrated several inches by thumb with moderate effort	500-1000	24-48
Stiff	Readily indented by the thumb but penetrated only with great effort	1000-2000	48-96
Very stiff	Readily indented by the thumbnail	2000-4000	96-192
Hard	Indented with difficulty by thumbnail	> 4000	> 192

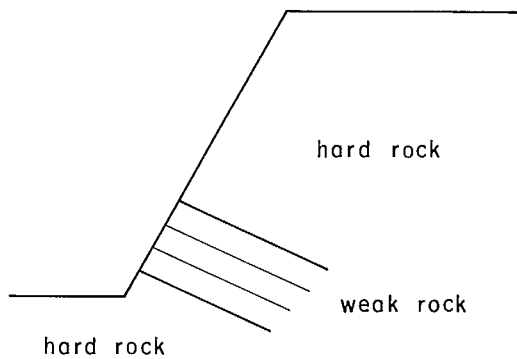


Fig 23 - Weak rock layer sandwiched between two hard rock formations.



Fig 24 - Principle of uniaxial testing.

ing these properties are described in Supplements 3-1, 3-2, 3-3, and 3-4.

Physical Properties

132. Physical properties such as porosity, density and water content, influence the mechanical characteristics of the rock substance. For example, the same type of rock substance has a lower strength and higher deformability when it is highly porous. Sandstones and carbonate rocks in particular display a wide range of porosities, and hence of mechanical characteristics. Igneous rocks weakened by weathering processes, have typically high porosities and are consequently highly deformable.

133. Density and porosity values are inter-related; a low-density rock is highly porous. In

some cases it is sufficient to quote porosity values alone; a complete description requires however both porosity and density values. The relationship between porosity and saturated bulk density for various sedimentary rocks is given in Fig 27 (21).

134. Presence of water within the pores of a rock further influences its mechanical characteristics. Strength and deformability properties of a rock substance are affected by its degree of saturation, through the pore water pressure that develops.

135. The weaker the material, the greater is the effect of these physical properties on the strength and deformability of the rock.

136. The physical properties in question are generally used for classifying and characterizing the rock substances. There are three main reasons the determination of these physical properties:

- a. they assist in developing a feel for the likely behaviour of the material;
- b. they may be helpful in interpreting test results;
- c. they may be helpful when comparing similar materials found either in various sectors of the same pit, or at other mining sites.

137. In addition, the density values of the materials are used directly in slope stability analyses when calculating the vertical forces or stresses acting within the slope wall. In Table 9 the density values of typical soils in their natural state are given, together with porosity and saturated water content (22). Figure 28 shows the density values of the most common rock types (21).

138. Although porosity, density and water content are mentioned most frequently, other related properties are also used on occasion. The appropriate definitions of the physical properties are:

- a. porosity, n , usually expressed as a percentage, is defined as the ratio of void volume to total or bulk volume
- b. density, γ , expressed in lb/ft^3 or kg/m^3 , is the ratio of bulk weight to total or bulk volume
- c. dry density, γ_d , expressed in lb/ft^3 or kg/m^3 ,

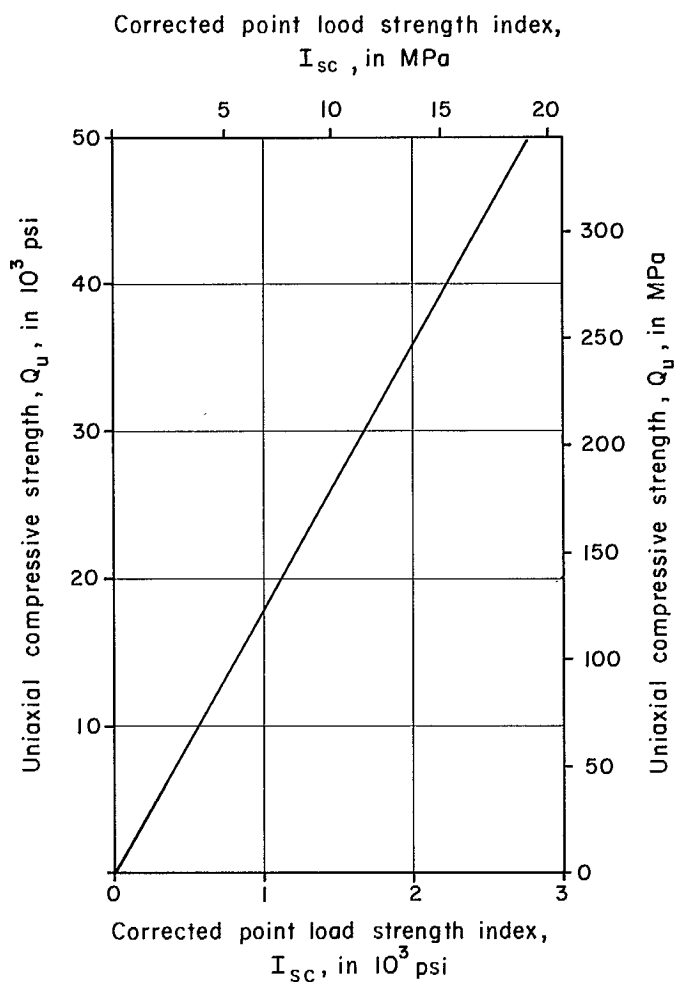


Fig 25 - Relationship between the corrected point load strength index and the uniaxial compression strength.

is the ratio of solid weight to total or bulk volume in a rock mass

- d. grain density, γ_g , expressed in lb/ft^3 or kg/m^3 , is defined as the ratio of solid weight to volume of solids in a rock mass
- e. specific gravity of a rock is defined as the ratio of grain density to the density of water
- f. water content, w , of a rock mass is defined as the ratio, usually expressed as a percentage, of the weight of water to the weight of dry solids in a rock mass; this term differs from the one usually used in assays, where $w = \frac{W_w}{(W_s + W_w)}$
- g. the degree of saturation, S_r , usually expressed as a percentage, is the ratio of the water volume in the rock voids to the total volume of

the voids.

139. The procedure for determining the porosity and density values in the laboratory is described in Supplement 3-1, para 15 to 24. This procedure also includes water content determination; so that all the related properties mentioned can also be calculated.

140. The density, γ , obtained by this laboratory method is useful for classification and characterization. When calculating the vertical stresses, it can be used only for obtaining an estimate.

141. The density of a rock mass in situ is established by field measurements. For soils and soil-like materials three equally suitable methods are available, and are described in Supplement

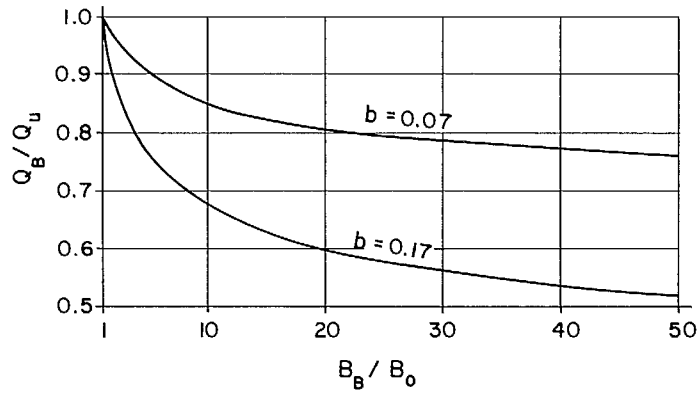


Fig 26 - Relationship between size of blocks and uniaxial compressive strength.

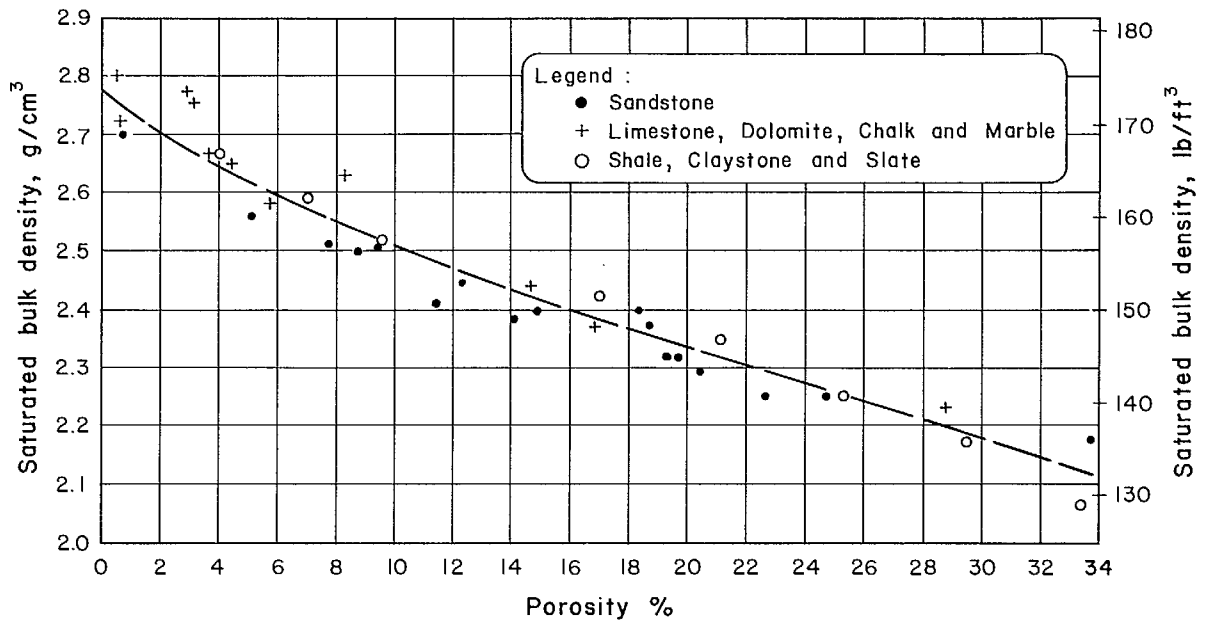


Fig 27 - Relationship between porosity and saturated bulk density of various sedimentary rocks.

3-3, para 24 to 27. The in situ density of a hard rock mass is established by the field method described in Supplement 3-3, para 28 to 32.

142. A situation can arise in which only the water content of the rock needs to be known. In this case the procedure described in Supplement 3-1, paras 8 to 14, should be followed.

143. A case typical of the foregoing arises when the effect of the water content on the shear strength of the rock substance is investigated. The triaxial tests are then always coupled with water content determination.

144. In triaxial testing of recompacted specimens, discussed in para 110 and 117, it is often necessary to establish the relationship between shear strength and recompacted density of the specimen. As the recompacted density depends

largely on the moisture content of the material being compacted, especially in case of fine material, specimen preparation must be combined with moisture-density relation tests. A description of these for fine materials is given in Supplement 3-4, para 17 to 23.

Deformation Properties

145. Deformation properties of the rock material are required for the following reasons:

- a. for use in stress analyses calculations of a slope by finite element methods
- b. for use as a tool in classifying and comparing rocks of various design sectors within the same pit, or of rocks from various pit walls
- c. as an aid in developing a feel for the likely behaviour of the material

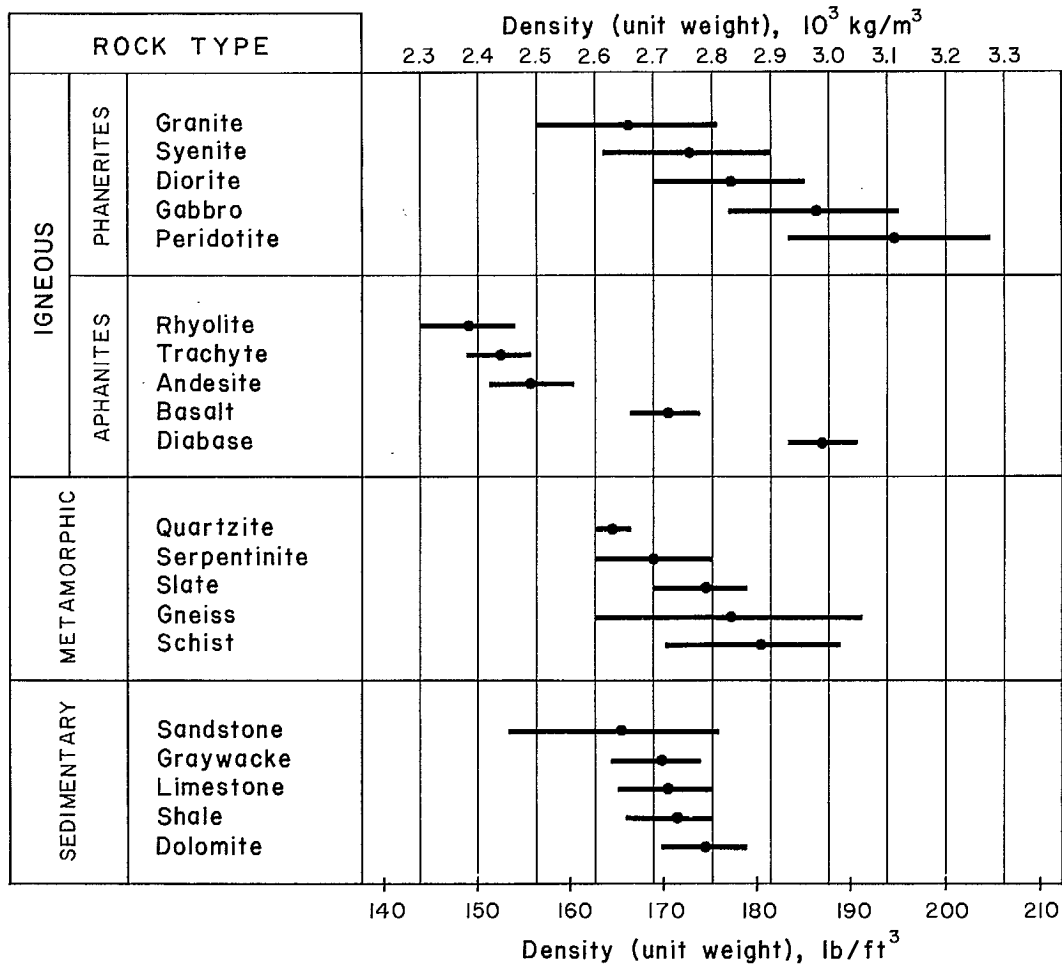


Fig 28 - Density values for the common rock types.

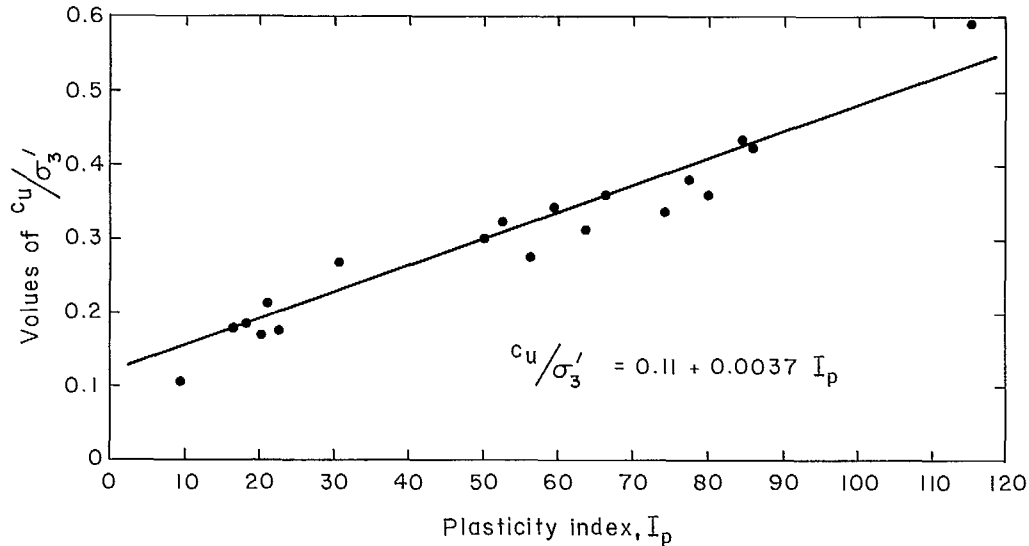


Fig 29 - Statistical relation between c_u/σ'_3 and plasticity index.

- d. as an aid in interpreting the results obtained for strength property determinations
- e. for use in designing appropriate mechanical support, or a controlled perimeter blasting system.

146. Deformation properties obtained by testing core specimens in laboratory, represent the upper limit values of the actual or in situ deformation properties of a rock mass. In the case of hard rocks, the deformation behaviour of the mass is so greatly affected by the presence of discontinuities that the deformation properties of the rock substance bear little relationship to that of the rock mass. If the substance is weak however, eg, in case of soil-like materials, laboratory results approximate more closely the in situ properties.

147. The method for determining the Young's modulus, E , is described in Supplement 3-2, para 8 to 15. The scope of this also includes the procedure required to obtain the uniaxial compressive strength, Q_u , of a cylindrical rock specimen. The testing method described in Supplement 3-1, para 47 to 55, basically designed to establish the uniaxial strength of a core

specimen, can also be used for determining Young's modulus.

148. To establish Poisson's ratio, μ , of cylindrical rock specimens, the testing procedure described in Supplement 3-2, para 16 to 23, is recommended.

149. The ultrasonic elastic constants can be determined by the testing method described in Supplement 3-2, para 24 to 37.

150. Deformation modulus, D , of a crushed rock material in a shear zone, or of a broken host-rock type of infilling material in a discontinuity plane, is determined by the test method which is part of that designed for the triaxial compressive strength or recompacted crushed rock specimens. This method is described in Supplement 3-2, para 73 to 90.

151. To obtain some measure of the in situ value of the deformation modulus, the dilatometer test performed in drill holes, is recommended. This test procedure is described in Supplement 3-3, para 7 to 10.

152. For practical purposes the tensile strength of a rock substance, σ_t , can be estimated from its uniaxial compressive strength, Q_u . The

values generally used are: $0.1Q_u$ or $0.12Q_u$. If a more accurate value is required however, the tensile strength is found by the Brazilian test as described in Supplement 3-1, para 67 to 74.

Index Properties

153. Index properties are generally used for classification and characterization. They are established for one or more of the following specific reasons:

- a. to obtain data which materially aid in the development of a feel for the likely behaviour of the material
- b. to obtain data for identifying materials with properties which could alter the stability configuration of a slope
- c. to obtain data which can either be used directly to estimate the strength properties, or can help in the interpretation of test results obtained to determine these strength properties
- d. to obtain data which can be used in comparing similar materials obtained, either from various sectors of the same pit or from other mining sites.

154. A large number of rock materials, notably those with a high clay content, are prone to swelling when exposed for even a relatively short time, to weathering by a process of alternate wetting or drying. When such materials are confined within a rock mass eg, as joint infillings, they can alter the stability conditions of the entire rock mass, due to the swelling pressure which develops with an increase in the water content. For example a swelling pressure up to 78 psi (537 kPa) has been measured for a certain type of clay infilling. The swelling ability of a material is important in designing underground structures. If material with such an undesirable property is already present in a wall sector, its effect on stability must be considered (23).

155. The swelling ability depends on the mineralogical composition as well as on the geological history of the material. Rocks with high clay contents, such as argillaceous sediments, usually exhibit the strongest tendencies to swell. There

are however several other rock types with swelling tendencies, such as pyrrhotite.

156. Among the two related indices the swelling pressure index, I_{sp} , is the more commonly used, it is determined by the method described in Supplement 3-1, para 25 to 32. The other related index is the swelling strain index, I_{ss} , established by the method described in Supplement 3-1, para 33 to 40.

157. The above indices are best used for classification and identification of the softer rock materials. For example, the swelling strain index should not be taken as the actual swelling strain that develops in situ, even under similar conditions of loading and water content. In cases of harder rocks these indices are used only if the rocks are in an advanced state of weathering.

158. Rock types that exhibit a tendency to swell are also usually prone to the effects of weakening or disintegration when exposed to processes of wetting and drying. This tendency is expressed by the slake-durability index, I_{d2} ; it is determined by the testing method described in Supplement 3-1, para 41 to 46. Rocks that disintegrate during these tests can be further characterized by using the soil consistency limit tests and the grain size distribution test.

159. Relationships between soil properties and the limits or indices of that soil can often be expressed by semi-empirical equations, such as shown in Fig 29. Using the established equations and knowing the limits, the properties of another soil can also be predicted. Since the limits are more easily determined than are compressibility, permeability, and strength, this approach sometimes proves useful (19). In the soil mechanics practice the relationships are often established between the approximate strength, the soil consistency and some easy method of identification. Such a relationship is shown in Table 10, which can be used to estimate the undrained shear strength of cohesive soils (24). The methods for establishing consistency limits and indices are described in Supplement 3-4, para 10 to 16.

160. Soil particle size and shape also have an influence upon the compressibility, permeability,

and the strength properties of the soil. The sizes of individual particles and their relative distributions, are determined by the method described in Supplement 3-4, para 4 to 9.

161. Use of the corrected point load strength index, I_{SC} , in terms of either psi or Pa, for predicting the uniaxial compressive strength of the rock substance has been discussed in para 124 and 125. This index is however more commonly used for the strength classification of rocks. The testing method for determining the corrected point load strength index is described in Supplement 3-1, para 56 to 66.

162. For easy reference, the above material properties are summarized in Table 11 which also contains references to the appropriate testing procedures.

163. A continuing effort is being expended to reduce the expenses involved in rock testing and to establish inexpensive index properties which could be used for characterizing rock masses and for predicting the behaviour of these masses. One such index property being used today is the Rock Quality Designation index, usually referred to as the RQD index (25).

164. The RQD index is defined as follows:

$$RQD \% = (L_p/L_t)100$$

where L_p = total length of the sound, unfractured, and longer than 4 in. (10 cm) core pieces in

a core run, using an NX size bit and core barrel;

L_t = total length of core run.

165. The following descriptive quality designation terms are assigned to the percentage values obtained:

RQD %	Quality designation
0-25	very poor
25-50	poor
50-75	fair
75-90	good
90-100	excellent

166. Practical experience shows that the RQD index can give a useful indication of the quality of a rock mass, provided that its use is only intended as a qualitative guide.

Time-Dependent and Plastic Properties

167. Very little attention has been paid in the past to investigation of the time-dependent deformational properties of various mine rocks. The testing procedures required for such studies are described in Supplement 3-2, para 91 to 96; the tests are lengthy, lasting for weeks, months, or even a year.

Table 11: Mechanical properties, other than shear and compression strength,
and testing procedures

Type	Property definition	Symbol	Testing procedures to obtain properties
Physical properties	porosity	n	Paragraphs 15 to 24, Supplement 3-1
	density (or unit weight)	γ	
	dry density	γ_d	
	grain density	γ_g	
	degree of saturation	S_r	Paragraphs 8 to 14, Supplement 3-1 and
	water content		Paragraphs 15 to 24, Supplement 3-1
	in situ density (soils and soil-like rocks)	γ	Paragraphs 24 to 27, Supplement 3-3
in situ density (hard rocks)	γ	Paragraphs 28 to 32, Supplement 3-3	
moisture density relation	-	Paragraphs 17 to 23, Supplement 3-4	
Deformation properties	Young's modulus	E	Paragraphs 8 to 15, Supplement 3-2 and Paragraphs 47 to 55, Supplement 3-1
	Poisson's ratio	μ	Paragraphs 16 to 23, Supplement 3-2
	Ultrasonic elastic constants:		
	Young's modulus	E_u	Paragraphs 24 to 37, Supplement 3-2
	modulus of rigidity (shear modulus)	G_u	
	Poisson's ratio	μ_u	
	deformation modulus (crushed rock)	D	Paragraphs 73 to 90, Supplement 3-2
deformation modulus (in situ)	D	Paragraphs 7 to 10, Supplement 3-3	
tensile strength	σ_t	Paragraphs 67 to 74, Supplement 3-1	

Table 11: Mechanical properties, other than shear and compression strength,
and testing procedures - cont.

Type	Property definition	Symbol	Testing procedures to obtain properties
	swelling pressure index	I_{sp}	Paragraphs 25 to 32, Supplement 3-1
	swelling strain index	I_{ss}	Paragraphs 33 to 40, Supplement 3-1
	slake-durability index	I_{d2}	Paragraphs 41 to 46, Supplement 3-1
Index properties	Consistency limits and indices:		
	shrinkage limit	w_s	Paragraphs 10 to 16, Supplement 3-4
	plastic limit	w_p	
	liquid limit	w_l	
	plasticity index	I_p	
liquidity index	I_l		
	grain size distribution	-	Paragraphs 4 to 9, Supplement 3-4
	corrected point load strength index	I_{sc}	Paragraphs 56 to 66, Supplement 3-1
	anisotropy index	I_a	

STATISTICAL ANALYSES OF TEST DATA

168. At this point, guidance is given on the test methods required to evaluate those mechanical properties which might be used when designing a slope.

169. The methods discussed in previous sections provide results relating only to the specific sample being tested. To obtain any design parameter values which might be representative of an entire design sector, several samples must be tested. The number of samples necessary depends on the degree of variation in the material located within the sector.

170. The design values representative of the entire design sector, are established by statistical methods. The test data are analyzed as to mean values and standard deviations.

171. Statistical analyses are demonstrated, hereunder with test results.

172. Irregularly shaped block samples from a Canadian open pit copper mine, that contained geological fractures, were tested in a laboratory for shear strength along the fractures. The three samples represented the variation of the wall material within a design sector, identified as diopsidic limey quartzite. The geological

descriptions of each sample are:

Sample A = limey quartzite with minor diopside

Sample B = limey quartzite with diopside

Sample C = diopsidic quartzite (26).

TEST RESULTS

173. Within each sample, an appropriate number of specimens were prepared and tested by the direct shear method as described in Supplement 3-2, para 55 to 63. The applied normal stresses ranged from 10 psi (69 kPa) to 660 psi (4547 kPa).

174. From the test reports, prepared in accordance with the specifications given in para 61 and 62 of Supplement 3-2, the values obtained for the residual friction angle, ϕ_r , peak friction angle, ϕ_p , dilatancy angle, d , and basic friction angle, ϕ_b , of each specimen are extracted and listed with the corresponding applied normal stresses in Table 12.

CALCULATION OF THE MEAN AND STANDARD DEVIATION

175. After including all the tested specimens of each rock sample, there were 63 residual friction angle values available ie, the observation number n is 63.

Table 12: Test results from a copper mine

σ_n psi	Sample											
	A				B				C			
	ϕ_r	ϕ_p	d	ϕ_b	ϕ_r	ϕ_p	d	ϕ_b	ϕ_r	ϕ_p	d	ϕ_b
10	37.5	66	35	31	28	75.5	32	43.5	40	71.5	26.5	46
	39.5	69	17.5	31.5	36.5	69	27.5	41.5	45	68.5	22	46.5
	40.5	67	13.5	53.5	41	60	27	33	32	65.5	28	37.5
15					29.5	69.5	39	30.5				
	42.5	58	20	38	--	65	30	35	43.5	64	22	42
	43	60.5	16.5	44	38.3	66	29.5	35.5	35.5	50	6	44
	41	62.5	15.5	47	42	52	22	30	40	59.5	18.5	41.5
20					34.5	63.5	34.5	29				
	39	46	7.5	38.5	38.7	60.5	27.5	33	44.5	59.5	17	42.5
	42.5	55.5	20	35.5	40	48.5	--	--	38.5	48	7	41
165	42.5	62	14	48	31.5	62.5	31	31.5	34.5	55	18	37
	34.5	42	7	35	32.2	58	19.3	38.7	46	56.5	20.5	36
	45.5	56.5	--	--	38.5	48	18.6	29.4	40	50.5	14	36.5
330					39	50.5	5	45.5	43.5	47.5	11.3	36.2
	33	40	6	34	26.5	53.5	7.1	46.4	39.5	51.5	16.3	35.2
	41.5	45	--	--	37.5	45	1.4	43.4	39	43.5	6.4	37.1
495					34.5	48	18	30	35	34	4.3	29.7
	34	38.5	2	36.5	29.6	47.5	7.1	40.4	42.5	45	4.8	40.2
	42.5	46	6.4	39.6	36	43	3.5	39.5	35	40.5	3.5	37
660	34.5	41	--	--	33	42	16.7	25.3	35	40	8.5	31.5
	33	38.5	2	36.5	39.9	40.5	2.8	37.7	32.5	40	2	38
	42.5	43.5	6.4	37.1	35	38	0	38	37	38.5	3.9	34.6
	34.5	38	--	--	27	34	10.6	23.4	32.5	38.5	8.5	30

176. The statistical sample mean, μ , is calculated as:

$$\mu = \frac{\sum_{i=1}^{i=n} x_i}{n} \quad \text{eq 15}$$

where x_i is the value of each individual observa-

tion, and n is the overall observation number.

177. The standard deviation, σ , is calculated as follows:

$$\sigma = \sqrt{\left[\frac{1}{n-1} \left\{ \sum (x_i - \mu)^2 \right\} \right]} \quad \text{eq 16}$$

or else by the algebraically equivalent form, which is more convenient for machine computations:

Table 13: Calculation of mean and standard deviation for the residual friction angle ϕ_r

x_i	x_i^2	x_i	x_i^2	x_i	x_i^2	x_i	x_i^2
26.5	702.3	34.5	1190.3	38.5	1482.3	42.0	1764.0
27.0	729.0	34.5	1190.3	38.5	1482.3	42.0	1764.0
28.0	784.0	34.5	1190.3	38.5	1482.3	42.5	1806.3
29.5	870.3	34.5	1190.3	39.0	1521.0	42.5	1806.3
29.6	876.2	34.5	1190.3	39.0	1521.0	42.5	1806.3
30.9	954.8	35.0	1225.0	39.0	1521.0	42.5	1806.3
31.5	992.3	35.0	1225.0	39.5	1560.3	42.5	1806.3
32.0	1024.0	35.0	1225.0	39.5	1560.3	42.5	1806.3
32.2	1036.8	35.0	1225.0	40.0	1600.0	43.0	1849.0
32.5	1056.3	35.5	1260.3	40.0	1600.0	43.5	1892.3
32.5	1056.3	36.0	1296.0	40.0	1600.0	43.5	1892.3
33.0	1089.0	36.5	1332.3	40.0	1600.0	44.5	1980.3
33.0	1089.0	37.0	1369.0	40.5	1640.3	45.0	2035.0
33.0	1089.0	37.5	1406.3	41.0	1681.0	45.5	2070.3
34.0	1156.0	37.5	1406.3	41.0	1681.0	46.0	2116.0
34.5	1190.3	38.3	1466.9	41.5	1722.3		
499.7	15695.6	750.8	20388.6	635.5	25255.1	650.0	28191.0

$$\sum_{i=1}^n x_i = 2356.0 \quad \sum_{i=1}^n x_i^2 = 89530.3$$

Observation number, $n = 63$

$$\text{Mean, } \mu = \frac{\sum x_i}{n} = \frac{2356.0}{63} = 37.4^\circ$$

Standard deviation,

$$\sigma = \sqrt{\left\{ \frac{1}{n(n-1)} [n \sum x_i^2 - (\sum x_i)^2] \right\}}$$

$$\sigma = \sqrt{\left[\frac{1}{63 \times 62} (63 \times 89530.3 - 2356^2) \right]} = 4.8^\circ$$

$$\sigma = \sqrt{\left[\frac{1}{n(n-1)} \left\{ n \sum_{i=1}^n x_i^2 - \left(\sum_{i=1}^n x_i \right)^2 \right\} \right]} \quad \text{eq 17}$$

178. The mean and the standard deviation for the residual friction angle are calculated by means of eq 15 and 16, and listed in Table 13. It is good practice to list the values of the individual observations in order of their magnitude.

179. Based on the results of the calculations

shown in Table 13, it can be said that the residual friction angle, ϕ_r , has a mean value of 37.4° and a standard deviation, a measure of its variation, of 4.8° .

180. Strictly speaking, equations 15 to 17 can be used only if the frequency distribution of the observations is normal. It is always advisable to compare the frequency distribution of the test results with the normal distribution.

COMPARISON OF RESULTS WITH NORMAL DISTRIBUTION

181. A comparison can be performed with the aid of Table 14 showing the Cumulative Normal Distribution (27).

182. Results of the calculation required for the comparison is demonstrated in Table 15. The necessary steps are:

- select convenient intervals for the x values, such as 27° , 29° , 31°etc
- count the cumulative frequency observations; $\sum i$, from column x of Table 13, where the residual friction angle values are conveniently listed in order of magnitude eg, the total number of observations when the residual friction angle values were found to be less than 29° is 3 ie, the x_i values of 26.5° , 27° and 28°
- calculate the cumulative frequency per cent values ie, $\sum i$ in terms of %
- calculate the $z = (x - \mu)/\sigma$ values for each selected interval of x eg,

$$z = (27 - 37.4)/4.8 = -2.17, \text{ or}$$

$$z = (39 - 37.4)/4.8 = 0.33$$

- calculate $F(z)$ per cent values using Table 14 eg,

$$F(-2.17) = 1.000 - 0.985 = 0.015 = 1.5\%, \text{ or}$$

$$F(0.33) = 0.629 = 62.9\%.$$

183. Results of the calculations presented in Table 15 are plotted in Fig 30. The difference between the cumulative normal distribution curve and cumulative distribution curve which could have been drawn by using the actual test results is negligible for practical purposes. It can thus be said that the test results of the residual friction angles have a normal distribution within the range of the given data. The difference can arise from the implicit assumption that the values go to the high and low extremes of normal distribution.

184. The final result obtained from statistical analyses on the test data for the range of residual friction angles, is presented in graphic form in Fig 31; it represents the residual shear

strength envelopes.

Table 15: Cumulative frequency distribution for the residual friction angle, ϕ_r

x	Cumulative frequency, $\sum i$	Cumulative frequency, %	$z = (x - \mu)/\sigma$	$F(z), \%$
27	1	1.6	-2.17	1.5
29	3	4.8	-1.75	4.0
31	6	9.5	-1.33	9.2
33	11	17.5	-0.92	17.9
35	21	33.3	-0.50	30.9
37	28	44.4	-0.08	46.8
39	35	58.7	0.33	62.9
41	45	71.4	0.75	77.3
43	56	88.9	1.17	87.9
45	60	95.2	1.58	94.3
47	63	100.0	2.00	97.7

INTERPRETATION OF RESULTS

185. Based on the information presented in Fig 30, the following conclusions can be drawn regarding residual friction angles obtained by the laboratory direct shear tests:

- there is an equal probability for the value of the residual friction angle being either less or more than 37.4°
- the probability that the value of ϕ_r is less than 32.6° is 16% (ie, one standard deviation towards the low values)
- the probability that the value of ϕ_r is less than 42.2° is 84% (ie, one standard deviation towards the high values)
- the probability that the value of the residual friction angle is within the limits of 32.6° and 42.8° is 68%.

186. To elucidate the meaning of the information presented in Fig 31, it is assumed that along a critically oriented fracture plane of the slope, because of previous movements the peak shear resistance is no longer available, and that the controlling parameter is the residual friction angle. Further, it is assumed that the normal

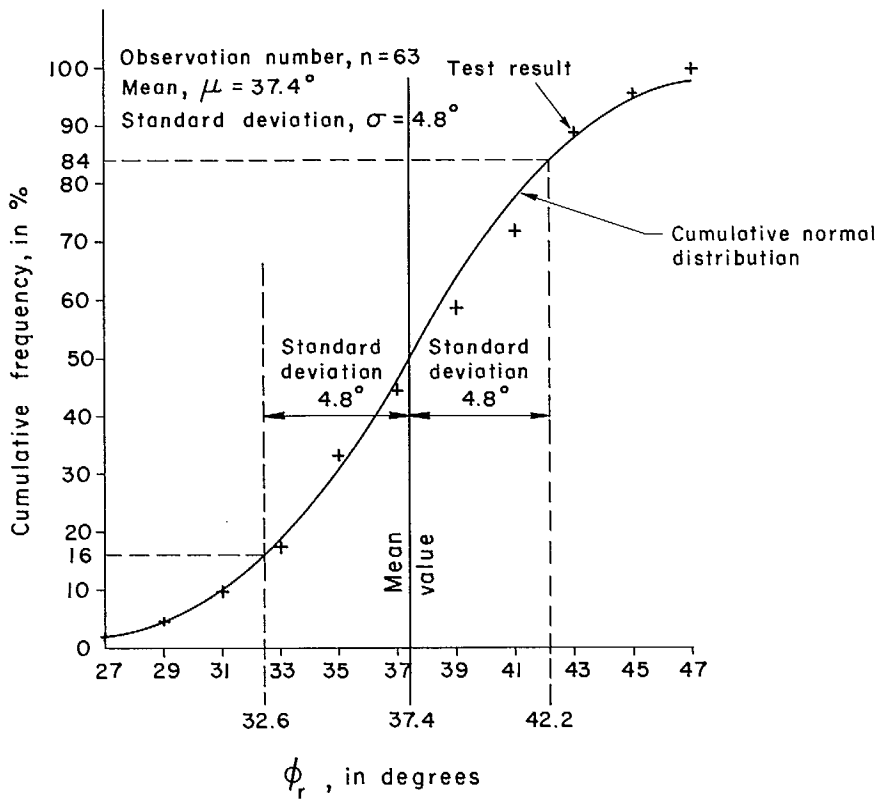


Fig 30 - Cumulative frequency distribution for the residual friction angles obtained by direct shear testing.

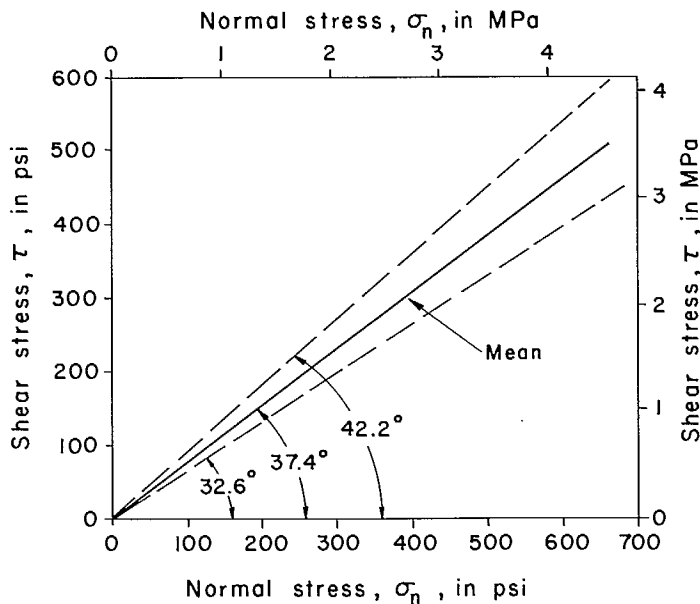


Fig 31 - Graphic representation of the statistically analyzed results of residual friction angle; showing the residual shear strength envelopes corresponding to the mean value of the friction angle and to the values increased and decreased by one standard deviation.

stress, σ_n , acting on the discontinuity plane is 300 psi (2.07 MPa). For these assumed conditions the following can be said about the shear resistance along the fracture plane:

- there is an equal probability that the shear resistance is either less than or more than, 229 psi (1.58 MPa)
- the probability that the value of the shear resistance is less than 192 psi (1.32 MPa) is 16%
- the probability that the value of the shear resistance is less than 272 psi (1.87 MPa) is 84°
- the probability that the value of the shear resistance is within the limits of 192 psi (1.32 MPa) and 272 psi (1.87 MPa) is 68%.

Statistical Analyses of Further Test Data

187. The above statistical analysis, demonstrated for the case of the residual friction angle, ϕ_r , needs also to be followed for each of the test results listed in Table 12.

188. The cumulative frequency distribution obtained for the basic friction angle, ϕ_b , is drawn in Fig 32. The basic friction angle has a statistical mean value of 37.3° with a standard deviation of 5.9°. The agreement between the cumulative normal distribution curve and the cumulative test distribution curve is not quite as good as that obtained for the residual friction angle (Fig 30). Considering that the basic friction angles are calculated only indirectly, the obtained agreement is however acceptable. The mean value of ϕ_b is almost identical with the mean value of the residual friction angle, $\phi_r = 37.4^\circ$; consequently they are interchangeable as discussed in para 76.

189. The values of both the peak friction angle, ϕ_p , and of the dilatancy angle, d , are affected by the applied normal stress, σ_n . These parameters must be statistically analyzed separately for each applied normal stress. The results of these analyses are shown in Table 16.

190. The sum or difference between two normal distributions is also a normal distribution. If μ_1 and μ_2 are the means and σ_1 and σ_2 are the standard deviation of two original distributions,

then the mean and standard deviation of the resultant distributions are respectively:

$$\mu = \mu_1 \pm \mu_2, \text{ and}$$

$$\sigma = (\sigma_1^2 + \sigma_2^2)$$

Table 16: Means and standard deviations for ϕ_p and d

σ_n , psi	ϕ_p		d	
	μ	σ	μ	σ
10	68.2°	4.0°	26.8°	7.7°
15	60.0°	5.3°	21.5°	8.3°
20	55.3°	6.4°	17.8°	8.5°
165	51.2°	6.9°	13.7°	6.2°
330	45.1°	6.2°	8.5°	6.2°
495	42.6°	3.0°	6.7°	4.6°
660	38.8°	2.5°	4.5°	2.2°

191. For example, if one adds to the normal distribution of the basic friction angle, ϕ_b , the normal distribution of the dilatancy angle, d , corresponding say to 165 psi normal stress, one should obtain the resultant distribution of the peak friction angle, ϕ_p , corresponding to 165 psi.

$$\text{ie, if } \mu_1 = 37.3^\circ, \sigma_1 = 5.9, \text{ and}$$

$$\mu_2 = 13.7^\circ, \sigma_2 = 6.2$$

$$\text{then } \mu = 37.3^\circ + 13.7^\circ = 51.0^\circ$$

$$\text{and } \sigma = (5.9^2 + 6.2^2) = 8.6^\circ$$

For practical purposes these results are close enough to the mean value of 51.2, and to the standard deviation of 6.9° of the peak friction angle at $\sigma_n = 165$ psi obtained by independently analyzing the corresponding test results (Table 16). The difference, especially in case of the standard deviation, is that none of the distributions of ϕ_b , or of d , or of ϕ_p , are exactly normal.

192. Using the statistical mean values obtained for peak friction angle, Table 16, the points of the peak shear strength envelope can be calculated

for each applied normal stress, as illustrated by the central curve in Fig 33. The points of the shear strength envelopes, corresponding to the peak friction angles increased by or decreased by

one standard deviation, can also be calculated. The meaning of these curves is similar to that of the residual shear strength envelopes, Fig 31, explained in para 186.

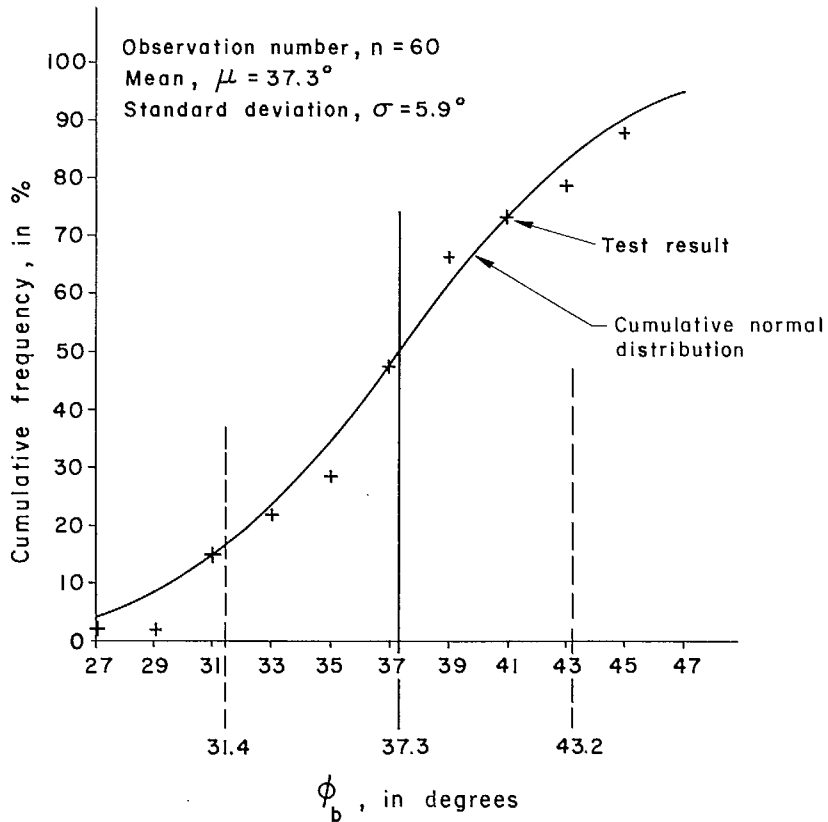


Fig 32 - Cumulative frequency distribution for peak friction angles obtained by direct shear testing.

REPORTING OF TEST RESULTS

193. The mechanical properties are determined to provide design parameters for use in the stability analyses of the slopes, and the rock properties which are useful in the slope design.

194. All design parameters such as shear strength, compressive strength, density, etc, must be provided in statistical terms; ie, their means and standard deviations must be known. Graphic presentations, where appropriate, as in Fig 30 and 31 are useful.

195. Mechanical properties, such as physical and index properties, are provided in terms of their ranges and average values.

REPORT SPECIFICATIONS

196. Preparation of a "Report on Mechanical Properties" is the final action to be taken within the mechanical properties determination task. The report must be prepared in such a way that it provides well documented answers to all questions posed during preparation of the test program.

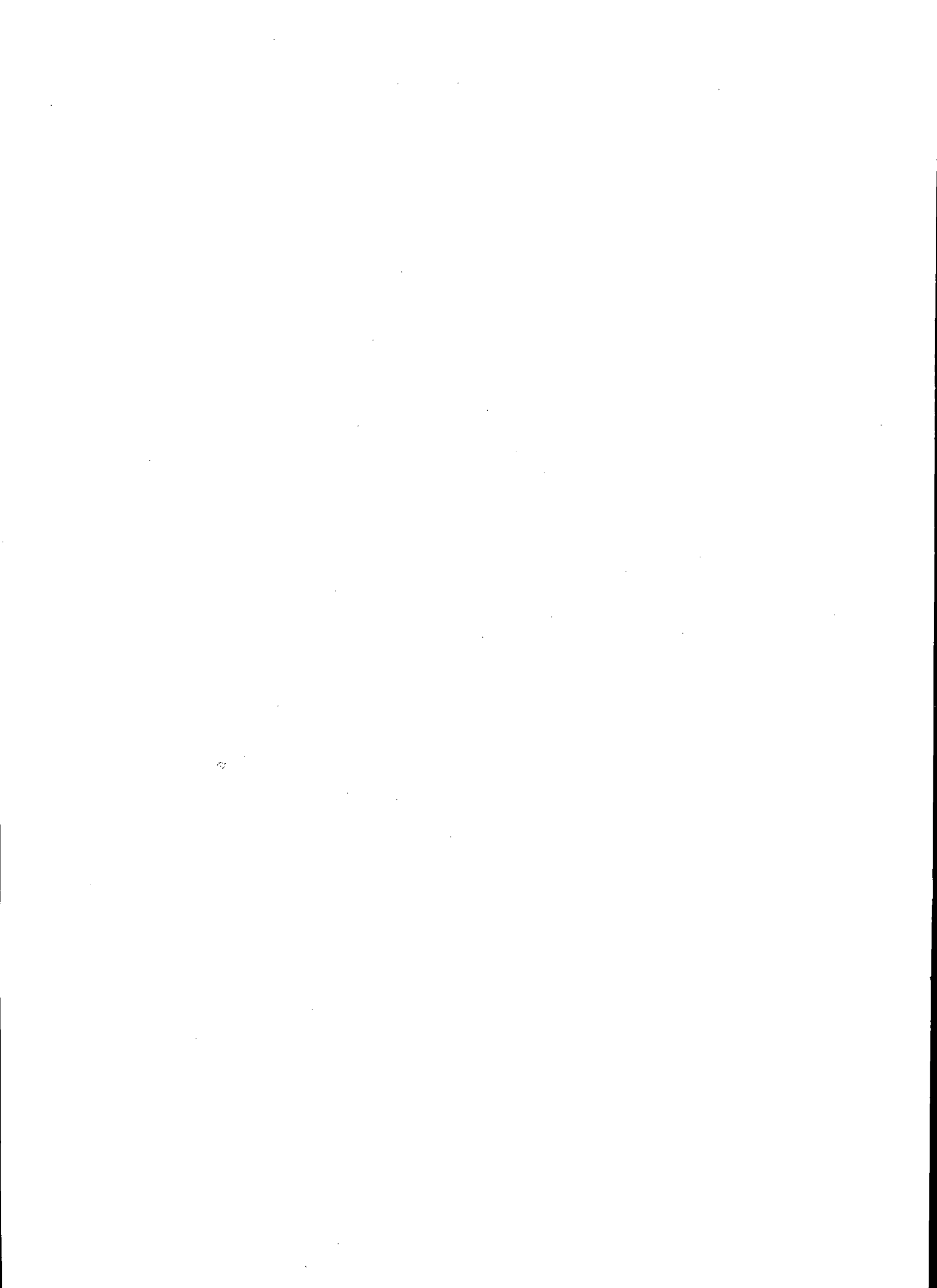
197. The following elements are considered important parts of a comprehensive report:

a. Title page - must include the name of the organization, report title, author, date, and

each copy must be numbered if controlled distribution is desired;

- b. Summary - must be a condensed version of the body of the report, written in language understandable by those members of mine management who may not be specialists in the field of rock testing, but who are nonetheless responsible for the work;
- c. Table of contents - must list all the various headings and sub-headings in the report as well as illustrations, tables, and graphs and their page numbers;
- d. Terms of reference or Purpose and scope - these must state the date of authorization for the work on determination of mechanical properties; a brief outline of the test program as given in Tables 1 and 2, and the basis upon which decisions were made during the preparation, as well as the names of the persons concerned in these decisions, must also be given.
- e. Mechanical Property A - this section must contain a technical presentation relating to a particular material property, including statements as to:
 - i definition of the mechanical property;

- ii value of the property which should be used in the slope design, presented in an appropriate form for the calculations ie, mean and standard deviations, average value and range, variations in a tubulated form, graphs, and relationships with other mechanical properties;
 - iii applicability of the mechanical property in question ie, a clear statement as to the exact design sector(s), or part of the design sector(s), to which this property is applicable, clearly defining the limitations and conditions of its use, such as the effect other factors may have upon it;
 - iv testing methods ie, referring to the appropriate test reports or to the origin of the data if previous experience is used;
 - v origin of the samples ie, where obtained, sampling method used, problems with sampling;
 - vi how the test data were analyzed ie, method used, reason for eliminating any test data, and detailed calculations etc;
 - vii recommendations for further testing or other investigations, and remarks eg, on the quality and reliability of the data presented.
- f. Mechanical Property B - this section is similar to Property A, but each mechanical property requires a separate section.
- g. Conclusions and Recommendations - these must present results of the test program in a consolidated form. The recommendations pertaining to further investigations needed for certain mechanical properties are summarized and presented in a suitable form for consideration by management. If the report is concerned with an early stage of mine development, the recommendation should also include an outline of the test requirements for the subsequent stages. This section must also specify how information regarding the eventual behaviour of the slope should be handled. In other words, the flow of the feed-back information needs to be specified together with the required procedures, in case unexpected changes occur in the rock material or structure.
- h. Acknowledgements - must provide information on assistance given to the author by individuals or organizations during the planning and execution stages of the test program.
 - i. Glossary - must provide a list of all terms, definitions, abbreviations, and symbols used in the report.
 - j. List of References - must give a list of all test reports and outside publications used in preparing the report. If appropriate, the reference should provide all necessary bibliographic details such as name of author, title, publication, volume, number, inclusive pages, and date of publication.
 - k. Distribution List - is usually the last page of the report, when distribution is to be controlled.



TEST PROGRAMS OF THE DEVELOPMENT STAGES

198. If properly designed, an open pit mine may experience only a limited number of cases of instability during its operating life. These cases are most likely to occur in those sections of the pit where the combination of geological and groundwater conditions is unfavourable.

199. As mentioned, programs for investigating the rock mass properties vary in accordance with the stage of mine development for which they are required. Although the type of properties required during the three phases is essentially the same, the approaches for obtaining them may differ. This is brought about by changes in site accessibility, changing availability of appropriate test samples, and accumulation of acquired experience during each of the three phases. Furthermore, the quality of the available data, and consequently the statistical reliability of the mechanical rock properties, also varies.

FEASIBILITY STAGE

General Considerations

200. At this stage, slope design work centres on establishing geometry of the prospective mine,

so that a meaningful analysis can be performed. Its purpose is to find out whether or not the mineral resources outlined by exploration, constitute a feasible mine. Within these terms of reference, the principal objective of slope design is to provide an estimate of the economically feasible ranges of slope angles for the various rock materials which are to constitute the walls of the prospective open pit.

201. This preliminary assessment relies primarily on the geological data made available by the exploration program which normally includes diamond drilling. At this stage, testing involves only a limited number of laboratory tests, mainly performed on drill cores.

Outline of Test Requirements

202. Interaction between the various activities, especially the triple one including the determination of the mechanical rock properties, structural geology investigations and slope design, has been emphasized throughout this chapter. Through the feasibility stage of development these activities must be even more closely integrated, partly because of the

preliminary nature of the slope design work, and partly because of the small budget usually available at this stage for detailed investigations.

203. The exploration program establishes the geological features of the wall materials, so that major structural domains can be defined. The major design sectors can then be outlined.

204. Using the geological information thus obtained, and following the steps given in para 26 to 35, a test program can be planned and executed. This program can obviously include only a limited number of the most suitable test types.

205. The test requirements depend upon the actual conditions found at the site. The importance of given types of tests consequently varies from site to site. The following discussion includes those test types which are usually required, and found to be adequate for most cases.

206. The principal aim of the tests is to establish reliable values for the selected mechanical rock properties, which can be used for estimating a range for the most likely slope angles, which in turn can be useful in the financial analysis. The number of specimens tested within each rock sample can usually be reduced to a minimum of three specimens.

Testing Discontinuities

207. The primary objective of tests performed during the feasibility stage is to establish the overall range of the strength parameters which apply to the local rock type and structural condition. In case of a fracture plane, the lowest value for the friction angle is the one given by the residual friction angle, ϕ_r . The easiest and most direct method to establish the residual friction angle is described in Supplement 3-2, para 46 to 54. The highest value of the friction angle is the one given by the apparent friction angle, ie, by $\Phi = \phi_r + d_0$. The value of the dilatancy angle at zero normal stress, d_0 , is determined by the method described in the Structural Geology chapter.

208. If a critically oriented discontinuity plane is filled with gouge material, the highest

value of the friction resistance is governed by the apparent friction angle, Φ , discussed above. To evaluate the lowest value of the shear resistance it should be assumed that the discontinuity planes contain thick gouge material; hence, the shear resistance is governed by the shear strength of the gouge material. If the infilling material is crushed host rock, then recompacted specimens are tested by the method described in Supplement 3-2, para 73 to 90. Should the gouge material be soil-like, the testing method described in Supplement 3-4, para 24 to 34, is appropriate.

Testing Rock Substance

209. The uniaxial compressive strength of the rock substance is used to classify the rocks within a pit, and to define and identify those areas where, due to the low shear strength of the rock substance, stability problems can be expected. The uniaxial compressive strength can be used to estimate the shear strength of the rock substance within regions of low normal stresses.

210. Choice of the test type to be used depends mainly on the rock substance, as well as on the testing equipment available. If the rock is solid and massive, core samples are tested either by the uniaxial compressive test method described in Supplement 3-1, para 47 to 55, or by the point load test method described in Supplement 3-1, para 56 to 66.

211. If the wall material consists of a highly fractured rock mass, its strength properties are evaluated by the triaxial testing method performed on recompacted samples as described in Supplement 3-2, para 73 to 90. Alternatively, the dilatometer test described in Supplement 3-3, para 7 to 10 can also be used. Actually, the latter test as performed in drill holes is designed for in situ deformation determinations; the test result thus obtained can also be used however to estimate the strength properties of a broken mass.

212. If the rock substance is weak and plastic and resembles soil more than rock, its strength properties are determined by the method used for soil as described in Supplement 3-4, para 24 to 34.

213. In most cases the density value of the rock mass used for calculating the vertical forces, can be estimated from Table 9 or Fig 28. In special cases, such as a large mass of an extremely weathered formation, the laboratory procedure described in Supplement 3-1, para 15 to 24 should be used to establish the density of the rock substance.

Testing Overburden

214. The presence of a thick, weak and badly drained overburden could lead to both stability and operating problems. The program must therefore include some simple tests which take

into account the quality of the overburden, even at the feasibility stage of mine development.

215. The consistency limits and indices of soil overburden are established by the methods described in Supplement 3-4, para 10 to 16; following this the soil strength can be estimated by using the appropriate graphs and tables, such as Fig 29 and Table 10.

Summary of Test Requirements

216. Table 17 shows a recapitulation of the tests usually required and which are also found to be adequate for the feasibility stage.

Table 17: Tests required during feasibility stage

Test requirement			
Subject	Properties	Rock condition	Testing procedure
Substance	Strength	solid, massive	Paragraphs 47 to 55, Supplement 3-1 or Paragraphs 56 to 66, Supplement 3-1
		highly fractured	Paragraphs 73 to 90, Supplement 3-2 or Paragraphs 7 to 10, Supplement 3-3
Rock		weak and plastic	Paragraphs 24 to 34, Supplement 3-4
	Density		Paragraphs 15 to 24, Supplement 3-1
Discontinuity	Strength	clean fracture joints	Paragraphs 46 to 54, Supplement 3-2 and Structural Geology Chapter
		joints filled with with gouge material	Paragraphs 24 to 34, Supplement 3-4 or Paragraphs 73 to 90, Supplement 3-2
Overburden	Strength		Paragraphs 10 to 16, Supplement 3-4

Data Presentation

217. The results are presented in the form of "Report on Mechanical Properties", as specified in para 196. This comprehensive report must include all the elements listed in para 197, with the provision that all results, including shear strength, compressive strength and density are to be given in terms of range values.

Utilization of Previous Experience

218. All previous experience represents important input data for the slope stability analyses at all development stages. In view of the specific characteristics of the feasibility stage, such as the preliminary nature of the design, scarcity of data, constraints on expenditures, hidden geological features, etc, proper use of all available experience is of particular importance.

219. The back-analysis of slides is a valuable method for estimating the full scale strength of a rock mass. Slopes become unstable only when their shear strengths along the surface of instability has been reduced nearly to their residual values. Consequently, the strength parameters deduced from back-analyses of unstable slopes provide lower limits for the in situ strength of the intact rock mass. In contrast, similar types of analyses conducted on excavated or natural stable rock slopes, will result in strength parameters which are closer to the upper values.

220. Using the method described in the Design chapter both the stable and the unstable slopes of any neighbouring open pit, as well as of any other rock excavations in the same type of rock and having similar geological features, are analyzed. Selecting the slope most applicable to the prospective mine is aided by the results thus obtained.

221. Published case histories of slope instability in similar types of rocks and geological conditions may also be helpful in estimating the possible range of applicable strength parameters. Laboratory and field test results as published in the rock mechanics literature, can also be used for the same purpose.

MINE DESIGN STAGE

General Considerations

222. The mine development stage has reached its design phase once the existence of an orebody has been established, financing for production arranged, and general outline of the pit and rate of its development have been decided.

223. The objective of the slope design within those terms of references is to determine the geometry of the mine boundaries ie, the location, slope angle and height of the various walls. This objective can be met only if the slope design is based on high quality and reliable geological, groundwater, and mechanical rock property data. All investigations aimed at obtaining these data during the mine design stage are consequently intensified, as compared with those of the feasibility stage.

Summary of Test Requirements

224. The purpose of tests performed during the mine design stage is to establish the means and standard deviations of the strength properties by testing an adequate number of samples. Other mechanical rock properties, in addition to those used during the preliminary design are also required now for a complete and comprehensive slope analysis and design.

225. The additional intensive structural geology investigations undertaken usually result in a structural map which displays redefined boundaries for the structural domains, and an increased number of design sectors. Changes in the available geological information must also be reflected by the testing program.

226. The mechanical rock properties required for the slope design are outlined in para 26 to 167. The testing task can only be said to have been successfully completed without wasting time and effort, once: (a) the requirements of a well planned test program have been completed, (b) the test results have been properly analysed, and (c) the results presented in an appropriate report.

227. The various sections of this chapter provide details of the work to be undertaken; for easy reference they are however summarized in Table 18, together with appropriate paragraph

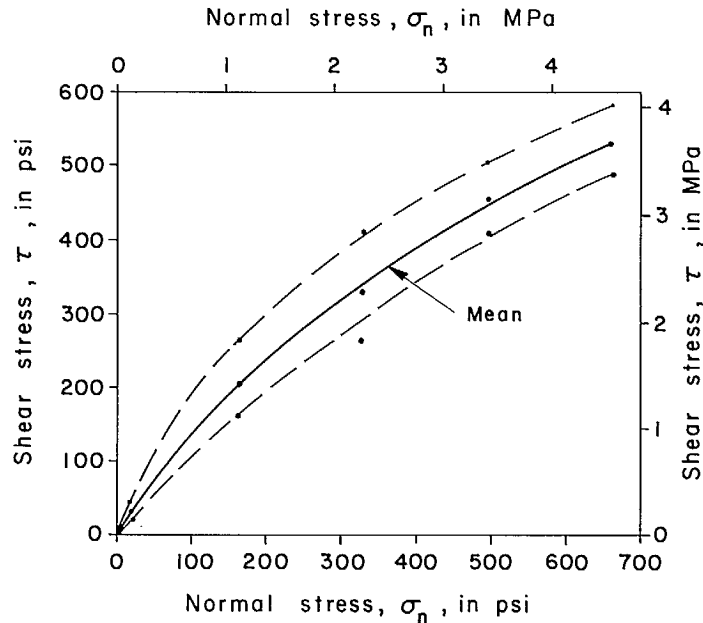


Fig 33 - Graphic representation of the statistically analyzed results of peak friction angles; showing the peak shear strength envelopes corresponding to the mean values of the friction angles and to the values increased and decreased by one standard deviation.

numbers.

OPERATING STAGE

General Considerations

228. During the operational life of a mine a situation may necessitate redesign of some of the pit wall sections if the assumptions contained in the original mine design have been outdated. These include changing commodity prices, new rock formations not detected by the original drill holes, drastic changes in ground conditions, or new ore discovered.

229. During the operating stage, information can be gathered on the structural features, groundwater conditions and mechanical behaviour of the wall material at relatively low cost. It should not be difficult to meet requirements for redesigning the pit wall, provided that an appropriately scaled observation program had been maintained throughout the operating stage. In any

case such a program is required for the continuous assessment of rock mass performance, and for comparing the actual behaviour of the slope with design predictions.

Outline of the Requirements

230. During the operating stage the testing task is two-fold:

- a. to verify or to modify, on the basis of in situ behaviour of wall materials, the design parameters previously obtained during the mine design stage;
- b. to perform the required tests if new rock formations or any drastic changes in rock mass quality become revealed.

231. To assess the performance of existing slopes a proper monitoring system must be designed and all relevant observations regularly processed and reported. Guidelines for designing and installing such a system are given in the Monitoring chapter; only those aspects of the

Table 18: Elements of a comprehensive test program

Work elements	Appropriate paragraph number
Planning of test program	26 to 36
Execution of test program	
Shear strength of discontinuities	39 to 94
Shear strength of substance	95 to 117
Compressive strength of a rock mass	118 to 129
Material properties other than strength properties	130 to 167
Statistical analyses of test data	
Test results	173 to 174
Calculation of mean and standard deviation	175 to 180
Comparison of results with normal distribution	181 to 184
Interpretation	185 to 186
Statistical analyses of further test data	187 to 192
Reporting of test results	
Report specifications	196 to 197

system are discussed here as they relate to the strength properties of the wall materials.

232. It has been observed from back-analyses of unstable slopes, that at the time of instability the shear strength of the rock mass often nearly equals its residual value ie, that at this stage the effect of interlocking is small or nonexistent. On the other hand, if the strength of the same mass is assessed by testing, the effect of interlocking is quite pronounced, with large apparent cohesion or a curved-line strength envelope, due to the well interlocked irregularities of the discontinuity planes; this interlocking feature must therefore be accounted for in the design parameters. These lead to the conclusion that slope instability in a rock mass is often of a progressive nature ie, that the actual failure is preceded by movements which decrease the strength along the surface of instability to a value close to the residual shear strength.

233. An optimal pit design is usually obtained

by assuming that no progressive instability will take place, and also by designing and installing a proper monitoring system as a safeguard. If information regarding movement is continuously available the loss of strength due to rock loosening or loss of interlocking can be properly estimated, and the time of probable instability predicted.

234. In summary, it is important to keep in mind that at the time of instability, the overall strength of the rock mass along the plane of movement may be much lower than previously determined. Since the progressive instability mechanism is difficult to assess in advance, a proper monitoring system may give the optimal solution.

235. A back-analysis of all slope instabilities which had occurred in the pit is important, so that the design data obtained from the testing program, and used for slope design, may be verified or modified. No opportunity should be missed to study all such cases for they can supply

valuable clues regarding instability modes and effective rock mass strength.

236. If the results of slope monitoring or of back-analyses verify the strength parameters obtained during the mine design stage, then no further tests are required during the operating stage. If the strength along a potential plane of instability is found however to be much less than previously expected, appropriate tests must be conducted on specimens representative of rock conditions along the potential plane of instability to verify its in situ strength.

237. After establishing the modified strength parameters, the geological information and the design analysis must be reviewed to identify other potentially suspect areas where the modified parameters might affect the stability of the walls.

238. Any additional tests which need to be undertaken, either because new rock formations had come to light or because unforeseen rock mass quality changes have been revealed, are performed in accordance with the procedures described. All additional data and information must be properly documented.

239. During the operating stage the use of a mechanical support system may become necessary. Available data on the strength and deformation properties of the rock mass, or on the plane of weakness usually suffice for designing such a system. In some cases additional tests might however be required. In view of the relatively

high cost of a mechanical support system, the costs of the additional tests, which may include large-scale in situ shear strength tests, become justifiable.

COSTS

240. The costs involved in determining mechanical properties depend on several interacting factors. The size of the open pit is one of these factors. Variations in the wall materials and in their condition are the other factors. Prevailing structural features and their degree of complexity greatly influence the test requirements and consequently the costs. The quality of the required data and consequently the statistical reliability of the mechanical rock properties vary in accordance with the current stage of development of the mine. As a result the required testing intensity also varies with the current stage of mine development, and so does the cost of testing.

241. Guideline figures for estimating the cost of investigating mechanical properties are given in Table 19. The amounts shown in the first lines indicate average costs and their variation as a function of pit depth and of the current development stage. The amounts in brackets indicate the range of costs which could arise as a result of variations in the above mentioned site conditions. The given cost figures are based on practical experience obtained by several companies; they are expressed in 1973 dollars.

Table 19: Budget guidelines

Stage	Depth 100 ft (30 m)	100 - 1000 ft (30 - 300 m)	> 1000 ft (> 300 m)
Feasibility	\$3000 (0 - 10,000)	\$30,000 (1000 - 100,000)	\$60,000 (10,000 - 200,000)
Mine design	\$5000 (0 - 10,000)	\$40,000 (1000 - 100,000)	\$60,000 (10,000 - 200,000)
Operating	\$5000 (0 - 10,000)	\$20,000 (1000 - 50,000)	\$50,000 (10,000 - 100,000)

CASE HISTORIES

242. The points discussed in para 198 to 238 are perhaps best illustrated by listing actual case histories. Three such histories will be discussed here. The first one exemplifies the mine design stage. The second one gives an example of a slope re-design problem when a pit was in its operating stage. The third case history provides an example of mechanical properties testing, undertaken for back-analysing rock slides.

MINE DESIGN STAGE

243. The mechanical property testing procedures described in this example were undertaken in connection with the design of the various slope angles required for the hanging wall of the Koolan Island open pit mine. The mine is located in Yampi Sound, Western Australia, and produces high grade hematite iron ore. The mine is situated on the southern slope of the island and eventually will be a little over one mile (1.6 km) long. According to present plans, the pit will extend down to a depth of 40 ft (12.2 m) below sea level, resulting in a maximum hanging wall slope height of about 400 ft (122 m). At present the mine is

in its early stages of production (28).

244. The geological cross-section of the mine site is shown in Fig 34. The rocks in the mine form a series of inclined layers consisting of:

- a. The Footwall Formation, which consists of a light brown fine grained quartzite in parallel-sided beds 1 to 10 ft (0.3 to 3 m) thick, containing also some interbedded shale layers with a thickness varying from a few inches up to about 2 ft (0.6 m). The quartzite is mostly hard and strong, but an estimated 20 per cent of it consists of friable material from which the siliceous cement has been leached. The leached material erodes easily.
- b. The Main Orebody is a tabular mass consisting of bluish-grey hematite, hematite conglomerate and hematite sandstone; it is largely friable, but also sufficiently strong to require blasting.
- c. The Hanging Wall schist, which consists of brown coloured chloritic and pelitic schists; the latter are soft and highly fractured with a well-developed schistosity parallel to the bedding and with minor interbedded hard quartzite layers.

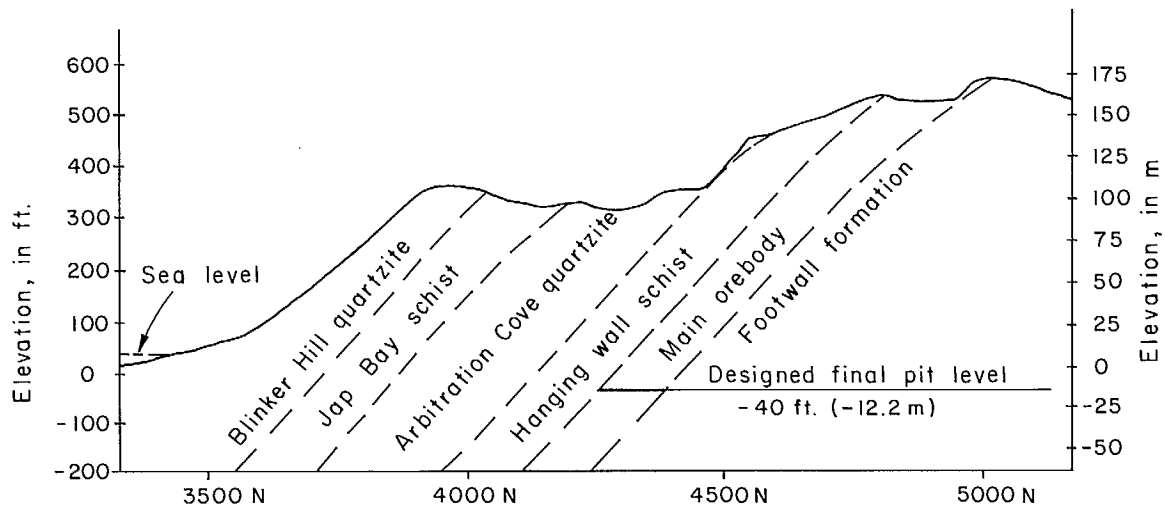


Fig 34 - Geological cross section of the mine site.

- d. The Arbitration Cove quartzite, which is a white to brown coloured fine grained quartzite with minor interbedded schist layers. The quartzite is hard and strong.
- e. The Jap Bay schist, which consists of brown coloured chloritic and pelitic schists; the latter are soft and highly fractured, but are interbedded with strong quartzite layers.
- f. The Blinker Hill quartzite, which is a white to brown coloured fine grained quartzite with minor interbedded schist layers. The quartzite is hard and strong.

245. No major faults or shear zones were found in the mine area; it is thought that defects in the rock mass consist of bedding planes and joints. The bedding strikes parallel to the pit axis at N 120°E, with an average dip of 45°SW at the western end and 55°SW at the eastern end.

246. Four sets of cross-joints can be identified, all having approximately the same direction, namely 90° to the bedding plane and inclined towards the pit. These structural features are equally dominant in the Blinker Hill quartzite, in the Arbitration Cove quartzite and also in the Jap Bay schist; they result in the 3-d-wedge type of instability shown in Fig 35. The orientation of joints in the Hanging Wall schist shows a little more variation.

247. For the hanging wall side 8 different design sectors were identified, namely the western end and the eastern end sectors within each of the four formations. The Footwall Formation showed a homogeneous structure and a homogeneous rock quality along its entire length; therefore, the entire footwall was taken to be a single design sector.

248. An appropriate number of randomly collected block samples, each containing one of the four joint sets, were tested for each type of rock formation. With the aid of a portable apparatus the direct shear test method was employed to determine the residual friction angles along the joint planes. The values of these friction angles, obtained for each joint plane separately, proved to be within a relatively narrow range. However, the range of the values showed some variation as a function of the rock type tested. All test data for each rock type were grouped and analysed together.

249. The cumulative frequency distribution curves thus obtained, together with the means and their standard deviations, are shown in Figs 36(a), 36(b), 36(c) and 36(d) for the Blinker Hill quartzite, Arbitration Cove quartzite, Hanging Wall schist and Jap Bay schist, respectively.



Fig 35 - Typical 3-d-wedge instability. Movement occurred along the two joint sets oblique to the face. The overhanging face is parallel to the bedding.

250. A similar testing procedure was used to evaluate the strength properties of the Footwall Formation. The duly analysed residual friction angle test data for the foot wall formation are shown in Fig 36(e).

251. A limited number of direct shear tests on ore material were also conducted, mainly for checking purposes. The analysed test data of the ore are shown in Fig 36(f).

252. With the aid of the test data thus obtained and making use also of the shear strength parameters obtained by back-analyses of the 3-d-wedge slides that had occurred in the pit, design analyses were performed for each design sector. The slope angles thus obtained for the hanging wall side are shown in Table 20.

OPERATING STAGE

253. The determination of mechanical properties, that is being described in this second example, is related to the redesign of the slopes of the Iron Monarch Mine near Whyalla, South Australia (29).

254. Commencing in 1914, the mine was originally developed in high grade ore outcropping along the crest of a ridge so that waste quantities were negligible and slope angles were not of great

importance. It is only in recent years that the mine has been excavated to below ground level on all sides, (Figs 37, 38 and 39) reaching its maximum depth of 470 ft (143 m) in 1971.

255. In 1971, the pit reached a turning point in its development. Deep drilling showed that the ore extended much deeper than had previously been anticipated. In addition an increasing demand for iron ore made it advantageous to seriously consider increasing the rate of production to such an extent that the pit would have bottomed-out within the next three years, should the existing pit limits and slopes (between 38 and 42 degrees) be maintained. This would have had the effect of "locking-in" the remaining ore, which could then have only been mined at the expense of high incremental stripping ratios. Furthermore, ore with a high manganese content would have been left along the north side of the pit, resulting in future blending difficulties. It was therefore necessary to revise the original mine plans and assess the possible advantages of changing the pit slopes. According to present plans the pit, when completed, will be roughly conical in shape with a maximum diameter of about 3000 ft (914 m) and a maximum depth of approximately 1000 ft (305 m).

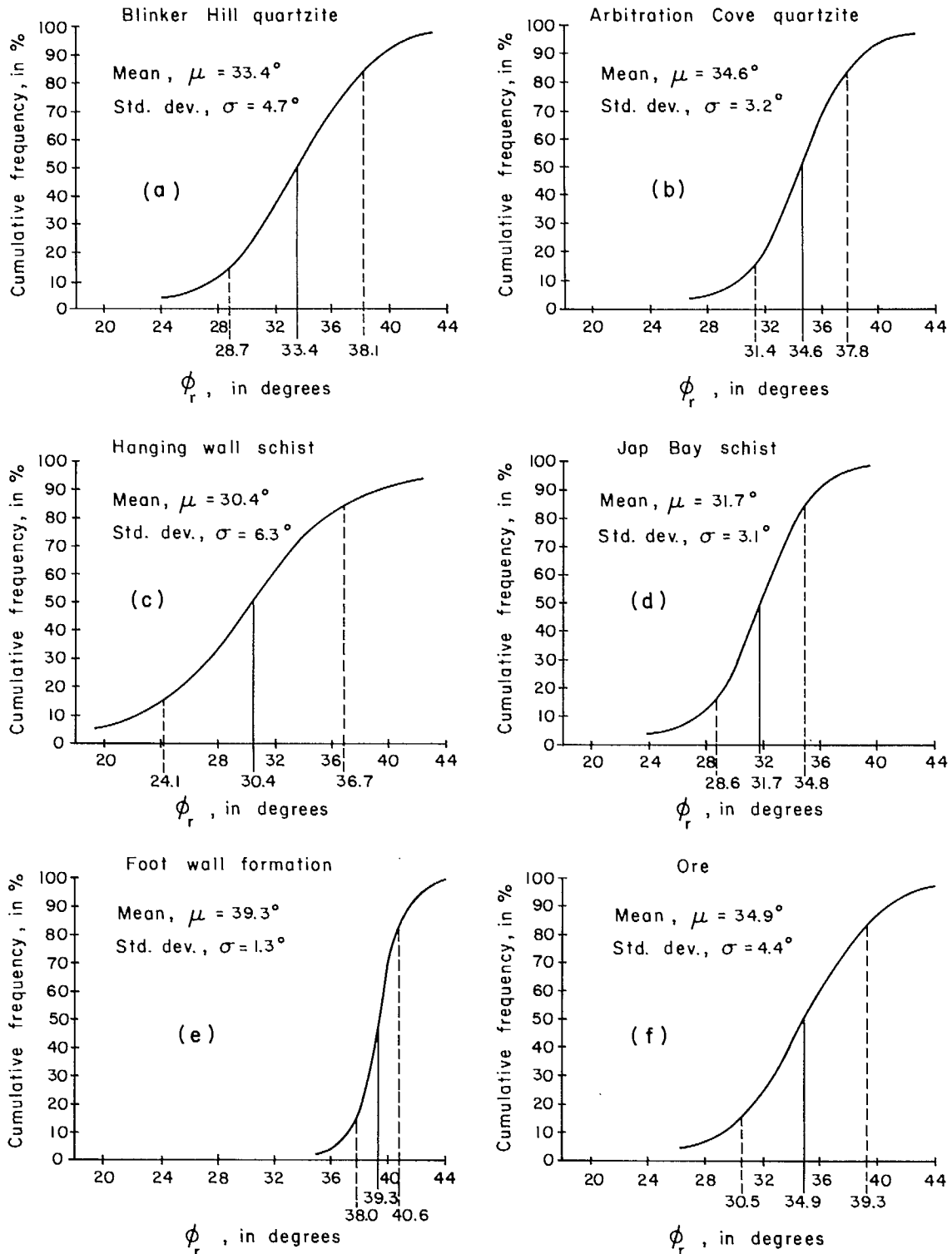


Fig 36 - Cumulative distribution curves of the residual friction angles, obtained for various wall materials: (a) Blinker Hill quartzite, (b) Arbitration Cove quartzite, (c) Hanging Wall schist, (d) Jap Bay schist, (e) Footwall formation, (f) Ore.

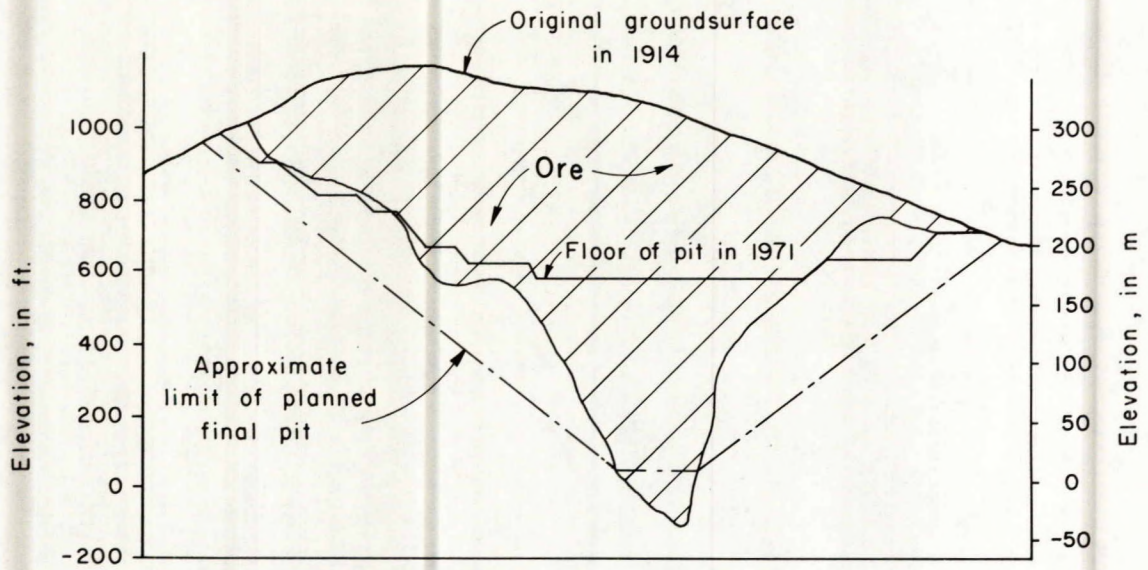


Fig 37 - Cross section through the orebody.

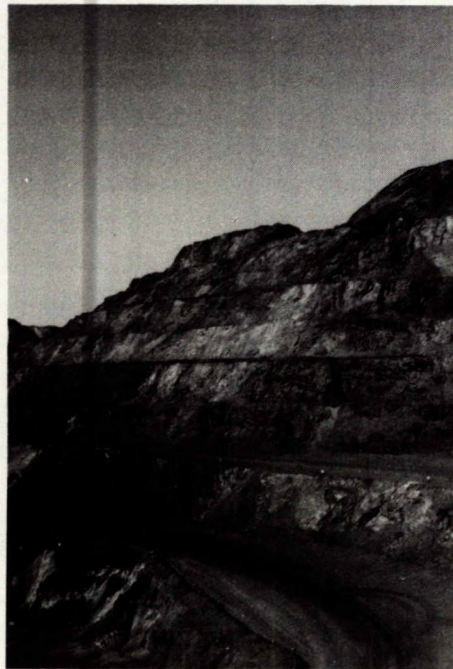
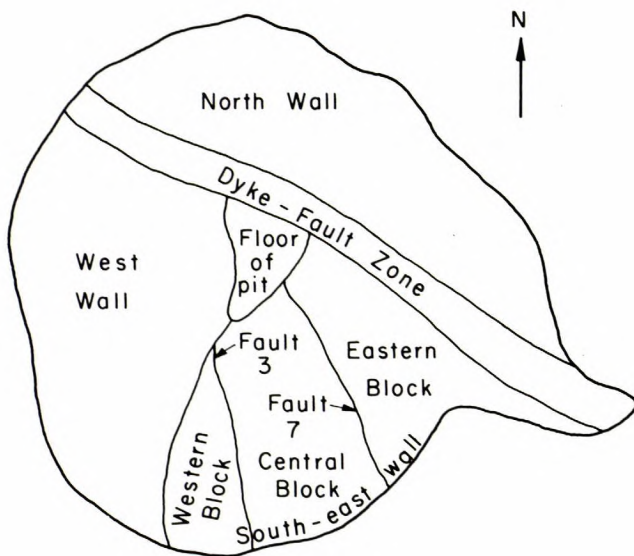


Fig 38 - SouthEast Wall of the Iron Monarch Mine. Highest point on the photograph is at elevation 1040 ft (317 m); pit floor was at elevation 570 ft (174 m) in 1971.



Fig 39 - NorthWall of Iron Monarch Mine with surrounding plain in background. Upper benches are in jaspilite, lower benches are in the amphibolite of the North Wall Dyke Zone. Loose covering 4th face from top is due to cutting-back operations. Crude extensometer in foreground is across a crack formed by minor slide on the south-east wall.



Scales:

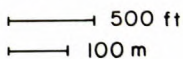


Fig 40 - Design sectors.

256. Analyses of the accumulated geological data indicated that the mine would have to be divided into six design sectors. Within each of these sectors the geological conditions and slope directions could be considered to be statistically homogeneous, for slope design purposes. The design sectors, as shown in Fig 40, were:

- a. North Wall,
- b. Dyke/Fault Zone,
- c. Central Block,
- d. Western Block,
- e. Eastern Block,
- f. West Wall.

257. The North Wall sector consists mostly of hard, strong, but jointed jaspilite. By means of geological investigations, which included inclined drilling into the jaspilite and trenching in the outcrop north of the pit, it was possible to conclude that the stability of the North Wall sector would be controlled by the orientation of the joints and by the shear strength along the critically oriented joints. The joint system causing concern strikes between $N 40^{\circ}W$ and $N 90^{\circ}W$ and dips at angles that range between 40 and 85 degrees towards the pit with a mean dip of about 65 degrees.

Table 20: Designed slope angles on the hanging wall side

Rock type	Overall slope angle excluding haul road	
	East end	West end
Blinker Hill quartzite	45°	45°
Arbitration Cove quartzite	40°	45°
Jap Bay schist	50°	50°
Hanging wall schist	47°	50°

258. The sheared, crushed and probably unstable amphibolite, by forming a soft toe along the north wall, represented the Dyke/Fault Zone design sector. The older amphibolite, wherever it is exposed in the existing pit, appears in the shape of a brownish dense clay. At greater depths though, namely in boreholes, it has a greenish-brown colour with a residual amphibolite texture; after a few days of storage in a core box though it is subject to spontaneous cracking. It is, therefore, very likely that this amphibolite would break down when exposed to the atmosphere in the pit. The younger amphibolite, which intrudes the older amphibolite and the ore, is a green crystalline rock of interlocking texture. It is not subject to spontaneous cracking when exposed to the atmosphere.

259. The Central Block sector is bounded by Faults 3 and 7. It consists of the south wall schist, with the schistosity dipping towards the pit at approximately 45 degrees. The stability of the sector is governed by the shear strength along the schistosity planes, as shown in Fig 41. The stability of the wall at the sector boundaries is also influenced by the faults, because here the occurrence of a 3-d-wedge type slide (Fig 42) can be anticipated.

260. Although the schistosity of the wall material contained in the Western Block has an overall dip towards the pit, its general appearance is analogous to that of a sheet of corrugated iron, with the axes of the corrugations plunging into the pit. The measured fold axes plunged at

an angle less than 35 degrees. Due to these structural features the Western Block is relatively more stable than the Central Block.

261. The material within and the structural features of the Eastern Block are similar to those of the Western Block; consequently its relative stability should also be similar. However, in case of the Eastern Block the negative effect of the convex wall geometry on the stability (Fig 40) also has to be considered.

262. The rock forming the West Wall is highly weathered feldspathic schist. One type of possible instability here is that of sliding along a set of inward dipping shear planes as shown in Fig 43. The second type of instability considered was that of a rotational circular slip controlled by the shear strength of the weathered soil-like feldspathic schist, with the latter resembling a dense clayey silt.

263. The test program, used to determine the mechanical properties, is shown in Table 21.

264. The test results thus obtained were analysed. A summary of these analysed test results is given in Table 22, expressed either in terms of mean values and standard deviations or by range values.

265. In addition to the aforementioned mechanical properties tests, the limited number of slides that occurred in the pit, were also analysed. From these back calculations the in situ friction angles and cohesion values were also obtained. The latter information, together with the analysed data concerning the dilatancy angles, d_0 , (which



Fig 41 - Instability of benches, near top of south-east wall, occurring along the schistosity planes.



Fig 42 - Wedge-shaped slide developing along faults on the south-east wall.

were measured in the field) is given in Table 23.

266. The slope analyses made use of the design parameters listed in Tables 22 and 23, and resulted in the slope design summarized in Table 24.

ANALYSIS OF SLIDES

267. The third example describes the determination of those mechanical properties that were required to perform a back-analysis of the slides which occurred in the Gibraltar East Pit of Gibraltar Mines Ltd., at McLeese Lake, B.C. (30).

268. The pit has been in operation since 1972. Its design called for uniform slope angles ranging

from 40 to 45°, with benches normally 90 ft (27.4 m) high and berms 45 ft (13.7 m) wide. At the time of back-analysis the pit was approximately 500 ft (152 m) deep.

269. The geological investigations, together with joint analyses, revealed the following structural features:

- a. Foliation in the quartz diorite is reasonably uniform and appears to be related to a regional stress pattern. It strikes at about 110° and dips southerly between 20° and 30° within the mine area. Joints parallel to the foliation are well developed, as shown in Figs 44 and 45.



Fig 43 - Face in deeply weathered feldspathic schists on the west wall. Small rock fall developed along shears dipping towards the pit.

These features have, therefore, a significant effect on slope stability along the north side of the pit.

- b. Three major joint sets along the north side of the Gibraltar East Pit, as follows:

Joint set "A" - joints parallel to sub parallel to the foliation (Figs 44 and 45). These joints are generally unfavourably oriented in relation to the pit wall, in the area of the instability that had occurred.

Joint set "B" - cross joints which strike approximately northeast and dip between about 60° and 90° northwest (Fig 44). These joints

are approximately normal to sets "A" and "C".

Joint set "C" - cross joints which strike approximately northwest and dip steeply northeast (Fig 44). These joints are generally discontinuous and are approximately normal to joint sets "A" and "B".

- c. Major faults occurring in this area generally appear to have similar orientations as the joint sets described above. A major set of faults in the pit strikes approximately northeast and dips between 50° and 80° southwesterly.

Table 21: Test program

Required mechanical properties	Testing method	Test results to be used for	Sampling source and method	Remarks
Jaspilite				
- shear strength along joint system dipping 65°	direct shear, residual and peak friction angles	North Wall sector	random, irregular block samples	
- density	laboratory and field measurements	input data for finite element analysis	random lump samples for <u>laboratory test</u>	Finite element model: (a) entire wall in jaspilite by removing the amphibolite
- elastic modulus	laboratory test		core samples from inclined holes and laboratory cores from block samples	
- Poisson's ratio	laboratory test			
- compressive strength	laboratory uniaxial			
Amphibolite				
- shear strength along shear surface	direct shear, residual friction	Dyke/Fault Zone sector	core samples from holes drilled into older amphibolite	limited number of tests for stability analysis of amphibolite slopes
- density		input data for finite element analysis		Finite element model: (b) toe of slope in amphibolite
- elastic modulus				
- Poisson's ratio				
- compressive strength				
- shrinkage limit	laboratory tests	characterization purposes		
- plasticity index				
South wall schist				
- shear strength along schistosity	direct shear, residual and peak friction angles	Central, Western and Eastern Block sectors	random, irregular block samples	
Feldspathic schist				
- shear strength along shears	direct shear, residual friction angle	West Wall sector, stability along shears	random, irregular block samples from more competent wall material	
- shear strength of substance	triaxial test on recompact specimens	West Wall sector, rotational circular slip instability	bulk samples from highly weathered wall material	
- shrinkage limite	laboratory test	characterization purposes		
- slake - durability index	laboratory test			

Table 22: Summary of test results

Mechanical property		Jaspilite	Amphibolite	South wall schist	Feldspathic schist
Residual friction angle, ϕ_r , deg		30.7 ± 2.3	23	27.5 ± 3.4	22.1 ± 1.3
Peak friction angle, ϕ_p , deg		30 to 34	-	31.6 to 38	-
Internal friction angle, ϕ , deg		-	-	-	35
Density, γ	lb/ft ³	207 ± 22	182 ± 9	-	-
	g/cm ³	3.31 ± 0.36	2.92 ± 0.15	-	-
Elastic modulus, E	10 ⁶ psi	10.2 ± 3.8	2.54 ± 1.63	-	-
	MPa	$70,327 \pm 26,200$	$17,513 \pm 11,238$	-	-
Poisson's ratio, μ		0.123 ± 0.072	0.243 ± 0.101	-	-
Uniaxial compressive strength, Q_u	10 ³ psi	49.3 ± 26.9	3.85 ± 1.94	-	-
	MPa	340 ± 185	27 ± 13	-	-

Table 23: Summary of additional design parameters

Design parameter		Jaspilite	Amphibolite	South wall schist	Feldspathic schist
Friction angle from back-analysis		40°	-	37.5°	34° - 35°
Cohesion from back-analysis	psi	-	-	-	5 - 30
	kPa	-	-	-	34 - 207
Dilatancy angle at zero normal stress, d_o	joints	$13.5^\circ \pm 11.6^\circ$	-	$7.2^\circ \pm 4.2^\circ$	$8.0^\circ \pm 5.6^\circ$
	fault, bedding	-	-	$8.5^\circ \pm 5.0^\circ$	-

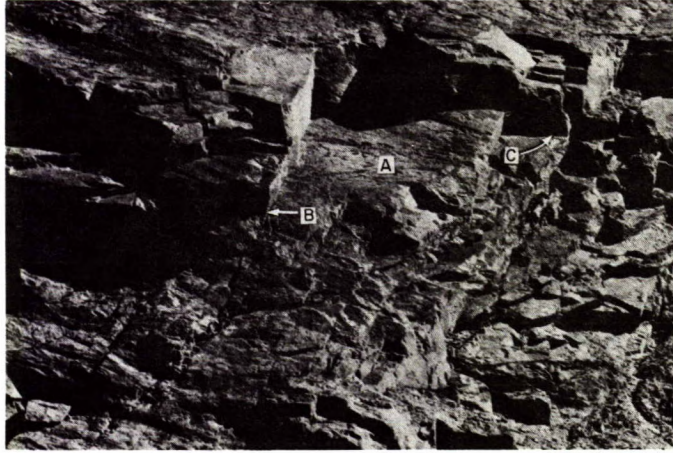


Fig 44 - View of three sets of structural discontinuities in Gibraltar East Pit.



Fig 45 - View of jointing parallel to foliation (A) on north wall of Gibraltar East Pit.

Table 24: Summary of slope angles and of bench geometry

Design sector	Overall slope angle	slope angle	Bench height	width
North wall	55°	72°	50 ft (15.2 m)	20 ft (6.1 m)
Dyke/fault zone	23°	-	-	-
Central block	42°	72°	25 ft (7.6 m)	20 ft (6.1 m)
Western block	42°	72°	25 ft (7.6 m)	20 ft (6.1 m)
Eastern block	42°	72°	25 ft (7.6 m)	20 ft (6.1 m)
West wall	35°	72°	25 ft (7.6 m)	2B ft (B.5 m)

270. The back-analysis included one large slide involving the total height of the wall sector and two much smaller 3-d-wedge type slides. Figure 46 shows the northeast wall where all three slides took place.

271. The slope forming material in the area of Slide No. 1 consists primarily of quartz diorite. The upper section of the slide consists of 90 to 180 ft (27 to 54 m) of weak leached cap rock, overlain by 45 to 90 ft (13.5 to 27 m) of glacial till, as shown in Fig 47. Foliation within the quartz diorite on this side of the pit is generally unfavourably oriented, as far as slope stability is concerned. Accordingly, instability appears to have developed, at least in part, along those joints which occur parallel to the foliation, ie along the joint set "A".

272. The planes of movement of both Wedge No. 2 and Wedge No. 3, shown in Figs 48 and 49 respectively, are faults parallel to joint set "A". Instability appears to have been caused by infiltration of surface water that flows into the pit over its crest. In the diagrammatic sketches of both wedges (Figs 50 and 51) the planes of movement are labelled as Plane "A". The other sides of both wedges belong to joint set "B", which dips

almost vertically. In Figs 50 and 51 these planes are labelled Plane "B". There is no shear along them; they act more like tension cracks.

273. Sampling was concentrated upon those discontinuities which were considered to play an important role in determining the stability of the three slides. Because of the relatively highly weathered character of the rocks in the north wall of the Gibraltar East Pit it was neither necessary, nor possible, to core; therefore all samples were taken in the shape of blocks.

274. A detailed summary of the sampling procedures used is given in Table 25. It contains the location and elevation of the respective samples and a description of the type of discontinuity sampled. Also given are the dip and dip direction of the sampled discontinuity.

275. The shear box apparatus used for all tests is shown in Fig 52; it was built at the University of Alberta. The normal load is provided by a freely suspended dead load system, while the shear load is generated mechanically by an electric motor and a gear box. The shear displacement, shear load and vertical displacement are recorded simultaneously by an electronic data acquisition system. The output of the three measured



Fig 46 - Salient features of the north-east wall of the Gibraltar East Pit showing Slide No. 1 and Wedges No. 2 and No. 3.



Fig 47 - View looking north of Slide No. 1 from the pit crest at elevation 3725.



Fig 48 - View of Wedge No. 2.



Fig 49 - View of Wedge No.3.

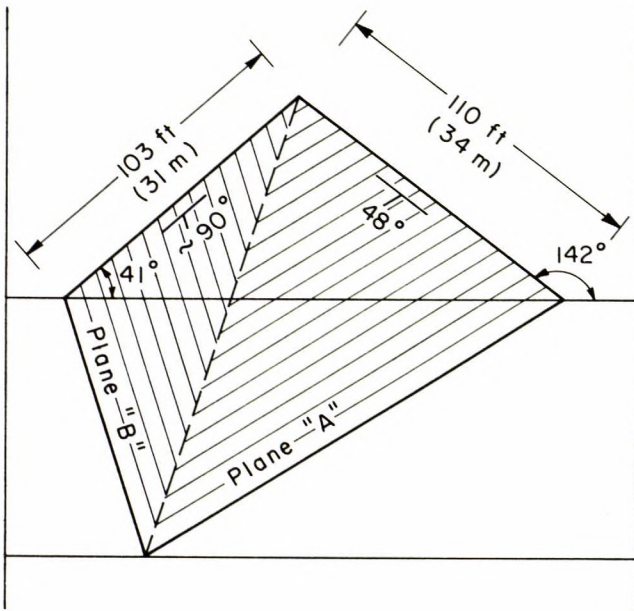


Fig 50 - Simplified geometry of Wedge No. 2.

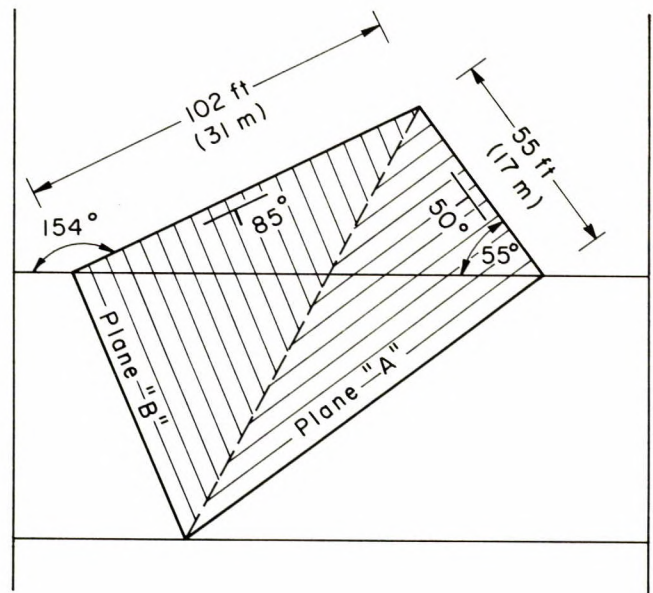


Fig 51 - Simplified geometry of Wedge No. 3.

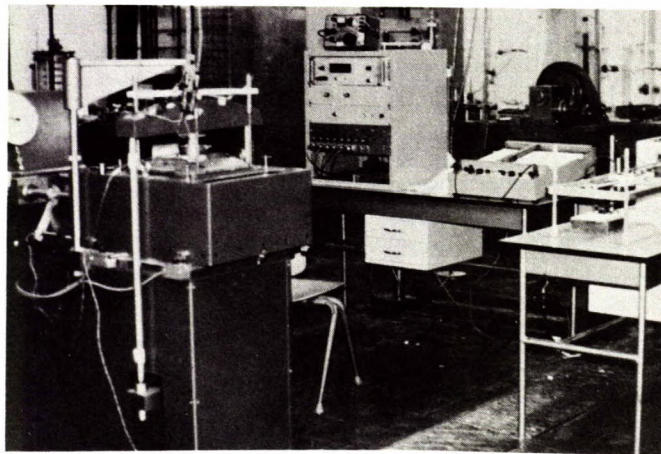


Fig 52 - Direct shear apparatus with the data recording system (built at the University of Alberta).

Table 25: Summary of block samples from Gibraltar Mine

Sample	Location	Elevation, ft	Type of discontinuity	Dip/dip direction
GB 1	Wedge No. 3	3643	Joint set "A", foliation direction; contains small amount of fine-grained material, orange stains, surface very irregular.	42°SW/208°
GB 2	Wedge No. 3	3643	Joint set "A", foliation direction; contains thin layer of fine-grained material, but no mineralization, surface very irregular.	41°SW/197°
GB 3	Wedge No. 2	3643	Joint set "A", foliation direction; highly weathered, surface very irregular.	Not measured
GB 4	Profile along Slide No. 1	3465	Joint set "A", foliation direction; contains thin layer of white weathered material, also some chalcopryrite, bornite and copper ore minerals, surface very irregular.	55°SW/207°
GB 5	Profile along Slide No. 1	3420	Joint set "A", foliation direction; contains thin layer of clay and pockets of minerals.	58°SW/212°
GB 6	Slide No. 1	3285	Sub-horizontal surface; contains large amount of chalcopryrite.	23°SW/189°
GB 7	Profile along Slide No. 1	3330	Joint set "A", foliation direction; contains traces of bornite and other minerals, rather soft, surface very irregular.	44°SW/189°
GB 8	Slide No. 1	3375	Sub-horizontal surface; contains fine-grained material, traces of slickenside.	25°NE/34°
GB 9	Profile along Slide No. 1	3375	Joint set "A", foliation direction; high degree of mineralization and some alteration, surface very irregular.	44°SW/209°
GB 10	Profile along Slide No. 1	3465	Joint set "A", foliation direction, weathered, brown coloured.	69°SW/204°

quantities is given in digital form. In addition to this the relationship between the shear displacement and shear load is recorded graphically, on an XY recorder.

276. The shear box used throughout the entire testing program measured 2 x 2 in. (4.1 x 5.1 cm). In order to fit the shear box size a block sample was cut into smaller specimens by means of circular diamond saw. These specimens were cast in forms, using KWIK-SET cement as casting agent.

277. Shearing was carried out starting from an initial position at which both halves of the specimen were in their original configuration. Shearing continued forward up to a maximum displacement of 0.25 in. (6.35 mm); then it was stopped and reversed. After returning to the initial position the shear movement continued in the opposite direction, again reaching a displacement of 0.25 in. (6.35 mm). This process was carried on until five cycles were completed. The rate of shear was kept constant during all tests at 1.33 mm/min, corresponding to a rate of strain of 2.6% per minute.

278. On an average 5 specimens were tested from within each sample at various normal stresses, ranging from 5 psi (34.5 kPa) to 50 psi (345 kPa).

279. The shear force mobilized at the point of

peak resistance during the first testing cycle was used to calculate the peak friction angle. The residual friction angles were calculated from the average of the shear forces that were recorded at the moment of passing through the initial position during cycles 2 to 5.

280. Analysis of the residual friction angle data yielded a mean value of $\phi_r = 24.0^\circ$, with a standard deviation of 6.4° . A graphical presentation of the data analysis is given in Fig 53.

281. The mean values, and their standard deviations, of the peak friction angles obtained are given in Table 26 as a function of the applied normal stresses.

Table 26: Mean and standard deviation values of peak friction angles

σ_n , psi	Mean, μ	Standard deviation, σ
5	65.6°	5.5°
10	56.1°	8.0°
30	53.2°	3.1°
50	47.3°	4.5°

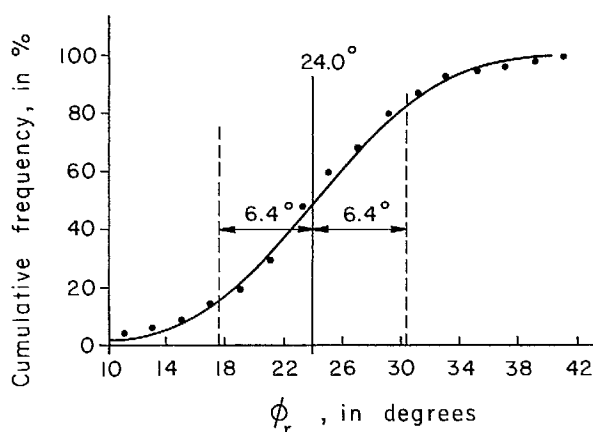


Fig 53 - Cumulative frequency distribution of residual friction angles of joint set "A".

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GLOSSARY

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

ALTERATION

Change of the mineral composition of a rock by hydrothermal solutions (eg kaolinization).

ANGLE OF INTERNAL FRICTION [D]

The maximum angle of obliquity between the normal and the resultant stress acting on a surface within a soil or rock.

ASPERITY

A high point on a discontinuity in rock. Asperities form the actual contact points along a discontinuity.

ATTERBERG LIMITS

- see plasticity index.

BED ROCK

The more or less continuous body of rock which underlies the overburden soils.

BEDDING

Repetitive layering in sedimentary rocks marking the boundaries of small lithological units or beds.

BENCH

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

BENCH ANGLE [D]

The angle of inclination of the bench face measured from the horizontal.

BENCH FACE

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

BENCH HEIGHT [L]

The difference in elevation between the grades of two succeeding downward production blasts.

BENCH WIDTH [L]

The horizontal ground left between each succeeding lift or blast.

BOUNDARY

Surface separating rock formations or zones of alteration.

COEFFICIENT OF UNIFORMITY [D]

The ratio of the 60% passing size of the material to the 10% passing size of the material, $U = D_{60}/D_{10}$.

COHESION [FL⁻²]

The portion of the shear strength, τ , indicated in Coulomb's equation, $\tau = c + \sigma \tan \phi$, by the term c ; ϕ is the angle of internal friction. It has the nature of an intergranular binding force. Sometimes referred to as apparent cohesion when it is known that the straight-line relationship is not valid but is assumed for a limited range of normal stress.

CONTACT

Surface between two rock types, one or both of which are not sedimentary.

CREEP

An increase in plastic strain with time is usually called creep.

CREST

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

CUMULATIVE DISTRIBUTION

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

DEFLECTION [L]

The movement of a point on a body.

DEFORMATION [L]

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

DEGREE OF SATURATION [D]

The ratio of the volume of water in the soil voids to the total volume of voids.

DENSITY (MASS DENSITY) [ML⁻³]

Mass per unit volume. Relative density - the density relative to its two limiting values.

DESIGN SECTOR

Sector of slope with uniform geology and similar geometry.

DIP [D]

The angle of a slope, vein, rock stratum, or borehole as measured from the horizontal plane downward. Where applicable, the dip is measured normal to the strike.

DISCONTINUITY

Surface of change of mechanical properties or composition in rock. This includes fracture surfaces, weakness planes and bedding planes, but is not restricted to mechanical discontinuities.

DISPERSION

The variability of observed data of a simple property.

DISPLACEMENT [L]

The straight line distance between two points or position.

DOMAIN

An area or volume in a body which passes statistical homogeneity or uniformity in respect to one or more variables.

ELASTICITY

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

EXTENSOMETER

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

- FACE**
The vertical, or near-vertical, surface of rock exposed by mining operations.
- FACTOR OF SAFETY [D]**
The ratio of the forces tending to prevent sliding to those tending to cause sliding.
- FAULT**
Surface of shear recognizable by the displacement of another surface that crosses it. Frequently rock on both sides of the fault is shattered resulting in infillings like fault breccia (coarse fragments), gouge (fine, soft powder) and mylonite (hard, fine-grained laminate). Walls often show striations from shear displacement (slickensides). With regard to orientation and displacement faults can be classified into thrust, normal and reverse faults. Numerous small faults or a confused zone of gouge, breccia or mylonite sometimes many kilometers wide, can be called a fault zone.
- FIELD STRESS**
See Stress.
- GROUND WATER LEVEL**
The level below which the pores and fissures of the rock and subsoil, down to indefinite depth are full of water.
- GROUT**
A mixture of cement and water or of cement, sand and water. May contain chemical additives to speed setting time or result in expansion on setting.
- HOMOGENEITY**
A characteristic of a volume or area of a substance where each element has the same property or value regardless of location.
- INFILLING (GOUGE)**
Material occupying the space between fracture or discontinuity surfaces.
- ISOTROPY**
Condition of having the same property in all directions.
- JOINT**
Fracture in rock mass along which there has been little or no displacement. Joints are surfaces of rupture (extension), which can be formed by crustal movements (regional joint systems), by shrinkage from cooling (columnar joints in basalt) or by elastic rebound of the earth's crust (sheet joints). Joints can follow bedding planes in weakly cemented layers and may be called bedding joints. A group of parallel joints is called a joint set and various joint sets form a joint system.
- JOINT ORIENTATION**
Mean planar surface representing the attitude of a joint. Plane can be described by a strike and dip or preferably by the dip vector.
- JOINT SET**
Group of parallel joints.
- JOINT SYSTEM**
Consists of two or more joint sets or any group of joints with a characteristic pattern.
- LAYER**
A bed or stratum of rock separated from the adjacent rock by a plane of weakness.
- LENGTH [L]**
Usually a long dimension parallel to the ore-body, unlike width and thickness.
- LINEATION**
The parallel orientation of geological features that are lines rather than planes, such as striae on slickensides, long axes of pebbles and cleavage/bedding plane intersections.
- MODULUS OF ELASTICITY (MODULUS OF DEFORMATION) [FL⁻²]**
The slope of the tangent (hence 'tangent modulus') of a stress-strain curve. The use of the

term Modulus of Elasticity is recommended for materials that deform in accordance with Hooke's Law, the term Modulus of Deformation for materials that deform otherwise.

MOHR'S CIRCLE

Graphical presentation of the state of stress at a specific point within a stressed material.

MOHR'S ENVELOPE

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

OVERALL SLOPE ANGLE [D]

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

PARAMETER

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

PLUNGE

Vertical angle between a horizontal plane and the maximum elongation of a body or the direction of a fold axis.

PLASTIC LIMIT [D]

A boundary region of water content representing a change in characteristic of the soil from those of a plastic to those of brittle material. This water content is called the plastic limit.

POISSON'S RATIO [D]

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

PORE WATER PRESSURE [FL⁻²]

Stress transmitted through the pore water.

PROBABILITY

It is concerned with unpredictable individual events but which are predictable in large numbers. It is the frequency ratio of occurrence of one event that can be expected in an infinitely large population of events. For less than an infinite population, it has the meaning of relative likelihood of occurrence.

PROBABILITY OF INSTABILITY

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

PROBABILITY OF SLIDING

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

PROGRESSIVE FAILURE

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

RANDOM SAMPLE

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

RELIABILITY

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

ROCK MASS

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

ROCK SUBSTANCE

The solid part of the rock mass, typically ob-

tained as drill core.

ROTATIONAL SHEAR SLIDE

A slide resulting from the yielding and redistribution of shear stresses in a soil so that a more or less circular surface of failure envelope develops before the cohesion breaks down and permits a comprehensive, circular sector of the slope to fail by rotating.

ROUGHNESS

Asperities of a rock surface on the scale of a hand specimen.

SAFETY FACTOR [D]

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

SAMPLE POPULATION

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

SCHISTOSITY

Surface of easy splitting in metamorphic rocks defined by the preferred orientation of metamorphic minerals.

SECTOR

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

SEGMENT

A vertical interval in a wall of one sector.

SHEAR FAILURE

Failure resulting from shear stresses.

SHEAR STRENGTH [FL⁻²]

The internal resistance offered to shear stress. It is measured by the maximum shear stress, based on original area of cross

section, that can be sustained without failure. Peak shear strength - at a certain level of shear stress the shear strength of the surfaces is exceeded and further displacement will take place without any further increase in shear stress. This limiting value defines the peak shear strength at that particular normal stress.

Residual shear strength - as the peak strength is exceeded, fracturing of interlocking projections on the surface occurs and the broken pieces are ground into detrital material as shear displacement continues. After a certain amount of displacement, the surface becomes slickensided and covered with gouge material and shear displacement takes place at a constant shear stress level. This shear stress is called residual shear strength.

SHRINKAGE LIMIT [D]

The decrease in volume stops at the water content at which the surface of soil becomes lighter and this water content is known as the shrinkage limit.

SLIDE

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

sector of the slope to fail by rotating.

SLICKENSIDES

Scratched surface, sometimes polished, resulting from shear movement.

SPECIFIC GRAVITY [D]

The ratio of the weight in air of a given volume of soil particles to the weight in air of an equal volume of distilled water at a temperature of 4°C.

STANDARD DEVIATION

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

STRESS [FL⁻²]

The force per unit area, when the area approaches zero, acting within a body. Effective Stress (Effective Pressure) (Intergranular Pressure), σ' [FL⁻²]: The average normal force per unit area transmitted from grain to grain in a granular mass. It is the stress that is effective in mobilizing internal friction. Field Stress [FL⁻²]: The stress existing in a rock mass independent of any man-made works. Residual Stress: Stress that exists in a formation owing to previously applied forces or deformations. Neutral Stress (Pore Pressure) (Pore Water Pressure) u , [FL⁻²]: Stress transmitted through the pore water (water filling the voids of the mass). Normal Stress [FL⁻²]: The stress component normal to a given plane. Principal Stress, σ_1 , σ_2 , σ_3 [FL⁻²]: Stresses acting normal to three mutually perpendicular planes intersecting at a point in a body, on which the shear stresses are zero. Major Principal Stress σ_1 [FL⁻²]: The largest (with regard to sign) principal stress. Minor Principal Stress, σ_3 [FL⁻²]: The smallest (with regard to sign) principal stress. Intermediate Principal Stress, σ_2 [FL⁻²]: The principal stress whose value is

neither the largest nor the smallest (with regard to sign) of the three. Shear Stress (Shearing Stress) (Tangential Stress), τ [FL⁻²]: The stress component tangential to a given plane. Total Stress, σ [FL⁻²]: The total force per unit area acting within a granular mass. It is the sum of the neutral and effective stresses.

STRESS CONCENTRATION [D]

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

STRIKE [D]

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

STRUCTURAL FEATURE

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

STRUCTURE

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

THICKNESS [L]

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

TOE

The bottom of a slope.

UNIT WEIGHT [FL⁻³]

Weight per unit volume.

Dry unit weight - that, when multiplied by the height of the overlying column of ground, yields the effective pressure due to the weight of overburden.

Saturated unit weight - The wet unit weight of

a granular mass when saturated.

Submerged unit weight - The weight of solids in air minus the weight of water displaced by the solids per unit volume of mass.

WASTE PILES

Refers to all waste embankments other than tailings or ore, resulting from a mining operation.

WAVINESS

Undulations of a rock surface on field scale.

WEATHERING

Breakdown of rock strength and mineral composition by chemical, organic and mechanical processes under atmospheric conditions.

WIDTH [L]

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

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

See Modulus of Elasticity.