ROCK MECHANICS OF THE BLACK STAR
OPEN CUT, MOUNT ISA

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the degree of
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by

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STATEMENT

The supporting papers submitted as Appendices D, E and F were done in collaboration with other people. My contribution to these, as well as other assistance received during the work, is clearly stated in the appropriate parts of the text and in the Acknowledgments. In all other respects, the work is entirely my own.

K.J. ROSENGREN.
ACKNOWLEDGMENTS

I am grateful to Professor J.C. Jaeger, my principal supervisor, for his guidance and encouragement throughout the course of this work. The geological aspects were supervised by Dr. C.B. Raleigh, and later by Dr. B.E. Hobbs, to whom I offer my grateful thanks.

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The field work was made possible by the cooperation and generous hospitality of Mount Isa Mines Limited, through Mr. E. Davies, Research Manager. In particular, I am indebted to Mr. K.E. Mathews and the Rock Mechanics Section for their helpful assistance in all phases of the work.

This thesis is dedicated to my wife. Without her patience and encouragement it would not have been possible.
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Chapter</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chapter 1</td>
<td>INTRODUCTION</td>
<td>1</td>
</tr>
<tr>
<td>Chapter 2</td>
<td>STATEMENT OF PROBLEM AND EXPLORATION METHODS</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>2.1 Outline of geology and mining methods</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>2.2 The proposed Black Star open cut</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>2.3 Exploration methods</td>
<td>14</td>
</tr>
<tr>
<td>Chapter 3</td>
<td>GEOLOGY AND MINERALOGY</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>3.1 Correlation between drill holes</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>3.2 Mineralogy</td>
<td>32</td>
</tr>
<tr>
<td>Chapter 4</td>
<td>STRUCTURAL ANALYSIS</td>
<td>39</td>
</tr>
<tr>
<td></td>
<td>4.1 Introduction</td>
<td>39</td>
</tr>
<tr>
<td></td>
<td>4.2 Stereographic projection</td>
<td>44</td>
</tr>
<tr>
<td></td>
<td>4.3 Analysis of bedding</td>
<td>51</td>
</tr>
<tr>
<td></td>
<td>4.4 Core orientation</td>
<td>55</td>
</tr>
<tr>
<td></td>
<td>4.5 Analysis of veins</td>
<td>58</td>
</tr>
<tr>
<td></td>
<td>4.6 Analysis of joints</td>
<td>62</td>
</tr>
<tr>
<td>Chapter 5</td>
<td>DYNAMIC ANALYSIS OF STRUCTURAL FEATURES</td>
<td>77</td>
</tr>
<tr>
<td></td>
<td>5.1 Introduction</td>
<td>77</td>
</tr>
<tr>
<td></td>
<td>5.2 Principles of dynamic analysis</td>
<td>79</td>
</tr>
<tr>
<td></td>
<td>5.3 Dynamic analysis of features in core</td>
<td>88</td>
</tr>
<tr>
<td></td>
<td>5.4 Dolomite twin lamellae</td>
<td>95</td>
</tr>
<tr>
<td></td>
<td>5.5 Other structural features</td>
<td>105</td>
</tr>
<tr>
<td>Chapter</td>
<td>Title</td>
<td>Page</td>
</tr>
<tr>
<td>--------------</td>
<td>------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>Chapter 6</td>
<td>FRICTIONAL PROPERTIES OF ROCK SURFACES</td>
<td>111</td>
</tr>
<tr>
<td>6.1</td>
<td>Introduction</td>
<td>111</td>
</tr>
<tr>
<td>6.2</td>
<td>Available methods and previous results</td>
<td>115</td>
</tr>
<tr>
<td>6.3</td>
<td>Direct shear tests on artificially prepared rock surfaces</td>
<td>125</td>
</tr>
<tr>
<td>6.4</td>
<td>Further double shear tests on small blocks</td>
<td>128</td>
</tr>
<tr>
<td>Chapter 7</td>
<td>MEASUREMENT OF FRICTION IN THE TRIAXIAL APPARATUS</td>
<td>142</td>
</tr>
<tr>
<td>7.1</td>
<td>Introduction</td>
<td>142</td>
</tr>
<tr>
<td>7.2</td>
<td>Elementary theory</td>
<td>143</td>
</tr>
<tr>
<td>7.3</td>
<td>Sliding with lateral displacement</td>
<td>148</td>
</tr>
<tr>
<td>7.4</td>
<td>Sliding with rotation</td>
<td>154</td>
</tr>
<tr>
<td>7.5</td>
<td>Apparatus and techniques</td>
<td>161</td>
</tr>
<tr>
<td>7.6</td>
<td>Triaxial tests on artificial joints</td>
<td>167</td>
</tr>
<tr>
<td>Chapter 8</td>
<td>STRENGTH OF JOINTS AND ROCK MATERIAL</td>
<td>175</td>
</tr>
<tr>
<td>8.1</td>
<td>Frictional properties of joints</td>
<td>176</td>
</tr>
<tr>
<td>8.2</td>
<td>Strength of rock material</td>
<td>185</td>
</tr>
<tr>
<td>Chapter 9</td>
<td>LABORATORY EXPERIMENTS ON A MODEL JOINTED ROCK MATERIAL</td>
<td>198</td>
</tr>
<tr>
<td>Chapter 10</td>
<td>SUMMARY AND CONCLUSIONS</td>
<td>201</td>
</tr>
<tr>
<td>10.1</td>
<td>Structural analysis</td>
<td>201</td>
</tr>
<tr>
<td>10.2</td>
<td>Strength of discontinuities</td>
<td>204</td>
</tr>
<tr>
<td>10.3</td>
<td>Slope stability analysis</td>
<td>206</td>
</tr>
<tr>
<td></td>
<td>REFERENCES</td>
<td>223</td>
</tr>
</tbody>
</table>
Appendix A  PHOTOGRAPHS OF DRILL CORE  A.1
Appendix B  COMPUTER METHODS IN STRUCTURAL ANALYSIS  B.1
Appendix C  COMPUTER ANALYSIS OF FRICTION TESTS  C.1
Appendix D  THE MECHANICAL PROPERTIES OF AN INTERLOCKED GRANULAR AGGREGATE  D.1
Appendix E  SIMPLE PHOTOGRAPHIC EQUIPMENT FOR BOREHOLE SURVEYING  back pocket
Appendix F  A MEDIUM-SCALE DIRECT FRICTION EXPERIMENT  back pocket
Chapter 1

INTRODUCTION

The application of static rock mechanics to mining is primarily concerned with design and support of openings in rock. In an underground mine, the openings must be designed to permit maximum extraction of ore at minimum cost consistent with safe operating practices. An analogous situation occurs in an open cut mine. Here the support is provided by the side batters which must be excavated as steeply as possible to minimize overburden removal. However, an overriding requirement is that they must remain stable throughout the life of the mine to ensure safety of personnel and equipment and also in many cases to permit the ore to be fully recovered.

This thesis is concerned with rock mechanics investigations into the hanging wall batter of the proposed Black Star open cut at Mount Isa, Australia. As an example of the economics involved, if the overall batter of this slope, which is 550 ft deep and 2000 ft long, could be steepened from 45° to 50°, overburden removal would be reduced by $1.5 \times 10^6$ cu. yds., which represents a monetary saving of some $1.5$ million. The incentive to apply
rational design procedures to such problems are therefore very great.

It is only in recent years that stability calculations for slopes in hard rock have been attempted. A factor which has emerged from these investigations is the great importance of structural defects in the rock mass. This point was clearly stated by Terzaghi (1962) who calculated that for an intact rock with the quite low unconfined compressive strength of 5000 psi, a vertical slope of some 4000 ft height should be stable. The fact that no such slopes exist and that much smaller slopes in much stronger rock have failed indicates that the stability is determined by the defects in the rock mass and not by the strength of the rock itself.

Most rocks, particularly those nearer the surface, are traversed by joints and faults which divide the rock mass into a system of nearly perfectly packed blocks, the size and shape of which are determined by the geometry of the jointing. It has been suggested, Wilson (1959), that if the size of the block is small compared with the size of the slope, the rock mass can be considered a granular aggregate and conventional soil mechanics methods applied to stability calculations. While this is probably true for extensively weathered rocks it may not be so for extensively jointed fresh rocks since the geometry of
the voids and the interlocking properties of the aggregate are quite different. A series of laboratory experiments on aggregates of calcite crystals, described later in this thesis, indicates that the strength of such an aggregate is seriously underestimated by soil mechanics experience.

An alternative approach, pioneered by Muller in Austria, is the systematic measurement and statistical analysis of rock defects. If the geometry of these defects together with the strength properties of the defects are known, it should be possible to calculate the factor of safety of any rock structure, including a rock slope. For complex three-dimensional situations, the stress distribution and interaction between the defects in the rock mass may be difficult to determine. In other cases, the geometry of the defects may be sufficiently simple to permit the estimate of the factor of safety by relatively simple calculations.

The approach adopted in the present work is therefore as follows. A specific practical problem of slope stability in rock exists and an attempt must be made to analyse it on a rational basis. The solution of the problem may be simple, difficult or even impossible. The category into which the problem falls cannot be determined
until some preliminary investigations are carried out. These investigations form the subject of this thesis and such matters as the development of new methods of stability analysis, which are a complete study in themselves, are specifically excluded.

The thesis falls fairly naturally into two parts. The first, Chapters 2 to 5, is concerned with the geological aspects of measuring and interpreting the structure of the rock. Chapter 2 is an introductory one giving a brief outline of the geology and mining methods at Mount Isa, the statement of the particular rock mechanics problem of the Black Star open cut, and the techniques used in obtaining the required information by diamond drilling. Chapter 3 is concerned with the geology and mineralogy of the rock formations encountered during drilling and classifies them into several zones as a basis for the structural investigations. Chapter 4 describes the methods of core orientation and the results of analysis of structural features in the core. Chapter 5 deals with the dynamic interpretation of the structural features and gives estimates of probable in situ principal stress directions.

The second part, Chapters 6 to 9, is concerned with the strength of the rock mass. The strength of rock
joints arises primarily from the frictional resistance under applied normal stresses. The knowledge of rock friction in general is quite meagre and in Chapter 6 the available information is briefly reviewed and a series of sliding friction tests on artificial rock surfaces are described. For testing the frictional properties of natural joints in drill core, the triaxial method is well suited and this method is analysed in detail in Chapter 7, including the theory for large displacements on the slip plane. The method is applied to the joints in the Mount Isa core in Chapter 8, in which a large number of triaxial compression tests on solid cores is also described. Chapter 9 describes the laboratory experiments on a model randomly jointed rock mass previously mentioned.

Finally in Chapter 10, the two rather separate parts of the thesis are brought together. It will be shown the geometrical situation is fairly simple and that useful predictions on slope stability can be made without complex methods of analysis.

The amount of geological detail in this thesis is considerably greater than that usually associated with an engineering investigation of this nature. No apology is offered for this. Of all the engineering disciplines, rock mechanics
is the one most closely associated with geology and advances in rock mechanics, particularly in the case of near-surface rocks, must be retarded unless full use is made of the vast fund of relevant knowledge available in the geological sciences.

Throughout the course of this work, maximum use was made of computer facilities. This is particularly important in structural analysis where many thousands of measurements are made and analysed and the procedures are laborious and time consuming. A particularly powerful computer method of fabric analysis was developed; the method and the computer programs are described as Appendix B. The computer was also extensively used in the analysis of strength tests.

Some of the work has already been published or is in course of publication. To avoid duplication, this work is briefly summarised in the body of the thesis and the papers included as Appendices.
Mount Isa is situated in north-west Queensland, Australia, at latitude $20^\circ 43'\text{S}$ and longitude $139^\circ 30'\text{E}$. The deposit was discovered in 1923 and mining was commenced in 1931 by Mount Isa Mines Limited (M.I.M.). After early vicissitudes, the mine is now the largest underground mine in Australia, with an annual production approaching 5 million tons of copper and lead-zinc-silver ore.

In this chapter, the geology and mining methods are briefly reviewed, as background to the remainder of the thesis. The rock mechanics problem of the proposed Black Star open cut is then detailed and the methods of obtaining structural information from the hanging wall batter area are described.

2.1 OUTLINE OF GEOLOGY AND MINING METHODS

The most recent general accounts of the geology and mining methods are those of Bennett (1965) and Davies (1967), respectively, from which most of the following is abstracted.

The rocks containing the Mount Isa orebody are Lower Proterozoic in age and occur on the western edge of the Precambrian belt of north-western Queensland. Of particular interest is the Mount Isa Group, a 15,000 ft. thick conformable
sequence of siltstone, shale and bedded carbonates, striking north-south and dipping steeply west in the vicinity of Mount Isa. From west to east and youngest to oldest, the members of the group are

- Magazine Shale
- Kennedy Siltstone
- Spear Siltstone
- Urquhart Shale
- Native Bee Siltstone
- Breakaway Shale
- Moondarra Siltstone

They are conformably underlain by a great thickness of quartzite and metamorphosed volcanics and unconformably (?) overlain by the Western Volcanics or "greenstone", a chloritic, metamorphosed sequence of volcanics and sediments.

Structurally the Mount Isa Group is situated on the western limb of a large north plunging anticline, the axis of which trends N-S about 12 miles east of Mount Isa. About 2000 ft. west of the mine the formations have been truncated by the Mount Isa Fault which strikes N-S and dips steeply west. This fault has been traced for 40 miles and movement is thought to have been west block up and north. On the western side of this fault is a sequence of quartzites and schists which are probably the stratigraphic equivalents of the rocks underlying the Mount Isa Group to the east.

The lead-zinc-silver and copper orebodies are adjacent but quite separate and both are confined entirely to the
Urquhart Shale formation which is a thin-bedded dolomitic and pyritic shale. The lead-zinc-silver ore occurs as fine grained argentiferous galena and sphalerite together with abundant pyrite and pyrrhotite in the bedding planes of the shale, and there is general agreement that it has a sedimentary origin. The copper ore occurs as chalcopyrite in large masses of deformed and recrystallised rock known as "silica-dolomite", which slightly transgress the enclosing shales. Bennett (1965) also suggests a sedimentary origin for the copper, but this is still somewhat controversial.

Within the mine, folding is well developed with axial planes striking slightly west of north and dipping nearly vertically, and with amplitudes of 6 to 150 ft. Major faults are of two types. The first, mainly strike faults, tend to occur as fault zones. The second occur as single fault planes and include bedding plane faults and the transverse faults, which offset other structures and are regarded as the youngest large scale structural features of the area. In the Urquhart Shale, jointing is well developed, and is commonly oriented at right angles to the bedding. The Urquhart Shale is commonly completely oxidised to a depth of 175 ft and a zone of carbonate leaching extends to a variable depth below this, Smith (1966).

Mining is concentrated in four sections of the mine named after original leases in the area. The Black Star section in the northern part of the mine contains the Black
Star lead orebody and the 650 copper orebody, Fig. 2.1(a). In this area, the country rock is fresh and strong and the ore has been mined by sub-level open stoping. In the footwall, further to the east, are the narrow Racecourse lead orebodies which are being mined by cut and fill methods. In the Black Rock or central section, Fig. 2.1(c), there is a concentration of transverse faults which has permitted leaching of carbonate to depths in excess of 200 ft. The 500 copper orebody in this area has been affected by this and is weak and porous; it is currently being mined by a sublevel caving method. In this area, also, the Black Rock secondary copper orebody occurs relatively near the surface. This deposit was mined by open cut methods but the open cut was forced to cease operations 30 ft. short of the planned depth of 520 ft. due to instability of the hanging wall (western) batter. In the southern or Rio Grande section, Fig. 2.1(b), the very large 1100 copper orebody terminates against a greenstone basement some 3000 ft. below the surface. The host silica dolomite is fresh and strong and the orebody is currently being developed for open stoping methods.

2.2 THE PROPOSED BLACK STAR OPEN CUT

The Black Star lead orebody was one of the original outcrops discovered in 1923. It proved to be very large, although of rather low grade, and large scale mining commenced
Fig. 2.1. Sections through mine area (after Davies (1967)). (a) northern section. (b) southern section.
Fig. 2.1(c). Section through central mine area (after Davies (1967)).
in 1931. The upper carbonate ore was mined by glory-holing to a depth of 100 ft. Below this the sulphide ore was mined by open stoping although the stopes were subsequently dry-filled. The present interest in open cutting derives from the high grade ore left as pillars and low grade ore not considered payable in the early mining. With modern excavation techniques an open cut to recover this ore could be an economic proposition.

A locality plan is shown in Fig. 2.2 and a section in Fig. 2.3. The orebody is tabular in shape, some 200 ft. wide, striking north-south and dipping $65^\circ$ west. The proposal is to open cut to below No. 5 level, a total depth of 550 ft. and a total excavation of some 30 million tons of rock. The limits of the open cut are fixed fairly rigidly by the following considerations.

(i) The eastern and western batters must be formed so that the bottom of the open cut coincides with the width of the orebody at that depth.

(ii) The top of the southern batter is fixed at approximately 6000N (mine coordinate) by the main shaft pillar. It is proposed to abandon the top portion of the Davidson shaft, which would be situated within the southern batter.

(iii) The top of the northern batter is fixed at approximately 8200N by uneconomic mineralisation further north.
Fig. 2.2. Surface plan of mine area (after Smith (1966)).
Fig. 2.3. Geological section looking north at 7200N showing outline of open cut.
There is therefore little opportunity to vary the location of the open cut to suit rock mechanics considerations. This is a general feature of mining projects which contrasts them to civil engineering works. The rock mechanics problem therefore reduces to selecting the optimum overall slope angles for the batters in view of the economic considerations discussed in Chapter 1.

In this work, study was concentrated on the western or hanging wall batter for the following reasons.

(i) The eastern and western batters were more likely to have a stability problem because of their long unsupported lengths. The northern and southern batters would be relatively short and arch shaped. Experience has shown that slopes of this configuration have enhanced stability because of induced circumferential compressive stresses.

(ii) The eastern batter appears to be a straightforward problem since the bedding direction (65° dip to west) coincides with the normal bench slope. Experience in the Black Rock open cut showed that the Urquhart Shale stood well under these conditions providing that the bedding planes were not undercut.

(iii) The western batter had become unstable and caused the cessation of operations in the Black Rock open cut, Edwards (1967). The actual final slope of this batter was 35½° so that without further investigation a slope much greater than
$40^\circ$ could not confidently be designed for the Black Star open cut.

The experience with the Black Rock open cut could not be taken over completely because:

(i) The Black Rock open cut was in the zone of deep oxidation and leaching and the rocks were porous and weak. Edwards (1967) quotes values of 200 psi for the unconfined compressive strength of kaolinised shale. So far as was known this situation did not occur at the Black Star site.

(ii) The western batter of the Black Rock open cut was directly over the caving operations in the 500 orebody, Fig. 2.1(c), and it is likely that this accelerated the deterioration of the batter.

(iii) As shown in Fig. 2.2, the Black Star open cut is situated further west into the Spear Siltstone formation and this was thought to be much less closely jointed than the Urquhart Shale.

(iv) The Black Rock open cut was abandoned in the face of an approaching wet season. It is possible that large scale drainage measures may have alleviated some of the deterioration of the batter.

In view of these different conditions, further investigations were considered justified.
2.3 **EXPLORATION METHODS**

None of the underground workings penetrated the area under consideration and the surface outcrop is very poor. The only information available was that from exploration prior to 1930. A series of east declined diamond drill holes had been put down to intersect the west dipping ore-body at depth. However, at that time the barren country rock was considered of no interest and the core was not kept. The drill logs are uninformative with entries such as "0-500 ft: barren shale". Since this time there has been no exploration in the area. Westerly diamond drilling had been carried out from the lower workings, but as shown in Fig. 2.5 the rock here is mainly silica dolomite which gives no information on the shales above. It was therefore decided that further exploration was necessary.

Two basic methods of exploration were available, development or diamond drilling, with the possibility of a combination of the two. On a cost basis, drilling was more attractive provided that the required information could be obtained. Recently, borehole movie or television cameras have been used for examining the walls of boreholes for structural features, Muller (1963), Rausch (1965), Hoek and Pentz (1968). However, such an instrument was not available for this project. Another possibility is orientation of the core for mapping the structural features. Orientation of core from short
holes is a relatively simple matter using a diamond scriber on the core barrel and oriented drill rods. For the long holes envisaged here orientation of the drill rods is impractical. Special core barrels containing a surveying instrument are under development, Young (1965), Rowley et al. (1964), but were not available for this work. On the other hand, the rocks in the area appeared to be very uniformly bedded and it was hoped that this could be used for orienting most of the drill core.

Diamond drilling, with orientation of the core, if possible, was therefore selected as the exploration method.

**Layout of diamond drill holes**

To enable the structure to be mapped in three dimensions at least three non-parallel drill holes are required. Ideally these should be mutually perpendicular. However, there are further restrictions in hole direction.

The core is oriented by means of the elliptical traces of the bedding on the core. A circular trace, obtained from a core drilled normal to the bedding, can give no orientation and the closer the elliptical trace approaches a circle the less precise must be the computed orientation. It is considered that the angle between the bedding plane and the core axis should be not greater than $65^\circ$; this gives an elliptical trace with major axis 10 percent longer than minor axis. On
the other hand, a hole drilled nearly parallel to the bedding is not economical since it penetrates only a small thickness of strata per unit length. Ideally, therefore, each hole should be inclined at between about 30° and 60° to the bedding planes.

The possible orientations of drill holes satisfying these requirements for bedding striking N-S and dipping 65° west are shown by the unshaded area in a lower hemisphere equal area projection, Fig. 2.4. This method of projection is explained in Chapter 4.

Bearing these factors in mind, suitable drilling sites were then investigated. Drilling from the underground workings was favoured since this would be done by M.I.M. drillers and greater supervision was possible. Drilling from surface sites was normally done by a contractor. Drilling sites were rather limited because the workings in this area were old and often inaccessible. The only suitable sites were on No. 4 level, some 350 ft. below surface, where two cross cuts in the middle of the area were accessible. It was decided to drill two intersecting horizontal holes, one trending north-west and one south-west. The holes were to be carried out to the vertical projection of the proposed top of batter, requiring each to be approximately 800 ft. long. After these holes were completed it was decided that a further hole was
warranted to explore the third dimension. The only other suitable location was a hole from the surface dipping 55° due east, approximately down the proposed batter line, allowing for expected deviation of the hole. The three holes were denoted A, B, C, in order of drilling, and are shown in Figs. 2.2 and 2.3 and on the projection, Fig. 2.4. Details are summarised in Table 2.1

### TABLE 2.1

<table>
<thead>
<tr>
<th>Hole</th>
<th>Mine description</th>
<th>Location of collar</th>
<th>Planned orientation</th>
<th>Depth</th>
</tr>
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<tbody>
<tr>
<td>A</td>
<td>U69 NW horizontal</td>
<td>No. 4 level</td>
<td>315° 0°</td>
<td>837 ft.</td>
</tr>
<tr>
<td>B</td>
<td>U79 SW horizontal</td>
<td>No. 4 level</td>
<td>225° 0°</td>
<td>763 ft.</td>
</tr>
<tr>
<td>C</td>
<td>Z74 E decline</td>
<td>Surface</td>
<td>90° 55°</td>
<td>790 ft.</td>
</tr>
</tbody>
</table>

So that the work would be of maximum value, the drilling requirements were very stringent. Firstly, 100 percent core recovery was necessary in order to detect faults and other zones of weakness. Ideally, for core orientation purposes, the pieces of core should fit together to form a continuous length. Since this was not always possible it was essential for the core to be undamaged, in the correct order and with the sense of withdrawal from the hole known. This last point
is very important since if the bedding is the only marker on the core, the orientation is ambiguous unless the sense of the core is known.

These requirements were approached from two angles, firstly, by the use of sophisticated drilling equipment and secondly by close supervision of the drilling operation.

**Drilling equipment**

Mr. D.G. Moye of the Snowy Mountains Authority (S.M.A.) was approached for advice on this aspect. He considered that, since the rocks were likely to be reasonably fresh, good results should be obtained by the standard S.M.A. methods. The equipment requirements, Moye (1967) are as follows:

(i) An hydraulic feed drilling machine must be used. This type of machine maintains a constant pressure on the bit and automatically adjusts its rate of advance to suit the rock being drilled. A screw feed machine, commonly used in underground drilling, causes the bit to advance at a constant rate irrespective of the type of rock and soft material is washed away before it can enter the barrel.

(ii) The drilling machine should be as large as possible to minimize vibration.

(iii) The largest size standard core (N-size) should be used. Larger core has a much better chance of surviving the drilling operation. It is also more suitable for mechanical testing.
(iv) The core barrel should have a stationary inner tube and bottom discharge bits to minimize core grinding and erosion of the core by the drilling water.

(v) The core barrel should have a removable split inner tube to enable the core to be extracted without disturbance.

The requirements for the core barrel were met by the Triefus NMLC triple tube core barrel with steel split inner tube, Fig. 2.5. An even more sophisticated version with a retractable cutting shoe for unconsolidated or highly weathered material was not considered suitable for the mainly hard rocks expected. The outer tube, 1, is screwed to the drill rods and carries the bit, N19, and reamer, N21. The nominal diameters of the hole and core are 2.945 in. and 2.045 in. respectively. The second tube, 13, carries the core lifter, 19, and remains stationary during drilling by means of the bearings 53 and 54. Fitting snugly within the second tube is the steel split inner tube, 14. At the end of a drilling run, the bit and reamer are removed and the inner tube containing the core is forced out of the second tube by water pressure applied to the coupling, 61. The inner tube can then be disassembled and the core remains undisturbed in one half of the tube.

A Mindrill M30 hydraulic feed drilling machine was used for the major part of the drilling.
Fig. 2.5. TRIEFUS NMLC triple tube core barrel with steel split inner tube.
Drilling operations

The requirements for drilling and handling of the core were quite different from those for normal exploration drilling in the mine. For this reason the initial part of the drilling was continuously supervised by the author. After a period, the drillers were considered sufficiently reliable and the work proceeded on a two shift basis. Average advance was 10 to 15 ft. per shift and each hole took approximately one month to complete. After each run, before the core was removed from the split inner tube, each piece of core was marked with an arrow showing the direction down the hole. The core was then carefully transferred to wooden core boxes with lids. Wooden marker blocks were used to mark the depth at the end of each run. Pieces of core were not broken to fit in the core boxes which were 5 ft. long. The core boxes were taken to the surface where the depth was marked on the core in Indian ink at 0.5 ft. intervals and on each separate piece of core. The core was transferred to plastic trays and packed in crates for transport to Canberra.

Generally, the drilling performance was very good. The drillers were enthusiastic and cooperative. Unfortunately, when the drilling was due to commence, the hydraulic feed machine was not available due to breakdown. Since the project had been delayed several months by the industrial
dispute at Mount Isa in 1965, it was desired to start immediately and a Mindrill E1500 screw feed machine was used for the first 300 ft. of hole A. This resulted in rather severe core loss in certain sections. However, when the hydraulic feed machine was installed, core losses became negligible. The importance of a hydraulic feed for this type of rock cannot be over-emphasised.

The Triefus core barrel performed well except that the core was often damaged due to blockage of the barrel by jointed rock. If a joint inclined at a small angle to the hole was encountered the core tended to wedge as it entered the barrel. Some trials were made with a Mindrill NMS core barrel. This was a double tube barrel with the following features.

(i) The second tube was split and held together by spring clips.

(ii) The inside of the split tube was chrome plated.

(iii) The core lifter was chrome plated and smooth faced rather than splined as is the usual design.

These features permitted this barrel to perform very well in jointed rock. The spring clips gave some flexibility to the inner tube and the chrome plating reduced friction between the barrel and the core. It was not extensively used since it cuts a much smaller core, 1.862 in. diameter.
However, a similar type of barrel cutting a full size NX core has now become available and would be the choice for future work.

At Canberra, the core was carefully pieced together and marked with a reference line. The core was photographed in 100 ft. intervals, from a height of 15 ft. to minimize photographic distortion. The core was laid out in five foot lengths with the foot marks coinciding to highlight core losses. The photographs are included as Appendix A.

Borehole surveying

Most diamond drill holes longer than 100 ft. tend to deviate from their planned direction, particularly if the rock is anisotropic. The mechanism of borehole deviation is not well understood, Logn (1965), but it is generally found that holes deviate towards the normal to bedding or schistosity; this is certainly past experience at Mount Isa. If the holes are drilled parallel to bedding they are unstable and may deviate in any direction.

There are certain techniques, such as stabilised core barrels and oversize rod couplings, designed to minimize deviation but these complicate the drilling. In the present work, the direction of the holes was not very critical and it was considered preferable to accept deviation and survey the holes when completed. A fairly accurate knowledge of
hole direction is required for core orientation.

Holes A and B were collared with a slight upwards inclination and their inclination at depth was measured by the simple method of pumping water through the drill rods and measuring the static head at the collar with a sensitive pressure gauge. In both holes, the inclination varied from 0 to 2 degrees.

Survey of the bearing of these holes was first attempted by a Tropari, the standard borehole surveying method at Mount Isa at that time. The Tropari is a small gimbal mounted magnetic compass with a clockwork mechanism which locks the compass needle and inclinometer at a preset time. It is common practice to use two or three instruments together as a mutual check on accuracy. Hole A was surveyed but gave very inconsistent results which were not satisfactory. It is thought that the delicate instruments were upset by the jerking which occurs during insertion and withdrawal from horizontal holes. There are also basic objections to locking of the magnetic needle, Elliston (1965).

Sophisticated multi-shot photographic surveying instruments have now been developed but at the time such an instrument was not available at Mount Isa. For this reason a simple photographic instrument was built at the Australian National University (A.N.U.). This instrument was designed
by Professor J.C. Jaeger and manufactured by Mr. W. McIntyre. The testing, calibration and field measurements were done by the author. The description of this and another instrument has been published and a reprint is included as Appendix E.

The deviations of holes A and B are shown in Figs. 2.6, (a) and (b). Unfortunately, Hole B had collapsed at a depth of 350 ft. and could not be surveyed past this point. There is some scatter in the readings since the mounting of the compass needle at that time was not entirely satisfactory; this has since been improved by the use of jewelled bearings. However, there is a clear indication that both holes deviated towards the bedding normal. Uniform deviations have been assumed and 1.5 and 2 degrees per 100 ft, respectively, best fit the result for holes A and B.

Hole C was satisfactorily surveyed by Tropari since the instrument could be smoothly inserted and withdrawn on a cable. The bearing of the hole gave a negligible deviation but the dip flattened from $57^\circ$ at the collar to $39^\circ$ at the bottom, again deviating towards the bedding normal. The dip of the hole was checked with the A.N.U. instrument, which gave identical results. However, being a multishot instrument it produced the results much more rapidly.
Fig. 2.6. Deviation (D) with depth (d) for the three holes.
Chapter 3

GEOLOGY AND MINERALOGY

In this chapter, the three drill holes are correlated to give a broad picture of the geology of the area and the rocks are divided into six zones as a basis for structural analysis. Structural features are only briefly mentioned since they are considered fully in Chapters 4 and 5. The mineralogy of the rock types is also briefly considered here.

3.1 CORRELATION BETWEEN DRILL HOLES

A cursory examination of the drill cores reveals that several different rock types can be recognised. The procedure adopted here was to locate boundaries between the rock types in each hole visually and these boundaries together with certain marker beds permitted correlation between the three holes.

The correlation is shown in plan in Fig. 3.1 and in section in Fig. 3.2. In each case the information from the present drilling was incorporated with existing information from the M.I.M. plan of No. 4 level and a section in a M.I.M. report by A.J. Weil (1965) respectively. The rocks were divided into six zones of approximately uniform rock type and these are summarised in Table 3.1 together with the various markers. Detailed drill logs are not presented since it is
Table 3.1

Summary of drill hole correlations

<table>
<thead>
<tr>
<th>WEST</th>
<th>Hole depth</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>A</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td></td>
<td>Massive light grey siltstone sedimentary (?) breccia</td>
<td>-</td>
<td>700</td>
<td>-</td>
</tr>
<tr>
<td>Zone 6</td>
<td>Thin bedded grey slate</td>
<td>700</td>
<td>565</td>
<td>190</td>
</tr>
<tr>
<td>Zone 5</td>
<td>Sedimentary breccia</td>
<td>660</td>
<td>535</td>
<td>220</td>
</tr>
<tr>
<td>Zone 4</td>
<td>Sheared black slate, bedding obliterated</td>
<td>-</td>
<td>450</td>
<td>375</td>
</tr>
<tr>
<td>Zone 3</td>
<td>Medium bedded black slate, grey mudstone and siltstone</td>
<td>240</td>
<td>218</td>
<td>510</td>
</tr>
<tr>
<td>Zone 2</td>
<td>Blue grey dolomitic siltstone</td>
<td>138</td>
<td>131</td>
<td>588</td>
</tr>
<tr>
<td>Zone 1A</td>
<td>Thin bedded non-pyritic shale with sedimentary structures</td>
<td>75</td>
<td>58</td>
<td>634</td>
</tr>
<tr>
<td>Zone 1B</td>
<td>Thin bedded pyritic shale (silica dolomite) (pyrite rib)</td>
<td>682</td>
<td>740</td>
<td>770</td>
</tr>
</tbody>
</table>

EAST
considered that the photographs of the core included as Appendix A, together with the following general description provide more graphic information.

**Marker beds**

Tuff Marker Beds (TMBs) are distinctive beds widely used for stratigraphic correlation at Mount Isa. They are of volcanic origin, Croxford (1964), and consist of 70 to 80 percent potash feldspar (microcline ?). They have a distinctive cherty appearance with characteristic "cross fractures" filled with quartz, dolomite or sulphides. Most are a few inches thick and individual beds have been traced over a strike length of 6000 ft. in the mine workings. A suspected TMB can be checked by etching with hydrofluoric acid and staining with sodium cobaltinitrite. A high potash feldspar content will be registered by a bright yellow stain.

In the drill core, the following TMBs were recognised.

(i) Two 1 in. TMBs approximately two feet apart in Hole C at depth 682 ft. These beds are associated with extremely pyritic shale and are correlated with two TMBs mapped in the hanging wall drive, Fig. 3.1. They were not encountered in holes A and B since these holes are collared further west.

(ii) A 2 in. TMB associated with the pyrite band, 647 ft. hole C, 53 ft. hole B. It could not be detected in hole A since the core here is oxidised and broken.
(iii) A 2 in. TMB at the boundary between Zones 1 and 2. This was detected in all three holes and also mapped in the V79 cross-cut, Fig. 3.1. It is considered by this writer to be the boundary between the Urquhart Shale and Spear Siltstone. The M.I.M. geological plan shows the boundary some 200 ft further west at this level.

(iv) A ¼ in. TMB at the boundary between Zones 3 and 4. This was detected in holes A and B. It is not present in hole C but there is a distinctive sequence of light and dark beds, around the TMB in the other holes, which definitely mark its position.

The other main marker bed is a band of interbedded pyrite and dolomite encountered in all three holes and also mapped in the V79 cross-cut, Fig. 3.1. This has been called the "pyrite band" to distinguish it from the "pyrite rib" a similar marker on the immediate hanging wall of the Black Star orebody, Weil (1965). This pyrite band also outcrops in the western wall of a trench excavated along the hanging wall of the orebody, Figs. 3.1 and 3.3(b). An interesting feature of this band is the core discing shown in hole C, 640 - 645 ft. This is similar to the discing shown by drill core from highly stressed areas, Jaeger and Cook (1963), Leeman (1964), Obert and Stephenson (1965). Since the depth of this core is only 500 ft below surface, it would be
Fig. 3.3. Photographs of trench on hanging wall of Black Star orebody. (a) eastern wall. (b) western wall.
surprising if high ground stress was the cause in this case. It is possible that this particular rock type is extremely susceptible to such discing under quite low stresses, but it is suspected that something in the drilling technique, such as excessively high bit pressure, is more likely to be the cause.

The zones into which the rocks were divided will now be described in more detail. Representative samples of core from the various zones are shown in Fig. 3.4.

**Zone 1**

This is typical thin bedded Urquhart Shale. It has been divided into two parts, Zone 1A and Zone 1B east and west of the pyrite band respectively. The shale in Zone 1A is very pyritic and the bedding planes are remarkably planar. Some bedding planes are very dark in colour and are probably rich in graphite. In Zone 1B the shale is non-pyritic, in the sense that there are no beds of pyrite, and sedimentary structures, notably current bedding and flame structures, Pettijohn and Potter (1964), are very common. These structures confirm that the bedding is not overturned, Croxford (1964), and give the bedding a contorted appearance.

In hole A, this zone is generally leached and there are several patches of complete oxidation, where the rocks are various shades of brown, red, yellow and white, Smith (1966),
Fig. 3, 4. Examples of core from various zones.
in spite of the fact that it is some 200 ft below the base of complete oxidation in this area. Core recovery in these sections was poor and the core was badly broken since at this time the screw feed drilling machine was being used. Hole B is quite fresh while parts of hole C appear leached. Typical exposures in the oxidised zone are shown in Fig. 3.3.

Hole C encountered silica dolomite from 740 to 770 ft. This is white "crystalline dolomite" with no relict bedding or fragments of shale and is the top of the 650 copper orebody host, Fig. 2.3. From 770 to 780 ft very pyritic bedded shale may represent the pyrite rib, Fig. 3.2, while from 780 ft to end of hole the bedding is crenulated, with abundant galena and sphalerite. This is the Black Star orebody.

Zone 2

The top of Zone 1 is defined by the 2 in. TMB and quite a distinct change in rock type. Zone 2 is a coarse grained blue-grey dolomitic siltstone with abundant dolomite and quartz veins. The rock is alternately thickly and thinly bedded, some beds being several feet thick and imparting a massive appearance to the core. The thin beds are carbonaceous and graphite coated bedding plane faults are very numerous. Stylolites, pressure solution phenomena common in carbonate rocks, Pettijohn (1957), are also very numerous. These are thin, dark, "wrinkled" planes parallel to bedding.
This rock is quite different from the Urquhart Shale and it is therefore considered that the 2 in. TMB is a convenient boundary between Urquhart Shale and Spear Siltstone.

Towards the top of this zone approximately 20 ft of completely oxidised rock was encountered in both holes A and B. This was thought to be the expression of a strike fault but no evidence of oxidation was detected at the expected depth in hole C. It is, therefore, probably only a local deep depression in the zone of complete oxidation down favorable bedding planes.

Zone 3

The lower boundary of Zone 3 is not well defined but is evidenced by finer grained rock than in Zone 2. The rock types of Zone 3 are siltstone, mudstone and slate, and become increasingly darker in colour towards the west. A feature of these rocks is the very large number of closely spaced parallel veins striking north south and dipping nearly vertically. These are discussed in more detail in Chapter 4.

Bedding is fairly uniform at about one foot spacing and the black slate in the western part of the zone has well developed cleavage which transgresses the bedding.

The eastern part of hole A in this zone is extensively leached and since the screw feed drilling machine was being used, core losses were severe and the core recovered is very
broken. However, this leached rock would be excavated and would not be present in the open cut batter at this level.

Zone 4

There is again no definite change in rock type between Zones 3 and 4 and the 1/4 in. TMB is taken as the boundary mainly because it is a convenient marker. The rock of Zone 4 is entirely a black slate with prominent cleavage and many dolomite-quartz veins as in Zone 3. Slightly west of the TMB the bedding is almost completely obliterated by these veins and the rock appears to have been severely deformed. This is best shown in hole C (refer photographs in Appendix A).

Zone 5

Zone 5 is a relatively narrow band of breccia. Part of this is a sedimentary or "intraformational" breccia, Pettijohn (1957), consisting of angular fragments of bedded shale and some patches with uniform thin bedding. Part of the breccia also contains large irregular blebs of coarse grained dolomite indicating that some remobilisation has taken place.

Zone 6

Zone 6 is a thin bedded light grey slate. The cleavage has the same orientation as in Zones 3 and 4 and transgresses the rather uniform thin bedding. In some parts the bedding
is distorted by irregular blebs of fine grained quartz.
In holes A and B the parallel, closely spaced dolomite veins are very prominent.

The only hole to fully penetrate Zone 6 is hole B. The western boundary is marked by another narrow band of breccia followed by massive light grey siltstone, with a few irregular veins.

3.2 MINERALOGY

The following is by no means a complete study of the mineralogy of the rocks but rather a general account of the changes in the proportions of major minerals through the sequence, which can later be related to the mechanical properties of the rocks. No attempt was made to study rarer minerals. There are two aspects of the mineralogy: (a) of the parent rock and (b) of the vein fillings. The vein fillings are generally coarse grained and can be studied adequately in thin section. On the other hand, the parent rock is generally very fine grained and microscopic study is difficult. For this reason, the X-ray diffraction method was applied.

X-ray diffraction measurements

The wavelength of X-rays is of the same order as the unit cell dimensions of crystals which therefore diffract X-rays. If the angle of incidence of the X-rays on the
crystal lattice satisfies the Bragg equation

\[ n \lambda = 2d \sin \theta \]

where
- \( n \) is a positive integer
- \( \lambda \) is wavelength of incident X-rays
- \( d \) is spacing between crystal planes
- \( \theta \) is angle between X-ray beam and the diffracting crystal plane

the diffracted rays are in phase and reinforce each other to give a high intensity.

In an X-ray diffractometer, Barrett and Massalski (1966), a sample is rotated at constant rate relative to an incident beam of nearly monochromatic X-rays. Rotating at exactly twice the speed of the sample is a detector, in the present case a proportional counter. When the Bragg equation is satisfied for a particular crystal plane spacing, a peak of diffracted X-ray intensity is recorded by the detector, which is transmitted via suitable circuits to a strip chart recorder. Each mineral has a distinctive combination of crystal plane spacings which are recorded as peaks on the record and which allow the mineral to be identified.

The apparatus used was a Philips PW1010 X-ray generator with PW1050 wide range goniometer and associated recording equipment. The experimental conditions were as follows:
CuKα radiation with Ni filter
X-ray tube operating at 40 kv and 16 ma.
Divergence slit 1° and receiving slit 0.2°.
Ratemeter range $4 \times 10^2$ and time constant 2 sec.
Scan speed 1 deg. $2\theta$/min.

Samples for analysis were in two forms (a) a powder compacted into an aluminium holder and (b) a slab of rock of the same size as the powder sample. Provided the grains are randomly oriented, the intensity of the peaks on the diffractometer chart should be proportional to the amount of mineral present. However, due to the effect of differential absorption of the X-rays in mixtures of different minerals, this relation holds only approximately.

From published work and examination of thin sections, six major minerals were likely to be present. These are dolomite, quartz, albite, microcline, chlorite and mica. A summary of X-ray determinations of samples in powder form unless otherwise noted, of samples from the different zones is given in Table 3.2. The various peaks are ascribed values of strong (S), medium (M), weak (N) or absent (-) and give an approximate indication of the proportions of minerals present, except for chlorite and mica which are affected by preferred orientation. No significant peaks of other minerals were detected.

The major minerals are quartz and dolomite. Both albite
Table 3.2
Summary of X-ray determinations

<table>
<thead>
<tr>
<th>Zone</th>
<th>Hole and depth</th>
<th>Description</th>
<th>Quartz</th>
<th>Dolomite</th>
<th>Albite</th>
<th>Microcline</th>
<th>Chlorite</th>
<th>Mica</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>B 6.4</td>
<td>pyritic shale*</td>
<td>S</td>
<td>M</td>
<td>-</td>
<td>M</td>
<td>-</td>
<td>W</td>
</tr>
<tr>
<td></td>
<td>A 29.5</td>
<td>pyritic shale*</td>
<td>M</td>
<td>-</td>
<td>W</td>
<td>W</td>
<td>W</td>
<td>W</td>
</tr>
<tr>
<td></td>
<td>B 62.8</td>
<td>non-pyritic shale</td>
<td>S</td>
<td>S</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>M</td>
</tr>
<tr>
<td></td>
<td>A126.0</td>
<td>non-pyritic shale (oxidised)</td>
<td>S</td>
<td>-</td>
<td>-</td>
<td>M</td>
<td>M</td>
<td>M</td>
</tr>
<tr>
<td>2</td>
<td>A159.1</td>
<td>siltstone</td>
<td>M</td>
<td>S</td>
<td>W</td>
<td>M</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>A179.0</td>
<td>siltstone</td>
<td>M</td>
<td>S</td>
<td>M</td>
<td>M</td>
<td>W</td>
<td>W</td>
</tr>
<tr>
<td></td>
<td>A181.0</td>
<td>siltstone</td>
<td>M</td>
<td>S</td>
<td>W</td>
<td>M</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>B164.4</td>
<td>siltstone</td>
<td>M</td>
<td>S</td>
<td>M</td>
<td>W</td>
<td>-</td>
<td>-</td>
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<td>3</td>
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<td>M</td>
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<td></td>
<td>A361.0</td>
<td>mudstone</td>
<td>M</td>
<td>S</td>
<td>M</td>
<td>W</td>
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<td>A391.5</td>
<td>black slate</td>
<td>S</td>
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<td>M</td>
<td>W</td>
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</tr>
<tr>
<td></td>
<td>B373.5</td>
<td>black slate-powder</td>
<td>S</td>
<td>M</td>
<td>W</td>
<td>W</td>
<td>M</td>
<td>M</td>
</tr>
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<td></td>
<td></td>
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<td></td>
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<td>S</td>
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<tr>
<td>4</td>
<td>A500.2</td>
<td>black slate</td>
<td>S</td>
<td>M</td>
<td>W</td>
<td>W</td>
<td>M</td>
<td>M</td>
</tr>
<tr>
<td></td>
<td>B430.0</td>
<td>black slate</td>
<td>S</td>
<td>M</td>
<td>W</td>
<td>W</td>
<td>M</td>
<td>M</td>
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<tr>
<td></td>
<td>C284.7</td>
<td>black slate</td>
<td>S</td>
<td>M</td>
<td>M</td>
<td>W</td>
<td>W</td>
<td>W</td>
</tr>
<tr>
<td>6</td>
<td>A801.0</td>
<td>grey slate slab</td>
<td></td>
<td></td>
<td>S</td>
<td>M</td>
<td>-</td>
<td>W</td>
</tr>
<tr>
<td></td>
<td></td>
<td>slab*</td>
<td>S</td>
<td>M</td>
<td>-</td>
<td>W</td>
<td>W</td>
<td>W</td>
</tr>
<tr>
<td></td>
<td>C140.3</td>
<td>grey slate slab*</td>
<td>S</td>
<td>M</td>
<td>-</td>
<td>W</td>
<td>S</td>
<td>S</td>
</tr>
<tr>
<td></td>
<td></td>
<td>slab*</td>
<td>S</td>
<td>M</td>
<td>-</td>
<td>W</td>
<td>W</td>
<td>W</td>
</tr>
<tr>
<td>(7)</td>
<td>C762.0</td>
<td>grey siltstone</td>
<td>S</td>
<td>W</td>
<td>M</td>
<td>W</td>
<td>W</td>
<td>M</td>
</tr>
</tbody>
</table>

* also peaks for pyrite, galena, sphalerite
and microcline are normally only minor constituents. The strong peaks shown by quartz are rather surprising since Bennett (1965) quotes quartz as being "minor" in the Spear Siltstone. To check whether quartz gives rise to peaks out of proportion to its quantity, several artificial mixtures of quartz and dolomite were prepared and X-rayed. For pure quartz and dolomite the major peaks, corresponding to spacings of 3.32Å and 2.88Å respectively, are of approximately equal size and in the mixtures are closely proportional to the composition of the mixture. Pure albite and microcline powders also give main peaks, at 3.18Å and 3.23Å respectively, of the same order. It must therefore be concluded that the X-ray results reflect the approximate proportions of these minerals.

The case of the platy minerals, chlorite and mica, is quite different. The proportions of these minerals increase in the rocks towards the west but powder samples show at best only moderate peaks. To examine the influence of these minerals on the well developed cleavage in the rocks west of Zone 3, small slabs were cut parallel and perpendicular to the cleavage plane. The faces of the slabs were ground flat and X-rayed in the usual manner. Three typical diffractometer records are shown in Fig. 3.5. The powder sample shows moderate peaks for chlorite and mica while slabs parallel
Fig. 3.5. Diffractometer charts for X-ray analysis of slate. Hole B, 373.5 ft. (Q = quartz, D = dolomite, C = chlorite, M = mica).
and perpendicular to the cleavage plane show very strong and very weak peaks respectively.

This is very convincing evidence that the cleavage in these rocks is caused by preferred orientation of these platy minerals parallel to the cleavage plane since the major X-ray peaks are various order reflections from the (00l) basal planes. The prominent peaks at 4.48, 4.44, 2.58 and 2.56Å for mica in the perpendicular slab are reflections from planes at a high angle to the base and which therefore tend to be at high angles to the cleavage plane. The cleavage would therefore be defined as "slaty cleavage", Turner and Weiss (1963). From the relative size of the peaks it appears that chlorite is more abundant than mica but this is not definite because of the preferred orientation effect. The peaks for quartz, dolomite, microcline and albite are much the same in all three records indicating that there is negligible preferred orientation of these minerals. Calcite was not detected in any of the samples examined.

Further evidence of the influence of platy minerals on the cleavage is shown by the scanning electron micrograph of a cleavage surface of the slate, Fig. 3.6. This micrograph was prepared by Dr. K.A. Gross, Defence Standards Laboratories, Melbourne and is reproduced with appreciation. The arrangement of platy minerals parallel to the cleavage surface are
Fig. 3.6. Scanning electron micrograph of cleavage surface in slate. Hole B, 373.5 ft. Magnification: X8500. (Photo by courtesy of Dr. K.A. Gross).

Fig. 3.7. Pressure shadows around pyrite cubes in slate. Hole A, 511.5 ft. Width of field 9 mm. (crossed nicols).
clearly visible. The mechanical properties of these rocks are therefore likely to be greatly influenced by this preferred orientation of chlorite and mica.

**Microscopic examination**

Study of thin sections, as far as it was possible, generally confirmed the X-ray determinations. The grain size of the coarsest grained rock, the siltstone of Zone 2, was of the order of 10 microns and most of the other rocks were much finer grained so that microscopic examination was difficult. Study was therefore restricted to coarse grained inclusions and the numerous veins. By far the most prominent vein mineral is dolomite with quartz and chlorite also important. Mica is relatively rare but where present is faintly pleochroic and so is probably phlogopite. The veins are discussed in greater detail in Chapters 4 and 5. In thin section, pyrite is ubiquitous and commonly occurs as euhedral crystals with "pressure shadows", Hills (1963), of oriented quartz and mica. A particularly fine example from Zone 4 slate is shown in Fig. 3.7.

**Staining methods**

Staining to detect potash feldspar in TMBs has already been mentioned. Staining is also a popular method for identifying carbonate minerals and a limited amount was done using the technique of Dickson (1965). This involves etching
with 1.5 percent HCL and staining with a mixture of alizarin red S and potassium ferricyanide in HCL, which stains calcite pink and iron-rich dolomite turquoise. In fact, no calcite was detected but the method proved very useful for distinguishing between quartz and dolomite veins in the core. I am grateful to Mr. H. Berry for preparing the reagents.

Summary

The geological picture deduced from the drilling is that of a uniformly dipping sequence of rocks ranging from shale to mudstone, siltstone and slate. They are sedimentary rocks which have been subjected to low grade metamorphism. The widespread occurrence of chlorite, together with albite and the presence of slaty cleavage, indicates that they belong to the greenschist facies, characteristic of low temperature, moderate pressure, regional metamorphism, Turner and Verhoogen (1960). The formation name Spear Siltstone is largely a misnomer since, in the area studied, it consists mainly of slate.
Chapter 4

STRUCTURAL ANALYSIS

4.1 INTRODUCTION

This chapter is concerned with the measurement, geometry and description of discontinuities in the rock. This type of work has been carried out in structural geology for many years but it is only fairly recently that it has been systematically applied to engineering problems. The requirements are also somewhat different, mainly in respect to the amount of detail involved. The most logical and systematic approach appears to be that of Muller (1963). A convenient precis of the fundamentals of the method is given by John (1962).

Firstly it is necessary to distinguish between the rock material and the rock mass. Rock material refers to the aggregate of mineral grains composing the rock, free from any defects such as bedding planes, joints, faults, etc. Apart from large bodies of some igneous rocks, rock material only exists in the form of rather small specimens. Rock mass refers to any body of rock complete with the defects considered above. As discussed in Chapter 1, the engineering behaviour of a rock mass is largely determined by the presence of these defects. The aim of engineering structural analysis is to systematically map and describe the defects so that the behaviour
To avoid future confusion, it seems worthwhile to define certain terms. The term 'joint' does not seem to have a clear definition in structural geology. Most textbooks, e.g. Hills (1963), Billings (1954), define a joint as a plane of separation in the rock along which there has been no visible displacement. If there has been visible displacement, the plane is classified as a 'fault'. However, if there are no markers it is not possible to say whether there has been displacement or not and the feature must logically be classified as a joint even though there may be slicken-sides indicating that it is actually a fault, Hills (1963).

To avoid this difficulty, in this thesis a joint is defined as any plane in the rock across which there is complete loss of cohesion. This definition implies no mechanism of formation. However, where there is evidence available, it is possible to classify some joints as extension fractures or some joints as faults. Following Griggs and Handin (1960) an extension fracture is defined as separation of a body across a surface normal to the direction of least principal stress with no offset parallel to the fracture surface. A fault is a localised offset parallel to a more or less plane surface of non-vanishing shear stress; this definition implies that all faults are not necessarily joints since faulting can...
occur without loss of cohesion.

On the other hand, a **vein** is defined as a planar feature of finite thickness within which there is material which differs from the surrounding rock either in grain size or composition or both. This definition again implies no mechanism of origin. Veins are of two main types:

(i) **Dilatational**, in which there is a component of displacement normal to the walls. This may be an open joint (extension fracture or fault) which has subsequently been filled and recemented, or the walls of the vein may have been forced apart by the entrance of the filling material.

(ii) **Replacement**, in which the parent rock near the walls of the vein has not suffered any relative displacement. It would normally only be possible to prove that a vein was a replacement type if an original marker passed through the vein.

In considering the effect of defects on the strength and deformation of a rock mass, the following characteristics may be important to a greater or lesser degree:

(i) whether defects are singular, such as a major fault, or whether they occur in families either systematically or randomly oriented;

(ii) nature of the defects, whether joints, veins, bedding planes, cleavage planes, etc;

(iii) orientation in space of the various families of defects;
(iv) spacing between defects in each family;
(v) continuity of individual defects in each family;
(vi) features of each defect such as
   (a) overall planarity,
   (b) small scale roughness of the surfaces,
   (c) coating on surface, if a joint,
   (d) type of filling, if a vein,
   (e) type of fracture, extension or shear;
(vii) strength of defect relative to that of the rock material.

Many of these characteristics are not independent. For example, faults tend to be more continuous and have different types of surfaces from extension fractures. The strength of the defects is the subject of later chapters of the thesis and will not be discussed further here. It is submitted, however, that in relation to any stability problem, the orientation of the defects is their most important property. If the defect is favorably oriented relative to the structure, the deleterious effects of the other properties are largely nullified. On the other hand, if the defect is unfavorably oriented, these effects are often exaggerated. The orientation of the defects has therefore received prime attention in this investigation.

A diamond drill core is a very small sample of the rock.
mass it penetrates. Its small size must inevitably make extrapolation of rock properties between core and rock mass very difficult. However, in some respects it is a very good sample since, provided core recovery is complete and at least two non-parallel holes are drilled, it gives a continuous and unbiased sample of the rock drilled. In any other method of exploration such as shaft or tunnel excavated by explosives, there is a very large number of fractures exposed on the walls, many of which are caused by blasting and unrelated to the defects in the rock mass. In such a case, short of measuring every visible discontinuity, which is usually impractical, considerable experience and judgement is necessary for an unbiased selection of those defects likely to influence the properties of the virgin rock mass. Of course, these larger openings have certain advantages over drill holes, notably in estimation of continuity of defects. The optimum method in terms of current technology is probably a drilled tunnel excavated by a mechanical mole.

The approach adopted in this work was to measure and classify every discontinuity encountered in the drill core. These were of three major types:

(i) joints;
(ii) veins;
(iii) planes of anisotropy, notably bedding and cleavage.
A fundamental concept of Muller's approach is the existence of so-called "homogeneous zones" with comparable geological and mechanical properties such as rock type, degree of weathering and rock structure. In this project, the six zones discussed in Chapter 3 fulfil these requirements and the information on rock defects was collated and analysed for each zone in each hole.

The remainder of this chapter is concerned with this analysis. However, since the stereographic projection is basic in all of this work, it is firstly described with particular reference to the special requirements in analysing drill core. This is followed by an analysis of bedding orientation and the method of orienting the drill core.

4.2 STEREOGRAPHIC PROJECTION

As a preliminary, certain definitions are given.

Strike of a plane is the direction of the line of intersection of the plane with a horizontal plane.

Dip of a plane is the angle between the plane and the horizontal plane, which is measured in the vertical plane at right angles to the strike. The sense of dip must also be stated. In this thesis, planes are specified by the parameters strike/sense/dip, e.g. 127NE35.

Trend of a line is the strike of the vertical plane containing the line.
Plunge of a line is the angle between the line and the horizontal plane measured below the horizontal in the vertical plane containing the line. There is no need to specify sense of plunge in this case.

In this thesis, lines are specified by the parameters trend/plunge, e.g., 348/42.

The stereographic projection is an extraordinarily powerful and elegant graphical method for the solution of problems in three dimensional geometry, widely used by geologists but practically unknown to engineers. It is extensively used throughout the thesis.

It is one type of spherical projection in which points on the surface of the sphere are projected onto the equatorial plane from a fixed point on the surface of the sphere. In structural geology, points on the lower hemisphere, only, are projected from the zenithal point, Z. P' is the stereographic projection of point P on the surface of the sphere, Fig. 4.1(a). Lines project as points, planes passing through the centre of the sphere project as great circles and all other planes project as small circles. The projection of a plane, 060SE30, is shown in Fig. 4.1(b), and P is the pole of the plane, that is, the projection of the normal to the plane. The circumference of the projection is called the primitive.

The power of the method lies in its ability to manipulate
Fig. 4.1. The stereographic projection
lines and planes in three dimensions. As simple examples, the line of intersection, \( L \), of two planes is given immediately by the intersection of the two great circles defining the planes, Fig. 4.1(c), and the angle, \( \theta \), between two planes is given by the angle between the respective poles measured along the great circle, Fig. 4.1(d). In practice, constructions are made on a transparent sheet revolving on a net ruled with meridional great circles and parallel small circles at 2° intervals. The methods are concisely described by Phillips (1960).

**Equal area projection**

A major disadvantage of the stereographic projection is that equal areas on the surface of the sphere do not necessarily project as equal areas. This makes the projection unsuitable for the statistical analysis of a large number of lines or planes, which is necessary in most problems of structural geology. For this reason a distortion of the stereographic projection, known as the equal area projection, is widely used. Its geometrical basis is given by Terzaghi (1965). The areal correspondence is achieved at the expense of geometric distortion since circles on the sphere in general project as ellipses, Vistelius (1966). However, if an equal area net is used, all the constructions available on a stereographic projection can equally well be carried out on
an equal area projection.

When a large number of planes are to be displayed on a projection it is convenient to plot only the poles to give a "scatter diagram" Fig. 4.2(a). The preferred orientation of the poles can best be observed by contouring their density on the projection. This is performed by systematically moving a counter over the projection and noting the number of poles falling within the counter at each position. In the usual Schmidt method, the counter is circular and its area is fixed at one percent of the area of the projection. Contours are drawn at given values of percent of total poles per one percent area of the projection, Fig. 4.2(b). The method is fully described by Turner and Weiss (1963).

For small numbers of poles and/or very weak preferred orientations, the Schmidt contours are often difficult to interpret since chance clustering of poles can give rise to maxima which have no real significance. A method devised by Kamb (1959) in which the area of the circular counter is increased as the number of poles decreases, smoothes out such effects and is gaining acceptance. Contours are drawn at integral numbers of standard deviations from the density expected from a uniform population of poles, Fig. 4.2(c). This method is discussed further in Appendix B.

Objections have been raised to the use of a circular
counter on the equal area projection because of the geometric distortion effect. Contouring with variable shape elliptical counters or use of the stereographic projection with variable radius circular counters has been proposed, Vistelius (1966). Although these procedures are theoretically sound, the additional complication is in general not justified by the precision of the source data, Fairbairn (1949).

Unless specifically stated otherwise, the lower hemisphere equal area projection and contouring with a one percent area circular counter are used throughout this thesis. All the plotting and contouring was done by digital computer. Full details of the computer programs are given in Appendix B.

Application to drill holes

The usual techniques have to be modified in the case of drill holes, Terzaghi (1965). If a hole is drilled into a rock mass containing several sets of joints of equal spacing, more intersections will be made with those joints nearly perpendicular to the hole than with those nearly parallel to the hole. In an extreme case, it is possible to drill a hole in a plane parallel to a joint set and not detect any joints at all. Terzaghi proposed the concept of the 'blind zone' of a drill hole which is the plane of which the drill hole is the normal. Joints with poles lying in or near this plane are nearly parallel to the drill hole and will not be adequately
intersected. The blind zones of the three holes A, B, and C are shown in Fig. 4.3. The only way to overcome this problem is to drill at least two non-parallel holes in any area.

Plotting of the joints in the drill core, without any correction for this effect, will therefore give a false impression of the intensity of the various joint maxima. If the true spacing of the joints is \( d \) and the joint plane is inclined at angle \( \alpha \) to the hole direction, Fig. 4.4, the apparent spacing, \( d' \), measured on the drill core is given by

\[
d' = \frac{d}{\sin \alpha}
\]  

(4.1)

Therefore, a correction for the effect can be made when contouring the scatter diagram by weighting each pole by the factor \( \frac{1}{\sin \alpha} \). In the normal method of manual contouring, Terzaghi recommends that the net be divided into a number of zones, bounded by concentric small circles centred on the hole axis, with approximately constant values of \( \sin \alpha \). However, since all this work was done by computer, it is quite simple to weight each pole by its exact value of \( \frac{1}{\sin \alpha} \) since \( \alpha \) is one of the orientation parameters supplied to the computer (see Section 4.3). A mathematical difficulty is that when \( \alpha \) is very small, the weighting factor becomes very large and has an unwarranted effect on poles lying near the blind zone. A maximum weighting factor of 5 has therefore been specified.
in the computer program. This is an arbitrary but reasonable solution to the difficulty. The variation of weighting factor with $\propto$ is shown in Fig. 4.5.

Where applicable, therefore, this correction has been made in the contoured projections, Fig. 4.2(d). There is little to be gained by the correction when there is nothing on the projection but a single point maximum and it has not been applied in such cases. The concept of this correction is not restricted to drill holes. Measurement of joints on a single planar outcrop gives rise to a 'blind spot' which is the pole to the plane of the outcrop. A similar difficulty also arises in the measurement of platy minerals such as mica on the universal stage of a microscope, Turner and Weiss (1963).

**Information on projections**

The convention used in this thesis for specifying additional information on projections is shown in the examples, Fig. 4.2. These may be summarised as follows:

(i) Top L/H corner - hole and zone, e.g. A.3 refers to hole A, zone 3.

(ii) Top R/H corner - depth of hole covered by measurements.

(iii) Bottom L/H corner - number of measurements on diagram. The number in brackets is the adjusted number of measurements after applying the Terzaghi correction.
procedure. Absence of this number means that the Terzaghi correction was not applied.

(iv) Bottom R/H corner - contour intervals used on diagram. For example, 1,3,6,9 means that contours are drawn to include areas containing >1, >3, >6 and >9 percent of the total poles per one percent area of the projection. The number above the contour intervals is the maximum contour value on the diagram.

Where the Kamb method of contouring has been used the word 'Kamb' is written above the contour intervals. Unless otherwise stated, a dashed great circle on the projection is the trace of the average bedding plane.

4.3 ANALYSIS OF BEDDING

It is clear from Figs. 3.1 and 3.2 that the strata boundaries and marker beds have remarkably constant orientation. This is particularly true for the pyrite band which has nearly a constant strike between holes A and B and the V79 cross cut, and constant dip, allowing for displacement in the correct sense by the T72 strike fault, between these holes and the surface outcrop. The 2 in. TMB shows similar good correlation. The average orientation of the bedding in the area is 355W70.

To check the uniformity of bedding on a finer scale for purposes of core orientation, the bedding plane traces on
the core were examined. If the bedding planes had uniform orientation and the holes were straight, the angle between the bedding trace and the core axis, herein termed the core bedding angle (CBA), would be constant. The actual variation of this angle, allowing for hole deviation, along the axis of the core will give an indication of the constancy of bedding orientation. In particular, if the area is folded, there will be cyclic variations in the core bedding angle, passing from a positive angle through zero to a negative angle. Of course, if bedding is the only marker it will not be possible to distinguish positive and negative angles, but the cyclic variation should still be present.

To test this possibility and to provide detailed bedding measurements for core orientation, the core bedding angle was measured at 3 in. intervals, where possible, along the core. These readings were averaged over one foot intervals and are plotted in Figs. 4.6, (a), (b), (c) for holes A, B and C respectively. The angle is not constant but fluctuates in a rather narrow range. The one foot average values were averaged over 10 ft intervals and these are presented in Fig. 4.7. For holes A and B, nearly all the measurements can be fitted to 10 degree spreads of strike and dip respectively, allowing for hole deviation. The measurements for hole C fit better to a steeper dip of about 80°. One possible reason is that the survey of this hole is
Fig. 4.6(a). One foot average core bedding angles.
Fig. 4.6(b). One foot average core bedding angles.
HOLE C

Fig. 4.6(c). One foot average core bedding angles.
Fig. 4/7. Ten foot average core bedding angles.
in error. However, as explained in Chapter 2, this hole was independently surveyed by Tropari and the ANU instrument, both giving identical results. The steeper dip must therefore be accepted as real. The nearly vertical dip near the top of this hole is certainly real since this was checked on a small outcrop near the hole collar.

Nevertheless, considering the size of the area explored, the bedding fluctuations are only second order variations. It is therefore concluded that the strata are in general uniformly oriented but with minor undulations in strike and dip as would be expected in any mass of rock of these dimensions. Core orientation using the bedding as a marker should therefore be reasonably successful.

The situation is theoretically the classical three borehole problem, Phillips (1960). The average bedding orientation could be determined from the intersection of the three small circles of radius \(90^\circ - \text{average CBA}\) drawn around the respective average hole directions. However, because of bedding undulations it was considered more logical to make best estimates of the bedding orientation for shorter lengths of each hole. Since the approximate orientation of the bedding is known this can be done independently for each hole.

Ten foot lengths of core were chosen for this procedure.
It is necessary to make an assumption regarding the strike or dip of the bedding or both. For example, it could be assumed that the strike is constant and that variations in core bedding angle are caused entirely by dip fluctuations, or vice versa. Alternatively both dip and strike could be assumed to vary in some pattern. The orientation of the hole relative to the bedding must also be considered. For example, in holes A and B, the core bedding angle is approximately equally sensitive to variations in both strike and dip. On the other hand, in hole C it is practically insensitive to changes in strike of the magnitude envisaged here and nearly all the variation must be due to dip fluctuations.

The procedure adopted was for hole C to assume strike constant at 355° and calculate the variations in dip. For holes A and B strike and dip were assumed to vary together in one degree intervals from the average 355W70. There are two possible combinations and one is appropriate to each hole. The total range of assumed bedding directions are shown in Fig. 4.8, and detailed in Tables 4.1, 4.2 and 4.3. These procedures are admittedly arbitrary but are designed to fit the observed core bedding angle into the minimum range of bedding orientation, while at the same time preserving geometric compatibility.
Fig. 4.8. Deviations of holes and ranges of assumed orientations of bedding.

Fig. 4.9. Location of reference line on core.
4.4. CORE ORIENTATION

The reference line on the core was defined by the major axes of the elliptical traces of the bedding on the core with a sense pointing down the hole, Fig. 4.9. The core was fitted together as closely as possible and laid out in approximately 50 ft lengths in 1\(\frac{1}{2}\) in. x 1\(\frac{1}{2}\) in. aluminium angle set on edge to form a vee. Since it was very difficult to handle the core if it was broken, fragments broken during the drilling were carefully fitted together using epoxy cement. The apex of each individual bedding trace was first marked and the reference line drawn as a continuous line of best fit through these marks.

The orientation of planar structures in the core was measured relative to this reference line and the core axis. The angle between the plane and the core axis was measured with a workshop protractor attached to a length of aluminium angle which lay along the side of the core. This angle (\(\alpha\)) could be read to an accuracy of 1 degree provided the feature was flat. If the feature was not flat the angle of an imaginary flat surface of best fit was measured. In fact, however, this was seldom necessary since nearly all the features were remarkably flat.

To measure the angle (\(\beta\)) between the major axis of the elliptical trace of the feature, sense pointing down hole, and the reference line, a device consisting of an annulus
which slid along the core was manufactured. It was centralised by six spring loaded ball bearings in two rows, which permitted the device to centralise on cores of slightly different diameter. A reference mark on the annulus was set against the core reference line and the angle measured with a protractor rotating on top of the annulus. This device worked quite well but was somewhat unwieldy and was eventually discarded in favor of a piece of paper graduated in degrees and which wrapped around the core. This had the disadvantage that it was only suitable for a fixed core diameter. However, there was little variation in core diameter so that this was not serious. The angle could be measured with an accuracy of ± 3° for a flat feature.

If the core is held vertically with a sense pointing down the hole and the reference line is considered to be zero degrees, the measured angles α and β are as shown in Fig. 4.10. Relative to the reference line, the apparent trend (T) and plunge (P) of a joint are given by

\[ T = \beta + 180^\circ \]  \hspace{1cm} (4.2)
\[ P = \alpha \]  \hspace{1cm} (4.3)

To determine the true position in space of the joint plane, the following manipulations are required.
Fig. 4.10. Geometrical relations of measurements on core.

Fig. 4.11. Three steps in rotation of core to true position in space. (a) V1 about vertical axis (b) ROT about east-west horizontal axis (c) V2 about vertical axis.
(i) The core axis must rotate to its true position in space, including correct sense.

(ii) The bedding plane must rotate to its true position in space.

The joint plane will then automatically rotate to its true position in space.

This can be accomplished on a stereographic or equal area net by one rotation about an inclined axis. However, as recommended by Phillips (1960), it is more convenient to use more than one rotation and rotate about a horizontal axis using the small circles on the net. For the computer solution to this problem it is convenient to standardize on an east-west rotation axis, so that three rotations are required:

(i) rotation about vertical axis (angle V1);
(ii) rotation about E-W horizontal axis (angle ROT);
(iii) second rotation about vertical axis (angle V2).

These three steps for a typical case are shown in Fig. 4.11. The magnitudes and senses of the three rotation angles can best be determined by considering the reverse problem, i.e. rotating the drill core from its true position to the reference position.

The rotation procedure shown in Fig. 4.11 is quite simple and can be readily performed on an equal area net.
However, when thousands of poles are to be rotated the procedure is very tedious and many errors are made, particularly when poles rotate to the primitive and must re-enter the projection at the antipodal point. For this reason, the problem was programmed for the computer and the method is detailed in Appendix B.

A summary of the average core bedding angle, hole direction, assumed bedding orientation and the three rotation angles for each 10 ft of each hole is given in Tables 4.1, 4.2 and 4.3.

4.5 ANALYSIS OF VEINS

A prominent feature of the drill core is the very great number of veins. Although they are not the primary defects in the rock it is essential that they be discussed first since it is not possible to classify the joints without reference to the veins. Veins must also be studied in their own right since they are likely planes of weakness in the rock, although this is not always the case.

Although there are some sections containing irregular masses of veins approaching a breccia, most of the veins are very planar. Logging of veins included the following information:

(i) orientation relative to core;
(ii) thickness of vein;
<table>
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<th>DEPTH</th>
<th>HOLE DIRECTION</th>
<th>CBA</th>
<th>ASSUMED BEDDING</th>
<th>ROTATION ANGLES</th>
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<td>30</td>
<td>352W67</td>
<td>+116 +090-041</td>
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<td>319/00</td>
<td>32</td>
<td>353W68</td>
<td>+115 +090-041</td>
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<td></td>
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* MEANS CORE ORIENTED FROM VI VEINS
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(iii) nature of filling material.

The filling material was also examined in greater detail by means of thin sections.

Orientation

Contoured equal area projections showing all veins measured in zones 1, 2, 3 and 6 are shown in Fig. 4.12. The Terzaghi correction procedure has been applied to all these diagrams. Zone 4 is not represented since there was no bedding from which the core could be oriented. Zone 5 is also not represented because this is a thin zone of sedimentary breccia, practically devoid of any veins, and which also cannot be oriented. The good agreement between all these diagrams, particularly in view of the fact that they were independently oriented, is very encouraging and indicates that the orientation procedure is sound.

The major feature of these diagrams is the point maximum of poles in the western part of the diagram, corresponding to planes striking north-south and dipping fairly steeply east. The dip of these planes steepens in the western zones 3 and 6. In most of the diagrams this maximum lies in a girdle defined approximately by the great circle projection of a plane striking north-south and dipping gently west. It will be shown later that this girdle, as such, has probably no significance. Particularly in zones
1 and 2, there is a weaker maximum at north and south poles of the projection, corresponding to planes striking east-west with almost vertical dip.

The spacing of the veins was not analysed in detail. However, Fig. 4.12, indicates that throughout the area, the average true spacing of veins is somewhat less than 1 in. This is a maximum value because there are many very fine veins visible in thin section, which are not apparent in hand specimen.

**Characteristics of veins**

Closer examination of the veins in thin section enables them to be classified into three major sets, which will be denoted V1, V2 and V3 respectively.

V1 veins are the major set defined by the point maximum in the western part of the projection. They are shear features which displace the bedding by small amounts, usually by a small fraction of an inch, Fig. 4.13. Quite often, they do not visibly displace the bedding at all. The widths vary from hairline up to 1 in. but most are less than 0.1 in. The thickness of a given vein tends to be less uniform than for the other sets, Fig. 4.16. The filling material is principally dolomite but quartz and chlorite are also important. The dolomite grains are almost invariably twinned and the quartz grains almost invariably show undulatory
Fig. 4.13. V1 vein faulting bedding. Hole A, 387.8 ft. Width of field 9 mm. (crossed nicols).
extinction and quite often contain "deformation lamellae", Carter, Christie and Griggs (1964). The chlorite is often segregated near the walls of the vein, Fig. 4.14, and some sections of these veins are exclusively chlorite, particularly in zones 2 and 3, Fig. 4.15. Pyrite is ubiquitous and is a major filling material in the Urquhart Shale.

V2 are represented by dolomite veins striking north south and dipping steeply. This is the maximum of poles near the north and south poles of the projection. These are extension features and are not older and are probably younger than set V1. The filling material is nearly pure dolomite. Quartz and pyrite are only minor and chlorite is generally absent. The widths are of the same order as set V1 but individual veins characteristically have very uniform width.

A good example of the relation between sets V1 and V2 is shown in Fig. 4.16. The V1 vein faults the bedding, which is stylolitic (see Chapter 3), but the V2 vein is not displaced. From examination of the intersections of these veins it should be possible to establish age relationships but this has proved very difficult since both fillings are mainly dolomite. It is only in rare cases, Fig. 4.17, that it can be shown that V2 cuts V1 and is therefore probably older and not of the same age. The generally different
Fig. 4.14. Segregation of chlorite near walls of V1 vein. Hole B, 373.8 ft. Width of field 0.8 mm. (crossed nicols).

Fig. 4.15. Part of V1 vein with entirely chlorite filling. Hole A, 153.7 ft. Width of field 0.8 mm. (crossed nicols).
Fig. 4.16. V1 vein (north-west) faulting bedding. V2 vein (north-east) is not displaced. Hole A, 151.7 ft. Width of field 9 mm. (crossed nicols).

Fig. 4.17. V2 vein (north-east) cutting dolomite twins in V1 vein. Hole A, 167.8 ft. Width of field 2.4 mm. (crossed nicols).
filling materials also support this conclusion.

V3 veins are approximately parallel to V2 but are filled exclusively with quartz and so are a different set. The quartz is quite distinct from that in the V1 veins since it is completely free from strain features. These are extension features and are undoubtedly younger than V1 or V2 since they cut both of these, Fig. 4.18. The maximum thickness is 0.2 in., but most are thinner than this. Their thickness is generally uniform but the thinner ones tend to die out in the rock, Fig. 4.19.

These pure quartz veins are obvious only in zone 2 although they may be represented by fine veins in the other zones. All the visible quartz veins in zone 2 of holes A and B were distinguished from the dolomite veins by the staining technique (Chapter 3) and poles are plotted in Fig. 4.20. Hole C was not analysed since the veins lie near the blind zone of that hole. The veins have a high preferred orientation but are by no means as numerous as the other sets. The small difference in orientation in holes A and B is probably a reflection of the precision of the orientation procedure.

4.6 ANALYSIS OF JOINTS

Breaks in the drill core may be classified as follows:

(i) joints, i.e. open fractures existing in the rock
Fig. 4.18. V3 quartz vein cutting V1 dolomite veins. Width of field 9 mm. (crossed nicols).

Fig. 4.19. V3 quartz veins cutting V1 dolomite veins. V3 vein dies out in rock. Width of field is 9 mm. (crossed nicols).
Fig. 4.20. Orientation of quartz veins, Zone 2. Dots - hole A, crosses - hole B.
before drilling commenced;

(ii) fractures along existing planes of weakness in the rock, produced by drilling;

(iii) fractures unrelated to planes of weakness in the rock, produced by drilling.

Fractures produced by drilling are the result of vibration of the core barrel, jamming of core in the barrel and breaking of the core with the core lifter at the end of each run. Type (iii) can usually be easily distinguished because they are generally curved and irregular, with fresh uncoated surfaces. Types (i) and (ii) both have planar surfaces and are systematically oriented but can usually be distinguished by the presence, or lack, of coatings on their surfaces, respectively. However, in some cases this may be very difficult since many weakness planes such as re-cemented faults have surfaces quite similar to those of open joints. Where any doubt existed they were classified as joints.

During measurement of the joints, the following information, additional to the orientation parameters, was noted.

(i) planarity of the surfaces - v. flat, flat, curved or irregular;

(ii) small scale roughness of the surfaces - v. smooth, smooth, rough or v. rough;
(iii) coatings on surface - clean, iron stained, chloritic, graphitic, etc.;
(iv) continuity of the joint across the section of the drill core;
(v) classification evidence - displacement of markers, slickensided or polished surfaces, etc.

Slickensides or other linear features may be oriented by measuring the angle between the lineation and the major axis of the elliptical face of the joint, in the plane of the joint. On the projection, the lineation can be plotted by measuring this angle, in the correct sense, from the dip direction along the great circle defining the joint plane. This point can then be rotated to its true orientation in space by the method described in Section 4.4.

Orientation

Contoured equal area projections of all joints measured in each zone are shown in Fig. 4.21. The Terzaghi correction procedure has been applied to these diagrams. Zone 5 is again not represented since it is a thin zone in which the core is difficult to orient. Although bedding is missing from most of zone 4, the V1 veins have such uniform orientation that it is possible to use them as a marker. The average orientation for zones 3 and 6 for each hole was estimated from the maxima in Fig. 4.12 and the mean of these
Fig. 4.21. Orientations of joints in different zones.
assumed to be the average orientation in Zone 4. A procedure similar to that used in the bedding analysis was then used to obtain an estimate of the vein orientation from the average "core vein angle" for each 10 ft length of core. This appears to have been successful since there is good agreement in the orientation patterns.

Three major sets of joints are indicated by Fig. 4.21.

(i) A point maximum of poles of planes striking north-south and dipping steeply west. This set will be called J1. This is in fact the bedding plane and the bedding joints are of major importance in zones 1 and 2. They are almost completely absent from zones 3 to 6 even though bedding traces are present on the core. This indicates that the bedding has almost no mechanical significance west of the zone 2 - zone 3 boundary. This only applies to fresh rock, since in weathered rock the bedding again becomes a primary weakness plane, which is reflected in zone 6, hole C, Fig. 4.21. This section of the hole is partly in oxidised rock above the base of complete oxidation.

(ii) A point maximum of poles of planes striking north-south and dipping between steeply east and vertical. This set will be called J2. This maximum tends to move towards the primitive in the western zones indicating that the joints become more nearly vertical.
(iii) A girdle concentration of poles in a great circle approximately coinciding with the trace of the bedding plane. The joint planes are therefore oriented at right angles to the bedding planes. This set will be called J3. The girdle is more or less filled in the various diagrams but there are point maxima near the centre of the projection and also near the south pole, corresponding to planes striking north-south and dipping gently east, and striking east-west and dipping steeply north, respectively. These sub-sets will be called J3a and J3b respectively.

The good correspondence between all these diagrams is again very pleasing. They also agree very well with a comprehensive series of measurements by Herget (1968b), of joints exposed in the mine workings in the Urquhart Shale. It would appear that the joint pattern is remarkably constant over the whole area.

**Characteristics of joint sets.**

Photographs of typical joint surfaces are given in Fig. 4.22.

Set J1, the bedding joints, are almost invariably flat and continuous across the core. Commonly they have dark coated surfaces, suggesting that they coincide with graphitic beds. The lack of bedding joints in zones 3 and 6 can probably be correlated with a much lower carbonaceous content in these
Fig. 4.22. Examples of typical joints
rocks. Many of these joints, particularly in zone 2, are quite definitely faults with prominent slickensides or polished surfaces and most of these have coatings of graphite. They appear to be the youngest structural features in the core since they truncate the V3 veins. Markers cannot be traced across the faults, indicating that the displacement is greater than the core diameter. The other bedding joints, not obviously faults, are difficult to classify but the majority of them are probably breaks on bedding planes during drilling. This is significant since they would be prominent planes of weakness during blasting or ripping operations.

Set J2, the north south striking, steeply easterly dipping joints, are also flat but are often not continuous across the core. They are parallel to the V1 veins and in fact many of the joints are probably breaks on the V1 veins produced by drilling, facilitated by the alignment of chlorite in the veins, Figs. 4.14 and 4.15. The discontinuous joints occur when the core does not break cleanly on the vein. However, there are many of these joints which are clearly of pre-drilling origin. The surface coating is almost invariably chlorite, although pyrite is also common in the Urquhart Shale. This shows close correlation with the V1 vein fillings. Some surfaces show faint slickensides or
polishing.

Set J3, the joints at right angles to the bedding, are again quite planar and fairly smooth, although a few have rough interlocking surfaces. Very commonly they have coatings or at least stains of limonite. Some have clean, fresh looking surfaces with small patches of white material, which was determined by X-ray diffraction as montmorillonite. With the exception of a small number in hole B, zone 1, these joints show none of the characteristics of faults and are undoubtedly extension fractures. Their most important characteristic is that very commonly they are not continuous across the width of the core. In zones 1 and 2 they often terminate against a bedding plane along which the core breaks. In the other zones, they simply die out and the core breaks along a fresh surface, generally along the cleavage.

Spacing of joints

Contoured equal area projections, with Terzaghi corrections, such as Fig. 4.21, can be used to determine average joint spacings directly. The number of poles represented by a given contoured maximum on the projection can be determined by multiplying the area between each pair of contours by the average contour value. Alternatively, once the maxima are determined by the contoured diagram, it is possible to return to the scatter diagram and count the
number of poles within a given area and correct for orient-
tation by means of an overlay with zones of constant average 
sin $\alpha$, as recommended by Terzaghi (1965). The latter 
method seemed simpler and was adopted.

Estimation of the average spacing in sets $J_1$ and $J_2$
is relatively simple since they are represented by point 
maxima on the projections. The set $J_3$ is more difficult to 
analyse because the joints have a wide range of orientations. 
The expedient adopted here was to allot each joint to one 
of the subsets $J_{3a}$ and $J_{3b}$ by dividing the girdle into two 
90 degree arcs, one extending $45^\circ$ from each end of the 
great circle (set $J_{3b}$) and the other (set $J_{3a}$) occupying the 
centre part of the girdle. This division, although some-
what arbitrary, was adopted because (i) there are maxima 
in the girdle in these areas, and (ii) the influence of 
these joints in the open cut batter will be approximately 
determined by these orientations, particularly set $J_{3a}$.

The average spacing in each joint set for each zone 
of each hole is given in Table 4.4. The average spacing for 
all the holes for each zone is also given. A striking 
point of this table is the narrow range into which the 
 spacings fall for all the joint sets. Maximum and minimum 
spacings are 6.2 ft and 0.3 ft respectively and 90 percent 
of the average spacings lie between 0.5 and 2.5 ft. The
**TABLE 4.4**

Average Joint Spacings (feet)

<table>
<thead>
<tr>
<th>Set J1</th>
<th>Zone</th>
<th>Hole</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>1.7</td>
<td>1.5</td>
</tr>
<tr>
<td>2</td>
<td>1.2</td>
<td>1.1</td>
<td>1.3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Set J2</th>
<th>Zone</th>
<th>Hole</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>1</td>
<td>0.9</td>
<td>0.8</td>
<td>3.2</td>
</tr>
<tr>
<td>2</td>
<td>1.6</td>
<td>6.2</td>
<td>2.0</td>
</tr>
<tr>
<td>3</td>
<td>1.0</td>
<td>1.0</td>
<td>1.2</td>
</tr>
<tr>
<td>4</td>
<td>1.9</td>
<td>-</td>
<td>3.5</td>
</tr>
<tr>
<td>6</td>
<td>2.1</td>
<td>0.8</td>
<td>2.1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Set J3a</th>
<th>Zone</th>
<th>Hole</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>1</td>
<td>2.5</td>
<td>2.6</td>
<td>1.7</td>
</tr>
<tr>
<td>2</td>
<td>0.8</td>
<td>0.8</td>
<td>0.6</td>
</tr>
<tr>
<td>3</td>
<td>1.0</td>
<td>0.8</td>
<td>2.2</td>
</tr>
<tr>
<td>4</td>
<td>2.4</td>
<td>-</td>
<td>0.6</td>
</tr>
<tr>
<td>6</td>
<td>2.3</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Set J3b</th>
<th>Zone</th>
<th>Hole</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>1</td>
<td>1.1</td>
<td>0.6</td>
<td>1.5</td>
</tr>
<tr>
<td>2</td>
<td>0.7</td>
<td>0.3</td>
<td>1.3</td>
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<tr>
<td>4</td>
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<td>-</td>
<td>1.6</td>
</tr>
<tr>
<td>6</td>
<td>1.6</td>
<td>1.5</td>
<td>2.2</td>
</tr>
</tbody>
</table>
small range of joint spacings between different sets indicate that the blocks comprising the rock mass will tend to have cubical rather than tabular shape.

The figures in Table 4.4 may be compared with joint spacings in the Urquhart Shale exposed in the mine workings, quoted by Herget (1968b). The same joint sets are represented and the average spacings are:

\begin{align*}
  J1 &\quad 0.3 \text{ ft} \\
  J2 &\quad \text{not measured} \\
  J3a &\quad 0.6 \text{ ft} \\
  J3b &\quad 0.7 \text{ ft}
\end{align*}

These spacings are all smaller than those measured in the drill core. This indicates that the shale in the mineralised zones is more closely jointed, which is quite probable, or alternatively that measurement of joints in walls of openings overestimates the true intensity. This is more likely to be true than the reverse situation because of the large number of extraneous fractures caused by blasting. However, Herget's results must be considered to be of the same order as those in the present work.

Muller defines the "unit rock block" as the smallest homogeneous rock unit produced by a system of geological separations intersecting a rock mass, John (1962). The average volume of a unit rock block is the product of the joint spacings in each of three dimensions. Average volumes
for the various zones are:

Zone 1: $1.1 \times 2.2 \times 0.9 = 2.2 \text{ cu.ft.}$
Zone 2: $1.2 \times 0.7 \times 0.6 = 0.5 \text{ cu.ft.}$
Zone 3: $1.0 \times 1.4 \times 1.3 = 1.8 \text{ cu.ft.}$
Zone 4: $2.3 \times 1.0 \times 1.8 = 4.1 \text{ cu.ft.}$
Zone 6: $1.3 \times 1.3 \times 1.7 = 2.9 \text{ cu.ft.}$

In the case of zones 1 and 2 where both J1 and J2 sets are present, the smaller of these two spacings was used. These calculations although very approximate, give a good idea of the average block size in the various zones. In practice they are likely to be maximum values since blasting will tend to reduce the natural rock blocks by splitting on bedding or cleavage.

**Continuity of joints**

Few joints exist as continuous flat planes over large areas. Major faults approach this situation, although the fault plane is normally undulating rather than flat. Many joints terminate against other joints to form a "brick wall" type of structure. Terzaghi (1962) defines joints as continuous if it is possible to construct sections through the rock mass which nowhere cut across intact rock. The sections may be approximately plane, irregularly warped, or stepped. It follows that sections following discontinuous joints must at some stage pass through sections of intact rocks which Terzaghi termed 'gaps'. The effective joint area ($A_i$) is
then defined by

\[ A_i = \frac{A_g}{A} \quad (4.4) \]

where \( A \) is the total area of a section.

\( A_g \) is the area of gaps in the section.

Muller adopts a similar approach. The "two dimensional extent" (\( K_2 \)) is equivalent to Terzaghi's \( A_i \). The "three dimensional extent" (\( K_3 \)) is defined as the total effective joint area per unit volume of the rock mass and is given by

\[ K_3 = \frac{K_2}{d} \quad (4.5) \]

where \( d \) is the spacing of the joint set.

The mathematical formulation given above can, in practice, be stated only in terms of average values with perhaps high and low limits. Terzaghi infers that such an analysis is of very limited use since it is impossible to measure the parameter \( A_g \). There is no doubt that it is an extremely difficult problem but he is being unduly pessimistic since it is possible to make at least an estimate of joint continuity by measuring joint traces in closely spaced openings of different orientation or by correlation between closely spaced boreholes.
Unfortunately, widely spaced boreholes, such as in the present investigation, are not well suited for the estimation of joint continuity. All that can be said is that a particular joint is, or is not, continuous across the section of the drill core. Examination of all the joints in a particular set will indicate broadly whether the set is continuous or not. On this basis J1, the bedding joints, are continuous and the other two sets J2 and J3, are discontinuous. It is not possible to say whether joints J2 and J3 could form a continuous but stepped section through the rock mass.

The only information regarding joint continuity at Mount Isa is the measurements of Herget (1968b) on the Urquhart Shale in the mine workings. The average and maximum continuity values ($A_1$ of Terzaghi or $\mathcal{K}_2$ of Muller) are as follows:

<table>
<thead>
<tr>
<th>Set</th>
<th>Average</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>J1</td>
<td>0.5</td>
<td>1.0</td>
</tr>
<tr>
<td>J2</td>
<td>not measured</td>
<td></td>
</tr>
<tr>
<td>J3a</td>
<td>0.03</td>
<td>0.3</td>
</tr>
<tr>
<td>J3b</td>
<td>0.07</td>
<td>0.5</td>
</tr>
</tbody>
</table>

These results may explain the wider spacing of joints measured in the drill core compared with those measured in the mine openings. Many of the discontinuous joints may not be intercepted by the drill hole where it passes through
4.6 OTHER STRUCTURAL FEATURES

Slaty cleavage

The well developed cleavage possessed by the rocks west of zone 2 was discussed in Chapter 3. It was shown by X-ray methods that the cleavage is a result of the preferred orientation of basal planes of the platy minerals chlorite and mica and must therefore be classified as slaty cleavage. This cleavage was in fact first noticed by the large number of parallel, freshly fractured breaks in some parts of the drill core.

The orientation of these fresh breaks in sections of the core where they are best developed is shown in Fig. 4.23. Because the contoured diagrams contain single point maxima, the Terzaghi correction was not applied. The maxima refer to planes striking slightly west of north and with a vertical dip. The agreement between individual diagrams indicates that the cleavage has very constant orientation in zones 3 to 6.

A very sensitive indicator of the presence of cleavage in a given piece of core is a simple "drop test". The piece of core is held horizontally about 12 inches above a concrete floor. When dropped, the core will break into a number of pieces separated by parallel fractures in the plane
Fig. 4.23. Orientation of slaty cleavage defined by fresh breaks in drill core.
of the cleavage.

The number of these cleavage fractures in the core is greatest in hole B and least in hole A. This can be related to the angle between the cleavage plane and the hole axis, being \(60^\circ\) in hole B and \(30^\circ\) in hole A. The breaks are essentially tensile fractures, with faint plumose markings, and are probably caused by alternating bending stresses due to flexing of the core barrel. The tensile stress on the cleavage plane is therefore greatest in hole B core. Hole C core, with an angle of \(40^\circ\) between cleavage plane and hole axis, show intermediate behaviour.

This slaty cleavage will clearly be of major mechanical significance in any structure excavated in these rocks.

**Singular structures**

The foregoing discussion has concentrated entirely on defects present in families or sets. Singular defects of large extent, in particular major faults, may have greater mechanical significance if unfavorably oriented with respect to the structure.

The bands of completely oxidised rock near the top of zone 2 in holes A and B were originally thought to be the expression of a major strike fault. However, the lack of similar evidence in hole C indicates that this is probably not so. Although there were several areas of
broken core in each hole there were none that could be definitely correlated as major structures. One weakness of widely spaced drill holes is that they cannot easily distinguish between a minor and major structure.

However, examination of the general geology of the area indicates that major faults, unfavorably oriented with respect to the open cut batter, are unlikely. Major faults in the mine area are strike faults, striking north-south, and transverse faults, striking approximately east-west, both with near vertical dips (Bennett, 1965). Both these types are represented in Figs. 3.1 and 3.2. The T72 strike fault is in the area of excavation and will not appear in the hanging wall batter. The three transverse faults shown in Fig. 3.1 were plotted from the M.I.M. geological plan of the level. Weil (1965) states that, with the exception of No.9 Glory Hole Fault, they do not extend far into the shale on either side of the orebody. If it continues far into the hanging wall, No.9 Glory Hole Fault should be intersected by hole B in zone 1b. There is no evidence for a major fault in this area of the hole but the jointing is very intense and this could be related to the fault. A band of powdered rock at a depth of 236 ft in hole B may also represent an intersection with a transverse fault. However, since their dips are predominantly vertical it is unlikely that the transverse faults will be important in the stability of the batter.
5.1 INTRODUCTION

Dynamic analysis in structural geology is concerned with the forces and stresses which have given rise to the various structures. Kinematic analysis, on the other hand, is concerned with the displacements and strains involved in the formation of the structures. Kinematic analysis has wide application in geology but is of only limited use in rock mechanics. However, dynamic analysis has considerably relevance because the stress system which formed the structural features may be the same as, or at least related to, the existing stress field, which is of major importance to any excavation formed in the rock.

For many decades it has been obvious that certain features in rocks such as folds, faults, certain joints, cleavage, etc., are the result of tectonic forces subsequent to the emplacement of the rock. Correlation of these features with a tectonic stress field was hampered for a long time by lack of knowledge of the mechanisms of fracture and flow in rocks. However, the advances in experimental rock deformation in the last few years have given great stimulus to the dynamic analysis of features in naturally deformed rocks.
For many geological structures it is now possible to determine fairly certainly the directions and relative magnitudes of the principal stresses during deformation. At this stage, it is not possible to determine absolute magnitudes of the principal stresses. A concise review of the current knowledge in this field is given by Friedman (1964). The processes considered relate both to brittle fracture and plastic flow of rocks. The former is mainly applicable to macroscopic structures and the latter to microscopic structures, but there is no firm division between them.

This chapter is therefore somewhat of a diversion from the main theme of the thesis. The justification for including such a study in the present work is that the techniques probably have a general application in rock mechanics, but many more case studies are required before this is certain. Knowledge of the mechanism of formation of the various structures also must inevitably lead to a greater understanding of their mechanical significance around excavations in the rock. Furthermore, the cost of obtaining the drill core was substantial and, since much of the core was destroyed during mechanical testing, it was considered that the maximum information should be obtained from the core before this occurred.
The principles of dynamic analysis, with particular reference to brittle fracture of rocks, are first considered and this information is applied to macroscopic features in the drill core. Plastic deformation of rock forming minerals is then considered and a dynamic analysis of twin lamellae in dolomite grains described. Finally, other features at Mount Isa, not encountered during the drilling, are briefly considered.

5.2 PRINCIPLES OF DYNAMIC ANALYSIS

The failure of rock under external stresses can occur by brittle fracture or by plastic flow or by a combination of both. Brittle fracture predominates in most rocks under conditions of low confining pressure and low temperature, in the form of shear fractures or extension fractures as defined by Griggs and Handin (1960). The experimental information on brittle fracture will be briefly reviewed as a background to dynamic analysis. Plastic deformation will be considered in Section 5.4. Compression is taken as positive with $\sigma_1 > \sigma_2 > \sigma_3$.

Experimental shear fractures

It has been found experimentally that most homogeneous and isotropic rocks under compressive stresses fail in shear and that failure can be expressed quite well by the
Mohr theory of fracture which states that the shear stress \( \tau \) is a function of the normal stress \( \sigma \) on the plane of failure. The form of the failure relation can be determined by a series of triaxial tests under different confining pressures. A particular case of the Mohr theory is the Mohr-Coulomb theory which predicts a linear relation between \( \tau \) and \( \sigma \) of the form:

\[
\tau = c + \sigma \tan \phi
\]  

(5.1)

where \( c \) is the cohesion

\( \phi \) is the angle of internal friction.

The relation (5.1) holds for many rocks over restricted ranges of stress and predicts that failure will occur on a plane inclined at angle \( \theta = 45^\circ - \phi/2 \) to \( \sigma_1 \). The same applies to the generalised Mohr theory except that the predicted angle \( \theta \) is not constant. Since \( \phi \) is generally not equal to zero the angle \( \theta \) is usually less than 45\(^\circ\).

Other predictions of the Mohr theory are:

(i) Failure will occur in a plane containing the intermediate principal stress, \( \sigma_2 \). Thus two conjugate shear planes, each inclined at angle \( \theta \) to \( \sigma_1 \), are predicted.

(ii) The failure stress is independent of the magnitude of the intermediate principal stress. This may not be true for many rocks, Hoskins (1967), Mogi (1967), but
does not invalidate the conclusions regarding fracture orientation.

In the usual "triaxial" test, with $\sigma_2 = \sigma_3$ or $\sigma_2 = \sigma_1$, failure is equally likely on any plane tangent to a core with axis along the $\sigma_1$ direction and included angle $2\theta$. In practice, however, usually only one shear plane forms, probably because no real rock is homogeneous and isotropic.

There is a great deal of experimental evidence to support these predictions. From a large number of triaxial compression tests on a large number of rocks, Handin and Hager (1957) found that the angle $\theta$ varied from $25^0$ to $35^0$ in 65 percent of cases and from $20^0$ to $40^0$ in 95 percent of cases. Borg and Handin (1966) found $\theta$ to average $28^0$ in 24 triaxial compression experiments. Many more examples could be cited, Handin (1966). Therefore, for most reasonably homogeneous and isotropic rocks, shear failure occurs on a plane inclined at less than $45^0$ to $\sigma_1$ and the failure plane contains $\sigma_2$. The angle $\theta$ is of the order of 30 degrees in triaxial compression and usually somewhat less in extension.

**Experimental extension fractures**

The other mode of brittle failure is by extension fracture. The commonest types are those formed in direct
tension tests or in Brazilian tests. In both these tests, \( \sigma_2 \) is tensile and the fractures form normal to \( \sigma_3 \).

However, extension fractures can also form in situations where all principal stresses are compressive, such as in the confined Brazilian test, Jaeger and Hoskins (1966). Vertical splitting is also common in triaxial compression tests under low, or zero confining pressure. In all cases, the fractures form normal to \( \sigma_3 \). Griggs and Handin (1960) conclude that propagation of cracks initiated by wedging is responsible for these fractures. More recently, Brown and Trollope (1967) have proposed that inter-particle tensile stresses are generated by lateral expansion of material at right angles to the \( \sigma_1 \) direction. Under certain conditions, these "effective tensile stresses" can cause failure in extension even though there are no external tensile stresses. The theory also explains the discrepancy between tensile strengths calculated from Brazilian tests and ring tests.

There appears, therefore, to be convincing experimental evidence that extension fractures can form normal to the direction of least principal compressive stress, which is the situation most likely to occur in a geological situation.
Effect of anisotropy

All the above discussion applies to reasonably homogeneous and isotropic rock. An important exception occurs in the case of rocks which have a well defined planar strength anisotropy, such as bedding, cleavage or schistosity. The modifications to the Mohr theory for such cases are given by Jaeger (1962). Depending on the orientation of the plane of weakness to the external stresses, failure may occur either in the weakness plane or through the surrounding rock. This theory is discussed further in Chapters 7 and 8.

In a comprehensive series of triaxial compression tests on slate, Donath (1961, 1964) found that failure occurred in the plane of the cleavage if it was oriented at between $15^\circ$ and $45^\circ$ to $\sigma_1$ and was influenced by the cleavage at angles up to $75^\circ$ to $\sigma_1$. If failure occurs through a plane of weakness, it also commonly ignores the direction of $\sigma_2$, Jaeger (1964). When the weakness plane is unfavourably oriented for failure, the rock will behave as though it were isotropic and shear fractures will form at the appropriate angles.

Extension fractures are somewhat different. Youash (1966) found that fractures in sandstone in direct tension occurred on the bedding planes for inclinations of the
bedding down to $30^\circ$ from $\sigma_3$. However, in a compressive stress field, shear fracture would preferentially occur under these conditions. Therefore, even in anisotropic rocks, extension fractures are likely to form only at nearly $90^\circ$ to $\sigma_3$.

**Application to dynamic analysis.**

Dynamic analysis is the reverse problem to that usually involved in studying the fracture of rock under a known stress system. The fracture orientations, and probably type, are known and it is required to deduce the stress system which cased them. The earliest application of dynamic analysis was probably Anderson's theory of faulting, Jaeger (1962). Using the Mohr-Coulomb theory and the experimental results discussed above, Anderson showed that:

(i) steeply dipping normal faults indicate that $\sigma_1$ was vertical at the time of formation;

(ii) transcurrent faults indicate that $\sigma_2$ was vertical at time of formation;

(iii) gently dipping thrust faults indicate that $\sigma_3$ was vertical at the time of formation.

In each case, the other two stress directions can be determined by the orientation and sense of movement of the fault.
If a joint can be identified as an extension fracture, it immediately follows that the $\sigma_3$ direction must have been nearly normal to the joint at the time of formation. There is no information on the $\sigma_1$ and $\sigma_2$ directions except that they must lie $90^\circ$ apart in the joint plane. If there is only one major orientation of joints in a particular set, this indicates that there was a unique $\sigma_3$ direction and that $\sigma_1 \succ \sigma_2 \succ \sigma_3$ or that $\sigma_1 = \sigma_2 \succ \sigma_3$ as in a triaxial extension test. If the normals to the joints occupy a variety of orientations in a plane, this indicates that there was no unique $\sigma_3$ direction and that $\sigma_1 \succ \sigma_2 = \sigma_3$ as in a triaxial compression test.

If a joint, which does not coincide with a plane of weakness in the rock can be definitely classified as a fault, it follows that the $\sigma_1$ direction was inclined at an angle $\theta$, which is of the order of $30^\circ$, to the joint plane at the time of formation. $\sigma_1$ must therefore lie on a small circle of radius $(90^\circ - \theta)$ and centred on the pole, $P$, of the fault, Fig. 5.1(a). $F$ is the trace of the fault plane. If the direction of movement, $S$, on the fault plane is known, $\sigma_1$ is then known to lie also in the plane containing $S$ and $P$, Fig. 5.1(b), and is one of the positions $\sigma_1$ or $\sigma_1'$. $\sigma_2$ is uniquely located as the pole of the plane $PS$ and lies in the fault plane $F$. If the sense of slip is also known, it is then possible to
Fig. 5.1. Dynamic analysis of a fault.

Fig. 5.2. Dynamic analysis of conjugate faults.
choose either $\sigma_1$ or $\sigma_1'$ as the most likely $\sigma_1$ direction. $\sigma_3$ is then automatically located $90^\circ$ from $\sigma_1$ in the plane PS, Fig. 5.1(c). Therefore, to completely specify the stress system, three types of data are required:

(i) orientation of fault plane;
(ii) slip direction in the plane;
(iii) sense of slip in the slip direction.

If all this information is not available it is only possible to obtain an ambiguous solution.

If two conjugate faults can be identified, no more information is required. The obtuse angle between the poles of the faults, $P_1$ and $P_2$, is equal to $(180^\circ - 2\alpha)$ and $\sigma_1$ is the bisector of this angle, Fig. 5.2. $\sigma_2$ is the line of the intersection of the fault planes and $\sigma_2$ lies $90^\circ$ from $\sigma_1$ in the plane $P_1 P_2$. If two joints intersect at a small angle, it cannot be immediately assumed that they are conjugate faults, since one may be a fault and the other an extension fracture formed by the same stress system, Friedman (1964).

If a fault coincides with a plane of weakness, all that can be said is that $\sigma_1$ was not normal or parallel to the fault plane. The angle $\theta$ may vary over a wide range in such a case. However, if a fault cuts across the plane of weakness at a high angle, all the above
discussion applies, including the fact that \( \theta \) is of the order of \( 30^\circ \). In addition, \( \sigma_1 \) must have also been nearly normal or parallel to the plane of weakness. If this were not the case, the fault would have preferentially formed in the weakness plane.

With regard to the data required to fully analyse a fault, the first requirement, the orientation of the fault plane, is easily measured. The direction of movement can be determined in two ways (a) by the direction of slickensides on the fault plane (b) or by the relative positions of markers on each face of the fault. Slickensides are notoriously unreliable indications of fault movements since they represent the last movement direction which may not coincide with the movement at formation of the fault. The results from slickenside directions must therefore be considered with caution. The sense of slip also cannot usually be obtained from slickensides alone, Paterson (1958), and requires an external marker. The other method requires a characteristic point on each face of the fault, or the displacement of two non-parallel planar markers, Phillips (1960). In either case, the sense of slip can also be determined.

In practice, dynamic analysis is essentially a statistical procedure and the results can only be considered
reliable if a large number of structural features can be statistically fitted to a given stress system. It would be unreasonable to expect that every single joint in a rock mass could be explained by the simple principles described above. Stresses are unlikely to be homogeneous, most real rocks are anisotropic to some degree and the fracture angle $\Theta$ is unlikely to be constant. Even with small samples under homogeneous stress, Borg and Handin (1966) found a great variety of subsidiary fractures formed at, and after, failure. This is likely to be more pronounced in rocks at depth where confining pressure is applied by comparatively rigid surrounding rock rather than by a hydraulic fluid as in a triaxial experiment. In a rock mass, the scale of the observer is such that these minor fractures could appear as important structural features.

5.3 DYNAMIC ANALYSIS OF FEATURES IN CORE

$V_1$ veins

These are quite definitely shear features and are the oldest features in the core. They have been known at Mount Isa for many years and were described as "cross-fractures" by Blanchard and Hall (1942). Herget (1968a) terms them "microfaults", which is a more graphic description.
Both these authors state that movement has been east block up and west. A relative movement in this general direction is also invariable in the drill core, but since there is only one other marker, the bedding, it is not possible to deduce the precise direction of movement. A dynamic analysis is shown in Fig. 5.3, assuming that the slip direction, $S$, pitches $45^\circ$ from the dip direction, $D$, on the fault plane to give the desired sense of movement. $\sigma_1$ then plunges south-east, $\sigma_3$ south-west and $\sigma_2$ north. $\sigma_1$ and $\sigma_3$ will plunge more nearly due east and west respectively, and $\sigma_2$ will be more nearly horizontal the greater the dip component of the slip direction.

A somewhat different approach to dynamic analysis of the "microfaults", equivalent to the $V_1$ veins, was made by Herget (1968a), who found that the poles to the microfaults formed an elongated maximum in the western part of the projection, Fig. 5.4. A similar tendency is evident in some of the diagrams in Fig. 4.12. Herget therefore concluded that, at the time of formation, $\sigma_1 > \sigma_2 = \sigma_3$ as in a triaxial compression test, and that the faults formed tangential to a cone centred on the $\sigma_1$ direction and with included angle $2\theta$. A small circle fitted to the maximum on the projection should therefore be centred on $\sigma_1$, and have a radius of $(90^\circ-\theta)$. He
Fig. 5.3. Dynamic analysis of V1 veins assuming movement direction north block up and west.

Fig. 5.4. Dynamic analysis of "microfaults" (after Herget (1968a)).
found that $\sigma_1$ coincided with the average bedding pole for the area, that is, that $\sigma_1$ was normal to the bedding, and that $\theta = 38^\circ$, which is a reasonable value. It would therefore appear that the V1 veins have mainly a dip component of slip which would bring the analysis of Fig. 5.3 into line with Herget's results.

The close spacing and small displacements of the V1 veins suggest that they were formed under high confining pressure and that the rock involved was constrained from deforming freely at its boundaries. If this were not the case, the shear strain would have been accommodated by a lesser number of larger faults. The situation is analogous to experimentally deforming a specimen in a thick metal jacket, Paterson and Weiss (1966). It is therefore likely that faulting occurred without loss of cohesion and that the V1 veins were never open joints.

The filling material probably migrated from the parent rock into the veins during deformation. This has some support from microscopic examination since many, but not all, of these veins appear to "merge" into the surrounding rock along their boundaries.

**J2 joints**

Many of these joint surfaces have slickensides which permit a dynamic analysis to be made. For those joints
which were originally V1 veins, the slickensides may be inherited from the veins or may have been caused by later movements.

The slickensided faults are most numerous in zones 3 to 6. A few, on which sense of slip can be measured by displaced bedding planes indicate that displacement has been east block up, as for the V1 veins. Assuming this sense for all the faults, deduced stress directions for the individual faults from zones 3 to 6, holes A and B, and also zone 1, hole A are shown in Fig. 5.5. The results agree well with Figs. 5.3 and 5.4, indicating that the slickensides were either formed with the V1 veins or later under the same stress system.

The situation in zone 1, hole B is rather different. This zone contains a large number of slickensided faults which do not fit the results of Fig. 5.5. Poles of all the faults, except bedding plane faults, with slickensides are shown in Fig. 5.6. On the basis of orientation and slickenside direction they have been divided into four groups shown on the figure. Groups 1, 2 and 3 are J2 joints and group 4 are J3 joints. The latter may be primary faults or may represent later shear movements on primary extension fractures.

The poles and slickenside directions in the individual
Fig. 5.5. Dynamic analysis of J2 faults. (a) Hole A, zones 3 to 6 (b) Hole B, zones 3 to 6 (c) Hole A, zone 1. Circles = \sigma_1, dots = \sigma_2, crosses = \sigma_3.
Fig. 5.6. (a) orientation of faults, hole B, zone 1 (b) slickenside directions of these faults.
groups are shown in Fig. 5.7. Groups 1, 3 and 4 are strike-slip faults and group 2 contains both strike-slip and dip-slip faults. The only group with a sense of slip is group 1, which displace the bedding and have a sense of west block north, similar to that postulated for the Mount Isa fault. Computed principal stress directions are shown in Fig. 5.7(e) to (h) for each group, there being two possibilities for each of $\sigma_1$ and $\sigma_3$ in the last three cases (refer Fig. 5.1(b)). Agreement is good for $\sigma_2$ which plunges steeply northwest. $\sigma_1$ dips gently south-west and $\sigma_3$ dips gently north-west or vice versa, with the former being more likely in view of Fig. 5.7(e). The dip-slip faults of group 2 fit the stress picture of Fig. 5.5.

**V2 and V3 veins**

Both these sets of veins have the characteristics of extension fractures. They strike east west and are nearly vertical, indicating that at the time of formation $\sigma_3$ was north south and horizontal. Since only one major orientation is represented in each set, $\sigma_1 > \sigma_2 > \sigma_3$ or $\sigma_1 = \sigma_2 > \sigma_3$ but nothing can be said of the orientation of $\sigma_1$ and $\sigma_3$ except that they both must have been in the plane of the veins. The different filling materials of the two sets indicates that they were caused by two separate
Fig. 5.7. (a) to (d) Orientations (dots) and slickenside directions (crosses) of four groups of faults in hole B, zone 1. (e) to (h) Dynamic analyses of these faults. Circles = \( \sigma_1 \), dots = \( \sigma_2 \), crosses = \( \sigma_3 \).
events, with at least $\sigma_3$ remaining fixed in direction.

**J3 joints**

These joints, with the exception of those from hole B discussed above, are also quite clearly extension fractures. Since their poles occupy a variety of orientations in the bedding plane, $\sigma_3 = \sigma_2$ and both must have been in the bedding plane. $\sigma_1$ must therefore have been oriented at right angles to the bedding. The fact that these joints are not filled indicates that they were probably among the last structures to form in the area. It is therefore likely that these joints formed upon the release of stress due to uplift and erosion during which $\sigma_1, \sigma_2$ and $\sigma_3$ retained their relative magnitudes. A mechanism of joint formation due to uplift is presented by Price (1966).

**Bedding plane faults**

These are very numerous in zones 1 and 2. Since they displace the V3 veins they appear to have a young age, although Herget (personal communication) concludes that they were formed as the result of bedding plane slip during folding. If this is so, the bedding plane faults in the core represent later movements on these early faults. Directions of all recognisable slickensides are plotted in Fig. 5.8. There is a concentration from hole B near the
Fig. 5.8. Slickenside directions of bedding plane faults. Circles = hole A, crosses = hole B, dots = hole C. B defines range of bedding orientations.
north pole and another for holes A and C plunging south. Unfortunately, there are only three readings from hole A since the core was badly damaged in these zones.

As would be expected, there is no definite conclusion from Fig. 5.8 since slip is likely to occur on the bedding plane faults under a wide range of stress orientations. This is reflected in the large spread of slickenside directions.

**Slaty cleavage**

Since the slaty cleavage in zones 3 to 6 is nearly parallel to the V1 veins, it is tempting to postulate that it was formed at the same time by a shearing process. The origin of slaty cleavage is by no means clear, but geological evidence from deformed markers, Billings (1954), and deformation experiments, Means and Paterson (1966), suggest that the preferred orientation of platy minerals is normal to \( \sigma_1 \). If this is the case, \( \sigma_1 \) must have been oriented east-west and horizontal during the formation of slaty cleavage at Mount Isa.

There is also evidence that the slaty cleavage postdates both V1 and V2 veins. Many of the veins in zones 3 to 6 have a "puckered" appearance. Examination of thin sections reveals that the puckers are caused by lenses of chlorite, cutting across the vein, and parallel to the
cleavage direction, Fig. 5.9. This effect is seen best in V2 veins but is also present in V1 veins if they are not nearly parallel with the cleavage. It seems likely therefore that the veins were in existence before the formation of the slaty cleavage.

The absence of slaty cleavage in zones 1 and 2 is probably related to the original mineralogy of these rocks, with quartz and dolomite predominant. The rocks in zones 3 to 6 probably originally contained a high proportion of clay minerals which gave rise to the chlorite and mica during low grade metamorphism.

5.4 DOLOMITE TWIN LAMELLAE

Most rocks consist of grains of the same, or different, minerals joined together at their grain boundaries. When a rock is stressed, each grain deforms at first elastically and with further increase in stress may deform plastically. Plastic deformation, or intracrystalline gliding, in crystals is the result of two processes:

(i) translational glide, or slip, in which atoms on either side of the slip plane are displaced by an integral number of unit lattice spacings;

(ii) twin glide, in which parts of the lattice rotate so that they form a minor image of the lattice across a twin plane.
(a) Puckered appearance of V2 veins in slate. Hole B, 373.8 ft. Width of field is 9 mm. (crossed nicols).

(b) Enlargement of part of (a) showing chlorite cutting through V2 vein. Width of field 0.8 mm. (crossed nicols).

Fig. 5.9
Both types of glide are the result of movement of dislocations through the crystal lattice under the influence of external shear stresses. Each mineral has a limited number of glide systems, each consisting of a glide plane containing one or more glide directions. The glide systems are related to the crystal structure and are independent of the external stress system. Gliding will occur when the resolved shear stress on the glide plane in the glide direction exceeds the critical value characteristic of that glide system at that temperature. The critical resolved shear stress is nearly independent of the normal stress on the glide plane, that is, the coefficient of internal friction is almost zero.

The resolved shear stress on a glide system is given by:

\[ \tau = (\sigma_1 - \sigma_3) \cos \chi \cos \lambda \]  

(5.2)

where \( \chi \) is the angle between \( \sigma_1 \) and the normal to the glide plane;

and \( \lambda \) is the angle between \( \sigma_1 \) and the glide direction. The quantity \( \cos \chi \cos \lambda \) is called the coefficient of resolved shear stress, \( S_0 \). It has its maximum value of 0.5 when both \( \chi \) and \( \lambda \) are equal to 45°. It can also assume negative values when the resolved shear stress is in
the opposite sense from that required for gliding.

There is very extensive literature on glide systems in minerals, mostly arising from rock deformation experiments. Much of this information is summarised by Turner and Weiss (1963), Friedman (1964), and Handin (1966). Calcite is the most ductile of the common rock-forming minerals and received the earliest and most extensive study. Dolomite is closely related to calcite and is of interest in the present context because twinned dolomite grains are common in the Mount Isa drill core.

Use of glide systems in dynamic analysis closely follows the principles given for brittle fracture in Section 5.2. Twinning has the greatest application since the change in orientation of the lattice is readily visible in thin section. Slip can only be observed indirectly by its effect on pre-existing markers, such as twin lamellae, in the grain. As for dynamic analysis of faults, three types of data are required:

(i) glide plane;
(ii) glide direction;
(iii) sense of shear in glide direction.

However, the situation is usually a good deal simpler since the glide direction and sense are fixed in many glide systems and it is usually only necessary to locate the glide plane.
The glide systems in dolomite have been studied in deformation experiments by Handin and Fairbairn (1955) and by Higgs and Handin (1959). Only two systems are known:

(i) translational glide on the (0001) basal plane with any of the 'a' axes as glide direction;
(ii) twin glide on the 'f' rhombohedra \{0\overline{2}21\}, with the normal to the intersection with (0001) as glide direction.

The relevant geometry of the dolomite crystal is shown in Fig. 5.10(a). The crystallographic c-axis, normal to the basal plane coincides with the optic axis, \(C_v\), which is in the centre of the projection. Since dolomite has trigonal symmetry there are three 'f' planes, the poles of which are \(f_1, f_2, f_3\), and there may be up to three sets of twins in any grain. The glide direction, \(g_1\), is shown for the \(f_1\) plane. The other visible feature in dolomite grains is the cleavage parallel to the \{10\overline{1}1\} rhombohedra which are designated \(r_1, r_2, r_3\). Relevant angles between these poles are

\[
\begin{align*}
\mathbf{f} \wedge \mathbf{C}_v &= 62\frac{1}{2}^\circ \\
\mathbf{f}_1 \wedge \mathbf{f}_2 &= 80^\circ \\
\mathbf{r} \wedge \mathbf{C}_v &= 44^\circ.
\end{align*}
\]

Twinning in dolomite has a negative sense (relative
Fig. 5.10. Geometry and dynamic analysis of twinning in dolomite.
to calcite which is positive) and the top part of the crystal moves down during shear, Fig. 5.10(b). In dynamic analysis of dolomite twinning it is assumed that $\sigma_1$ is oriented so that $S_0$ has its maximum value of 0.5. This is equivalent to assuming that twinning was caused by the minimum possible value of the external stress. Stresses oriented so that $S_0$ is less than 0.5 can still cause twinning but the magnitude must be greater so that the critical resolved shear stress can be exceeded. This procedure was first applied to calcite by Turner (1953) who introduced the terms "compression" and "tension" axes, denoted by C and T respectively. "Extension" would be more correct for the latter term. The computed C and T axes should statistically define the $\sigma_1$ and $\sigma_3$ directions respectively.

Taking into account the sense of shear, for a grain containing one set of twin lamellae, the C axis is located 45° from f towards $C_V$ in the $C_Vf$ plane, Fig. 5.10(c). The T axis is 90° from C in the same plane. If the grain contains two or three sets of lamellae, most workers, using mainly calcite, have applied the above procedure to the best developed set of lamellae. If two or three sets are equally well developed this is not very logical and Nissen (1964) has modified the procedure for calcite. A similar
analysis for dolomite is as follows:

(i) A grain with no lamellae indicates that $T$ is parallel to $C_v$. $S_0$ on each $f$ plane is then $-0.41$;

(ii) For one set of lamellae, the original Turner technique is applicable. $S_0$ for both $C$ and $T$ axes is $+0.5$;

(iii) For two equally developed sets of lamellae, the $C$ axis is chosen so that $S_0$ on the third $f$ plane is zero. The situation is shown in Fig. 5.10(d). $S_0$ on the other two $f$ planes is $+0.42$ for $C$ and $+0.32$ for $T$.

There are many other possibilities but the above is the most logical compromise and follows Nissen's approach. In the general case, where $C_v$, $f_1$ and $f_2$ are measured, the bisector of the acute angle ($80^\circ$) between $f_1$ and $f_2$ is the $T$ axis. It is also the pole of the plane containing the $C$ axis, which is $27^1_1^0$ from $C_v$ towards the intersection of the two planes, Fig. 5.10(e).

(iv) A grain with three equally developed sets of lamellae indicates that $C$ is parallel to $C_v$. $S_0$ on each $f$ plane is then $+0.41$.

The above procedures must inevitably lead to a certain scatter of $C$ and $T$ axes about the $\sigma_1$ and $\sigma_3$ directions since twinning can occur even when $C$ and $T$ are not oriented as assumed. A further source of scatter is that each grain does not necessarily experience the same
stresses as the aggregate as a whole. In fact, continuity of strain makes this impossible although a large number of grains will be subjected statistically to the applied external stress system. These factors make it essential that a large number of different grains be examined. In addition, the computed C axis is only $17^\circ$ from $C_v$. Therefore, a preferred orientation of $C_v$ in the rock will give rise to a preferred orientation of C axes which may have no dynamic significance. Reliable dynamic inferences can therefore only be obtained when the grains are randomly oriented, or nearly so, Turner (1962).

Coarse grained dolomite in thin sections from the drill core often shows f twinning, Fig. 5.11. The twins have the characteristic lenticular appearance of deformation twins rather than the stepped appearance of growth twins and therefore should have some dynamic significance. The lamellae are very fine and sharp when viewed on edge and would therefore be termed "untwinned" lamellae, Christie (1958). The presence of dolomite twins in these rocks is somewhat surprising since twinning was produced in deformation experiments only at temperatures above $400^\circ$C. Friedman (1964) states that they are rare except in intensely deformed metamorphic rocks. The Mount Isa rocks by no means fit that description. However, it is probable that
(a) Twinning in dolomite grains.
Width of field 0.8 mm.
(crossed nicols).

(b) Dolomite grain tilted on universal stage so that twin planes are vertical. Width of field 0.6 mm. (plane polarised light).

Fig. 5.11
low strain rates, which were not investigated in the deformation experiments, would permit twinning at lower temperatures. It seems to be a general principle that decrease in strain rate is equivalent to increase in temperature in rock deformation.

Two thick dolomite veins from zone 2 were selected for analysis. The dolomite in these veins is fairly coarse grained and is more suitable for microscopic examination. These veins, denoted X and Y, belong to sets V1 and V2 respectively and had the following locations and orientations:

Vein X : hole A, 181.0 ft, orientation 356E58
Vein Y : hole B, 165.4 ft, orientation 117N61.

Poles to these veins are shown in Fig. 5.12.

In order to avoid any bias due to preferred orientation, three mutually perpendicular thin sections were cut from vein X and two from vein Y. The microstructure of the two veins is quite different, Fig. 5.13. Vein A has fairly uniform coarse grained dolomite with minor quartz and pyrite. Vein Y has some coarse dolomite grains in a matrix of fine grained dolomite and fragments of siltstone.

The optic axis, $C_v$, and the poles to the twin planes were measured on a petrographic microscope fitted
Fig. 5.12. Orientation of two dolomite veins studied.
Fig. 5.13. Photomicrographs of dolomite veins. Width of field 9 mm. (crossed nicols).
with a universal stage, Turner and Weiss (1963). The optic axis is measured by rotating the grain to extinction so that $C_v$ is either vertical or horizontal and east-west. The pole to a twin lamella can be measured by rotating the grain so that the lamella assumes its minimum width and sharpest focus. The pole is then oriented in a horizontal position. Readings of the scales on the universal stage permit the poles to be plotted on a projection. In this case plotting was performed by computer.

For dynamic analysis, grains were classified as having 1, 2, 3 or no sets of lamellae. This classification was based on the "spacing index" of the lamellae, $S_p$, which is the number of per millimetre normal to the lamellae plane, Turner and Weiss (1963). Any set of lamellae with $S_p < 10$ was ignored. For grains containing more than one set, any set in which $S_p$ was less than half that in the next most abundant set was ignored. This was quite an arbitrary classification to fit the four cases of dynamic analysis discussed previously. It has been found in both calcite and dolomite that the spacing index is closely related to $S_0$, the coefficient of resolved shear stress. Grains with a high spacing index are therefore more likely to have been favorably oriented with respect to the external stress system. The majority of the grains had spacing
indices of less than 100, with a few up to 200.

Statistics of the measurements are given in Table 5.1.

Table 5.1

<table>
<thead>
<tr>
<th>Vein</th>
<th>X</th>
<th>Y</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. grains measured</td>
<td>240</td>
<td>140</td>
</tr>
<tr>
<td>0 sets lamellae</td>
<td>22%</td>
<td>36%</td>
</tr>
<tr>
<td>1 set lamellae</td>
<td>64%</td>
<td>46%</td>
</tr>
<tr>
<td>2 sets &quot;</td>
<td>12%</td>
<td>18%</td>
</tr>
<tr>
<td>3 &quot;</td>
<td>2%</td>
<td>-</td>
</tr>
</tbody>
</table>

Vein X therefore has a high proportion of twinned grains, perhaps indicating that it was subjected to somewhat higher levels of stress. The spacing indices of grains in vein X also tend to be higher than those in vein Y.

For each grain examined, the C and T axes were computed as shown in Fig. 5.10. Each thin section was then rotated to its true geographical position and the results for the three, (vein X), or two (vein Y), thin sections combined. The results of the dynamic analysis are summarized in Figs. 5.14 and 5.15. Contouring has been done by the Kamb method, since it is best able to cope
Fig. 5.14. Dynamic analysis of dolomite lamellae, vein X.
Fig. 5.17. Dynamic analysis of dolomite lamellae, vein Y.
with the scatter of results inherent in the method. The
distribution of c-axes is shown in (a) of each diagram.
They appear to have significant preferred orientations
but tests in which random trends and plunges of poles
were contoured by the computer indicates that the Kamb
method regularly produces concentrations up to 6σ from
random populations. This is because a random population
is not necessarily uniform. The grain orientations may
therefore be regarded as very nearly random.

Contoured diagrams of the compression and tension
axes derived from the dynamic analysis are shown in (b)
and (c), respectively, of each figure. Vein X has a
strong concentration of T axes and a weaker concentration
of C axes. The reverse is true for vein Y. For both
veins, the $\sigma_1$ directions defined by the C axes are in
good agreement and plunge due east. In vein X, $\sigma_3$
plunges due west. In vein Y, $\sigma_3$ defined by the maximum
is north-south and horizontal but the T axes also form
a girdle normal to $\sigma_1$, indicating that $\sigma_2 = \sigma_3$ as in a
triaxial test.

5.5 OTHER STRUCTURAL FEATURES

There are several major structural features at Mount
Isa which were not encountered in the drilling. These are:
(i) folding;
(ii) strike faults;
(iii) transverse faults;
(iv) silica dolomite masses.

These will not be discussed in detail but it is of interest to see whether they fit the structural picture provided by the minor structures.

Folds

Although there does not appear to be any folds in the area investigated, folding is common in the mine area and the Mount Isa Group is on the limb of a very large anticline, Bennett (1965). The axial planes of all the folds are north-south and vertical. There is, as yet, no basis for the dynamic analysis of folds but it seems reasonable that folds should be formed by compressive stress nearly normal to the axial plane. If this were the case, $\sigma_1$ would have been east-west and horizontal.

Strike faults

These are major structures at Mount Isa which strike north-south and dip nearly vertically. Murray (1961) classifies three types:

(i) Mark I with displacement east block up;
(ii) Mark II with displacement west block up;
(iii) Mark III with displacement east block up.

Murray describes them as dip-slip faults so that for
Mark I and III faults, \( \sigma_1 \) must have been plunging to the east and for Mark II faults \( \sigma_1 \) must have been plunging to the west. In either case \( \sigma_2 \) must have been oriented north-south and horizontal. The Mount Isa Fault is a Mark II fault, and the T72 fault, Fig. 3.2, is a Mark III fault. In general, however, the strike faults have not been studied very closely and more work is needed.

**Transverse faults**

These faults have been studied in more detail since they displace the orebody and effect mining operations. They are also responsible for the deep leaching in the Black Rock area. They occupy a wide range of orientations, Foy (1964), but the majority strike ENE with a displacement north block east, or strike ESE with a displacement north block west. There appears to be little doubt that they are conjugate faults. Mapping by Foy indicates that the ENE faults dip steeply north while the ESE faults dip steeply south. The situation is therefore as shown in Fig. 5.16, with \( \sigma_1 \) plunging west, \( \sigma_2 \) plunging east and \( \sigma_3 \) north-south and horizontal.

**Silica-dolomite masses**

The origin of these masses is controversial and probably very complex. However, because of their mineralogy it is likely that they were formed in situ by
Fig. 5.16. Dynamic analysis of transverse faults.
recrystallization of the Urquhart Shale and possibly the lower members of the Spear Siltstone. A striking feature is that the appearance of the silica dolomite in thin section is very similar to that of many of the veins in the drill core studied in this investigation. Copper mineralisation was noted in rather irregular dolomite veins at depth 280 ft in hole B. It is therefore suggested that more detailed study of structures in the Spear Siltstone may provide valuable information on the origin of these economically important bodies of rock.

Conclusions

A summary of the principal stress directions deduced from dynamic analysis of the various structures is given in Table 5.2. To trace the complete stress history of the area it is necessary to know the age relations of the structures. Of the features in the core, the V1 veins appear the oldest and J3 joints the youngest. Of the major structures, the folding and the strike faults are early and the transverse faults are later. There is, however, no information connecting the ages of the minor and major structures.

Although the stress directions listed in Table 5.2 are not consistent with a single stress system, the directions of the stresses, without regard to their relative
Table 5.2

Summary of dynamic analyses

<table>
<thead>
<tr>
<th>Feature</th>
<th>Principal stress directions</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\sigma_1$</td>
</tr>
<tr>
<td>Folding</td>
<td>E-W horiz. (?)</td>
</tr>
<tr>
<td>Strike faults</td>
<td></td>
</tr>
<tr>
<td>Mark I, III</td>
<td>plunge east</td>
</tr>
<tr>
<td>Mark II</td>
<td>plunge west</td>
</tr>
<tr>
<td>Transverse faults</td>
<td>plunge east</td>
</tr>
<tr>
<td>J1 veins</td>
<td></td>
</tr>
<tr>
<td>J2 joints</td>
<td>plunge east</td>
</tr>
<tr>
<td>J2 faults (hole B, zone 1)</td>
<td>plunge west</td>
</tr>
<tr>
<td>V2 veins</td>
<td></td>
</tr>
<tr>
<td>V3 veins</td>
<td>in E-W vertical plane</td>
</tr>
<tr>
<td>J3 joints</td>
<td>plunge east</td>
</tr>
<tr>
<td>Slaty cleavage</td>
<td>E-W horiz.</td>
</tr>
<tr>
<td>Dolomite lamellae</td>
<td></td>
</tr>
<tr>
<td>Vein X</td>
<td>plunge east</td>
</tr>
<tr>
<td>Vein Y</td>
<td>plunge east</td>
</tr>
</tbody>
</table>
magnitudes, are reasonably constant. The three common
directions are north-south horizontal, plunging east and
plunging west. In particular, one of $\sigma_2$ or $\sigma_3$ appears
to have been north-south and horizontal throughout the
tectonic history of the area. It is therefore reasonable
to assume that the above three directions have been constant
principal stress directions, but that the relative magni­
tudes of the stress have been changing over geological
time.

The presently existing stress system in the mine
area is more likely to be related to that which caused
the younger structures, such as J3 joints and V3 veins
and probably the transverse faults. All these structures
fit a consistent stress system with $\sigma_1$ plunging east,
$\sigma_2$ plunging west and $\sigma_3$ north-south and horizontal.
The dolomite lamellae in vein Y also fit this system and
the lamellae in vein X and the V1 veins fit a similar
system with $\sigma_2$ and $\sigma_3$ interchanged.

The only published stress measurements at Mount Isa
are those of Hoskins (1967) in silica dolomite on 13 and
14 levels in the Rio Grande area, using borehole deformation
gauge, "doorstopper", and flatjack methods. Since there
was insufficient information to determine the principal
stresses uniquely, Hoskins assumed, presumably intuitively,
that one principal stress was oriented north-south and horizontal. On this assumption the principal stresses were calculated to be

\[ \sigma_1 = 3100 \text{ psi plunging } 45^\circ \text{ east} \]
\[ \sigma_2 = 2400 \text{ psi north-south horizontal} \]
\[ \sigma_3 = 1800 \text{ psi plunging } 45^\circ \text{ west} \]

There is therefore good agreement between these stress directions and those deduced from dynamic analysis.

Hoskins (1967) also measured dolomite twin lamellae from near the stress measurement sites but obtained inconsistent results. Many more measurements are required to establish the consistency of this method.

The most difficult feature to explain is the slaty cleavage. Since it is parallel to the axial plane of the major folding, it would be expected to have been formed during the folding. However, the evidence that it appears to postdate the V2 veins, Fig. 5.9, indicates that it is younger than the folding since Herget (1968a) considers the microfaults to be late stage structures. The other possibility is that the V1 and V2 veins predate the folding and that slaty cleavage was formed during the folding. Many of the V1 veins do, in fact, have small scale fold structures with the cleavage as axial plane. Further work, both in the field and in elucidating the mechanism of formation of slaty cleavage will be required to resolve this question.
Chapter 6

FRICIONAL PROPERTIES OF ROCK SURFACES

6.1 INTRODUCTION

Previous chapters of this thesis have concentrated on the description and geometry of the discontinuities in the rock mass. The next essential step is to investigate the strength characteristics of the joints under simple stress systems since it is the combination of geometry and strength of the discontinuities which determines the reaction of the rock mass to external stresses.

The frictional properties of the joints in a rock mass are a fundamental problem in rock mechanics. It assumes its greatest importance in shallow or surface structures such as the foundations of a dam or the slopes of an open cut since in these cases the normal stresses are generally of a relatively lower order than in deep excavations. In the latter case, the joints may have little influence except in a narrow zone around the opening and elastic behaviour may be assumed to apply with a high degree of confidence.

At the present time, however, knowledge of the frictional properties of rocks is very limited. Moreover, there is no generally accepted method for determining these properties. In the case of diamond drill core, sliding in the triaxial
test is a simple and obvious, although somewhat indirect, method which is presently subject to some uncertainties. These are examined in detail in Chapter 7 and the method is applied to the natural joints in the drill core in Chapter 8.

However, in order to gain some understanding of the phenomena involved and also to perfect the experimental technique, it was considered desirable to perform some preliminary experiments on expendable artificially prepared surfaces. These exhibited interesting properties in their own right and are described in the latter part of this chapter. As an introduction to these, and to the following two chapters, the nature of friction and the available methods for measuring friction in rocks, with the results so far obtained, are firstly briefly reviewed.

The laws of friction

The classical laws of friction, namely that frictional resistance is proportional to the normal force and independent of the nominal area of contact, were first proposed by Leonardo da Vinci and subsequently rediscovered by Amontons some 200 years later, Bowden and Tabor (1964).

Amontons' first law can be stated by

\[ F = \mu N \]  \hspace{1cm} (6.1)

where \( F \) is the frictional resistance, \( N \) is the normal force and \( \mu \) is the coefficient of friction. Dividing each
side of eqn 6.1 by $A$, the nominal area of contact, leads to

$$\tau = \mu \sigma$$ \hspace{1cm} (6.2)

where $\tau$ and $\sigma$ are the shear and normal stresses respectively.

Amontons' laws are explained qualitatively by the adhesion theory of friction developed by Bowden and Tabor (1950, 1964). In this theory it is postulated that contact between two surfaces occurs only at the tips of asperities so that the true area of contact is much less than the nominal area of contact. Due to the high local normal stresses, welding occurs at the tips of the asperities and frictional resistance arises from the forces necessary to shear these contacts. It can be shown that $\mu$ is equal to the ratio of shear strength to yield stress of the material and to a first approximation this is a constant.

This theory was developed primarily for metals and most of the experimental verification has been done on metals. The theory is not very plausible for very brittle solids such as quartz and Byerlee (1967b) has recently presented a theory invoking brittle fracture of asperities. There seems little doubt that the asperity concept is fundamental in any theory of friction.

As will be shown later, the simple relation 6.2 often does not fit the frictional behaviour of rocks particularly
well. Very commonly it is found that $\mu$ (defined by $\tau/\sigma$) decreases with increasing normal stress. Some workers have fitted their results to a power law

$$\tau = \mu_0 \sigma^n$$  \hspace{1cm} (6.3)

where $\mu_0$ and $n$ are constants. There is some theoretical justification for such a law since Archard (1957) showed that if the deformation of surface protuberances is partially elastic the true area of contact is proportional to $(\text{load})^n$ where $2/3 < n < 1$.

On the other hand, it is also commonly found that, over a large range of normal stresses, $\tau$ and $\sigma$ are linearly related by an equation of the form

$$\tau = c + \mu \sigma$$  \hspace{1cm} (6.4)

or alternatively

$$\tau = c + \sigma \tan \phi$$  \hspace{1cm} (6.5)

where $c$ is a constant which may be identified with the cohesion in the Coulomb criterion of failure and $\phi$ is the angle of friction, Jaeger (1962). It is considered that this relation, where applicable, is preferable to the power law since (i) it has mathematical advantages in being linear, (ii) it is directly comparable with the Coulomb criterion for failure in soils and rocks and (iii) the cohesion term, $c$, has a simple physical significance as the shear strength of interlocked asperities. The relation 6.4 will be used in discussing
the results of the present work.

**Peak and residual strength**

To complete this preamble, it seems worthwhile to point out the distinction between "peak" and "residual" frictional resistance. In many cases the shear stress necessary to initiate sliding may be considerably greater than that necessary to sustain it. In an extreme case a vein may be filled with a material of high strength such as quartz or dolomite or an open joint may have large interlocking asperities which must be sheared before sliding can proceed. This concept was formalised for soil mechanics by Skempton (1964) and has since been applied to rock mechanics problems by Deere et al. (1966) and others. It must not be confused with the difference between static and dynamic coefficients of friction leading to "stick-slip" relaxation oscillations, which are discussed later.

### 6.2 AVAILABLE METHODS AND PREVIOUS RESULTS

The available information will be briefly discussed in terms of the various methods used.

**Direct shear tests on minerals**

The classical method of measuring friction, Bowden and Tabor (1950), is shown in Fig. 6.1(a) and consists of a flat surface, A, which slides under a spring loaded button, B, of the same, or another, material under a given normal load N.
Fig. 6.1. Methods used for measuring the frictional properties of rocks.
The frictional force \( F \) is measured by the deflection of the spring \( S \). The samples are usually small and the normal load \( N \) does not exceed a few tens of pounds. However, a wide range of variables, including temperature, speed of sliding and surface contamination can be investigated. Bowden and Tabor give information on the frictional properties of some minerals including rock salt, diamond, graphite and mica but these are not very typical and were studied mainly because of their unusual frictional behaviour.

The most comprehensive study of the friction of minerals was made by Horn and Deere (1962) with a similar type of apparatus. The major conclusion was that massive structured minerals (quartz, microcline, calcite) had low coefficients of the order of 0.1 when oven dried and much higher values (0.4 to 0.8) when saturated with water. On the other hand, layer-lattice minerals (muscovite, phlogopite, biotite, chlorite serpentine, steatite, talc) showed the reverse behaviour, the coefficient of friction being of the order of 0.5 when dry and 0.25 when wet. These values apply to "very smooth" surfaces and low normal loads up to 10 lb. Further observations were that quartz, when saturated, showed pronounced "stick-slip" behaviour and that increased surface roughness increased the friction but decreased the antilubricating effect of water on quartz.
Other studies of the friction of minerals, notably quartz, have arisen in soil mechanics from attempts to correlate the friction between individual grains with the frictional properties of an aggregate. Penman (1953) measured the friction between two fragments of quartz in a shear box, Fig. 6.2(b), and obtained values of $\mu = 0.65$ when saturated and 0.19 when oven dried for normal stresses up to 126 psi. These results are consistent with Horn and Deere's. For "much higher" normal stresses, $\mu$ decreased to 0.35 when saturated. Bishop (1954) obtained $\mu = 0.41$ for quartz pebbles. Rowe (1962) measured the friction between quartz sand and a quartz block and found that $\mu$ varied from 0.58 for silt to 0.4 for pebbles under saturated conditions.

These values for quartz show inconsistencies which could probably be resolved by considering variations in surface roughness in the different tests. This view is supported by the experiments of Byerlee (1967b) designed to test his theory of friction based on brittle fracture. This theory predicts that polished surfaces of brittle materials should have coefficients of friction of the order of 0.1. The higher values commonly measured for ground surfaces are ascribed to interlocking of asperities. Using a direct shear apparatus, Byerlee showed that the coefficient of friction of Westerly granite increased from 0.2 to 0.6 as the
CLA (centre line average) roughness increased from 20 to 300 microinches, while the coefficient for a freshly fractured surface was between 0.8 and 1.3. Further experiments sliding quartz, microcline, hornblende and calcite on sapphire showed a similar dependence of friction on surface roughness.

**Direct shear tests on rocks**

The shear box, Fig. 6.1(b), commonly used in soil mechanics work is a simple and versatile machine. The normal load, N, is usually not greater than a few hundred pounds and is applied by a dead weight hanger which is free to translate with the top part of the box. Maximum normal stresses of the order of 200 psi can be applied to a 2 in. square sample. In rock mechanics work, the interest generally lies in much higher normal stresses of the order of a few thousand psi and the normal load in a direct shear machine must be applied by a hydraulic jack. Since one half of the shear box must translate across the face of the jack some method, such as roller bearings, is required to eliminate friction in the apparatus.

The largest and most sophisticated machine of this type is operating at Imperial College, London, Hoek and Pentz (1968). Normal and shear load capacities are 100 tons and maximum specimen size is 15 in. x 12 in. Friction in the apparatus is eliminated by a PTFE bearing surface. Tests on large samples of natural joints in porphyry showed that
both peak and residual strengths followed the linear law (eqn 6.4) with $c$ of the order of 300 psi and 50 psi and $\mu$ equal to 0.60 and 0.51 respectively.

Another large machine with a normal load capacity of 60 tons and maximum sample size of 16 in. square has been built in Yugoslavia. Tests on bedding planes and joints in limestone, Krsmanovic and Langof (1964), showed the residual coefficient of friction to range from about 0.2 for clay filled joints to over 1.0 for very rough stratification planes. The latter showed a peak strength up to twice the residual strength which was achieved after some 1½ in. of sliding. Tests on intact samples of sandstone, limestone and conglomerate, Krsmanovic (1967), gave residual coefficients of friction of the order of 0.7.

Ripley and Lee (1961) used a conventional soil mechanics shear box to study the behaviour of both ground and naturally rough surfaces of sandstone, siltstone and shale. The ground specimens showed coefficients of friction in the range 0.47 to 0.60. The rough surfaces showed residual coefficients which were somewhat higher, but when corrections were made for the effect of riding over large asperities, the results could be correlated with those for the ground surfaces. This procedure is analogous to the "energy correction" of soil mechanics in which a correction is made for the work done by
the sample expanding against the normal stress if the sample dilates, or alternatively for the work done on the sample by the normal stress if the sample consolidates during shear. Such corrections are normally not significant in rock mechanics since volume changes are generally very small. However, they may be important for shearing of rough natural joints since Hoek and Pentz (1968) demonstrated varying amounts of dilation depending on the value of the normal stress.

In this regard an interesting series of direct shear tests on plaster of Paris specimens was reported by Patton (1966). The specimens were cast with various types of interlocking saw-tooth asperities. It was found that the frictional behaviour was of a dual nature. Under low normal stresses the surfaces rode up over the asperities and the slope of the $\tau/\sigma$ relation was a straight line through the origin with slope ($\phi + i$), where $i$ is the average inclination of the teeth from the shear plane. Under higher normal stresses the asperities were sheared and the failure relation was another straight line with slope $\phi$ and an intercept on the $\tau$ axis corresponding to the strength of the teeth. Patton therefore suggests that for real rock surfaces a single linear failure relation is unrealistic in view of the possibility of multiple modes of shear failure. However, it is felt that in view of the uncertainties involved in the practical
application of any friction measurements, such refinements are unwarranted at this stage.

The direct shear method is also the one most commonly used in in-situ tests for determining rock strength.

**Double shear tests**

A convenient adaptation of the direct shear test is the double shear test, Fig. 6.1(c). In this case, a flat block C is sandwiched between two other blocks A, B supported on packers P and the normal load N supplied by jacks in a loading frame. This system eliminates the difficulty of translating across the face of the jack described above, but only the average frictional resistance on each face can be measured. It is therefore preferable that the faces be identical and for this reason the method is virtually limited to artificially ground rock surfaces.

Maurer (1965) used this method for testing the frictional behaviour of surface of various rocks after fracture in the same apparatus. The samples were small, 0.5 x 1.0 inch in section. His results were expressed in the form of the power law (eqn 6.3) and constant $\mu_o$ was found to vary from 3.7 to 60.0 and the exponent $n$ from 0.46 to 0.80. If the results are replotted in terms of normal and shear stresses it is found that the linear law (eqn 6.4) applies reasonably well to most of the rocks. Values of $\mu$ range from 0.35 to
0.90 and c from 200 psi to 2400 psi under normal stresses up to 15,000 psi.

The double shear apparatus was used for 12 inch square blocks with artificially ground surfaces by Hoskins, Jaeger and Rosengren (1968). These tests were part of the present work and will be discussed separately in Section 6.3.

**Triaxial tests**

Although direct shear tests are theoretically the simplest for determining frictional properties, the need for specialised apparatus has limited their application. This restriction does not apply to sliding in the triaxial test which was introduced by Jaeger (1959) and has since been used by many other workers. In this method, a length of core containing the joint, natural or artificial, is jacketed in rubber tubing and tested under confining pressure in a conventional triaxial apparatus. Provided that the joint is favourably oriented, failure will occur by sliding along the joint rather than in the solid rock and from the load-displacement curve the frictional properties may be deduced. This method will be critically examined in Chapter 7 and only the results previously obtained will be reviewed here.

Jaeger (1959) used 2 in. diameter cores and confining pressures from 3000 to 15,000 psi. For plaster filled joints, the $\tau-\sigma$ failure line was slightly curved and $\mu$ varied
from 0.40 to 0.78. For sawn and ground bare surfaces violent stick-slip oscillations occurred and $\mu$ varied from 0.53 to 0.61. For natural fracture surfaces of porphyry, marble, sandstone and gneiss, $\mu$ varied from 0.52 to 0.86. Wet surfaces of sandstone and gneiss gave slightly lower values of $\mu$. The intercept $c$ was not measured accurately but in all cases was less than 3000 psi. Similar tests using lower confining pressures on natural joints in quartz monzonite, Lane and Heck (1964) yielded $c = 200$ psi and $\mu = 0.62$. Handin and Stearns (1964) used 0.75 in. diameter cores and confining pressures up to 30,000 psi and obtained $\mu = 0.4$ for ground surfaces of dolomite.

A comprehensive series of experiments was conducted by Byerlee (1967a) on ground and fractured surfaces of Westerly granite. He used very small cores, 0.6 in. and 0.5 in. diameter, and very high confining pressures up to 150,000 psi. For both ground and fractured dry surfaces the average friction parameters were $c = 7500$ psi and $\mu = 0.6$. For ground surfaces with additional pore water pressure, $\mu$ remained at 0.6 but $c$ was reduced to 1500 psi; this was attributed to reduction in strength of the asperities by water. Byerlee also recorded stick-slip behaviour when sliding on shear fractures.

Murrell (1966) measured the frictional properties of fracture surfaces of sandstone in the course of a more compre-
hensive study on the strength of rocks. He fitted his results to the power law (eqn 6.3) with constants $\mu_0 = 2.10$ and $n = 0.89$. His results for $\sigma < 20,000$ psi can also be fitted to the linear law (eqn 6.2) with $c = 500$ psi and $\mu = 0.70$.

Raleigh and Paterson (1965) measured the residual friction on faults in serpentinite and found that $\mu (= \tau/\sigma)$ reduced from 0.73 to 0.40 as the confining pressure increased from 11,000 to 75,000 psi.

**Rotating friction tests**

The final method that has been used in measuring friction is that of one surface rotating against a stationary slider, Fig. 6.1(e), or two discs rotating in opposite directions about their common axis, Fig. 6.1(f). The frictional resistance can be calculated by measuring the torque $T$ required to rotate the cylinder under a given normal load $N$.

The first method, using a very sophisticated apparatus but small normal loads, was used by Rae (1963) for a sandstone wheel and sandstone and limestone sliders. For the sandstone slider $\mu = 0.7$ and was independent of sliding speed and for the limestone slider $\mu$ reduced from 0.75 to 0.2 as the speed increased from 0 to 15 ft/sec. A machine using the second method has been built in South Africa but details of its construction and the results obtained have not been published.
The very great advantage of these rotating methods is that they are capable of infinite displacements without change in geometry and are thus well suited for studying the variation of friction with displacement, which is of great importance for movements along joints in a rock mass. Once again, however, specialised apparatus is required.

6.3 DIRECT SHEAR TESTS ON ARTIFICIALLY PREPARED ROCK SURFACES

It will be evident from the preceding brief review of the literature on rock friction that, although the measured coefficients of friction fall within a relatively narrow range, there are many inconsistencies and gaps in the data. These may be summarised as follows.

(i) The relation between frictional behaviour and the nature and roughness of the surfaces is not very clear. In many cases there appears to be surprisingly little difference in behaviour between ground and naturally rough surfaces. In particular, little attention has been paid to the measurement of surface roughness.

(ii) There is very little information on the variation of friction with duration of sliding. It may be expected that after a certain amount of sliding the surfaces will become damaged and slickensided with production of loose detrital material. Alternatively, initially rough surfaces may be made smoother by shearing of protuberances and asperities. In any
case, it would not be surprising to find a large variation in friction with displacement.

(iii) Many of the experiments were made on extremely small surfaces and these may not be relevant to the large surfaces in question in rock mechanics.

(iv) The role of stick-slip oscillations in sliding and their relation to surface roughness is still obscure. They have been suggested as a mechanism for earthquakes by Brace and Byerlee (1966) and so deserve further investigation.

(v) Many of the measurements have used the triaxial method. It will be shown in Chapter 7 that certain difficulties associated with this test need to be clarified.

For these reasons it seemed worthwhile to measure frictional properties of large surfaces of different rocks with controlled surface roughness. This work was done in collaboration with E.R. Hoskins and Professor J.C. Jaeger and a paper has been written for publication in the International Journal of Rock Mechanics and Mining Sciences. A reprint of this paper is included as Appendix F, and the information given therein will not be repeated. However, for completeness, the main conclusions will be briefly summarised.

(i) The sliding behaviour for any rock is profoundly influenced by the degree of surface finish. "Rough" surfaces show steady sliding whereas "smooth" surfaces exhibit stick-slip oscillations. The critical roughness separating the
two types of behaviour is of the order of 100 microinches.
It must be emphasized that these "rough" surfaces are still
very smooth and flat compared with most natural joint surfaces.

(ii) The stick-slip oscillations appear to be similar to
those observed in metallic friction and are due to the co­
efficient of dynamic friction, $\mu'$, being less than the coeffi­
cient of static friction, $\mu$. The frequency of the
oscillations is determined by the difference $(\mu - \mu')$ and the
stiffness of the testing machine. Since these oscillations
only appear with very smooth surfaces, it is not clear whether
they have any practical significance in rock mechanics.

(iii) All the surfaces showed significant variations in
friction with displacement. In every case, the force required
to maintain sliding slowly increases with displacement, the
rate of increase depending on the type and roughness of the
surface. Some surfaces, such as sandstone, reach a steady
value after less than 0.1 in. of sliding. Others, such as
trachyte, show a rising characteristic even after 0.5 in.
of sliding.

(iv) For all the surfaces tested, the linear law (eqn 6.4)
was applicable for normal stresses up to 1000 psi. $\mu$ varied
from 0.51 to 0.75 for rough surfaces and from 0.18 to 0.63
for smooth surfaces. In both cases, c was less than
200 psi.
(v) The difference in friction between rough and smooth surfaces of the same rock varied with rock type. For example, $\mu$ for trachyte was the same in both cases but for gabbro it varied from 0.18 (smooth) to 0.66 (rough). Wet surfaces appeared to show the same behaviour as dry although this point was not exhaustively investigated.

(vi) All the rocks, except marble, showed rather uniform surface behaviour. Rough surfaces showed slickensides with a fine layer of powdered rock while smooth surfaces appeared unaffected except for a slightly higher degree of polish in some cases. On the other hand, the two marbles showed a great deal of slickensiding, with transfer of material between the faces, in a few isolated spots covering some 25 percent of the surface. This was attributed to the well-known ductile behaviour of calcite under relatively low stresses.

6.4 FURTHER DOUBLE SHEAR TESTS ON SMALL BLOCKS

It was apparent from the large double shear tests that the variation in friction with displacement was quite large for certain of the rocks tested. A few preliminary triaxial tests on ground surfaces showed that a rising friction characteristic was also often obtained and since it was intended to use this method for larger displacements than had previously been attempted (Chapter 7), it was clearly important
to separate the effects of changing geometry in the apparatus from the inherent behaviour of the rock. For this reason it was considered that further experiments on this aspect should be performed.

Since a large number of experiments would be required, and in view of the high cost of preparing the 12 in. square blocks, these tests used smaller sliding areas 3 in. square. One advantage of this size was that they were more nearly equal to the size of specimen (2 in. diameter cylinders) to be used in the triaxial tests.

Apparatus

The experimental set-up is shown in Fig. 6.2. The two outer blocks \( L, M \), are 3 in. square \( \times 1\frac{1}{2} \) in. thick and rest on steel packers \( P, P \), 1 in. thick. The centre block, \( N \), is 4 in. long, 3 in. wide and \( 1\frac{1}{2} \) in. thick and is loaded through the steel bearing block \( Q \). Normal force is provided by the 3 in. diameter piston jack, \( J \), and reaction is provided by the heavy steel end blocks, \( A, B \), through two \( 1\frac{1}{2} \) in. diameter tie rods \( D \). Since a fairly rigid piston jack was used instead of the flexible flat jacks in the larger experiments, the block \( M \) is loaded through a spherical seat, \( S \), to eliminate any wedging action caused by slight non-parallelism of the rock blocks. To protect the jack from tilting on the spherical seat, a third steel block, \( C \), freely sliding on
Fig. 6.2. Diagrammatic layout of small double shear apparatus.
the tie rods, is provided. The whole arrangement rests on the lower platen of a 50 ton capacity Avery testing machine.

The load-displacement recorder of the testing machine, which records the displacement of the lower platen through a cord and pulley system, was modified by inserting additional pulleys to give a magnification of 17.5 to 1 on the displacement axis. In the large friction experiments, an X-Y recorder was operated by displacement transducers attached to the machine recorder (to measure load) and between the blocks (to measure displacement). The former arrangement was not very satisfactory because the inertia of the system caused large time lags in measuring sudden changes of load. For this reason a small load cell was manufactured.

This load cell consists of a 3/4 in. long hollow cylinder (2 in. o.d. x 1 in. i.d.) of high tensile steel (Balfours SD50). Cemented to the inner surface at 90 degree intervals are four electrical resistance strain gauges (Philips PR9834K/05, 300 ohm foil), alternate gauges being axial and circumferential to the cylinder. The axial gauges connected in series act as the active arm and the circumferential gauges connected in series act as the dummy arm of the half-bridge which is connected to a Philips PR 9300 direct reading strain gauge bridge. The dummy gauges are actually partially active since these gauges go into tension when the load cell is
compressed along its axis and thus increase the sensitivity. The three wires, active, dummy, and common are led out through three small holes drilled in the wall of the cylinder and the bore of the cylinder is filled with Dow-Corning Silastic 583 RTV cold-vulcanizing silicone rubber to protect the gauges. The capacity of the cell is at least 50 tons. As shown in Fig. 6.2, the load cell, L, is used with another spherical seat, T, to ensure uniformity of loading and the output from the strain gauge bridge is connected to the Y-axis of a Houston HR-100 X-Y recorder. The X-axis of this recorder is fed by a Sanborn 7DCDT-250 direct current displacement transducer set between the platens of the testing machine.

Specimens and procedure

Since it was not desired that this work become too involved, testing was concentrated on only two rocks, both commercial building stones

(i) "Bowral trachyte" - actually a microsyenite, Joplin (1964), consisting of interlocking tabular crystals of orthoclase of grain size 0.5 to 1.0 mm., which constitute some 80% by volume. The other major mineral is aegerine-augite which has been extensively altered to siderite and calcite. It is moderately strong with an unconfined compressive strength of about 25,000 psi. This rock was used in the large friction tests and was chosen for further study because
it showed a rather pronounced variation in friction with displacement.

(ii) "Narrandera quartzite" - an orthoquartzite or silica-cemented sandstone containing 98% quartz, together with fragments of fine-grained sedimentary rocks. The quartz grains are 0.1 to 0.2 mm. dia. and have interlocking, sutured boundaries. It is very strong with an unconfined compressive strength of over 40,000 psi. This rock was chosen for study since it is almost entirely quartz, a mineral of great practical importance in rock mechanics.

Photomicrographs of these rocks are shown in Fig. 6.3.

Each of these rocks was tested with "smooth" and "rough" surfaces, prepared by grinding with carborundum grit on a cast iron lap and then on a glass plate. Roughness was measured with a Brush Model 185 Surfindicator, Hoskins et al. (1968). The properties of these surfaces are given in Table 6.1.

<table>
<thead>
<tr>
<th>Rock</th>
<th>Surface</th>
<th>Carborundum powder</th>
<th>Roughness microinches</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trachyte</td>
<td>Smooth</td>
<td>400 grit</td>
<td>50 ± 5</td>
</tr>
<tr>
<td>Trachyte</td>
<td>Rough</td>
<td>60 grit</td>
<td>500 ± 50</td>
</tr>
<tr>
<td>Quartzite</td>
<td>Smooth</td>
<td>600 grit</td>
<td>130 ± 10</td>
</tr>
<tr>
<td>Quartzite</td>
<td>Rough</td>
<td>60 grit</td>
<td>500 ± 50</td>
</tr>
</tbody>
</table>
Fig. 6.3. Photomicrographs of rocks used in friction experiments. Width of field 9 mm. (crossed nicols).
Since the variation in friction with displacement was of primary interest, a standard, although arbitrary, method of testing was adopted as follows:

(i) Under a given normal load, the middle block was slid for the full inch displacement.

(ii) The rig was dismantled as carefully as possible and the blocks reset without touching the surfaces.

(iii) The middle block was slid in approximately 0.1 in. steps under various normal stresses in the following order:

(a) Normal stress used in initial sliding to re-establish conditions.

(b) Six or seven equally spaced normal stresses between 400 and 3000 psi.

(c) Normal stress used in initial sliding.

For all this work, the sliding speed was approximately 1 in. per hour. The results are summarised below.

**Initial sliding (first inch)**

Sketches of typical testing machine records for the four cases are shown in Figs. 6.4 and 6.5, all under a normal load of 14,100 lb.

For smooth trachyte, Fig. 6.4(a), start of sliding is sharply defined at $\mu = 0.35$. The shear stress then increases fairly rapidly with steady sliding, although in some cases a few irregular stick-slips occur. After 0.2 to 0.4 in.
Fig. 6.4. Testing machine records for double shear tests on (a) smooth and (b) rough, trachyte. $2F = \text{shear force}, \; X = \text{displacement}$. Normal load 14,100 lb.
Fig. 6.5. Testing machine records for double shear tests on (a) smooth, (b) rough, quartzite. $2F =$ shear force, $X =$ displacement. Normal load 14,100 lb.
displacement, regular stick-slip oscillations suddenly commence and continue thereafter. The oscillations quickly reach a stable amplitude although they may be interspersed with larger or smaller ones. The shear stress continues to rise but at a decreasing rate and after 1 in. displacement $\mu$ is of the order of 0.8. After slip the surfaces show faint slickensides but there is little detrital material and the measured roughness is little altered.

For rough trachyte, Fig. 6.4(b) behaviour is generally similar but there is no sharply defined start of sliding and regular stick-slip oscillations do not develop. The shear stress increases with displacement and after 1 in. displacement $\mu$ is again about 0.8. After slip, the surfaces are intensely slickensided and covered with fine white powder. When this is removed, the measured roughness has decreased to about 300 microinch.

For smooth quartzite, Fig. 6.5(a), start of sliding is sharply defined by a stick-slip and regular oscillations continue thereafter. The shear stress slightly decreases and the amplitude of the oscillations tends to increase as sliding progresses. However, the effect of displacement is much less marked than in the trachyte. The coefficient of friction is very much lower, of the order of 0.3. After slip there is no visible damage to the surfaces except that they appear to be more highly polished. The measured roughness
is little altered.

The behaviour of rough quartzite, Fig. 6.5(b) is quite different. Start of sliding is not sharply defined but the load displacement curve quickly establishes itself with a slight downwards slope and $\mu$ is of the order of 0.7. Sliding is steady for about 0.5 in., after which regular stick-slip oscillations with gradually increasing amplitude develop and these continue thereafter. After slip, the surfaces are slickensided and covered with fine white powder. The measured roughness has decreased to about 200 microinch.

**Subsequent sliding**

Typical X-Y records for stages of the second run, after the blocks have been reset, are given in Figs. 6.6 to 6.9, for the same specimens as in Figs. 6.4 and 6.5. For purposes of comparison, the three records in each figure refer to the same stages under the same normal loads, which are shown on each record. The numbers in brackets refer to the previous sliding history of the specimens. The displacement scales on all these records are the same but the load scales are different.

These records are largely self-explanatory and interest centres mainly on the shape of the stick-slip oscillations. When compared with similar records of the large friction experiments, Hoskins et al. (1968), the oscillations are more nearly of the theoretical shape because of the fast response
Fig. 6.6. Typical X-Y records for second run on smooth trachyte. Previous sliding histories (a) 1.0 ins, (b) 1.1 ins, (c) 1.5 ins. Normal loads (a) 14,100 lb, (b) 3500 lb, (c) 17,500 lb.
Fig. 6.7. Typical X-Y records for second run on rough trachyte. Previous sliding histories (a) 1.0 ins, (b) 1.1 ins, (c) 1.5 ins. Normal loads (a) 14,100 lb, (b) 3500 lb, (c) 17500 lb.
Fig. 6.8. Typical X-Y records for second run on smooth quartzite. Previous sliding histories (a) 1.0 ins, (b) 1.1 ins, (c) 1.5 ins. Normal loads (a) 14,100 lb, (b) 3500 lb, (c) 17,500 lb.
Fig. 6.9. Typical X-Y records for second run on rough quartzite. Previous sliding histories (a) 1.0 in, (b) 1.1 ins, (c) 1.5 ins. Normal loads (a) 14,100 lb, (b) 3500 lb, (c) 17,500 lb.
of the load cell.

In each record, the oscillations are extremely uniform indicating a constant difference between the coefficients of static (μ) and dynamic (μ') friction. The smooth trachyte characteristically shows two types of oscillation, Fig. 6.6(a), but each of these is of uniform amplitude. In general, the large amplitude oscillations are favored as sliding progresses, Fig. 6.6(c). On the other hand, for both smooth and rough quartzite, the oscillations have a uniform amplitude which, however, tends to slightly increase as sliding progresses. This difference in behaviour may possibly be explained by the fact that the quartzite is more nearly a mono-minerallic rock with a smaller grain size.

The quantity (F-F*)/X where F and F* are the slip and stick loads respectively and X is the displacement during the slip is approximately constant and of the order of 0.8 x 10^6 lb/in. This is in good agreement with the estimated stiffness of the testing machine, which is of the order of 10^6 lb/in.

The small peak in Figs. 6.6(b), 6.7(b) and 6.9(b) is characteristic of nearly all rock surfaces tested when the normal stress is reduced from a previously higher value. This is presumably caused by the surfaces being more closely interlocked at the higher normal stress.
Normal-shear stress relations

Stresses for the second run of all the tests were plotted individually and \( c \) and \( \mu \) estimated from the straight lines of best fit. The results are summarised in Table 6.2. The coefficients of friction quoted are as follows:

(i) \( \mu \) is slope of line corresponding to slip stress and is the coefficient of static friction;
(ii) \( \mu^* \) is slope of line corresponding to stick stress;
(iii) \( \mu' \) is the coefficient of dynamic friction and since the present apparatus approximates the simple system of Fig. 6.1(a), is equal to the mean of \( \mu \) and \( \mu^* \), Hoskins et al. (1968).

**TABLE 6.2**

<table>
<thead>
<tr>
<th>Rock</th>
<th>Surface</th>
<th>Test No. in normal stress psi</th>
<th>Running-in normal stress psi</th>
<th>( \mu )</th>
<th>( \mu^* )</th>
<th>( \mu' )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trachyte</td>
<td>Smooth</td>
<td>10 420</td>
<td>100</td>
<td>0.82</td>
<td>0.62</td>
<td>0.72</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8 850</td>
<td>120</td>
<td>0.75</td>
<td>0.53</td>
<td>0.64</td>
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<tr>
<td></td>
<td></td>
<td>12 1700</td>
<td>80</td>
<td>0.80</td>
<td>0.58</td>
<td>0.69</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6 2550</td>
<td>50</td>
<td>0.86</td>
<td>0.58</td>
<td>0.72</td>
</tr>
<tr>
<td></td>
<td>Rough</td>
<td>21 850</td>
<td>0</td>
<td>0.87</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>25 1700</td>
<td>0</td>
<td>0.86</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Quartzite</td>
<td>Smooth</td>
<td>14 1800</td>
<td>0</td>
<td>0.29</td>
<td>0.16</td>
<td>0.22</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16 2700</td>
<td>0</td>
<td>0.31</td>
<td>0.14</td>
<td>0.22</td>
</tr>
<tr>
<td></td>
<td>Rough</td>
<td>27 1800</td>
<td>0</td>
<td>0.68</td>
<td>0.61</td>
<td>0.64</td>
</tr>
<tr>
<td></td>
<td></td>
<td>29 2700</td>
<td>0</td>
<td>0.72</td>
<td>0.63</td>
<td>0.67</td>
</tr>
</tbody>
</table>
These results, which may be regarded as residual values, are averaged graphically in Figs. 6.10 and 6.11 for trachyte and quartzite respectively. The average parameters are given in Table 6.3.

Table 6.3

<table>
<thead>
<tr>
<th></th>
<th>c psi</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Trachyte</td>
<td>Smooth</td>
<td>50</td>
<td>0.83</td>
<td>0.71</td>
</tr>
<tr>
<td></td>
<td>Rough</td>
<td>0</td>
<td>0.87</td>
<td></td>
</tr>
<tr>
<td>Quartzite</td>
<td>Smooth</td>
<td>0</td>
<td>0.30</td>
<td>0.22</td>
</tr>
<tr>
<td></td>
<td>Rough</td>
<td>0</td>
<td>0.70</td>
<td>0.66</td>
</tr>
</tbody>
</table>

Shear and normal stresses for virgin sliding of smooth trachyte are shown in Fig. 6.12 for the following conditions:
(i) initiation of sliding;
(ii) 0.5 in. displacement;
(iii) 1.0 in. displacement.

These three cases fit straight lines reasonably well with \( c = 50 \) psi and \( \mu = 0.35, 0.75 \) and 0.85 respectively. The value for 1 in. displacement is not significantly different from the residual value and that for 0.5 in. displacement is only slightly lower.

Conclusions

Although specific conclusions have been given above the following general statements can be made:
Fig. 6.10. Residual $J-\tau$ diagrams for trachyte (a) smooth, (b) rough. Symbols refer to different tests.
Fig. 6.11. Residual \( J - \tau \) diagrams for quartzite (a) smooth, (b) rough. Symbols refer to different tests.
Fig. 6.12. $\sigma$-$\tau$ diagram for smooth trachyte. Dots = initiation of sliding, crosses = after 1 in of sliding, circles = after 2 in sliding.
(i) Certain rocks (e.g. Bowral trachyte) show a marked variation in friction with displacement whereas others (e.g. Narrandera quartzite) do not. There is little doubt that this behaviour is caused by wear during the sliding process. It is significant that both smooth trachyte and smooth quartzite show similar $\mu$ values for initiation of sliding (0.35 and 0.30). The quartzite is a very hard rock and there is negligible change in the nature of surface as sliding progresses and $\mu$ does not alter. On the other hand, the trachyte is much softer and the surface damage causes the friction to rise as sliding progresses. A puzzling point, however, is that the wear of the smooth trachyte surfaces is not nearly as great as that of the rough surfaces even though the residual friction values are very similar. The behaviour in terms of stick-slip oscillations is also quite different. Approximately one inch of sliding is required for the trachyte to reach the residual condition, and the lower friction values obtained by the large double shear tests arise from the smaller displacements in these experiments.

(ii) Although the residual value of $\mu$ for trachyte appears to be largely independent of roughness, that for quartzite is certainly not so. Although this point was not tested, it is probable that intermediate values of roughness would show intermediate friction behaviour and also that smoother surfaces
would give lower values of $\mu$ down to the minimum of 0.1 to 0.15 predicted by Byerlee's (1967b) theory. As suggested earlier, much of the conflicting friction data on rocks (in particular, quartz) could be resolved by considering differences in surface roughness. It is probable, however, that $\mu = 0.7$ is a maximum value for quartz-rich rocks since much higher values have not been reported in the literature (Section 6.2).

(iii) Stick-slip behaviour is dependent primarily on surface roughness, as shown by Hoskins et al. (1968), and also on rock type. For trachyte, beyond a certain roughness, stick-slip oscillations are evidently not possible whereas they occur in quartzite even with an initial roughness of 500 microinch. It is likely, however, that the stick-slips in quartzite develop as the surfaces are made smoother by wear as shown by the gradually increasing amplitude of the oscillations, Fig. 6.6(b). It is not possible to reconcile the present results with those of Brace and Byerlee (1966) who found stick-slip behaviour for a variety of surfaces, ranging from polished to rough fracture surfaces, with Westerly granite, a rock consisting of 66% feldspar and only 28% quartz, Byerlee (1967b). The oscillations in their experiments were not nearly as regular as in the present ones and it is possible that some other mechanism such as sudden fracture of protruding grains is involved. This is feasible because the normal stresses used were very much
higher than in the present experiments. Also, it is possible that experimental inadequacies with the triaxial method were a contributing factor (see Chapter 7).

Briefly, these conclusions may be summarised by saying that friction measurements on ground rock surfaces are not reproducible or meaningful unless the surface roughness and previous sliding history of the surfaces are specified.
Chapter 7

MEASUREMENT OF FRICTION IN THE TRIAXIAL APPARATUS

7.1 INTRODUCTION

The technique of sliding on a pre-determined plane of weakness in the triaxial test appears to have been first used by the U.S. Bureau of Reclamation (1954) for testing the bond strength between concrete and rock. It was adapted for measurement of sliding friction between the surfaces of a joint by Jaeger (1959) and has since been used extensively for this purpose (refer Chapter 6). The popularity of this method is due to certain advantages, notably:

(i) The apparatus is simple and usually readily available.
(ii) Minimal specimen preparation is required.
(iii) High normal stresses can be readily applied by the confining pressure.
(iv) Pore pressures can be more readily introduced than in other types of apparatus.

Against these advantages there are some uncertainties which require consideration.

(i) The normal stress is not constant during a test but varies with the shear stress.
(ii) The nominal area of contact between the faces is continually changing as sliding proceeds.
(iii) Unknown lateral stresses may be set up at the ends of the cylinder during a test.

These points make the test difficult to interpret, particularly for large displacements, and the method is generally regarded as not being a good one when large displacements are involved. As shown in Chapter 6 friction measurements on certain rock surfaces may not be meaningful unless the variation with displacement is established.

In this chapter, the usual elementary theory for the initiation of sliding is extended to include large displacements on the shear plane both with lateral translation and with rotation of the two halves of the specimen. The apparatus used and a series of sliding tests on artificial joints, designed to perfect the experimental techniques and test the theoretical predictions, are then described.

7.2 ELEMENTARY THEORY

Consider a cylinder containing a planar joint inclined at angle \( \alpha \) to its axis, subjected to principal stresses \( \sigma_1 \) and \( \sigma_3 \), Fig. 7.1(a). The normal and shear stresses on the plane are given by Jaeger (1962):

\[
\sigma = \sigma_1 \sin^2 \alpha + \sigma_3 \cos^2 \alpha = \sigma_3 + (\sigma_1 - \sigma_3) \sin^2 \alpha \quad (7.1)
\]

\[
\tau = (\sigma_1 - \sigma_3) \sin \alpha \cos \alpha \quad (7.2)
\]
Fig. 7.1. Sliding in the triaxial test.

Fig. 7.2. $\sigma$-$\tau$ diagram for sliding in the triaxial test.
Eqns 7.1 and 7.2 can be combined to give

\[ \sigma = \sigma_3 + \tau \tan \alpha \]  \hspace{1cm} (7.3)

In frictional studies, the fundamental relation is that between \( \tau \) and \( \sigma \) during sliding. It is therefore convenient to plot the stress path on a \( \sigma-\tau \) diagram, which is also the Mohr diagram, Fig. 7.2. The assumed relation for slip in the joint is the straight line DE with intercept, \( \phi = \text{artan} \alpha \), which is the linear relation, eqn 6.2. Under hydrostatic pressure \( \sigma_3 \), before the differential stress \( (\sigma_1-\sigma_3) \) is applied, the shear stress on the plane is zero and the normal stress equal to \( \sigma_3 \), point A in Fig. 7.2. As the differential stress increases, both normal and shear stresses increase, following the path AC inclined at angle \( \alpha \) to the \( \tau \) axis, which is the relation eqn 7.3. Distances such as AL along the stress path are equal to \( (\sigma_1-\sigma_3) \sin \alpha \) and these are also the chords of the Mohr circles originating at A. Eventually the stress path will reach the failure line DE and sliding will commence under the normal and shear stresses corresponding to point C.

This representation shows how the differential stress required to cause sliding increases with confining pressure and also with \( \alpha \). AH is the stress path for a direct shear test, \( \alpha = 0 \) and \( \sigma \) constant. As \( \alpha \) increases towards
the intersection point \( C \) becomes poorly defined and instability may occur leading to very large changes in differential stress for small changes in \( \alpha \). An example will be given later. Clearly, if \( \alpha = 90^\circ - \phi \), the stress path will never intersect the failure line and sliding on the joint is impossible. In this case, the differential stress will increase until the Mohr circle based on \( A \) reaches the failure envelope for the intact rock at which stage fracture will occur in the solid rock ignoring the joint. Depending on the strength of the rock this situation can occur for values of \( \alpha \) somewhat less than the critical value. The lower limit of \( \alpha \) is determined by the geometry of the cylinder and is of the order of \( 25^\circ \).

If the joint is filled or sufficiently interlocked, stresses corresponding to point \( C \) will be insufficient to initiate sliding. The stresses will then follow the path \( AB \) to point \( K \), at which stage shear failure of the filling or shearing of the asperities will occur and the stresses will return along \( AB \) to point \( C \) defining the residual strength of the joint. The peak strength defined by point \( K \) lies on the line defining the failure criterion of the filling or of the asperities, which may be assumed to have the linear form

\[
\tau = c_0 + \mu_o \sigma
\] (7.4)
where $c_0$ is the "peak" cohesion and $\mu_0$ is a coefficient of internal friction. The situation is no doubt a good deal more complex than this in view of Patton's (1966) work on multiple modes of shear failure but additional refinements seem unjustified at this stage. In any case, each joint must be regarded as an individual so that only one experiment leading to failure may be performed and $c_0$ and $\mu_0$ cannot both be determined. Since $\mu$ and $\mu_0$ are likely to be of the same order a simple approximation is to assume $\mu = \mu_0$ so that the failure line FG through K is parallel to DE and the intercept OF is equal to $c_0$, defined as the "peak cohesion".

The above analysis refers strictly only to the initiation of sliding with frictionless end conditions for the cylinder. As sliding proceeds, the situation is complicated by the change in geometry of the system and the change in contact area between the surfaces. If rigid platens are used in the triaxial bomb the ends of the sample must translate, Fig. 7.1(b), and work must be done by the differential stress to overcome frictional resistance at the platens. Most previous investigators have used one spherical seat in the system. However, as Jaeger (1959) pointed out, as sliding proceeds the situation becomes that of Fig. 7.1(c) and it is impossible to maintain uniform normal stress across the
joint. The sample then usually fails by secondary splitting. The only other way to maintain the specimen geometry is to use two spherical seats, Fig. 7.1(d).

Both the systems in Fig. 7.1(b) and (d) have been used in the present experiments and the theory for large displacements for both of them is set out below. Firstly, however, the effect of change in area of contact will be considered.

Change in contact area with sliding

If the relative displacement along the joint is s, the ends of the cylinder come together by a, and the axis of the cylinder is displaced by b, Fig. 7.3(a). These quantities are related by

\[ a = s \cos \alpha \]  
\[ b = s \sin \alpha \]

(7.5)  
(7.6)

The elliptical faces of the joint are displaced by distance s and the true area of contact is \( A_1 \), Fig. 7.3(b). On each face of the joint an area \( A_2 \) is no longer in contact. Corresponding areas on a horizontal projection are \( A_1' \) and \( A_2' \), Fig. 7.3(c). These are given by

\[ A_1' = \frac{D^2(2\theta - \sin 2\theta)}{4} \]  
\[ A_2' = \frac{D^2(\pi - 2\theta + \sin 2\theta)}{4} \]

(7.7)  
(7.8)
Fig. 7.3. Change in contact area during sliding.
where $D$ is the diameter of the cylinder and $\theta$ is as shown in Fig. 7.3(c) and given by

$$\cos \theta = \frac{a \tan \alpha}{D} \quad (7.9)$$

The areas on the joint faces are then given by

$$A_1 = \frac{D^2(2\theta - \sin 2\theta)}{4 \sin \alpha} \quad (7.10)$$

$$A_2 = \frac{D^2(\pi - 2\theta + \sin 2\theta)}{4 \sin \alpha} \quad (7.11)$$

A convenient approximation when $s$ is small can be calculated by reference to Fig. 7.3(d). The area $A_2$ is given approximately by

$$A_2 \approx \frac{\pi D^2}{\sin \alpha} + D.s - \frac{\pi D^2}{\sin \alpha} = D.s \quad (7.12)$$

and

$$A_1 = \frac{\pi D^2}{\sin \alpha} - D.s \quad (7.13)$$

There is, therefore, an approximately linear relation between true area of contact and small displacements on the shear plane.

7.3 SLIDING WITH LATERAL DISPLACEMENT

In this part of the work it is convenient to work in terms of resultant forces. Stresses can be calculated at
any time if the appropriate areas are known.

Consider, firstly, a solid in the shape of two displaced half cylinders of diameter $D$ and cross sectional area $A$, immersed in a fluid under pressure, $\sigma_3$. The resultant forces in the plane of the paper are shown in Fig. 7.4(a) and are

$$P_1 = \sigma_3 A$$ (7.14)

$$P_2 = \sigma_3 \frac{A}{\tan \alpha}$$ (7.15)

$$P_3 = \sigma_3 A_2$$ (7.16)

where $A_2$ is given by eqn 7.11. Other forces act on the curved surfaces of the cylinder, but since they are equal, opposite and collinear they may be omitted from further discussion. Since the solid is under hydrostatic pressure, the forces $P_1$, $P_2$ and $P_3$ must be in complete equilibrium and it is possible to calculate the point of action of force $P_3$, if required, without knowing the centroid of the area $A_2$. Therefore, any additional non-hydrostatic forces applied to the body must themselves be in complete equilibrium.

We now consider a cylinder sliding in a triaxial test with differential load $R_1$ and lateral force $R_2$ due to frictional restraint at the platens, Fig. 7.4(b). The point of action of $R_1$ is indeterminate. However, since the
Fig. 7.4. Resultant forces acting during sliding with translation.
non-hydrostatic forces must be in complete equilibrium
the resultants of $R_1$ and $R_2$ at each end of the cylinder
must be collinear and therefore the line of action of the
resultant must make an angle, $\psi = \arctan \left( \frac{R_2}{R_1} \right)$, with the
axis of the cylinder. In fact, the point of action of $R_1$
is not relevant to this problem and for the symmetrical
case, Fig. 7.4(b), it will be assumed that the resultant
passes through the centroid of the contact area, $A_1$.

The lower half of the specimen is shown in Fig. 7.4(c)
with the forces $P_1$, $P_2$, $P_3$ from the confining pressure,
$R_1$, $R_2$ from the differential load and $N$ and $F$ from the
upper half of the specimen. Resolving along and at right
angles to the cylinder axis, we obtain

$$\begin{align*}
N \sin \alpha + F \cos \alpha &= R_1 + P_1 - P_3 \sin \alpha \quad (7.17) \\
N \cos \alpha - F \sin \alpha &= R_2 + P_2 - P_3 \cos \alpha \quad (7.18)
\end{align*}$$

In the particular apparatus used here, the ends of the
specimen and the platens are sealed from the confining fluid
by a rubber jacket and $R_2$, the frictional force at the
platen, is given by

$$R_2 = k(R_1 + P_1) \quad (7.19)$$

where $k$ is the coefficient between rock and platen, which is
assumed to be constant. Using eqn 7.19, eqns 7.17 and 7.18 can be rearranged to give

\[ N = (R_1 + P_1)(\sin \alpha + k \cos \alpha) + P_2 \cos \alpha - P_3 \]  
\[ F = (R_1 + P_1)(\cos \alpha - k \sin \alpha) - P_2 \sin \alpha \]

The normal and shear stresses can be obtained by dividing by the instantaneous contact area (eqn 7.10) but the expression becomes rather complicated.

One case of interest, however, is that of the initiation of sliding when \( A_1 = \frac{A}{\sin \alpha} \) and \( P_3 = 0 \). Eqns 7.20 and 7.21 then reduce to

\[ \sigma = \sigma_1 \sin \alpha (\sin \alpha + k \cos \alpha) + \sigma_3 \cos^2 \alpha \]  
\[ \tau = \sigma_1 \sin \alpha (\cos \alpha - k \sin \alpha) - \sigma_3 \sin \alpha \cos \alpha \]

These equations are only valid for \( \sigma_1 > 0 \). When the ends of the cylinder are frictionless, \( k = 0 \), they reduce to eqns 7.1 and 7.2.

As an example of the effect of platen friction, values of the ratio \( (\sigma_1 - \sigma_3)/\sigma_3 \) required to initiate sliding for the case \( \alpha = 40^\circ \) and various values of \( \mu \) and \( k \) are given in Table 7.1.
TABLE 7.1

<table>
<thead>
<tr>
<th>k</th>
<th>((\sigma_1 - \sigma_3)/\sigma_3) for values of</th>
<th>0.3</th>
<th>0.4</th>
<th>0.5</th>
<th>0.6</th>
<th>0.7</th>
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<tbody>
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<td>0</td>
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<td>0.81</td>
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<td>5.09</td>
</tr>
<tr>
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<td>0.84</td>
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<td>1.81</td>
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<td>5.25</td>
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<tr>
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<td>4.46</td>
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<td>1.14</td>
<td>1.73</td>
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<td>3.86</td>
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<td>10.90</td>
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<td>1.61</td>
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<td>16.50</td>
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<tr>
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<td>7.93</td>
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<tr>
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<td>7.75</td>
<td>34.60</td>
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<tr>
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<td>6.60</td>
<td>32.00</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
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</tbody>
</table>

* Sliding on joint not possible

It is evident that unless these frictional effects are eliminated, or at least precisely known so that corrections may be made, very significant errors may be introduced by assuming that there are no end effects on the specimen.

Eqns 7.22 and 7.23 may be rearranged to yield

\[
\sigma = \sigma_3 \left(\frac{\cos \alpha}{\cos \alpha - k \sin \alpha}\right) + \tau \left(\frac{\sin \alpha + k \cos \alpha}{\cos \alpha - k \sin \alpha}\right) \quad (7.24)
\]

for \(\tau > 0\), which may be compared with eqn 7.3.
Since a differential load-displacement curve is the output from a test, it is also instructive to investigate the variation of differential stress with displacement for a rock with a constant value of \( \mu \). Although most rock surfaces show a cohesion intercept when the results are reduced in terms of constant \( \mu \), for purposes of illustration the equations are somewhat simplified, but of the same form, by ignoring the cohesion term. Therefore, putting \( F = \mu N \), and using eqns 7.20 and 7.21, we have

\[
R_1 = \frac{P_1 \left\{ (\mu+k)\sin\alpha - (1-\mu k)\cos\alpha \right\} + P_2 (\sin\alpha + \mu \cos\alpha) - \mu P_3}{(1-\mu k)\cos\alpha - (\mu+k)\sin\alpha}
\]

(7.25)

The stress obtained by dividing \( R_1 \) by \( A \), the cross-sectional area of the cylinder, is not a principal stress because there are shear stresses from force \( R_2 \) on the end of the cylinder. However, since it is the differential stress measured by the testing machine it is convenient to retain the notation \((\sigma_1-\sigma_3)\) for comparison with the normal frictionless case. This is given by

\[
\sigma_1-\sigma_3 = \sigma_3 \left\{ \frac{\mu}{\sin\alpha + ks\sin\alpha + \mu k\cos\alpha - \frac{\mu k A}{1-\mu k\cos\alpha - (\mu+k)\sin\alpha}} \right\}
\]

(7.26)

The only variable in eqn 7.25 is \( A_2 \) and therefore as sliding proceeds, \( A_2 \) increases and \((\sigma_1-\sigma_3)\) decreases. A
computer program was written to solve eqn 7.26 for any
nominated values of $\mu$, $k$ and $\alpha$ and for various values of
axial displacement. Typical differential stress-displacement
curves predicted by this equation for the case $\alpha = 40^\circ$ and
several values of $k$ and $\mu$ are shown in Fig. 7.5. Since
the relations are linear, this indicates that the change in
area relation, eqn 7.13, is a very good approximation.

7.4 SLIDING WITH ROTATION

This is the case shown in Fig. 7.1(d) with a spherical
seat at each end of the specimen. The geometric constraints
of this system are:

(i) Each half of the specimen must rotate about the centre
of its respective spherical seat.

(ii) The centres of the spherical seats must move along the
axis of the apparatus. This only applies if the outer
parts of the spherical seats are fixed relative to the
apparatus. If the spherical seats are free to translate
laterally it is possible to have the more complicated
situation of combined rotation and lateral translation.
However, only the case of pure rotation will be considered
here.

With regard to the geometry of the spherical seats there
are three basic cases.
Fig. 7.5. Theoretical load-displacement curves for sliding with translation for 2 in. diameter cylinder and $\alpha = 40^\circ$.
$X =$ axial displacement.
**Case A:** The centre of each spherical seat may lie within its respective half of the specimen, Fig. 7.6(a).

**Case B:** The spherical seats may be reversed so that the centres lie outside of, but on the same side of the shear plane as, the respective halves of the specimen, Fig. 7.6(b).

**Case C:** The centres of each spherical seat may lie on the opposite side of the shear plane to its respective half of the specimen, Fig. 7.6(c).

There is no restriction on the radii of the spherical seats and the two seats need not have the same radius. Any combination of the three cases above is possible, but not necessarily practical. Three combinations are shown in Fig. 7.7. The only inadmissible situation is that where the centres of the two seats coincide, in which case the system is unstable since the specimen can rotate as a rigid body. Clearly, if the centres are close together, the system is quasi-stable since large rotations will be required to accommodate small displacements on the slip plane.

In cases A and B, the shear plane rotates away from the axis of the apparatus and the centres of the spherical seats approach each other as shearing proceeds. In case C, the shear plane rotates towards the axis and the centres move apart. The case of pure translation discussed previously
Fig. 7.6. Three basic cases of sliding with rotation.
is a special case of case C, with seats of infinite radius.

Each half of the specimen rotates through the same angle, which is readily calculated. If the axial displacement is \( x \), the displacement on the shear plane \( s \), and the original separation of spherical seats \( d \), we have in the triangle OKL, Fig. 7.6(a), for cases A and B,

\[
\begin{align*}
KL &= \frac{s}{2} \\
OL &= \frac{d}{2} \\
OK &= \frac{d-x}{2} \\
\angle OLK &= \alpha
\end{align*}
\]

from which

\[
\delta = \arcsin \left( \frac{dsin\alpha}{d-x} \right) - \alpha \quad (7.27)
\]

and

\[
s = (d-x) \frac{\sin \delta}{\sin \alpha} \quad (7.28)
\]

For case C, the equivalent equations are

\[
\delta = \alpha - \arcsin \left( \frac{dsin\alpha}{d+x} \right) \quad (7.29)
\]

and

\[
s = (d+x) \frac{\sin \delta}{\sin \alpha} \quad (7.30)
\]

The equations 7.27 to 7.30 hold for unsymmetrical spherical
seats, Fig. 7.7, and also for the case when the shear plane
does not pass through the middle of the cylinder. The change
in area of contact can then be calculated using eqns 7.5, 7.9,
7.10 and 7.11.

Next, consider the resultant forces acting during sliding.
The forces $P_1$, $P_2$, $P_3$, from the confining pressure are the
same as in the translation case and are again in complete
equilibrium. The resultant non-hydrostatic forces at the
ends of the specimen must also again be collinear but in
this case their point of action is more precisely defined.
The distribution of stress in the spherical seat is statically
indeterminate but it is known that the resultant stress at
each point must be inclined at angle $\phi_s$ to the radius
vector, where $\phi_s = \arctan(k)$ is the angle of friction for
sliding at faces of the seat. Irrespective of the distri­
bution of stress in the seat, the resultant force is very
nearly tangent to the "$\phi$-circle", which is a circle centred
on the centre of the seat and of radius $r \sin \phi_s$, where $r$
is the radius of curvature of the seat, Taylor (1948). The
line of action of the resultant forces is therefore tangent
to both $\phi$ circles, Fig. 7.6. This method can be extended
to non-symmetrical cases, Fig. 7.7. However, if the seats
are non-symmetrical or if the shear plane does not pass
through the centre of the specimen, the resultant force on
the face of the joint does not pass through the centroid of
the contact area. This implies a non-uniform distribution of stress on the joint.

The equilibrium of one half of the cylinder will be considered for case A only since the other two cases automatically follow. The resultant non-hydrostatic force is resolved into two components, $R_1$ along the axis of the apparatus and $R_2$ at right angles, Fig. 7.6(a). In this case $R_1$ is not strictly the "differential load" but since this is the force measured by the testing machine it will be treated as such. Resolving, we have

\[
N \sin(\alpha + \delta) + F \cos(\alpha + \delta) = R_1 + P_1 \cos \delta + P_2 \sin \delta - P_3 \sin(\alpha + \delta)
\]

(7.31)

\[
N \cos(\alpha + \delta) - F \sin(\alpha + \delta) = R_2 - P_1 \sin \delta + P_2 \cos \delta - P_3 \cos(\alpha + \delta)
\]

(7.32)

Now, from the geometry of Fig. 7.6(a), $R_2$ is a definite ratio of $R_1$. However, since the spherical seats are inside the rubber jacket with the specimen, the confining pressure also contributes to the normal stress on the faces of the seat and a more general relation is therefore

\[
R_2 = k'(R_1 + P_1)
\]

(7.33)

where, in general, $k'$ will be little different from $k$, 

the coefficient of friction in the spherical seat. Eqns 7.31, 7.32 and 7.33 then yield

\[ N = R_1 (\sin(\alpha+\delta) + k' \cos(\alpha+\delta)) + P_1 (\sin\alpha + k' \cos(\alpha+\delta)) + P_2 \cos\alpha - P_3 \]  
\[ F = R_1 (\cos(\alpha+\delta) - k' \sin(\alpha+\delta)) + P_1 (\cos\alpha - k' \sin(\alpha+\delta)) - P_2 \sin\alpha \]  

(7.34)  
(7.35)

The normal and shear stresses can be obtained by dividing \( N \) and \( F \) by the instantaneous contact area, \( A_1 \).

At the initiation of sliding, \( A_1 = \frac{A}{\sin\alpha}, P_3 = 0, \delta = 0 \) and the equations reduce to eqns 7.22 and 7.23 for \( k' = k \).

As in the lateral translation case, for a rock with constant \( \mu \) the differential load during sliding is given by

\[
R_1 = P_1 \left\{ \frac{\mu \sin\alpha - \cos\alpha + \mu k' \cos(\alpha+\delta) + k' \sin(\alpha+\delta)}{(1-\mu k') \cos(\alpha+\delta) - (\mu + k') \sin(\alpha+\delta)} + P_2 (\mu \cos\alpha + \sin\alpha) - \mu P_3 \right\}
\]

(7.36)

and in terms of stresses

\[
\sigma_1 - \sigma_3 = \sigma_3 \left\{ \frac{\mu}{\sin\alpha} + k' \sin(\alpha+\delta) + \mu k' \cos(\alpha+\delta) - \frac{A_2}{(1-\mu k') \cos(\alpha+\delta) - (\mu + k') \sin(\alpha+\delta)} \right\}
\]

(7.37)

The whole of the previous discussion applies directly to case B and also to case C provided that \( \delta \) is replaced by \(-\delta\).
The variation of \((\sigma_1-\sigma_3)\) is not obvious in this case since both \(A_2\) and \(\delta\) are variables. A computer program was written to solve eqn 7.36 for any nominated values of \(\mu\), \(k'\), \(\alpha\) and \(d\) (distance between centres of spherical seats) for various values of axial displacement for any of the cases A,B,C. Some typical results are plotted in Fig.7.8 for \(\alpha = 30^\circ, 40^\circ, 50^\circ, \mu = 0.3\) and 0.7. These curves apply to a 2" dia. x 4\(\frac{1}{2}\)" long cylinder and the dimensions of the spherical seats are:

<table>
<thead>
<tr>
<th>Case</th>
<th>Radius</th>
<th>Distance between centres</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1(\frac{1}{4})&quot;</td>
<td>3(\frac{1}{4})&quot;</td>
</tr>
<tr>
<td>B</td>
<td>1(\frac{1}{4})&quot;</td>
<td>6(\frac{3}{4})&quot;</td>
</tr>
<tr>
<td>C</td>
<td>5(\frac{3}{4})&quot;</td>
<td>6(\frac{3}{4})&quot;</td>
</tr>
</tbody>
</table>

These are approximately the proportions of the cases in Fig. 7.6. For comparison the translation case is also shown (curve T) and in all cases \(k' = k = 0.005\).

It is clear from these curves that the interpretation of load-displacement curves, even for a rock with constant \(\mu\), is by no means straightforward. The shape of the load-displacement curve depends on

(i) the geometry of the system
(ii) the angle \(\alpha\)
(iii) the value of \(\mu\)
(iv) the value of \(k\) (shown in Fig. 7.5)
Fig. 7.8. Theoretical load-displacement curves for sliding with rotation for 2 in. diameter cylinder and $k = 0.005$. (a) $\alpha = 30^\circ$, (b) $\alpha = 40^\circ$, (c) $\alpha = 50^\circ$. $X$ = axial displacement.
and is a result of the interaction of the change in contact area factor and the change in shear angle factor. The effect of change in contact area alone is shown by curve T while curves A and B, and C, show the added influence of rotation of the shear plane away from and towards the load axis respectively.

A surprising result is the large influence of $\mu$ on the shape of the curves. For $\mu = 0.3$ the shape (and magnitude) is little influenced by $\alpha$ whereas from $\mu = 0.7$, variations in $\alpha$ have profound effects.

7.5 APPARATUS AND TECHNIQUES

When the work commenced, it was considered that the system using two spherical seats was more likely to succeed in practice than the translation method. Although lubricated ends consisting of latex sheet and silicone grease on polished platens had been successfully developed for soil testing, Rowe and Barden (1964), it was felt that such a system would be quite unsuitable for the large axial stresses required in rock mechanics. It was visualised that for low friction translation, a system of hardened steel rollers would be required and this would introduce practical difficulties because of its fixed translation direction.

Accordingly, two identical spherical seats with a radius of $1^{1/4}''$ were manufactured. These seats were lubricated with Molybond GA-50 anti-sieze compound and during the course
Fig. 7.9. Frictional properties of Molybond GA-50 measured on 2 in diameter discs in double shear apparatus.
of this work it was discovered that this material had extraordinarily low friction properties under high normal stresses. It was then that the lateral translation method was considered practical and in view of its inherently simpler nature it was used on most of the later testing.

The frictional properties of Molybond GA-50 were investigated in the double shear apparatus, Fig. 6.2. Three 2 in. dia. hardened steel discs were lubricated and the middle one displaced by a shear force. The results of these tests are shown in Fig. 7.9 in which the vertical scale has a 50 to 1 exaggeration. There was some scatter of results but the ranges of shear forces are shown by the vertical bars. Obviously, any linear relation must have a significant cohesion intercept but in view of the very low friction it was considered sufficiently accurate to use a zero intercept. A value $k = 0.005$ fits the results fairly well.

This very low value was unexpected since Bowden and Tabor (1964) quote $\mu$ of the order of 0.1 for MoS$_2$ on steel in air. Molybond GA-50 consists of 50 percent MoS$_2$ and a greasy binder, which appears to be completely squeezed from between the surfaces under moderate normal pressures. The MoS$_2$ remains as a dry lubricant and it is considered that the mechanism of lubrication is one of plastic shear rather than frictional sliding. It is known that most crystals
deforming plastically have coefficients of internal friction near zero (ref. Chapter 5). This explains the nature of Fig. 7.9. The value \( k = 0.005 \) was adopted in analysis of tests.

The triaxial bomb used in the experiments was a USBR type for 2 in. dia. cylinders, described by Paterson (1958). In view of the importance of water pressures in practical applications of sliding on joints, the bomb was modified to permit circulation of water under pressure through the specimens, Fig. 7.10. The piston, \( P \), of the bomb was provided with two drainage ducts \( A, B \), and the circuit was completed by the coil, \( C \), of \( 1/16 \) in. bore copper tubing, which offers no restraint to deformation of the specimen. This tubing was sealed to the piston and to the bearing block, \( E \), by small O-rings compressed by a small collar soldered on to the tubing. Since it is very difficult to prepare half cylinders with parallel ends an unlubricated spherical seat, \( F \), was included to provide initial alignment. This and the two \( 3/16 \) in. thick hardened steel discs lubricated with Molybond, \( D \), at each end of the specimen were centrally drilled to permit passage of water. The whole system was enclosed by \( 1/8 \) in. thick soft rubber jacket, \( J \), sealed at each end by hose clips (not shown).

An innovation in this apparatus was the method of piping pore water into the joint. In an apparatus such as this, the pore pressure cannot exceed the confining pressure
Fig. 7.10. Diagrammatic section through triaxial bomb showing pore pressure connections.
since the jacket has no rigidity. However, experiments showed that for pore pressures up to very near this maximum, water will not flow between jacket and sample, presumably because the soft rubber penetrates every crevice on the side of the specimen. Therefore, water must flow through the sample itself and most rocks, including some high porosity sandstones, are virtually impermeable in terms of the pressures and testing rates used in these experiments. This problem was overcome by cutting slots, S, at each end of the specimen and connecting these to the joint plane by 1 mm. bore plastic tubes, T, cemented with epoxy cement into slots cut in the sides of the cylinder. With this arrangement, water could be freely circulated through the joint plane and any required pore pressure applied. By measuring the pressure at each end of the joint by gauges, G, the uniformity of pressure in the joint could be checked.

In tests without pore pressure, the end block E was replaced by a solid one and the drainage duct, B, plugged. If required, strain gauges could be attached to the specimen and the leads brought out through the drainage duct, A. In some experiments, a 2 in. dia. load cell, similar to that described in Chapter 6 but with the three wires emerging along its axis, was inserted between the piston and the upper discs, D. For tests with two spherical seats, the discs, D,
Fig. 7.11. Photograph of pressure bench.
It was soon realised that the normal method of controlling confining pressure in short term tests by means of screw press and needle valve was inadequate for sliding friction tests. Each test occupied one to two hours and very close control over the confining pressure was required. For this reason an hydraulic accumulator, A, was manufactured. This consisted of a steel cylinder with a finely honed small diameter piston, loaded by dead weights through a hanger. If the piston clearances are suitable (about 0.005 in.) a small leak lubricates the piston, which is continually oscillated to prevent sticking by a fork and eccentric cylinder driven by a small geared motor. The piston had a stroke of 4 in. and two pistons were made, one 3/8 in. dia. and one 1/4 in. dia. These two pistons permitted maximum pressures of 4000 psi and 8000 psi respectively with a total weight of some 400 lb. The accumulator was connected in series with the oil pressure supply and during a test provided very fine control of confining pressure with very little attention. Maximum confining pressure used in the friction tests was of the order of 4000 psi.

The tests were performed in the 50 ton Avery testing machine. Load displacement curves were recorded by the machine recorder. In some tests the Houston X-Y recorder
was connected to the internal load cell and to a DCDT between the platens. However, this was an added complication and was only used for a few tests with stick-slip oscillations.

Reduction of results was programmed for the computer. Coordinates from the load-displacement curve were punched on cards, together with information on confining pressure, pore pressure and specimen geometry. The computer made the necessary corrections for end friction, change in contact area and rotation of the sample (for spherical seats), and calculated effective normal and shear stresses for each point. Graphs showing the variation of $\tau/\sigma$ with displacement on the slip plane and the relation between effective normal stress and shear stress were then plotted on the line printer. A complete description of this program is given in Appendix C.

7.6 TRIAXIAL TESTS ON ARTIFICIAL JOINTS

A number of tests was made on ground surfaces, mainly of quartzite, in order to perfect the experimental technique and also to check the theoretical predictions of previous sections. These tests were mainly on rough surfaces since it was felt that stick-slip oscillations complicate the situation and are of limited significance for natural surfaces.

Narrandera quartzite

It was shown in Chapter 6 that this rock shows rather classical behaviour in that $\mu$ varies little with displacement.
It should therefore be a good material for testing the performance of the apparatus since theoretical load-displacement curves were presented in Sections 7.3 and 7.4. Qualitatively, behaviour in both direct shear and triaxial tests is similar. For smooth surfaces, sliding commences with stick-slip oscillations which continue thereafter. Rough surfaces show steady sliding which, after a few tenths of an inch displacement, grades into stick-slips of small amplitude.

Typical differential load-axial displacement curves for rough quartzite are shown in Fig. 7.12, (a), (b), (c) for lateral translation under different confining pressures, while (d) is an example with two spherical seats (case A). Curves I and II of (b) are identical tests done with and without lubricated discs respectively. Corresponding values of $\mu$ calculated from these curves on the assumption that $k = 0.005$ are 0.71 and 0.85 respectively; this gives an indication of the error that can arise if end effects are not considered. From Table 7.1 the actual value of $k$ which must be operating to produce curve II is of the order of 0.1, which is a reasonable value.

Of Fig. 7.12 (a), (b) and (c), only (c) resembles the theoretical curves of Fig. 7.8. It is evident that other factors influence the measured differential load. These may include the following.
Fig. 7.12. Typical differential load \((R)\) axial displacement \((X)\) curves for sliding of rough quartzite in triaxial test. (a), (b) Curve I, (c) lubricated discs, (d) two spherical seats. Curve II of (b) unlubricated discs.
(i) Under low confining pressure the thick rubber jacket may offer some restraint to movement on the shear plane. Although the rubber has a low modulus it would be held against the rock surface by friction so that the gauge length actually extended by the deformation of the sample would be quite small.

(ii) Another effect of the thick jacket is that, particularly under low confining pressures, it may not deform sufficiently for the force $P_3$, Fig. 7.4, to be fully effective. From eqn 7.25 this would have the effect of increasing the differential load required to maintain sliding.

(iii) Under low axial loads, the friction of the Molybond is probably higher than the assumed value of 0.005. Fig. 7.9 suggests that $k$ could be 0.01 or greater under these conditions.

(iv) The lateral forces induced during sliding may cause additional friction at the O-ring of the triaxial bomb. The initial load on the piston is measured before the sample is loaded and therefore does not include any lateral component. However, few tests with the internal load cell indicate that this effect is not large.

Fig. 7.12(d) for two spherical seats also shows these effects since the slope of the curve is greater than that predicted by Fig. 7.8.
Values of normal stress and shear stress at sliding for a large number of specimens of rough quartzite are shown in Fig. 7.13. The tests include dry surfaces with lateral translation and with spherical seats, and also tests with pore pressure in the joint. Plotted also are the results from the direct shear tests. It is seen that there is good agreement and all tests scatter around a common line with $c = 0$ and $\mu = 0.70$. It may therefore be concluded that the triaxial test may be used with confidence and that the additional errors discussed above are of second order. It is also clear that this rock obeys the effective stress law very well.

Only a few tests were made on the smooth surfaces. A typical X-Y record made by the internal load cell is shown in Fig. 7.14 for $\alpha = 30^\circ$ and $\sigma_3 = 1000$ psi. The stick-slip behaviour is similar to that in the direct shear tests and $\mu = 0.35$ which is in good agreement.

**Bowral trachyte**

Only a few exploratory tests on this material were made to check the behaviour against that in the direct shear tests. Behaviour was again qualitatively similar, smooth surfaces showing stick-slip and rough ones steady sliding. A typical X-Y record for smooth trachyte with $\alpha = 40^\circ$ and $\sigma_3 = 1000$ psi and two spherical seats is shown in Fig. 7.15. At initiation
Fig. 7.13. \( \sigma^i - \tau \) diagram for many tests on rough quartzite. Dots = dry surfaces with lubricated discs, circles = pore pressure with lubricated discs, triangles = dry surfaces with two spherical seats, squares = dry surfaces in double shear test.
Fig. 7.14. X-Y record for smooth quartzite in triaxial test with two spherical seats. \( \alpha = 30^\circ \), \( \sigma_3 = 1000 \) psi. \( R = \) differential load, \( S = \) slip displacement.

Fig. 7.15. X-Y record for smooth trachyte in triaxial test with two spherical seats. \( \alpha = 40^\circ \), \( \sigma_3 = 1000 \) psi. \( R = \) differential load, \( S = \) slip displacement.
of sliding $\mu = 0.39$, again showing good agreement.

It was shown in Chapter 6 that smooth trachyte had a fairly large difference ($\mu - \mu'$) so that the stick-slip oscillations were of large amplitude. A good example of the instability which can arise in the triaxial test is shown in Fig. 7.16 which are X-Y records for smooth trachyte with two spherical seats as in Fig. 7.6(a) and $\alpha = 50^\circ$, $\sigma_3 = 1000$ psi.

The three diagrams (a), (b), (c) are records of tests in which the specimen is replaced after each run. For the first run (a), $\mu$ is initially 0.38 and builds up with stick-slips to 0.70. Now $\mu = 0.7$ corresponds to $\phi = 35^\circ$ so that the maximum value of $\alpha$ for sliding is $55^\circ$. On the second run as the rotation of the sample effectively increases $\alpha$ from $50^\circ$ the amplitude of the stick-slips rapidly increases. Referring to Fig. 7.2 at slip the differential stress must return along the stress path $CA$ until it intersects the line of slope $\arctan (\mu^*)$ corresponding to the stick phase. Clearly, if $\alpha$ is very close to $90^\circ - \phi$ the stress drop will be very large if there is a large difference between $\mu$ and $\mu^*$ and this is the situation occurring here. Finally, Fig. 7.16(c) is a record of the third run where the shock from the load drop of some 65,000 lb. was sufficient to stop the testing machine. On removal, the specimen was quite intact. The peak of Fig. 7.16(c) corresponds to $\mu = 0.76$. 
Fig. 7.16. X-Y records for smooth trachyte in triaxial test with two spherical seats. $\alpha = 50^\circ$, $\sigma_s = 1000$ psi. Specimen replaced and retested in (b) and (c). $R =$ differential load, $S =$ slip displacement.
Comparison with other methods

The use of lubrication to reduce end friction in the triaxial method has not been previously reported in the literature. It is therefore of interest to compare the present method with methods used by other investigators. It was shown in Fig. 7.12(b) that lack of lubrication in the lateral translation method increased the apparent value of $\mu$ from 0.71 to 0.85, an increase of some 20 percent.

The method most commonly used is that with one spherical seat, Fig. 7.1(c). This seat is usually exposed to the confining fluid and is therefore lubricated to some extent. The normal stress on the seat is also reduced by the magnitude of the confining pressure. To compare results obtained from both methods, two identical specimens of rough quartzite were tested, one with lubricated discs and one with a single spherical seat. Both were tested in a stage test (see Chapter 8) in which the confining pressure is rapidly incremented as soon as slip commences. The machine recorder curves are shown in Fig. 7.17(a) and (b) for lubricated discs and single spherical seat respectively. In (a) the specimen was initially tested at $\sigma_3 = 1480$ psi. After steady sliding was achieved $\sigma_3$ was reduced to 410 psi and incremented in steps to 2580 psi. In (b), $\sigma_3$ was increased in four steps from 410 psi to 1480 psi, then reduced and the
Fig. 7.17. Testing machine records for sliding of rough quartzite with $\alpha = 40^\circ$ in triaxial test.
(a) lubricated discs, (b) one spherical seat.
$M =$ total machine load, $X =$ axial displacement.
process repeated. In this case, the differential load did not reach a steady value and $\sigma_3$ was increased at each stage as soon as it was obvious that slip had occurred. Stick-slip oscillations occurred during the second run of (b).

Shear and normal stresses, calculated from the usual elementary formulae, eqns 7.1 and 7.2 are shown in Fig. 7.18. The results for the lubricated discs case define a linear relation with $c=0$ and $\mu = 0.70$. The beginning and end of each stage were plotted for the spherical seat case. For the first run $\mu = 0.85$ and for the second $\mu = 0.92$ approximately. These values are of the same order as that obtained from the unlubricated translation case.

In order to investigate the change in geometry in the single spherical seat case, a piece of paper was inserted at each end of the specimen during assembly. After the test, the piece at the end nearest the spherical seat was unmarked, indicating that there had been no differential movement and that the spherical seat had operated. However, the paper at the piston end was intensely scored at one side indicating that the sample had been tilting on the platen. It would therefore appear that this case is rather similar to that with two spherical seats, but, of course, the end conditions are unknown.

It would seem therefore that neglect of end lubrication leads to an overestimation in the value of $\mu$ by some 20%.
Fig. 7.18. $\tau-\sigma$ diagram for sliding of rough quartzite in triaxial test. Dots = lubricated discs. Circles and squares = first and second run respectively with one spherical seat.
This may be important if the values are used in design because it means that the factor of safety against sliding will be some 20% less than that imagined. However, the situation will vary with the type of apparatus and experimental conditions. The only way to eliminate these uncertainties is to ensure that the ends of the specimen are fully lubricated. Discs lubricated with Molybond are a simple and effective method.

One further point is that interaction between the ends of the specimen and the platens may contribute to some of the effects observed during sliding. Some exploratory experiments with composite specimens in the direct shear apparatus, Fig. 6.2, indicate that stick-slip oscillations commonly occur in the sliding of quartzite on steel. It is therefore possible for stick-slip to occur at unlubricated platens in a triaxial test. This effect may have contributed to the stick-slip oscillations recorded for very rough surfaces of granite by Brace and Byerlee (1966).
Chapter 8

STRENGTH OF JOINTS AND ROCK MATERIAL

The strength of a jointed rock mass is primarily related to the strength of, and the interaction between, the joints intersecting the mass. Open joints obtain their strength from the frictional resistance developed between their faces under the influence of applied normal stresses. For a complexly jointed rock mass, the interference between various sets of joints may impart a higher strength. However, it is clearly of fundamental importance to measure the basic frictional resistance of the joints.

In very hard rocks, the strength of the rock material may be sufficiently great that the rock blocks may be regarded as rigid bodies interacting along their joint surfaces. However, if the rock is relatively weak, it may be possible for failure planes to pass partly or entirely through the rock material. In the present case, the rock material appears reasonably strong but it possesses two prominent planes of weakness, the bedding and the slaty cleavage, as well as numerous veins which may influence the strength of the rock blocks. It is therefore important that this aspect be investigated.

This chapter therefore describes sliding friction
tests on natural joints from the drill core and further tests to determine the variation in strength of the rock material in various parts of the core.

8.1 FRICTIONAL PROPERTIES OF JOINTS

The joints encountered during the drilling were described and classified in Chapter 4. These joints showed a wide range in the characteristics which would be expected to influence frictional properties, such as surface roughness and surface coatings. On appearance, they fell into two main classes.

(i) Faults, which tend to have smooth, polished or slickensided surfaces and mainly chlorite or graphite coatings. These are represented by the J2 faults and the bedding plane faults.

(ii) Extension fractures, which tend to have rough, interlocking surfaces and coatings of limonite, pyrite, quartz, dolomite, or occasionally they may be clean. These are represented principally by the J3 joints. These classes are not mutually exclusive for some faults have quite rough surfaces and vice versa.

The main aim of the present work was to cover as wide a range of joint surfaces as possible, in order to gain some idea of the variation to be expected in frictional properties of natural joints. It was shown in Chapter 6
that most previous work on rock friction has been done on artificially prepared surfaces and that the information on natural surfaces is quite meagre. The drill core was therefore inspected and representative joints of suitable inclination chosen for study. The core was cut to correct length, the ends flattened, and the joints slid in the triaxial apparatus using the techniques described in Chapter 7. In addition to these naturally occurring joints, sliding tests were also made on suitable surfaces of failure in solid cores which had been fractured in triaxial compression tests, which are described in Section 8.2.

From the outset, it became apparent that different joints showed vastly different behaviour during the initial stages of sliding. On this basis they could be divided into four groups with some intermediate behaviour. The types of initial load-displacement curve are shown in Fig. 8.1 and described in Table 8.1.

Type A behaviour is that which would be expected from rough surfaces with interlocking asperities, the peak load being that required to shear the asperities before steady sliding can proceed under stresses defined by the residual strength of the joint. Type B behaviour is also readily explained as a special case of Type A in which the peak strength coincides with the residual strength. Both these
Fig. 8.1. Different types of initial behaviour for sliding of natural joints. $F =$ differential load, $X =$ axial displacement.
types of behaviour have previously been reported by Krsmanovic and Langof (1964) and Hoek and Pentz (1968) for large surfaces of natural joints.

Type C and D behaviour is apparently less common since it does not seem to have been previously reported in the literature, either for natural joints or artificially prepared surfaces. As shown in Chapter 6, Type C behaviour is quite characteristic of artificially prepared surfaces of certain rocks, notably Bowral trachyte. Also, as will be shown later, it is very common behaviour for certain natural joint surfaces in the present experiments.

Table 8.1

<table>
<thead>
<tr>
<th>Type of behaviour</th>
<th>Characteristic joints</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. No slip until peak load then gradual drop off to residual value</td>
<td>Joints with large interlocking asperities; bedding planes with cross ripples; faults with cross slickensides or grooves.</td>
</tr>
<tr>
<td>Fig. 8.1(a)</td>
<td></td>
</tr>
<tr>
<td>B. Well defined initial slip which continues at constant load</td>
<td>Joints with hard, fairly smooth surfaces; (also Narrandera quartzite).</td>
</tr>
<tr>
<td>Fig.8.1(b)</td>
<td></td>
</tr>
<tr>
<td>C. Well defined initial slip which continues with rising load</td>
<td>Relatively rough, chlorite or graphite coated surfaces; very smooth hard surfaces; (also Bowral trachyte).</td>
</tr>
<tr>
<td>Fig.8.1(c)</td>
<td></td>
</tr>
<tr>
<td>D. Continuous curvature of load-displacement curve</td>
<td>Faults with smooth or polished chloritic surfaces.</td>
</tr>
<tr>
<td>Fig.8.1(d)</td>
<td></td>
</tr>
</tbody>
</table>
Fig. 8.2. Testing machine record for specimen No. 802 with variations in confining pressure and pore pressure. Initial confining pressure 1810 psi. $M = \text{total machine load, } X = \text{axial displacement.}$
Fig. 8.3. $J-	au$ diagram for specimen No. 802. Symbols refer to different confining pressures. $c_0 = 1000$ psi, $c = 200$ psi, $\varphi = 29^\circ$, $\mu = 0.55$. 
given in Fig. 8.2, which is a load-displacement curve from the testing machine recorder. This joint (No. 802, \( \alpha = 39^\circ \)) was an extension fracture, with fine scale interlocking grooves and a limonite coating, from hole A, zone 2. The initial portion, AB, of the curve corresponds to the thrust on the piston from the confining pressure of 1810 psi, before the differential load was applied. Under zero pore pressure, differential load reached a peak at C and then reduced to a residual value at D. After a steady load was reached, the pore pressure was increased to 1500 psi, E, and then successively reduced to 1000, 5000 and 0 psi, F to H. Further stages were done by varying both confining pressure and pore pressure. Changes in confining pressure also change the initial thrust on the piston, which is registered by the machine recorder, whereas changes in pore pressure do not have this effect. This particular experiment was terminated by puncturing of the rubber jacket after 0.5 in. displacement on the slip plane. As noted in Chapter 7, the jacket is the limiting factor on displacement in most of these experiments.

The effective normal-shear stress relation for this test is shown in Fig. 8.3. These results were reduced by the computer program described in Appendix C. Point K corresponds to the peak stress at initiation of sliding.
Fig. 8.4. Testing machine record for specimen No. 221. Initial confining pressure 1480 psi. $M =$ total machine load, $X =$ axial displacement.
Fig. 8.5. J-I diagram for specimen No. 221. c = 120 psi, $\phi = 27^\circ$, $\mu = 0.50$. 
Fig. 8.6. Testing machine record for specimen No. 227. Initial confining pressure 1810 psi. $M =$ total machine load, $X =$ axial displacement.
Fig. 8.7. $\sigma$-$\tau$ diagram for specimen No. 227. 
$c = 0$, $\phi = 27^\circ$, $\mu = 0.5$. 
Fig. 8.8. Testing machine record for specimen No. 253. Confining pressure 1480 psi. $M =$ total machine load, $X =$ axial displacement.
Fig. 8.9. $\sigma$-$\tau$ diagram for specimen No. 253.
$\varphi = 34^\circ$, $\mu = 0.67$. 
The latter was very smooth and polished, and both were tested without pore pressure. The normal-shear stress relations are shown in Figs. 8.7 and 8.9 respectively. In both these cases slip, as estimated by departure from linearity of the load-displacement curve, commences at point, P, on the stress path. In Fig. 8.7, the stresses move along the stress path to the residual strength line. In Fig. 8.8, a steady differential load was not obtained in the initial slip. Assuming c=0, the maximum load falls on a residual strength line with $\phi = 34^\circ$, Fig. 8.9.

The results of all the sliding friction tests on natural joints are summarised in Table 8.2. The letters A, B, C, D refer to the initial slip behaviour, Fig. 8.1, and tests including pore pressure are calculated in terms of effective normal stresses. The parameters for most of the joints showing type D behaviour are estimated from a single point at the maximum differential load, since further stages were not possible in the displacement available in the apparatus. Photographs of typical joints after sliding are shown in Fig. 8.10.

Features which emerge from this tabulation are:

(i) Despite the wide variations in types of surface, the residual coefficients of friction nearly all fall within the narrow range 0.5 to 0.7, that is, $\phi$ between $27^\circ$ and $35^\circ$. The major exceptions are the graphite coated faults which,
<table>
<thead>
<tr>
<th>SPEC Numb</th>
<th>PEAK</th>
<th>RESIDUAL</th>
<th>INIT SLIP</th>
<th>PORE PRESS</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>250</td>
<td>50</td>
<td>800</td>
<td>-</td>
<td>32</td>
<td>0.63</td>
</tr>
<tr>
<td>827</td>
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<td>500</td>
<td>180</td>
<td>31</td>
<td>0.60</td>
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<tr>
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<td>200</td>
<td>29</td>
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<tr>
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<td>48</td>
<td>1000</td>
<td>50</td>
<td>28</td>
<td>0.53</td>
</tr>
<tr>
<td>* 51</td>
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<td>110</td>
<td>36</td>
<td>0.73</td>
<td>B</td>
</tr>
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<td>100#</td>
<td>18#</td>
<td>0.32#</td>
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* FRACTURE SURFACE FROM TRIAXIAL TEST
# $\sigma < 3000$ PSI
$\sigma > 3000$ PSI

F = FAULT
E = EXTENS FRACT
B = BEDDING
T = TRIAXIAL FRACT
### Table 8.2 (Continued)

**Summary of Friction Tests on Natural Joints**

<table>
<thead>
<tr>
<th>SPEC NUMB</th>
<th>PEAK $C_0$</th>
<th>$C$</th>
<th>$\phi$</th>
<th>$\mu$</th>
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<th>PORE PRESS</th>
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</tr>
<tr>
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<tr>
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<td></td>
<td>SMOOTH CHL</td>
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<tr>
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<td>-</td>
<td>34</td>
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<td>D</td>
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<td></td>
<td>SMOOTH CHL</td>
</tr>
<tr>
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<td></td>
<td>ROUGH CHL,GRAPH</td>
</tr>
</tbody>
</table>

* Fracture surface from triaxial test

# $\sigma < 3000$ PSI

$\sigma > 3000$ PSI
Fig. 8.10. Examples of typical joints after sliding in the triaxial apparatus.
as would be expected, give lower values of 0.2 to 0.3. Three of these faults gave curved failure relations which can be fitted to two straight lines for $\sigma$ greater or less than 3000 psi. The flattening of the failure line no doubt reflects a change in mechanism of slip from frictional sliding to plastic deformation of the graphite crystals, which are probably aligned nearly parallel with the fault plane from previous displacements on the fault. Some of the rough surfaces gave surprisingly low values of $\mu$ considering that ground surfaces of quartzite, which are much smoother, gave $\mu = 0.7$. This effect occurs mainly with joints in the harder rocks and the reason is probably that after the initial sliding, the surfaces contain a great deal of debris; subsequent moving is likely to occur partly by rolling on these particles rather than by pure sliding.

(ii) The majority of the joints show type B or C behaviour in the initial sliding. Type C behaviour, which is characteristic of Bowral trachyte (Chapter 6), is therefore likely to be quite common in natural joints. This in fact is a reverse behaviour from those joints showing peak strength, which was previously considered to be the most likely behaviour in rough natural joints, Deere et al. (1966). Types A and B behaviour seem to be characteristic of
extension fractures, and types C and D of faults, although this is by no means the invariable rule.

(iii) Residual values of cohesion are mainly in the range 0 to 200 psi. Very few joints gave a zero intercept.

(iv) Peak cohesion, where present, is of the order of a few hundred psi, with a maximum of 1000 psi in these experiments. These tests refer only to open joints; veins are considered in the next section.

(v) There is no significant difference between tests run dry and those with pore pressure. It would appear that these joints obey the effective stress law. If the effective stress law was not fully obeyed, the computed coefficients of friction should be on the high side and this is not the case. The only joint which showed a low value of $\mu$ under pore pressure was No. 239 which had a polished chloritic surface. This would be expected since Horn and Deere (1961) found that $\mu$ for chlorite decreased when saturated.

(vi) Sudden drops of load during sliding were observed in a few cases. These were very similar to the stick-slip oscillations described in Chapter 6 but were only isolated events. They occurred on very flat smooth surfaces and may be caused by the same mechanism.

The major deficiency of this work is that the size of joint tested is still extremely small compared with the
size of a joint influencing an engineering structure in rock. The longer wavelength undulations in natural joint planes are not taken into account and it is felt that the friction values quoted are, at least in terms of peak strength, minimum values. However, it is probable that the residual values are fairly realistic. Certainly the present results are of the same order as those obtained on large specimens in the Imperial College direct shear machine, Hoek and Pentz (1968). The triaxial method offers several advantages in terms of ease of testing and there is no reason why it should not be applied to large diameter cores, if a suitable triaxial bomb is available.

8.2 STRENGTH OF ROCK MATERIAL

The determination of realistic strength values for natural rocks is a matter of some difficulty. Most laboratory studies of the fracture of rock use small specimens from a single block of carefully chosen, homogeneous and isotropic rock, such as granite, or alternatively a carefully chosen anisotropic rock, such as roofing slate. There is every justification for using such materials in fundamental studies, since the number of uncontrolled variables in the experiments must be kept to a minimum. Even under these conditions, however, the result of a single test is of doubtful value and a number of tests under different
conditions is required to obtain meaningful strength properties.

In most practical problems, particularly those associated with mining, there is little control over the type of rock in which an excavation is made. This point has been discussed with regard to the Black Star open cut in Chapter 2. The difficulties discussed above are magnified many times and the problem reduces to making a best estimate of the strength of the rock by whatever means are available. Various simple index properties such as density, porosity, hardness, etc., are discussed by Miller (1965). However, at the present time, these properties are of little value unless there is an established correlation between them and strength parameters of the particular rock under consideration. The problem therefore reduces to making some sort of direct strength tests.

Two requirements are clearly important in studying the strength of natural non-homogeneous and anisotropic rocks. 

(i) Samples tested should be as large as possible.
(ii) A large number of samples should be tested.

For obviously anisotropic rock such as that at Mount Isa an additional requirement is that the variation of strength with direction should be investigated.

In the present case, the only samples available are
approximately 2 in. diameter diamond drill cores. This sets an upper limit on the size of sample which can be tested. The number of samples is only limited by the expense in preparation and the time required for testing. This makes a simple test using a simple specimen shape very desirable.

Possible methods of testing considered were:

(i) unconfined compression tests on lengths of drill core;
(ii) triaxial compression tests on lengths of drill core;
(iii) triaxial or unconfined compression tests on small cores drilled from the drill core;
(iv) Brazilian tests on drill core or samples cut from the core.

The first two methods minimize sample preparation but there is little control over the orientation of anisotropy and planes of weakness. The third permits the effect of the anisotropy to be investigated but involves additional sample preparation and the resulting samples are quite small. The last method is somewhat indirect in that the tensile strength, rather than shear strength is measured.

Unconfined compression tests on lengths of core is undoubtedly the simplest approach. However, with anisotropic rocks, the unconfined test is rather unsatisfactory since samples tend to split or break irregularly and their
interpretation is difficult. The compromise adopted here was to concentrate on triaxial compression tests on lengths of core with lesser numbers of triaxial tests and Brazilian tests on small oriented samples. The main requirement was to determine the relative strengths of the rock in the various zones with regard to their relative behaviour in the open cut batter.

Lengths of core were therefore selected for testing, and sawn into 5 in. lengths. The ends were ground flat and parallel and the samples tested in the triaxial bomb described in Chapter 7. One external spherical seat, as described by Paterson (1958), was used in all these tests. The function of this spherical seat was to provide alignment during initial loading and the considerations of Chapter 7 are not relevant. Sliding on the fracture surfaces was not carried out because very often they intersected either platen and the results would not have been meaningful. Instead, suitable fracture surfaces were retested with lubricated ends and these results are included in Table 8.2.

Since the minimum strengths were likely to be of greatest practical interest, the majority of specimens were chosen so that the plane of weakness was oriented at a favorable angle for failure. Therefore, most tests were performed on core from hole A in which the bedding, V1 veins
and slaty cleavage were all inclined at from 20° to 40° to the core axis. No attempt was made to select particularly good or bad rock, except that likely failure planes were located near the centres of the specimens where possible. A number of cylinders was tested from each zone in hole A. Since the rock in zone 1 was leached, further fresh specimens from zone 1 of hole B were tested. In addition, a few tests were made on slate from hole B in which the cleavage was oriented at a high angle from the core axis, and also on samples of silica dolomite from hole C. A total of 210 triaxial tests on the NMLC core (2.045 in. diameter) was made. A further 78 triaxial tests were made on EX (0.875 in. diameter) cores drilled at various angles to bedding and cleavage from hole C, zone 1 and hole A, zone 3, respectively. The maximum confining pressure used was 5000 psi since higher pressures were not considered relevant to the present problem.

The usual method of analysing the results of triaxial tests on rocks is to plot Mohr circles to define the failure envelope. This method assumes that failure occurs in a plane inclined at $45^\circ - \frac{\emptyset}{2}$ to the core axis where $\emptyset$ is the slope of the Mohr envelope. This is approximately true for isotropic rock (refer Chapter 5). Moreover, it is also usually found that the results for tests in which the sample
splits or fails on an irregular surface are consistent with those which fail on a planar surface whose inclination is predicted by the Mohr-Coulomb theory.

In the case of anisotropic rock, the failure plane is generally influenced by the orientation of the plane of weakness relative to the directions of principal stress. It is possible to plot separate Mohr envelopes for each orientation of the planes of weakness, Hoek (1967). Alternatively, failure stress can be plotted against angle of inclination of the planes of weakness to give a family of curves for different confining pressures, Donath (1964), Jaeger (1965). These curves generally show a minimum strength when the planes of weakness are oriented at about 30° to $\sigma_1$. This is predicted by the Mohr-Coulomb theory, Jaeger (1962), and the problem has also been treated in terms of Griffith theory with a preferred orientation of major cracks, Hoek (1964), Walsh and Brace (1964).

A somewhat different approach, influenced by the theory for sliding on a joint in the triaxial test, Chapter 7, is used in this work. The plane of weakness is equivalent to an interlocked or filled joint and stresses greater than the residual strength of the fracture surface will be necessary to initiate failure. It is therefore appropriate to calculate the shear and normal stresses on the actual
plane of failure by means of eqns 7.1 and 7.2. These stresses, when plotted on a $\sigma-\tau$ diagram, will define the failure relation for the plane of weakness, irrespective of its orientation. This approach was used by Donath (1961) who found a single linear failure relation fitted failure on the plane of cleavage in Martinsburg Slate over a wide range of inclinations. Comparison with the residual strength line for sliding on the fracture surface and the Mohr envelope for failure through the rock ignoring the plane of weakness will give a good indication of the influence of anisotropy on the strength of the rock.

The results of the triaxial tests are tabulated and displayed graphically as follows:

2 in. diameter cylinders

<table>
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<th>Hole</th>
<th>Zone</th>
<th>Table</th>
<th>Figure</th>
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<tbody>
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### TABLE 8.3

**SUMMARY OF TRIAXIAL COMPRESSION TESTS**

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THIN BEDDED SHALE (LEACHED)  
BEDDING APPROX 30 DEG TO AXIS

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**HOLE A ZONE 2 2.045IN DIA**  
DOLOMITIC SILTSTONE  
BEDDING APPROX 40 DEG TO AXIS

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## TABLE 8.4
### SUMMARY OF TRIAXIAL COMPRESSION TESTS

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SILTSTONE, MUDSTONE, SLATE  
VI VEINS APPROX 30 DEG TO AXIS

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SUMMARY OF TRIAXIAL COMPRESSION TESTS

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BLACK SLATE  
CLEAVAGE APPROX 25 DEG TO AXIS

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### HOLE C ZONE 1 2.045IN DIA

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<th>FAILURE-PLANE SIGMA</th>
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Results of triaxial compression tests on 2 in. dia. cylinders. Modes of failure: dots = on veins, crosses = on bedding.
Fig. 8.13. Hole A, zone 3.

Fig. 8.14. Hole A, zone 4.

Results of triaxial compression tests on 2 in. dia. cylinders. Modes of failure: dots = on vein, circles = on cleavage, half circles = other.
Fig. 8.15. Hole A, zone 5.

Fig. 8.16. Hole A, zone 6.

Results of triaxial compression tests on 2 in. dia. cylinders. Modes of failure: dots = on vein, circle = on cleavage, half circle = other.
Fig. 8.17. Hole B, zone 1.

Fig. 8.18. Hole B, zone 4.

Results of triaxial compression tests on 2 in. dia. cylinders. Modes of failure: dots = on vein, crosses = on bedding, half circles = other.
Fig. 8.19. Hole A, zone 3.

Fig. 8.20. Hole C, zone 1.

Results of triaxial compression tests on 7/8 in. dia. cylinders. Modes of failure: dots = on vein, crosses = on bedding, circles = on cleavage, half circles = other.
On each of these diagrams, a straight line with $\phi = 30^\circ$ is drawn as the approximate residual strength relation. Reference to Table 8.2 shows that this value of $\phi$ is a reasonable average for the residual strength of the fracture surfaces. Since the figures are drawn at different scales, parallel lines at cohesion intervals of 10,000 psi are also drawn to aid comparison.

The results fall into two main groups.

(i) For failure on V1 veins or cleavage in zones 3, 4 and 6, hole A, the results scatter in quite a narrow band parallel to the residual strength line. This indicates that the angle of internal friction for the plane of weakness is also of the order of $30^\circ$. The "peak cohesion" defined in Chapter 7 is of the order of 2000 psi.

(ii) For failure on bedding or V1 veins in zones 1 and 2 there is a much greater scatter and the strengths reach much higher values. The results also lie approximately parallel to the residual strength line but this is less definite. On this basis, peak cohesion is of the order of 5000 psi. The results for the sedimentary breccia (zone 5) are also of this order.

The greater scatter in zones 1 and 2 is related to the spacing of planes of low cohesion relative to the sample size. In zones 3 to 6 the V1 veins and cleavage are
closely spaced and each sample has a high chance of containing at least one plane of weakness on which failure will preferentially occur. In zones 1 and 2, bedding planes with low cohesion are more widely spaced and many samples containing only bedding planes with quite high strength. If samples of larger diameter, say 2 ft, were tested the high strengths in zones 1 and 2 would not be recorded. On the other hand, the strength of rock from zones 3 to 6 would be only slightly reduced.

In some cases, failure did not occur on a plane of weakness. Particularly in the quite strong shale and siltstone, failure occurred on a plane at a small angle to the core axis. Clearly the Mohr-Coulomb failure criterion is inadequate for predicting fracture orientations in such cases and it is possible that the interparticle "effective tensile stresses" postulated by Brown and Trollope (1967) are operating under the relatively low confining pressures used here. In any case, the basis of plotting as for failure on a plane of weakness seriously underestimates the strength of these samples relative to the others.

The angle of internal friction of most isotropic rocks is 45° or higher under low confining pressures. The only samples on which this can be checked is the silica dolomite, Table 8.9. These results are plotted on a $(\sigma_1 - \sigma_3), \sigma_3$
diagram in Fig. 8.21 together with a larger number of tests on silica dolomite from Hoskins (1967). There is considerable scatter but the two sets of results are clearly consistent. Using the equations relating this diagram to the $\sigma$-$\tau$ diagram given by Raleigh and Paterson (1965), linear failure relations with various values of $\phi$ are superimposed on the plotted results. In fact, $\phi = 45^\circ$ seems to fit these results reasonably well. For this reason, these samples which failed irregularly or on a low angle plane were assumed to have Mohr circles tangent to a failure envelope with $\phi = 45^\circ$ and these tangent points are plotted on the appropriate diagrams. This is quite an arbitrary assumption but is an attempt to make the results comparable with those for failure on the plane of weakness. In nearly all cases, the strengths of these samples is much higher.

The mode of failure was of three major types:

(i) entirely through a weakness plane - these almost invariably gave low cohesion values;

(ii) partly through a weakness plane or stepped on two adjacent planes, the remainder of the fracture giving the impression of tensile failure. These are similar to the tensile/shear fractures postulated by Hoek (1967) and show intermediate cohesion values;

(iii) unrelated to a weakness plane, usually when such a
Fig. 8.21. Strength of silica dolomite. Dots = present work, circles = results from Hoskins (1967). Lines are failure relations with different values of $\phi$. 
plane was not present in the particular specimen. These failures show the highest strength values.

In many cases, particularly in the Urquhart Shale, failure occurred on a bedding plane in preference to closely spaced, favorably oriented veins. In general, the veins are not prominent planes of weakness in zones 1 and 2 since they are filled mainly with dolomite and quartz. This is by no means the case in the other zones where chlorite is an important vein mineral. Another interesting feature was that failure in samples of sedimentary breccia occurred on nearly identical planes, indicating that slaty cleavage is moderately well developed in this rock.

**Brazilian tests**

A smaller number of Brazilian tests was also performed. These were done in the early stages of the investigation to assess the suitability of this test for rapid determination of rock strength. It has the virtue of being simple but the relation of the results to compressive and shear strengths is not clear.

The method was eventually abandoned in favor of the triaxial compression tests described above. However, the results obtained are of interest and are described briefly below.
1.385 in. diameter cylinders were drilled at right angles to the NMLC cores so that the bedding or cleavage formed an axial plane of the cylinder. It was possible to cut two, approximately 0.6 in. thick discs from each of these cylinders. One of each pair of these discs was tested with bedding or cleavage parallel to, and normal to the loading direction, respectively.

Most of the Brazilian tests were made on discs from hole A, zone 3. Histograms showing the scatter of computed tensile strengths are shown in Fig. 8.22. In all cases, the tensile strength was computed from the usual formula for isotropic elasticity. This is not really applicable in the present case but there is no solution available for anisotropic material. The results should therefore be regarded as approximate only. A few tests were also made on thin bedded shale from hole C. The results may be summarised as follows:

<table>
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<th>No Samples</th>
<th>Mean tensile strength (psi)</th>
<th>Stand. Dev. (psi)</th>
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<tr>
<td>Hole A</td>
<td></td>
<td></td>
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<tr>
<td>Zone 3</td>
<td>↓ cleavage</td>
<td>87</td>
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<tr>
<td>Zone 3</td>
<td>↓ cleavage</td>
<td>85</td>
<td>1880</td>
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<tr>
<td>Hole C</td>
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</tr>
<tr>
<td>Zone 1</td>
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Fig. 8.22. Frequency (F) of tensile strengths (σ_T) measured in Brazilian tests. Hole A, zone 3. (a) perpendicular to cleavage, (b) parallel to cleavage.
The anisotropy of strength is clearly defined. However, the scatter of individual values about the means is quite large and it was not uncommon to find the tensile strength normal to the cleavage greater than that parallel to the cleavage in a pair of discs from the same cylinder. It is felt that the size of sample is too small for reliable results to be obtained.
Chapter 9

LABORATORY EXPERIMENTS ON A MODEL

JOINTED ROCK MATERIAL

Progress in understanding the mechanical behaviour of a jointed rock mass has been hampered by the difficulty in measuring the material properties. The average joint spacing in most rock masses is at least several inches and may be many feet. Sample sizes must be several times greater than these dimensions if they are to be representative of the behaviour of the rock mass and not merely of the rock material. For this reason, in situ testing of large samples has proved popular, Nose (1964). However, since such tests are very expensive and time consuming, usually only a small number can be performed and the results may not be as representative as desired. They are also of limited application in understanding the fundamentals of rock mass behaviour.

An alternative is to study models of rock masses. One approach is to use regularly shaped unit blocks manually stacked together, but fairly extensive sample preparation is required particularly if real rock, rather than a castable rock-like material, is used. During attempts to produce a perfectly packed aggregate from an originally intact rock by heating and quenching, it was discovered
that when coarse grained marble is heated to around \( 600^\circ C \),
the anisotropy of thermal expansion in calcite produces
thermal stresses sufficient to cause almost complete grain boundary separation. The resulting material is a low porosity aggregate of perfectly packed calcite crystals, the behaviour of which may be relevant to that of randomly jointed rock masses.

The laboratory experiments on this material were done in collaboration with Professor J.C. Jaeger and a paper on the subject has been submitted for publication. This paper is included as Appendix D. In this work, I was solely responsible for developing the method of sample preparation, for the unconfined and triaxial compression tests and for the permeability experiments. The other experiments and the discussion were done jointly.

The major conclusion from this work is the very great influence of relatively small compressive stresses on the strength of a perfectly packed granular aggregate. Prediction of the behaviour of such an aggregate on the basis of soil mechanics experience with sands will lead to a serious under-estimation of the strength. Randomly jointed rock masses can therefore be expected to show quite high strengths in compressive stress fields, as predicted by Terzaghi (1962).
A major objection to the application of these results to practical problems is that few rock masses possess random jointing. However, equally few rock masses possess perfectly regular jointing and with further investigation it may be possible to predict the behaviour in intermediate situations from that of these two rather idealised limiting cases.
Chapter 10

SUMMARY AND CONCLUSIONS

This chapter briefly reviews the conclusions presented in the body of the thesis with particular attention to these factors which deserve further investigation. The information on geometry and strength of the joints is then applied to some simple calculations on slope stability.

10.1 STRUCTURAL ANALYSIS

The technique of using oriented diamond drill core for structural analysis was, in general, very successful. Probably no other method could give equivalent detail on rock structures over such a large area for the same amount of expenditure. It is pertinent, however, in the light of experience gained, to examine the limitations of the present approach and how more useful information could be obtained, for the moment disregarding the cost factor. These limitations may be summarised as follows:

(i) The orientation procedure was successful only because of the uniformity of large scale geological structures in the area and the presence of a persistent marker, the bedding, on the core. The method could not be used in igneous rocks or in closely folded sedimentary or metamorphic rocks.

(ii) The task of analysing the core was extremely time
consuming and laborious. The method could not be considered for large scale routine use unless several trained technicians were available. Of course, the economic advantages of rational stability analysis may make such a course quite feasible. Analysis of the information, once obtained, is not a difficulty provided a computer is available.

(iii) The method is not very suitable for detecting major structures, unless they have a profound effect on large sections of core. For instance, a major fault which cleanly cuts through the rock cannot be easily distinguished from a very minor structure.

(iv) Continuity of joints cannot be quantitatively determined. This is because the drill core has such a small cross-section. All that can be said is that certain joints are continuous, or discontinuous to some unknown degree.

In view of these factors, the following recommendations are made for more effective exploration.

(i) At least one major opening should penetrate the area to detect major structures such as faults, and which could be used as a base for more detailed exploration.

(ii) Short drill holes (up to 100 ft long) should be drilled from this opening to explore the larger volume of
surrounding rock. A large number of short drill holes is more economical than fewer long holes since more time is spent in productive drilling rather than in withdrawing and inserting the drill rods. This is extremely important when drilling from underground openings since the drill string must be broken into very short lengths.

(iii) With short holes, orientation of the core independently of any markers should be practical using oriented drill rods and a scribe on the core barrel.

(iv) An alternative method of core orientation and for structural analysis in its own right is examination of the walls of the borehole. Borehole television and movie cameras for this purpose have already been mentioned and there is no doubt that they have wide application. Disadvantages of these instruments are that they are usually complex, fragile and costly. The borehole periscope described by Krebs (1967) appears to be a simple and effective alternative for holes up to 100 ft long. In the author's opinion however, no amount of examination of borehole walls can be as effective and detailed as examination of the drill core, if oriented; rather are the two methods complementary.

(v) Determination of continuity of joints is the most difficult problem of all. Very little information can be
obtained by measuring traces of individual joints in tunnels or shafts. The most effective method is probably correlation of closely spaced holes drilled from openings. Very many holes are required for this to be effective. The most logical approach would be detailed studies of relatively small volumes of rock with cross-checks on the results.

While the above discussion has stressed the limitations of long hole drilling for structural purposes, there is no doubt that it can give very valuable preliminary information on which to base a detailed study. The deciding factor in the amount of detail obtained must always be the economic consideration.

10.2 STRENGTH OF DISCONTINUITIES

Sliding in the triaxial test has been given detailed attention in the investigation. The reasons for this are listed in Chapter 7, the main one being that it is an ideal method for making large numbers of tests on joints from diamond drill core, which are the most common types of samples. The technique using lubricated ends on the samples permits sufficient shear displacements so that joints from NX core give reasonable results. There is no doubt that tests on large samples of joints in a machine such as at Imperial College must give more realistic information. However, these tests are quite expensive to
conduct and there is probably justification for only a small number of machines of such complexity and expense in the world. The main requirement at the present stage is to determine how well the results from NX core agree with those from large samples in a shear box. Triaxial tests on 6 in. diameter cores are probably also within the capabilities of many laboratories.

With regard to the results obtained in the friction experiments, the most important is that the frictional properties of a wide variety of joint surfaces are all very similar. How much of this effect can be attributed to the small sample size is not yet known. However, since most large joint surfaces in a rock mass are more irregular than those tested, it would be expected that peak cohesion would be underestimated in the triaxial friction tests. It was further demonstrated that many joint surfaces show a gradual development of frictional resistance with displacement. If this effect is also characteristic of large scale joint surfaces, progressive creep without catastrophic failure would be expected on these joints. On the other hand, those joints which show well defined peak strength are likely to lead to catastrophic failure if this peak strength is exceeded.
10.3 SLOPE STABILITY ANALYSIS

There are two basic approaches to the analysis of slope stability in soils and rocks. Firstly, if the stress distribution within the slope and the strength and deformation characteristics of the material are known, it may be possible to predict from first principles the failure surfaces and the conditions under which failure will occur. This approach is the ultimate one but the problem has so far proved intractable, mainly in regard to stress analysis. Photoelastic models can give useful qualitative information, but in the case of jointed rock masses a linearly elastic model material cannot be expected to give quantitative results; in some cases, the results may be quite misleading. Future advances may be expected in the analysis of regular packings of idealised unit blocks, Trollope (1961), and in the application of the finite element method, Goodman and Taylor (1966).

The second approach is the trial and error location of the most likely failure surface of predetermined shape. This is the limit equilibrium method which has been used in soil mechanics for several decades. In soil slopes, the failure surface is usually assumed to be an arc of a circle, although other shapes such as a logarithmic spiral or any arbitrary surface can be used. For rock slopes
this "slip circle" approach is unlikely to be applicable unless the rock is closely and randomly jointed, as is the case in some igneous rock masses. In such a case, the strength parameters to be used in analysis must be determined from large specimens, with dimensions several times greater than the maximum particle size, or estimated from tests on model materials such as the granulated marble described in Chapter 9.

At the present time, the most tractable approach to the analysis of rock slopes with well defined patterns of jointing is the limit equilibrium method. The blocks of rock material are regarded as rigid bodies sliding on joint planes, the geometry and strength of which are determined by methods similar to those described in this thesis. The method is a basically simple problem in statics, although the computations may become very involved for complex three-dimension joint systems. The most advanced approach so far is the technique of vector analysis developed by Wittke (1965). However, in many cases, relatively simple statical calculations will give useful information on the stability of a proposed slope.

The concept of factor of safety in slope stability is rather controversial. It has been variously defined with respect to slope height, slope angle, material density,
cohesion, angle of friction and total shear strength. These methods in general give different values of the factor of safety \( F \) for a slope which is not on the point of failure. However, it is considered that the most logical definition is

\[
F = \frac{\text{maximum available resisting force}}{\text{resultant force tending to cause collapse}}
\]

This definition is often difficult to apply for curved failure surfaces but this is not the case for planar slip planes and will be used below.

As a preliminary, it is informative to study the simple two-dimensional case shown in Fig. 10.1(a). The slope has height \( H \) and angle \( \beta \) and a planar joint inclined at angle \( \alpha \) intersects the toe of the slope. The weight of rock above the joint plane is \( W \) and it is assumed that the strength \( S \) of the joint can be expressed by the linear relation

\[
S = c + \sigma \tan \phi \tag{10.1}
\]

If we ignore all external forces such as those from water pressure or vibrations, the factor of safety is given by

\[
F = \frac{W \cos \alpha \tan \phi + c.(AB)}{W \sin \alpha} \tag{10.2}
\]
Fig. 10.1. Three cases of sliding on single planar joint.
Expressing the length $AB$ and the weight of the block in terms of the geometry of the slope and the bulk density, $\gamma$, of the rock, we have for

$$F = \frac{\tan \phi}{\tan \alpha} + \frac{2c \sin \beta}{\gamma H \sin \alpha \sin (\beta - \alpha)} \quad (10.3)$$

Therefore, the factor of safety is a function of both the height $H$ and angle $\beta$ for given values of $\alpha$, $c$ and $\phi$. For $c = 0$ the slope will be stable only if $\alpha < \phi$.

Returning to the specific problem of the Black Star open cut, two cases must be considered:

(i) the overall stability of the batter with height 550 ft and slope angle of the order of 45°;
(ii) the stability of individual bench faces, which for operating reasons would be designed on the final batter with height 70 ft and angle approximately 70°, if there were no over-riding stability considerations.

The three major sets of joints described in Chapter 4 can now be examined as potential failure surfaces in both these cases.

(i) The bedding joints, $J_1$, cannot be primary surfaces of failure in either case since they dip into the batter. They can, however, be important secondary failure planes which separate blocks tending to slide on other joints. In particular, the bedding plane faults are likely to be
100 percent continuous and have zero tensile strength. In addition, since the maximum in situ stress is likely to be nearly normal to the bedding (Chapter 5), the bedding joints will tend to open more than will the other joints upon relief of stress caused by excavation.

(ii) The set J2 dip, in general, from 70° east to vertical. Therefore, in general, they also will not affect the stability in either case. However, there are individual joints in this set which have a flatter dip and which may affect the stability of individual benches, particularly in zone 1 and 2 rocks. The fact that they are not continuous will alleviate their affect and they are not regarded as important primary failure planes. Again, however, they will be important secondary failure planes.

(iii) The J3 joints normal to the bedding, are likely to be the most important primary failure planes in the batter. In particular, the set striking north-south and defined by the maximum J3a in the girdle, Fig. 4.21, is exactly the case shown in Fig. 10.1(a). The wide range of orientation of points in set J3 also means that tetrahedral blocks of a wide variety of shapes have questionable stability. Both these cases are considered below.

In view of the presence of joints J1 and J2, the failure of the complete block shown in Fig. 10.1(a),
which will be termed case A, is unlikely. More likely modes of failure are shown in Figs. 10.1(b) and (c) with secondary failure on the J1 joints (case B) and J2 joints (case C), respectively. In Fig. 10.1(c) the J2 joints have been assumed vertical. For case B, the expression for factor of safety is identical with that for case A, eqn 10.3. For case C, it is given by

$$F = \frac{\tan \phi}{\tan \alpha} + \frac{2c \tan \beta}{H \tan \alpha (\tan \beta - \tan \alpha)} \quad (10.4)$$

It can be shown that for any given values of the variables, the factor of safety given by eqn 10.4 is always less than that given by eqn 10.3 for the non-trivial case $\beta > \alpha$. Case C is therefore the most critical.

It was shown in Chapter 8 that the values of $\phi$ for most joints were in the range 25° to 35°. Since the J3 joints are statistically normal to the bedding which dips 60° to 80° west, the angle $\alpha$ is in the range 30° to 10°. Therefore, for the majority of the joints and stability can be maintained by friction alone. However, for individual joints with $\alpha > \phi$ stability will be maintained only by cohesion on the joint plane.

In assessing the stability of the batter as a whole, the worst combination of $\alpha$ and $\phi$ likely to occur on a significant number of joints is $\alpha = 35°$ and $\phi = 25°$. 
For $H = 550$ ft and $\gamma = 165$ lb/cu.ft, factors of safety for different values of $\beta$ can be calculated from eqns 10.3 and 10.4. Various values for the three cases are shown in Table 10.1. It is evident that quite small values of cohesion are sufficient to significantly increase the stability of the batter. For example, for $\beta = 50^\circ$ and $c = 50$ psi, the factor of safety is 1.2 in the worst case. This would be acceptable for a relatively short term mining operation, when it is considered that the majority of J3 joints will have $\alpha < \phi$. As can be seen from Table 8.2 this cohesion value is obtained in nearly all the friction tests on natural joints. Therefore a slope with the geometry quoted above theoretically should have a safety factor greater than unity even if the critically oriented joint is continuous.

However, it would appear that the above analysis is conservative. The J3 joints are characteristically discontinuous. The discontinuity can assume two forms which cannot be distinguished by the exploration described earlier in the thesis. Firstly, the joint may form a single plane containing "gaps", Terzaghi (1962), of solid rock which must be sheared before slip can occur. The effective cohesion, $c_i$, is given by

$$c_i = c \frac{A}{E}$$  \hspace{1cm} (10.5)
Table 10.1
Factors of safety for sliding on a single planar joint

\[ \alpha = 35^\circ \quad H = 550 \text{ ft} \]
\[ \phi = 25^\circ \quad \gamma = 165 \text{ lb/cu.ft} \]

<table>
<thead>
<tr>
<th>Values of F</th>
<th>$c$ (psi)</th>
<th>$\beta = 40^\circ$</th>
<th>$\beta = 45^\circ$</th>
<th>$\beta = 50^\circ$</th>
<th>$\beta = 55^\circ$</th>
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<tr>
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<tr>
<td>0</td>
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<td>0.67</td>
<td>0.67</td>
<td>0.67</td>
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<td>0.83</td>
<td>0.81</td>
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<tr>
<td>20</td>
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<td>1.12</td>
<td>1.00</td>
<td>0.95</td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>1.90</td>
<td>1.35</td>
<td>1.16</td>
<td>1.10</td>
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<tr>
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<td>2.31</td>
<td>1.57</td>
<td>1.32</td>
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<tr>
<td>50</td>
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<tr>
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<td>1.81</td>
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<tr>
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<td>2.14</td>
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<td>2.93</td>
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<td>2.09</td>
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<tr>
<td><strong>Case C</strong></td>
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<td></td>
</tr>
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<td>0.89</td>
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<td></td>
</tr>
<tr>
<td>100</td>
<td>3.37</td>
<td>2.17</td>
<td>1.77</td>
<td>1.57</td>
<td></td>
</tr>
</tbody>
</table>
where \( c \) is cohesion of intact rock
\[
A_g \quad \text{is area of gaps in section}
\]
\[
A \quad \text{is total area of the section.}
\]
From the triaxial compression tests (Chapter 8) the cohesion of intact rock across bedding or cleavage was found to be several thousand psi. Clearly then, \( c_i \) can give a considerable additional contribution to stability even if \( A_g \) is quite small.

Secondly, the joint surface may be continuous but stepped to form a "brick wall" structure, Terzaghi (1962), Fig. 10.2. There are three different conditions here

(i) For \( c = 0 \) and \( \alpha < \phi \), the maximum stable slope angle is \( 90^\circ \) and independent of \( H \).

(ii) For \( c = 0 \) and \( \alpha > \phi \), the maximum stable slope angle is \( (\alpha + \delta) \) and independent of \( H \). \( \delta \) is a function of the relative dimensions and staggering of the blocks and is given by

\[
\tan \delta = \frac{L}{M} \quad (10.6)
\]

The staggering distance, \( L \), is unknown in detail but statistically must be equal to half the average joint spacing in that direction. It was shown that the rock blocks in the present case are approximately equi-dimensional. Therefore,
Fig. 10.2. Effect of staggering of joints on slope stability.
\[ \tan \delta = \frac{1}{2} \]

and \[ \delta = 26^\circ \]

and with an average value of \( \alpha = 30^\circ \), the stable slope, even with zero cohesion, is \( 56^\circ \).

(iii) For \( c > 0 \) and \( \alpha > \phi \), the maximum stable slope angle is a function of both the height, \( H \), and the cohesion, \( c \). The situation is different from that shown in Fig. 10.1, since although the basic stable slope angle is \( (\alpha + \delta) = \psi \), individual blocks slide on joint planes inclined at angle \( \alpha \). For failure of the complete block with the bedding planes as secondary failure plane, Fig. 10.2(a), the factor of safety for \( \beta > \psi \) is

\[ F = \frac{\tan \phi}{\tan \alpha} + \frac{2c \cos \delta \sin \beta}{\delta H \sin \alpha \sin(\beta - \psi)} \quad (10.7) \]

Similar expressions can be obtained for the case where the J2 joints are secondary failure planes, Fig. 10.2(b) or for partial block failures similar to those shown in Figs. 10.1(b) and (c). However, the single failure plane, Fig. 10.1, is clearly the more critical in all cases, provided that failure through intact blocks does not occur. The strength of rock at right angles to bedding or cleavage, Chapter 8, indicate that this is unlikely in the Black Star
open cut.

The effect of cohesion on the stability of individual bench slopes can be calculated as for the overall batter above. For \( \alpha = 35^\circ \), \( \beta = 70^\circ \), \( \phi = 25^\circ \), \( H = 70 \) ft, the factors of safety for the most critical case, eqn 10.4, are

\[
\begin{align*}
    c = 0 \text{ psi}, & \quad F = 0.67 \\
    c = 10 \text{ psi}, & \quad F = 1.15 \\
    c = 20 \text{ psi}, & \quad F = 1.63 \\
    c = 30 \text{ psi}, & \quad F = 2.11
\end{align*}
\]

Therefore, very small values of cohesion are again sufficient to impart quite high factors of safety. The acceptable factor of safety for these slopes needs to be higher than that for the overall slope since their smaller dimensions make it more probable that a continuous plane of low friction could occur.

One further point is that the whole of the previous discussion refers to the simple two-dimensional case with no restraint on the vertical faces of the sliding block. In practice, the cohesion on these surfaces will give additional restraint to the blocks. On the other hand, if clay-filled joints are encountered, their cohesion is likely to be negligible and factors of safety in nearly all cases would be less than unity. Although no significant
clay seams were detected in the drilling, close inspection would be required during the excavation of the open cut to confirm that none are present. In addition, fairly frequent minor rock falls must be expected from the faces of the slopes when individual blocks resting on continuous joints have their support removed by excavation.

**Stability of tetrahedral wedges**

The wide range of orientations of J3 joints normal to the bedding, makes it probable that in many cases the mode of failure will be by sliding of a tetrahedral wedge on two inclined joints. A typical situation is shown in Fig. 10.3(a). Partial block failures, as in Fig. 10.1(b) and (c) are similarly possible.

If contact is maintained on both joint faces during sliding, there is only one possible slip direction, which is the line of intersection of the two planes. Since the joint poles lie in a great circle girdle, Fig. 4.21, the line of intersection of all combinations of joints is defined by the pole of the plane containing the joint poles, i.e. the pole of the bedding plane. Two typical joints J1 and J2 with poles P1 and P2 are shown on the equal area projection, Fig. 10.3(b), with the line of intersection, K.

The analysis of the stability of such a wedge follows
Fig. 10.3. Tetrahedral block formed by intersecting J3 joints.
exactly that given above for blocks sliding on a single inclined plane, except that the geometry is more complex. However, a method recently proposed by John (1968) permits a rapid graphical appraisal of the situation. For the moment, neglecting cohesion and considering only frictional resistance, when the wedge is on the point of sliding, the resultant force on the wedge from each plane must lie in a plane containing the pole of the joint plane and the slip direction. When friction is fully mobilised the resultant forces, $R_1$ and $R_2$, must be located at $\theta$ degrees along the great circles from the respective poles of the joint planes in the correct sense, Fig. 10.4(a). For equilibrium to be maintained, the plane containing the resultant forces $R_1$ and $R_2$ must also contain the direction of the resultant external force on the wedge. If gravity is the only external force, this means that $R_1$ and $R_2$ must lie in a vertical plane. The case of different coefficients of friction on the two planes is easily included.

An example is shown in Fig. 10.4(b) for two J3 joints intersecting to give $\alpha = 30^\circ$. In this case, the angle of friction mobilised on each joint plane to preserve stability is $22^\circ$. For a given value of $\alpha$ it can easily be shown that the angle of friction mobilised increases from zero to a maximum as the two joint poles move from the
Fig. 10.4. Graphical analysis of stability of tetrahedral block.
primitive (corresponding to two parallel east-west vertical planes) towards coincidence in a vertical plane normal to the batter (corresponding to a single failure surface as shown in Fig. 10.1). Therefore, the tetrahedral wedge case is not as critical as the single sliding surface case in this problem where the poles of J3 joints lie in a great circle girdle. This is not necessarily so in a general case where there are no restraints on the orientation of the joint planes.

To determine the influence of cohesion on the joint planes it is necessary to calculate their areas, which is a fairly complex problem in three-dimensional geometry. However, the angular relations can be easily determined on a stereographic projection. A simple example is shown in Fig. 10.5 with corresponding angles on the block diagram in (a) and on the projection in (b). Problems of any complexity can be handled in this manner. There is no doubt that the stereographic projection is fundamental in all of this type of work. Moreover, if a large number of cases have to be analysed it is a relatively simple matter to program the constructions on the projection for the computer, using methods similar to those described in Appendix B.
Fig. 10.5. Use of stereographic projection in determining angular relations for tetrahedral blocks.
Effect of water pressure in joints

All of the above discussion has referred to sliding on dry joints. It was shown in Chapter 8 that the frictional properties of the joints are not significantly altered by mere wetting, but the effect of water pressure in the joints on stability must be considered. These pressures, analogous to pore pressures in soils and termed "cleft water pressures" by Terzaghi, have two distinct effects.

(i) They reduce the frictional resistance to sliding on a plane in accordance with the effective stress law.
(ii) They increase the forces tending to cause instability by hydrostatic pressure in cracks in the rock mass.

The effects of cleft water pressures can easily be incorporated into a stability analysis provided their magnitude is known. The details are fully discussed by Muller (1964) and Terzaghi (1962) and will not be considered here.

However, it is clear from the stability calculations given above that any cleft water pressures will seriously reduce the factor of safety for sliding on the J3 joints. It is therefore essential that the cleft water pressure surface be kept permanently below the deepest potential slip plane at various stages of open cut development. Mount Isa is situated in a very arid climate and with the
drainage provided by the mine workings the static water table is likely to be well below the bottom of the open cut. However, high rates of precipitation occur during the wet season and it is possible that dangerous transient cleft water pressures may be set up. For this reason adequate drainage provisions must be an integral part of any batter design.

The most practical method of cleft water control is probably by drainage galleries driven into the batter. The number and spacing of these can be determined by methods such as the electrical resistance analogue described by Hoek and Pentz (1968). However, the input data, such as permeability of the joint system, can only be determined by field measurement with piezometers. Although this is extremely important it was not considered during the present investigation since at the present time (1968) the Black Star open cut is not firmly committed because of metallurgical difficulties and the depressed price of lead. Further expense in field measurements could therefore not be justified at this stage. However, if, and when, it is decided to proceed with the open cut, installation of piezometers at appropriate locations behind the batter must have high priority.
Dynamic stresses

The other factor which has not received attention is the effect of dynamic stresses from earthquakes or blasting on the stability of the batter. Mount Isa is in a stable area and the occurrence of a large earthquake is extremely unlikely. However, if large scale blasting is carried out the effects may be significant.

This matter is discussed by Hoek and Pentz (1968). It is first necessary to measure the magnitudes and directions of the accelerations produced by blasting by a suitable seismic instrument. The effect is then considered in stability analysis by including an additional body force of appropriate magnitude and direction. The influence of these forces on sliding of wedges is also discussed by John (1968). However, in general, it is not considered that this will be an important factor since the magnitudes of the accelerations can be controlled by reducing the size of individual blasts.

Conclusions

The simple stability analyses given above show that major sliding is only likely on the J3 joints and that acceptable factors of safety can be obtained if fairly small cohesion values are assumed for the joint planes.
Since these are likely to be present, there seems no reason why an overall batter slope of 50° should not be used in preliminary batter design, provided that it is accepted in principle that adequate drainage measures will be provided. In the author's opinion it would not be prudent to assume a higher angle at this stage since the various assumptions are largely untested on a large scale. In fact, however, it is possible that much steeper batters are not geometrically possible since the haul road and minimum berm widths must be included in the batter design. The further information on structure and other variables obtained as excavation proceeds will permit progressive amendments to any proposed design to suit particular circumstances.
REFERENCES


HILLS, E.S. (1963) Elements of structural geology, Methuen: London.


Appendix A

PHOTOGRAPHS OF DRILL CORE

The core is laid out in nominal five foot lengths with the foot marks coinciding, in order to highlight core losses. The hole and depth are marked on each photograph and the scale is given by the black and white marker, with one foot graduations, shown in the middle of each photograph.
It has been repeatedly stressed in this thesis that manipulations of lines and planes in three dimensions can be most conveniently performed on a stereographic projection. Statistical analysis by contouring on an equal area projection is also a relatively simple procedure. A difficulty arises, however, when a large amount of data must be analysed since the procedures, although simple, become very laborious and time consuming. This is particularly true when rotations about non-vertical axes are involved, such as in the present method of core orientation. Computer processing of the data was therefore considered. The programs which have evolved are quite general and very powerful and have been extensively used by other workers in structural analysis at the Australian National University.

The programs were written in FORTRAN IV(G) for the IBM 360/50 computer at the A.N.U. Computer Centre. (I am indebted to Dr. R. Underwood for my introduction to FORTRAN programming and for helpful advice in the early stages of the project.) Essentially they consist of a main program, which varies with the type of data supplied
and output required, and which calls various subroutines for rotating, plotting, and contouring the data by different methods. These subroutines are as follows:

(i) ROTATE - rotates poles about any axis specified by three nominated rotation angles.

(ii) SCATTR - plots a scatter diagram of the rotated poles.

(iii) SCHMDT - contours density of rotated poles according to the standard Schmidt method.

(iv) TERZAG - contours density of rotated poles according to the Schmidt method, including the Terzaghi correction procedure.

(v) KAMB - contours density of rotated poles according to the Kamb method.

(vi) COUNT - called by SCHMDT, TERZAG and KAMB to perform the actual counting out procedure.

(vii) CONTUR - called by SCHMDT and TERZAG to calculate contour intervals and set up symbolic data for plotting.

(viii) CIRCLE - called by all the plotting subroutines to insert the primitive circle and titles on the projections.

Listings of all these programs are appended.

Main programs

A variety of programs has been written for different
purposes but only two are listed to illustrate the method. The purpose of these programs is to read each pair of measured parameters, such as trend and plunge of a pole, which is then rotated by subroutine ROTATE. The rotated trends and plunges are stored in two parallel arrays TREND and PLUNG, respectively, which are input data to the plotting subroutines. These arrays have adjustable dimensions in the subroutines, so that any number of readings up to the maximum declared dimensions of these arrays may be automatically processed. The readings are also automatically counted by the main program.

Program DRILL CORE is specifically designed to analyse measurements from the drill core as described in Chapter 4. Input data are lists of the two angles \( \alpha \) and \( \beta \) (ref. Section 4.4), in this case punched one pair per card. At the head of each batch of data is a header card with data description, followed by a card with the three rotation angles \( V_1, \) ROT, \( V_2 \); if rotation is not required, a blank card is inserted and subroutine ROTATE is not called. For each reading, the Terzaghi weighting factor is calculated and stored in a third array, CORRN for subsequent input to subroutine TERZAG.

Program FABRIC EIGHT is a general purpose program designed to process a wide variety of structural data
including:

(i) universal stage measurements (inner vertical axis and north-south tilt axis readings);
(ii) trends and plunges of poles;
(iii) dip directions and dips of planes;
(iv) strikes and dips of planes.

The type of data to follow is signified by the digits 0, 1, 2 or 3, respectively, in col. 41 of the header card for each batch of data. In this program, the data is punched eight readings per card.

Other programs have been written for specific problems, including one which computes the angles between c-axes of quartz grains measured on a universal stage.

Subroutine ROTATE

This subroutine is based on a program by McIntyre (1963). I am grateful to Professor McIntyre for sending me a copy of his unpublished report. Input data are trend and plunge of pole to be rotated and the three rotation angles; output data are trend and plunge of rotated pole. The method is quite independent of any method of projection since computations are performed relative to the surface of the reference sphere. However, the method is more easily explained with reference to a
projection, Fig. B.1. The basic case is shown in (a) where the pole $Q$, with trend $T$ and plunge $P$, rotates about an E-W horizontal axis through angle $\alpha$ to pole $Q'$. The E-W axis is fixed as the horizontal rotation axis to simplify the program logic and is obtained by the first rotation $(V1)$ about the vertical axis.

The plane $ROQR$ containing $Q$ rotates to a new position $R'O'Q'$ and it follows that:

(i) the angle between the two lines, of which $O$ and $Q$ are the poles, remains constant, that is

\[ \angle O'Q' = \angle OQ = 90^\circ - P = b \]

(ii) the angle between the two planes $ROR$ and $SON$ remains constant, that is

\[ \angle NOQ = T \]

and

\[ \angle OO'Q' = 180^\circ - T = \gamma \]

(iii) the poles $O$ and $Q$ rotate through the same angle, that is

\[ \angle OO' = \angle OQ' = a. \]

It is therefore possible to solve the spherical triangle $OO'Q'$ for the side $OQ' = c$ and the angle $O'OQ' = \beta$ by means of the equations

\[ \cos c = \cos a \cdot \cos b + \sin a \cdot \sin b \cdot \cos \gamma \quad (B.1) \]
Fig. B.1. Rotation of poles about east-west axis on lower hemisphere projection.
and

\[ \cos \beta = \frac{\cos b - \cos a \cos c}{\sin a \cdot \sin c} \quad (B.2) \]

The trend \((T')\) and plunge \((p')\) of the rotated pole are given by

\[ T' = \beta \quad (B.3) \]

\[ p' = 90^\circ - c \quad (B.4) \]

For the computer program it is essential that cosine formulae be used since this is the only way of distinguishing between acute and obtuse angles in the solution of the equations.

If the side \(c\) is greater than \(90^\circ\), the pole has rotated into the upper hemisphere, Fig. B.1(b), and the conjugate pole is computed, since it is desired to project only the lower hemisphere in the plotting subroutines.

To simplify the program logic all rotations are confined to the "eastern" hemisphere. The mirror image is taken of poles in the western half, the computations performed, and another mirror image taken to give the final result. Rotations about a vertical axis are simply performed by adding the rotation angle to the trend of the pole.

This program can rotate poles at the rate of approximately 400 per minute on the IBM 360/50.
Plotting of output data

An important feature of all the plotting and contouring subroutines is that they output a standard 20 cm. diameter equal area projection. This allows additional constructions to be performed, if required, on a standard net. Since a graph plotter was not available, all plotting was done on the line printer and it is convenient to consider the general principles of this technique.

The elements of a two-dimensional array can be considered as the nodes of a two-dimension grid even though the array is not physically stored in this fashion in the computer. To display data graphically on the line printer it is only necessary to compute values at the appropriate "grid points" and then print the array. The only limitation is that the symbols printed must conform to the printer positions and stepped rather than continuously curved graphs are produced.

A circle such as the primitive of a projection can be displayed by plotting the array in which the circle is inscribed. One difficulty with most line printers is that the character spacings are different for rows and columns. For the IBM 1403 printer used here, the spacings are

10 characters per inch in rows

and

8 characters per inch in columns.
Therefore, a square array is printed as a rectangle and the inscribed circle as an ellipse. Consequently, to plot a circle, a rectangular array with an inscribed ellipse must be set up in the computer. In this case, the 20 cm. circle requires a rectangular array of 78 columns x 63 rows. All computations are performed on the 79 x 79 array ACONT which is stored in COMMON and used by all the subroutines.

Subroutine SCATTR

The radial distance \( r \) of a pole with trend \( T \) and plunge \( P \) from the centre of a projection of radius \( R \) is given by, Fairbairn (1949),

\[
r = \sqrt{2} \ R \sin\left(\frac{90^\circ - P}{2}\right)
\]

for the equal area projection, and

\[
r = R \tan\left(\frac{90^\circ - P}{2}\right)
\]

for the stereographic projection. The cartesian coordinates of the pole are then given by

\[
x = r \sin T
\]

\[
y = r \cos T
\]

In this program, the \( x \) and \( y \) axes of the array have unequal scales so that a square, with inscribed circle,
is printed. The coordinates of each pole are rounded off to the nearest element of the array, which is incremented by one for each pole. When all the poles have been processed, the element values are converted to appropriate symbols and the array printed. This step is necessary to permit that part of the projection with no poles to be blank and also for elements with greater than 9 poles to be represented by a single symbol. Subroutine CIRCLE is called to define the projection by stars at 10 degree intervals around the primitive.

Contouring subroutines

For contouring to be rigorous it should be performed on the surface of the reference sphere, since there are difficulties associated with the size and shape of a counter on the stereographic and equal area projections, respectively, as described in Chapter 4. However, in geology, there are very few cases where these errors are significant since the source data usually has measuring errors of several degrees. For this reason, the somewhat simpler method of contouring on the projection using a circular counter was adopted.

The counting-out procedure is based on the method of Spencer and Clabaugh (1967), and is a combination of the Schmidt and Mellis methods of manual contouring,
Turner and Weiss (1963). The cartesian coordinates of each pole is computed as in SCATTR and a counting circle is effectively centred on the computed point. This is achieved by computing the distance between the pole point and each neighbouring grid point; if the distance is less than the radius of the counting circle the grid point is incremented by a certain amount. In SCHMDT and KAMB this amount is unity and in TERZAG it is equal to the weighting factor for that pole. Following Spencer and Clabaugh (1967), only grid points in the immediate vicinity of the pole are tested in order to save computer time. If the pole falls within one counting circle radius from the primitive, the counting circle breaches the primitive and an imaginary conjugate pole, one projection diameter distant through the centre, is also counted out. Grid point values outside the primitive are meaningless and are returned to zero when all poles have been counted out.

In SCHMDT and TERZAG, the area of the counting circle is one percent of the area of the projection. In KAMB, the area of the counting circle \( a \) varies with the number \( N \) of poles to be contoured and is given by

\[
a = \left( \frac{9}{N+9} \right) A
\]

where \( A \) is the area of the projection.
A great advance over previous methods is that shaded contoured diagrams are immediately produced by the computer. This is achieved by allotting each grid point in the array a shading symbol appropriate to its value. In SCHMDT and TERZAG, the contour intervals are determined by the maximum contour value and are as follows:

<table>
<thead>
<tr>
<th>Maximum Value</th>
<th>Contour Intervals</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 7</td>
<td>1, 2, 3, 4, 5</td>
</tr>
<tr>
<td>&gt; 5 and &lt; 10</td>
<td>1, 2, 4, 6, 8</td>
</tr>
<tr>
<td>&gt; 10 and &lt; 15</td>
<td>1, 3, 6, 9, 12</td>
</tr>
<tr>
<td>&gt; 15 and &lt; 20</td>
<td>1, 4, 8, 12, 16</td>
</tr>
<tr>
<td>&gt; 20</td>
<td>1, 5, 10, 15, 20</td>
</tr>
</tbody>
</table>

where all contour values are expressed in terms of percent of total poles per one per cent area of the projection. These intervals are quite arbitrary and can be varied if desired. In KAMB contours are at intervals of \(2\sigma\) where \(\sigma\) is the standard deviation from the expected density of a random population, Kamb (1959).

Since it would be rather difficult to perform the counting-out procedure on a grid with different scales on the axes, the whole procedure is performed on a square array. In order to print a circular projection, approximately every fifth row of the square array is removed before printing. This is equivalent to com-
pressing the ellipse into a circle and does not involve any approximation since the array values are merely shading symbols and not actual values at this stage.

The structure of the subroutines is influenced by the amount of common calculation in the three methods. Thus, the counting-out procedure is common to all three and is performed by subroutine COUNT but the counting circle radius and increment value are specified by each calling program. The contouring method is identical in SCHMDT and TERZAG and is performed by subroutine CONTUR, while a different method of contouring is included in KAMB. Plotting of the primitive and part of the title common to all methods is performed by subroutine CIRCLE.

Examples of output from the program are shown in Figs. B.2, B.3, B.4 and B.5. These are the printouts from which Fig. 4.2 was prepared. Total storage requirements depend on the declared dimensions of the arrays TREND, PLUNG and CORRN, but for 1000 poles is 66,000 bytes.
C K. J. ROSENGREN PROGRAM DRILL CORE
C THIS PROGRAM PROCESSES DATA MEASURED ON DRILL CORE. INPUT DATA THREE
C OF ROTATION ANGLES AND PAIRS OF MEASUREMENTS OF
C (1) AT=ANGLE BETWEEN FEATURE AND REFERENCE LINE
C (2) AP=ANGLE BETWEEN FEATURE AND CORE AXIS
C DIMENSION TREND(1000), PLUNG(1000), CORRN(1000)
C COMMON ACONT(79,791), CODE(40), NV1,NROT,NV2
C
P1=3.141593
C READ HEADER CARD AND ROTATION ANGLES
R READ1.11 CODE=NEND
1 FORMAT(40A1,11)
   IF(NEND.GT.1) GO TO 7
   N=0
4 READ(I,2)MV1,NROT,MV2
2 FORMAT(14,1X,14,1X,14)
   IF(MV1.GT.900) GO TO 6
   NV1=MV1
   NV2=MV2
   NROT=MROT
C READ MEASURED DATA, CALCULATE TERZAGHI CORRECTION FACTOR, CONVERT TO POLE
C ON PROJECTION, ROTATE AND STORE ROTATED POLE, COUNT NUMBER OF READINGS
5 READ(I,3)AT,AP
3 FORMAT(11,9X,F5.0,F4.0)
   IF(N.GT.8) GO TO 4
   SINAP=SIN(AP*PI/180.)
   IF(SINAP.LT.0.21) SINAP=0.2
   AT=AT+180.
   IF(AT.GE.360.) AT=AT-360.
   CALL ROTATE(AT,AP,NV1,NV2,NROT,TR,PL)
   N=N+1
   TREND(I)=TR
   PLUNGN=PL
   CORRN(I)=1./SINAP
   GO TO 5
6 CONTINUE
C CALL SUBROUTINES
C CALL SCATT(N,TREND,PLUNG)
C CALL SCHMT(N,TREND,PLUNG)
C CALL TERZAGH(N,TREND,PLUNG,CORRN)
C CALL KAMB(N,TREND,PLUNG)
GO TO 8
7 STOP
END
C K. J. ROSENGREN PROGRAM FABRIC EIGHT

C THIS IS A GENERAL PURPOSE PROGRAM FOR ANALYSIS OF FABRIC DATA
C READINGS ARE PUNCHED EIGHT PER CARD
C HEADER CARD HAS ANY DESCRIPTIVE DATA IN COLS 1 TO 40, A DIGIT IN
C COL 41 INDICATES TYPE OF DATA TO FOLLOW
C BLANK OR 0 DENOTES UNIVERSAL STAGE MEASUREMENTS (INNER VERTICAL
C CIRCLE AND N-S TILT, TILT TO WEST POSITIVE)

DENOTES TEND AND PLUNGE OF POLES
2 DENOTES DIP DIRECTION AND DIP OF PLANES
3 DENOTES STRIKE AND DIP OF PLANES
DIMENSION TREN! (1000), PLUNG (1000), AT (8), AP (8)
COMMON HCONT (79, 79), CODE (40), NVL, NROT, NV2
C READ Header DATA, CONVERT TO POLES (IN PROJECTION, ROTATE AND STORE
C ROTATED POLES, COUNT NUMBER OF READINGS
99 N=0

READ (1) CODE, NEND
1 FORMAT (40A1, I)
   IF (NEND .EQ. 0) GO TO 93
2 READ (1) NV1, MROT, MV2
3 FORMAT (3I4)
   IF (NV1 .GT. 900) GO TO 97
   NV1 = NV1
   NROT = MROT
   IF (NV1 .EQ. 0 .AND. NROT .EQ. 0 .AND. NV2 .EQ. 0) MM=1
4 WRITE (3, 52) CODE, NEND, NV1, NROT, NV2
52 FORMAT (1X, 'RECORDED ROTATED POLE')
10 READ (1, 5) (AT (I), AP (I), I = 1, 8)
   5 FORMAT (8(F4.0, F4.3))
   CONTINUE
7 J=1-1
   GO TO 9
8 CONTINUE
   CT = AT (I)
   CP = AP (I)
   IF (NEND .EQ. 3) GO TO 41
   IF (NEND .EQ. 2) GO TO 21
   IF (NEND .EQ. 1) GO TO 11
   AT (I) = 270 - AT (I)
   AP (I) = -AP (I)
   GO TO 12
11 AT (I) = AT (I) + 270.
   AP (I) = -AP (I)
   GO TO 12
21 AT (I) = AT (I) + 180.
   AP (I) = 90 - AP (I)
12 IF (AT (I) .GT. 900) AT (I) = AT (I) + 360.
   IF (AT (I) .LT. 0) AT (I) = AT (I) - 360.
31 BT = AT (I)
   BP = AP (I)
   IF (MM .EQ. 1) GO TO 13
   CALL ROTATE (BT, BP, NV1, NV2, NROT, TR, PL)
   GO TO 14
13 TR = BT
   PL = BP
14 TREND (N+1) = TR
   PLUNG (N+1) = PL
   N = N+1
   CONTINUE
N = N+1
   GO TO 10
9 N = N+1
   GO TO 2
97 CONTINUE
C CALL PLOTTING SUBROUTINES
C WRITE (1) S1 IN, CT, CP, TREND (N+1), PLUNG (N+1)
51 CONTINUE
   N = N+1
   CONTINUE
GO TO 99
99 STOP
END
SUBROUTINE ROTATE

This subroutine rotates poles on a lower hemisphere spherical projection.

Input data from calling program are trend (AT) and plunge (AP) of pole to 3f.

Rotated and three rotation angles.

(1) NV1 = rotation about vertical axis, clockwise positive.

(2) NROT = rotation about horizontal east-west axis, positive angle between 0 and 180 deg, clockwise looking east along axis.

(3) NV2 = second rotation about vertical axis, clockwise positive.

Output data are trend (TR) and plunge (PL) of rotated pole.

All input/output angles in degrees.

SUBROUTINE ROTATE(AT,AP,NV1,NV2,NROT,TR,PL)

PI = 3.141593
HPI = PI/1.0
PI2 = PI/2.0
AF = PI/180.0
VF = NV1*AF
V2 = NV2*AF
ROT = NROT*AF
AT = AT*AF
AP = AP*AF

C FIRST ROTATION ABOUT VERTICAL AXIS

IF(AT < PI/2) AT = AT*PI2

IF(NROT = 0 OR 180) BYPASS SPHERICAL TRIANGLE

IF(NROT = 0) GO TO 102

IF(NROT = 180) GO TO 106

POLE LIES IN WESTERN HEMISPHERE, TAKE MIRROR IMAGE AND PUT J = 1

IF(AT < PI/2) GO TO 101

J = 1

C CALCULATE KNOWN SIDES AND ANGLES AND SOLVE SPHERICAL TRIANGLE

101 GAMMA = PI/2 - AT

SIDE = HPI - AP
SIDEC = ROT
SSA = SIN(SIDE)
CSA = COS(SIDE)
SSB = SIN(SIDEC)
CSB = COS(SIDEC)
CSC = COST(GAMMA)

IF(CSC < 1.0 AND CSC > -1.0) SIDE = COS(GAMMA)

IF(CSC < 1.0 AND CSC > -1.0) SIDE = ARCCOS(CSC)

IF(CSC > 1.0) SIDE = 0.0

C COMPARE PLUNGE OF ROTATED POLE, IF SIDE > HPI PLKF HAS ROTATED

C INTO UPPER HEMISPHERE, SO COMPUTE CONJUGATE POLE, ALSO CHECK IF UNROTATED

IF(SIDE > HPI) GO TO 102

PL = HPI - SIDE
TR = ETA
GO TO 103

102 IF(J.EQ.1) TR = PI2 - TR
GO TO 106

103 IF(J.EQ.1) TR = PI2 - TR
GO TO 106

104

PL = AP
GO TO 106

105

C SECOND ROTATION ABOUT VERTICAL AXIS

106 TR = TR + V2

IF(J.EQ.1) TR = TR + PI2

IF(TR = PI) TR = TR - PI2

IF(TR = PI2) TR = TR - PI2

PL = PL/AF

RETURN

END
SUBROUTINE SCATT(n, trend, plunge)

DIMENSION trend(n), plunge(n), symbol(35), scat(15)

COMMON acont(79, 79), code(40), nv1, nrot, nv2

DATA blank, 0, code(0), symbol, scat, 79, 79, 40, nv1, nv2 / *:

DO 211 k = 0, n
  DO 221 l = 1, m
    DO 211 i = 1, 360
     精 = trend(i) * af
      pl = plunge(i) * 0.5 * af
      xx = rad * sin(i)
      yy = rad * cos(i)
      x = xx * c.73 * s.c.5
      y = 32.5 - yy * c.65
      k = x > 0.5
      l = y < 0.5
      acont(k, l) = acont(k, l) + 1
  END 221
  CONTINUE

CONVERT GRID VALUES TO APPROPRIATE SYMBOLS

DO 214 k = 0, 79
  DO 214 l = 1, 64
    IF (acont(k, l) < 0) acont(k, l) = 0
    IF (mpot ne 0) acont(k, l) = blank
    mpot = acont(k, l)
    IF (mpot) acont(k, l) = symbol(i)
  END 214
  CONTINUE

C INSERT PRIMITIVE AND TITLES AND PRINT PROJECTION

CALL CIRCLE

WRITE(3, 281) code, nv1, nrot, nv2, (acont(k, l) * k = 1, 79, l = 1, 64)

RETURN

END

SUBROUTINE CIRCLE

DIMENSION hem(16), star(35)

DATA symbol, star / *:

DO 911 i = 1, 360
  sin = sin(i * af)
  cos = cos(i * af)
  x = 15.5 * sin(i * af)
  y = 32.5 + x * cos(i * af)
  k = x > 0.5
  l = y < 0.5
  acont(k, l) = star
  CONTINUE

RETURN

END
C.  K. J. ROSENGREN SUBROUTINE SCHMT

C THIS PROGRAM CONTOURS POLES ON AN EQUAL AREA PROJECTION USING THE
C SCHMT METHOD WITH A ONE PERCENT AREA CIRCULAR COUNTER
C INPUT DATA ARE TRENDS AND PLUNGE AND NUMBER OF POLES
C SUBROUTINE SCHMT(TREND, PLUNG, CINT, KINT)
C DIMENSION TREN(D), PLUN(D), CINT(5), KINT(5)
C COMMON ACNT, COUNT, CODE, COUNT, NCT, NVE
C DATA TITLE, CINT, HINT, PINT, TINT
C *HINT., HINT, PINT, TINT
C AF=, 1.11093/180.
C 411 K=1,70
C 411 L=1,70
C 411 COUNT=0.

411 CONTINUE

C DEFINE COUNTING INCREMENT AND COUNT OUT PROJECTION
R=3.90
ADD=1.
DO 417 J=1,5
T=TREND(J)*AF
P=90.-PLUN(J)*AF
CALL COUNT(T,P,ADD)
417 CONTINUE

C SET UP CONTOUR INTERVALS, CONVERT TO SYMBOLS AND PRINT PROJECTION
AM=3.
CALL CONTUR(AN,CINT,KINT,PMAX)
CALL CIRCLE
TO 425 K=66,79
I=65
ACONT(I,01)=TITLE(1)

425 CONTINUE

WRITE(*,440) CODE,NCT,NCT,NCT,NCT,NCT,NCT,NCT,NCT,NCT,NCT

440 WRITE(*,440) CODE,NCT,NCT,NCT,NCT,NCT,NCT,NCT,NCT,NCT

RETURN
END

C.  K. J. ROSENGREN SUBROUTINE COUNT

C THIS PROGRAM CONTORS OUT POLES ON AN EQUAL AREA PROJECTION, INPUT DATA
C TREN AND PLUNGE OF EACH POLE, COUNTING INCREMENT AND RADIS OF
C COUNTING CIRCLE
C SUBROUTINE COUNT(TREND, PLUNG, COUNTING INCREMENT AND RADIUS OF
C COMMON ACNT, COUNT, CODE, COUNT, NCT, NVE
C DATA TITLE, CINT, HINT, PINT, TINT
C *HINT., HINT, PINT, TINT
C AF=, 1.11093/180.
C 411 K=1,70
C 411 L=1,70
C 411 COUNT=0.

C COMPUTE RECTANGULAR COORDINATES WITH ORIGIN AT (1,1), DEFINE LIMITS C OF SUB-ARRAY, TEST DISTANCE FROM POLE TO EACH GRID POINT.
C RAD=12.979.*SIN(P)
C 1X=RADCOS(P)
C 1Y=RADSIN(P)

1X=X+40.
1Y=Y-40.

IF(1X.LT.1) X=1.
IF(1X.GT.99) X=99.
IF(1Y.LT.1) Y=1.
IF(1Y.GT.99) Y=99.

CALL COUNTING(1X-1,1Y-1,1X+1,1Y+1,1X-1,1Y+1,1X+1,1Y-1)

RETURN
END
C THIS PROGRAM COMPUTES CONTOUR INTERVALS FOR SCHMIDT METHOD AND APPLYING THE TEZIAGH CORRECTION PROCEDURE.

C INPUT DATA ARE TREND AND PLUNGS AND NUMBER OF POLES WITH APPROPRIATE WEIGHTING FACTORS.

C DEFINING COUNTING INCREMENT AND COUNT OUT PROJECTION.

C SET UP CONTOUR INTERVALS, CONVERT TO SYMBOLS AND PRINT PROJECTION.

C COMPUTE MAXIMUM CONTOUR VALUE AND CONTOUR INTERVALS.

C CONVERT EACH CP TO VALUE TO APPROPRIATE SHADING SYMBOL.

C COMPRESS CIRCLE INTO ELLIPSE FOR PRINTING BY REMOVING APPROX.

C EVERY FIFTH ROW.

C SUBROUTINE TERZAGH.

C DEFINED COUNTING INCREMENT AND COUNT OUT PROJECTION.

C SETUP CONTOUR INTERVALS, CONVERT TO SYMBOLS AND PRINT PROJECTION.

C COMPUTE MAXIMUM CONTOUR VALUE AND CONTOUR INTERVALS.

C CONVERT EACH CP TO VALUE TO APPROPRIATE SHADING SYMBOL.

C COMPRESS CIRCLE INTO ELLIPSE FOR PRINTING BY REMOVING APPROX.

C EVERY FIFTH ROW.
C subroutine KAM9

This program contours poles on an equal area projection using the
KAM9 method with a variable area circular counter.

Input data are trends and plunges and number of poles.

Subroutine KAM9: Input Trends, Plunges, Symbols.

Dimension A (input trends), P (output trends), S (symbols).

Common: A (input trends), P (output trends), S (symbols).

Data: AKAM9 (input), AM (output), AS (symbols).

C

Define counting increment, compute radius of counting circle

C and standard deviation

T = T(1) * AF

Call: COUNT(1, P, AM, R)

511 CONTINUE

C Define counting increment, compute radius of counting circle

C and standard deviation

A = 9. * N + 9

S = SORT(A)
R = S(39)
E = S(A)

DO 517 I = 1, N
T = T(1) * AF
F = [9C - P(UNGL1)] - 0.5AF

CALL COUNT(1, P, ADD, R)

517 CONTINUE

C Zero all grid points outside primitive

DO 518 K = 1, 79
DO 518 L = 1, 79

IF (AKAM9(K, L) .GT. CINTI1) GO TO 513

ACON1K, L) = 0

513 CONTINUE

C Compute maximum grid value and replace grid values by

C appropriate shading symbols

DO 533 I = 1, 60
CINTI1 = S16 * 8 + I

533 CONTINUE

C Compress circle into ellipse for printing by removing approx

C every fifth row

DO 520 J = 1, 79
DO 520 K = 1, 79

ACON1K, L) = AKAM9(K, L)

520 CONTINUE

C Insert primitive and titles and print projection

CALL CIRCLE

DO 525 K = 65, 79
ACON1K, 61) = AKAM9(K, L)

525 CONTINUE

DO 526 K = 65, 79
ACON1K, 77) = AS(1)

526 CONTINUE

WRITE(3, 540) CODE, NROT, NV1, NV2, N, AF, SIG, SMAX,

# (ACON1K, L), L = 1, 79

540 FORMAT(15X, 40A1, 13X, N, 5X, 14X, N, 4X, 3X, F, 3X, F)

RETURN
Appendix C

COMPUTER ANALYSIS OF FRICTION TESTS

The various corrections to be made to the results of triaxial friction experiments are detailed in Chapter 7. These do not entail complicated mathematics but rather a good deal of laborious computation. Since a large number of tests was performed, a simple but comprehensive computer program for rapid reduction and plotting of results was written. A variation of this program was used to compute the theoretical curves of Figs. 7.5 and 7.8.

The program was written in FORTRAN IV(G) for the IBM 360/50 computer in the A.N.U. Computer Centre. It consists of a main program which reduces the results, and two plotting subroutines which plot graphs of the variation of friction with displacement on the slip plane and the normal-shear stress relations, respectively. Listings of the programs are appended. Total storage required is 52,700 bytes.

Input data to the main program are points scaled off the load-displacement curve on the testing machine recorder. The coordinates of each point are:

(i) total load registered by machine, including initial thrust on piston;

(ii) axial displacement in terms of tenths of an inch of recorder movement since the beginning of that stage.
These coordinates are punched one pair per card together with the appropriate confining pressure, pore pressure and stage number. This involves a good deal of repetitive input data but is very flexible; a more rigid method with less input data could be used if desired. At the head of each batch of data cards is a master card containing sample description, the angle \( \chi \), specimen diameter and the initial distance between the centres of spherical seats, if applicable. For cases A and B, (Chapter 7), this distance is a positive number; for case C it is negative. If this number is omitted the results are calculated for lateral translation.

**Main program**

The stages in the computation are as follows:

(i) recorder displacement in each stage is converted to actual cumulative axial displacement, and cumulative slip displacement and the angle \( \delta \) are calculated using eqns 7.5, 7.6, 7.27, 7.28;

(ii) true area of contact is calculated using eqns 7.9, 7.10, 7.11;

(iii) differential load, \( R_1 \), is calculated and normal and shear forces on the slip plane calculated from eqns 7.34 and 7.35;

(iv) effective normal and shear stresses are calculated
and the results stored for input to the plotting subroutines;
(v) required data is listed by printer - an example of the output formal is shown in Fig. C.1.
(vi) plotting subroutines are called.

Subroutine FPLOT

This subroutine is called by the main program when all the source data has been processed and listed. Input data to FPLOT are four arrays, EFNORM, SHEAR, SLIP and STAGE, containing the effective normal stress, shear stress, slip displacement and stage symbol for each point on the load displacement curve. Each ratio $\tau/\sigma$ and slip displacement are converted to appropriate symbols in the array GRAPH, all other elements of which are blanks except for the graph axes. GRAPH is then printed. An example of the output is given in Fig. C.2.

Subroutine SPLOT

This subroutine is similar in principle to FPLOT and outputs a graph showing the relation between effective normal stress and shear stress for each point. Even though the line printer spacings are unequal on the two axes, the subroutine adjusts the output format so that the graph has equal scales of 1000 psi per inch on both axes. An example is shown in Fig. C.3. This example is for a type C joint and the stress path leading to the residual line is also plotted.
<table>
<thead>
<tr>
<th>CONF PRESS</th>
<th>PORE PRESS</th>
<th>AXIAL DISP</th>
<th>SLIP DISP</th>
<th>CORR AREA</th>
<th>DELTA DEG</th>
<th>TOTAL LOADS</th>
<th>LOADS INIT</th>
<th>DIFF</th>
<th>FORCES SHEAR</th>
<th>STRESSES SHEAR</th>
<th>EFF NORM STAGF</th>
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</thead>
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<td>0.0</td>
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<td>0.0</td>
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</tr>
<tr>
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<td>0.033</td>
<td>4.553</td>
<td>0.0</td>
<td>41000</td>
<td>21857</td>
<td>18134</td>
<td>13683</td>
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<td>1228</td>
<td>3027</td>
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<td>4.528</td>
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<td>18134</td>
<td>13683</td>
<td>14312</td>
<td>1228</td>
<td>3027</td>
</tr>
<tr>
<td>1420</td>
<td>0.033</td>
<td>0.037</td>
<td>4.505</td>
<td>0.0</td>
<td>41000</td>
<td>21857</td>
<td>18134</td>
<td>13683</td>
<td>14312</td>
<td>1228</td>
<td>3027</td>
</tr>
</tbody>
</table>

Fig.C.1
Fig. C.3
C K.J. OSENGREN FRICITION RESULTS TRANSLATION OR ROTATION

COMMON CODE(100), GRAPH(110, 64), NALPHA
DIMENSION EFFNORM(100), SHEAR(100), STAGE(100), SLIP(100)
DIMENSION CX(20), SYMBOL(20)
DATA SYMBOL/'1','2','3','4','5','6','7','8','9','A','B','C','D',
      'E','F','G','H','J','K','L'/
AK=0.005
P=1.1593
C READ HEADER CARD WITH SPECIMEN DESCRIPTION, ANGLE OF JOINT, DIAMETER OF SPECIMEN, DISTANCE BETWEEN CENTRES OF SPHERICAL SEATS (IF ROTATION) AND
C SET UP HEADINGS
3 READ(1, 1) CODE, NALPHA, D, AL
1 FORMAT(10A4, 12, 10, 0, F 0.0)
IF(NALPHA.GT.90.) GO TO 4
WRITE(3, 10) CODE, NALPHA, D
10 FORMAT(' A', I0A4, 10X, 'ALPHA=', I2, D=*, F5.3, *INS')
IF(AL.EQ.0) GO TO 13
WRITE(3, 14) AL
14 FORMAT('0', 'ROTATION*K=0.005*', 5X,
      '*DISTANCE BETWEEN CENTRES*', F5.2, *INS')
GO TO 16
13 WRITE(3, 15)
15 FORMAT('0', 'TRANSXATION*K=0.005')
16 WRITE(3, 17)
17 FORMAT('0', 'CONF PORX AXI2 SLIP CORR DELTA', 9X, 'LOADS',
      '5X', 'FORCES', 7X, 'STRESSES', 'EFF RATIO STAGE',
      '*', 'PRESS PESSP DISP DISP AREA DEG TOTAL INIT',
      '*DIFF SHEAR NORM SHEAR NORM NORM')
18 CONTINUE
PM=0.
RalphA=NALPHA*PI/180.
R=3(RALPHA)
COS=R COS(RALPHA)
TANA=TAN(RALPHA)
C CALCULATE CUMULATIVE AXIAL DISPLACEMENT, ROTATION OF SPECIMEN,
C SLIP DISPLACEMENT
7 AX=AX/175.
J=0.
CX1(J)=CX1(J)+AX
1 IF(AL.EQ.0) GO TO 18
IF(AL.LT.0) SN=AL*C
IF(AL.GT.0) SN=AL-C
SIN=AK*SINA/C
RBETA=ARSIN(SIN)
RDELTA=RBETA-RALPHA
DELTA=DELTA*180./PI
C K.J.FOSENGREN FRICTION RESULTS TRANSLATION OR ROTATION (CONTINUED)

COSB = COS(RFETA)
SIND = SIN(RDELT)
COSD = COS(RDELT)
SX = AL * SIND / SINB
IF(AL.LT.0) SX = -SX
X = SX * COSA
GO TO 19
18 Y = C
SX = SX / COSA
C CALCULATE CHANGE IN AREA AND COEFFICIENTS IN NORMAL AND SHEAR
C FORCE EQUATIONS
19 Y = X * TANA
COST = Y / D
RTHETA = ARCCOS(COST)
RTHETA 2 = 2 * RTHETA
SINT2 = SIN(RTHETA 2)
SEG1 = RTHETA 2 - SINT2
SDE = SEG1 * D * DP
A1 = SEG1 / (4 * SINB)
A2 = (A - SDE / 4) / SINB
IF(AL.NE.0) GO TO 20
SINB = SINB
COSB = COSB
DELTA = 0
20 R1 = SINB * AK * COSB
B2 = COSB - AK * SINB
C1 = SINB * AK * COSB
C2 = COSB * AK * SINB
C CALCULATE DIFFERENTIAL LOAD, FORCES P1, P2, P3, NORMAL AND SHEAR FORCES.
C NORMAL AND EFFECTIVE NORMAL STRESSES
INIT = 11.9 * NSIG3 + 200
IR1 = LOAD - INIT
P1 = NSIG3 * A
P2 = P1 / TANA
P3 = NSIG3 * A2
FN = R1 * B1 + P1 * C1 + P2 * COSA - P3
FT = R1 * B2 + P1 * C2 - P2 * SINB
SF = FT / A1
SN = FN / A1
SNP = SN - NPORE
C STORE DATA FOR PLOTTING AND PRINT DATA
EFNORM(N) = SNP
SHEAR(N) = ST
SLIP(N) = SX
STAGE(N) = SYMBOL(M)
UM = ST / SNP
IFT = FT
IFN = FN
IST = ST
ISN = SN
ISNP = SNP
WRITE(3, 11) NSIG3, NPORE, C, SX, A1, DELTA, LOAD, INIT, IR1, IFT, IFN,
IST, ISN, ISNP, UM, M
GO TO 5
C CALL PLOTTING SUBROUTINES
5 CALL EBLT(N, EFNORM, SHEAR, STAGE, SLIP)
CALL SPLTIN(EFNORM, SHEAR, STAGE)
GO TO 3
4 STOP
END
C K J. ROSENGREN SUBROUTINE F PLOT

SUBROUTINE F PLOT(N,EFNORM,SHEAR,STAGE,SLIP)
COMMON CODE(10),GRAPH(118,64),N ALPH A
DIMENSION EFNORM(N),SHEAR(N),STAGE(N),SLIP(N)
DIMENSION DIGIT(11),ZERO(N),RATIO(16),DISP(24)
DATA BLANK/' /,VERT/' /,DASH/' /,POINT/' /,
DATA DIGIT/1,2,3,4,5,6,7,8,9,0,11/,DATA ZERO/0,0,0,0,0,0,0,0,0,0,0/,DATA RATIO/P,A,T,I,H,0,1,2,3,4,5,DATA DISP/S,L,D,M,E,N,T,C,H,B,E,S/
C SET UP AXES AND TITLES FOR GRAPH
DO 21 K=1,118
DO 21 L=1,64
GRAPH(K,L)=BLANK
21 CONTINUE
DO 22 I=1,11
L=(I*10)+7
K=L-1
GRAPH(K,2)=ZERO(I)
GRAPH(K,2)=POINT
GRAPH(K,3)=DIGIT(I)
GRAPH(K,3)=VERT
22 CONTINUE
DO 23 I=6,117
GRAPH(I,4)=DASH
23 CONTINUE
GRAPH(7,2)=ZERO(I)
GRAPH(5,4)=ZERO(I)
DO 24 J=1,16
L=(J*10)+7
K=L-1
GRAPH(K,3)=ZERO(I)
GRAPH(K,4)=POINT
GRAPH(K,5)=DIGIT(J)
GRAPH(K,5)=DASH
24 CONTINUE
DO 25 I=52,67
K=L-1
GRAPH(I,1)=RATIO(K)
25 CONTINUE
DO 26 J=23,46
L=J-22
GRAPH(I,1)=DISP(L)
26 CONTINUE
DO 27 J=3,64
GRAPH(I,1)=VERT
27 CONTINUE
C CONVERT DATA POINTS TO ARRAY ELEMENTS AND PRINT GRAPH
DO 29 I=1,N
UM=SHEAR(I)/EFNORM(I)
SX=SLIP(I)
K=(UM*100.+7.5)
L=(SX*100.+4.5)
IF(K.GT.118.OR.L.GT.64) GO TO 31
GRAPH(K,L)=STAGE(I)
GO TO 29
31 WRITE(3,32) EFNORM(I),SHEAR(I),STAGE(I),SLIP(I)
32 FORMAT(11,*,POINT IS OFF SCALE,F8.0,F8.0,3X,A1,F9.3)
29 CONTINUE
WRITE(3,12) CODE,N ALPH A,GRAPH
12 FORMAT(11,*,10A4,*,ALPHA=*,12,/*,18A1))
RETURN
END
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Appendix D

THE MECHANICAL PROPERTIES OF AN INTERLOCKED
LOW-POROSITY AGGREGATE*

by

K.J. ROSENGREN and J.C. JAEGER

SYNOPSIS

If coarse grained marble is heated to around 600°C the anisotropy of thermal expansion of calcite causes almost complete separation at grain boundaries. The resulting material retains its shape, and consists of a mass of crystals in contact, with a porosity of about 4 per cent, very small direct tensile strength, and the mechanical analysis and permeability to water of a sand. It may be regarded as a laboratory model of randomly jointed rock and perhaps of bad and broken rock in general. It has frequently been suggested that soil mechanics theory may be applied to such rock.

The present paper examines the mechanical properties of the heated marble and shows that they are very different from those of soils. A small amount of confining pressure varies the triaxial strength rapidly, the initial slope of

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the Mohr envelope being of the order of 65° and the strength finally increasing to over 80 percent of that of the original rock. Young's modulus also increases with confining pressure but only to about 30 percent of that of the original rock. Even if one principal stress is tensile, a perpendicular compressive stress greatly increases the strength. Model "plate bearing" tests give the same value of Young's modulus as compression of cylinders, suggesting that this may be true for full scale tests on bad rock. Permeability is found to decrease rapidly with confining pressure, and slightly with uniaxial stress. Pronounced effects of size on strength are observed which appear to deviate from the usual power or Protodyakonov laws at small sizes.

INTRODUCTION

Much work in rock mechanics is concerned with the behaviour of poor and closely jointed rock, but very little is known about the mechanical behaviour of such material and in the past attempts have been made to apply soil mechanics theory to it, Morrison and Coates (1955). We have for some time been attempting to produce a material with a similar fabric, but on a smaller scale, whose mechanical properties could be studied in the laboratory. In the course of this, one of us (K.J.R.) noticed that if coarse grained marble is
heated to around 600°C the grains become almost completely separated at the grain boundaries and the resulting material consists of a number of calcite crystals, fitting together at their boundaries, which can be crumbled with the fingers. This effect, which was observed by Lord Rayleigh (1934) and is well known with metallic alloys (Boas and Honeycombe, 1947), is caused by anisotropy of thermal expansion which is particularly great for calcite, being 1.84 percent expansion parallel to the c-axis and 0.22 percent contraction perpendicular to the c-axis over the range 20-600°C, Rosenholtz and Smith (1949). Unfortunately, calcite is the only common rock-forming mineral which possesses this property to a useful degree.

The material obtained in this way, will, for want of a better term, be referred to as "granulated" marble. It has a porosity of about four percent, a negligible uniaxial tensile strength of about 5 psi, and very interesting mechanical properties which will be described in the paper. Its unconfined compressive strength (measured on 5" x 2" cylinders) is of the order of 2300 psi and its strength increases very rapidly with confining pressure to nearly that of the original rock. Its Young's modulus (tangent modulus) also increases with confining pressure, but remains considerably less than that of the original rock. Quite
small compressive components of stress are sufficient to change the properties of the material considerably; as an extreme example, the diametral compression test gives "tensile strengths" of the order of 50 psi. The permeability of a jacketed specimen decreases rapidly with increase of confining pressure and decreases, but much less rapidly, with uniaxial compression in the direction of flow.

It is apparent that the behaviour of this material is intermediate between that of soil and solid rock. If it can be accepted as a model for broken and closely jointed rock, it suggests that the ordinary Coulomb theory of soil mechanics cannot be applied to such rock and that relatively small components of compressive stress greatly enhance its strength. Since the grain boundaries are randomly distributed, the present material cannot be regarded as a model for regularly jointed rock; it may be relevant to material which has been shattered by blasting, and Terzaghi (1962) has remarked that some rocks, such as granite, may show an almost random pattern of jointing of which the microstructure of crystalline rocks provides a model.

Various model experiments may be performed on the material. For example, it will be shown that a model plate bearing test on it gives about the same value for Young's modulus as is obtained in triaxial tests, a result which goes some way
towards justifying the use of the plate bearing test on bad rock underground. It is also, as will appear later, an interesting substance on which to study the effect of size on strength.

**SPECIMEN PREPARATION**

The marble used here is the rather coarse-grained Wombeyan marble studied by Paterson (1958). Specimens were cut from blocks chosen to have a grain size of the order of 1 to 2 mm but the material is rather inhomogeneous and occasional large crystals may affect the results from smaller specimens. In all cases, specimens were cut to the required shape and examined for homogeneity before heating.

A considerable number of preliminary experiments was made to determine the optimum conditions of heating required to obtain the desired effects. Calcite breaks down chemically at elevated temperatures and it was observed that heating above $650^\circ C$ in air caused some breakdown at the surface of specimens. However, it was found that heating to $750^\circ C$ in an atmosphere of $CO_2$ produced no granulation beyond that observed at $600^\circ C$ so the simple method of heating in air to $600^\circ C$ was finally adopted as standard. During heating, the samples initially emitted a continuous 'pinging' sound, presumably caused by tensile failure of the grain boundaries. After a time, this sound ceased,
suggesting that grain boundary separation was complete.

Microscopic examination of the granulated material shows that the grain boundaries are usually reasonably plane and that almost all grain boundaries have opened up (Fig. 1). Effects are shown much better for this coarse grained material than for the much finer grained Carrara marble which retains a tensile strength of about 100 psi after equivalent heat treatment.

The increase of volume, as obtained from the measurement of the dimensions of specimens before and after heating, was 3.5 percent. Also, Paterson (1963) has shown that the porosity of the original marble is of the order of 0.5 percent so that the porosity of the final treated material is about four percent. Although the granulated material may be crumbled with the fingers, specimens retain their shape on heating and no major cracks appear. They do not break up if handled with care even in such processes as insertion into jackets.

Uniaxial tensile strength was measured on "dog-bone" specimens with metal contacts attached to their ends with araldite. Load was applied by slowly pouring lead shot into a pan. The average uniaxial tensile strength was 5 psi. Fractures were entirely along grain boundaries, showing the ends of a number of calcite grains.
(a) Photomicrograph showing microstructure of granulated marble. Width of field 10 mm. (crossed nicols).

(b) Enlargement of part of (a) showing detail of open grain boundaries. Width of field 2.5 mm. (plane polarised light).

Fig. 1
A particle size distribution curve determined by sieving the crumbled material is shown in Fig. 2. On the basis of particle size, the aggregate would be described as a coarse sand but with the major difference that the porosity is extremely small.

**UNCONFINED COMPRESSION**

This was measured on cylinders 5" long and 2" in diameter for comparison with the triaxial results given in the next section. Its mean value was 2300 psi so that the ratio of the uniaxial compressive to tensile strength is of the order of 400. Failure was by bulging followed by gradual crumbling at the outer surface. A typical stress-strain curve is shown in Fig. 3 which is a complete stress-strain curve in which the material still holds considerable load in the descending portion of the curve.

Axial strain was measured from the displacement of the platens, but, because of the nature of the material it was difficult to measure lateral displacement or Poisson's ratio. Following Barnard (1964) circumferential strains were measured by wires wrapped around the circumference and attached to dial gauges. Strains were measured at three points, Fig. 3 (inset). The gauges began to record small strains at about 70 percent of the maximum load and there was a rapid increase at around 90 percent of it. The
Fig. 2. Particle size distribution for "granulated" marble

Fig. 3. Axial and circumferential strains in unconfined compression. Positions of circumferential gauges are shown inset. Curve I: axial strain. Curve II: circumferential strain, gauge B. Curve III: circumferential strain, mean of gauges A and C.
circumferential strains are shown by the dotted lines in Fig. 3. In this case the centre gauge recorded less strain than the other two; in other tests the gauges behave in a different manner depending on the pattern of bulging near failure. The behaviour is very similar to that of concrete as described by Barnard (1964) but both axial and circumferential strains are much larger.

TRIAXIAL COMPRESSION

Triaxial tests were made on cylinders 5" long and 2" in diameter with rubber jackets. From the nature of the material it was impossible to attach strain gauges to it, so strain in the specimen was measured from the relative displacement of the platens with a correction for distortion in the apparatus.

A typical testing machine recorder curve showing the effects of cycling the load is shown in Fig. 4 for a confining pressure of 415 psi. This is a complete load-displacement curve showing a continuous transition to a "residual strength" of 3600 psi, corresponding to sliding across a single shear plane of failure inclined at 31° to the axis.

Stress-strain curves for a number of different confining pressures are shown in Fig. 5. From these, the maximum differential stress attained, \( \sigma_{\text{max}} \), and the tangent modulus
have been determined and are given in Table I. In most of
the curves of Fig. 5, testing was stopped before a residual
stress was attained so that the specimens could be examined.

The variation of $\sigma_{\text{max}}$ with the confining pressure $\sigma_3$
is shown in Fig. 6 together with an average of a large number
of measurements which have been made on various samples of
solid Wombeyan marble. It appears that there is a very
rapid increase of $\sigma_{\text{max}}$ with confining pressure for
$0 < \sigma_3 < 500$ psi while for $\sigma_3 > 1000$ psi, $\sigma_{\text{max}}$
is greater
than three quarters of the value for the original marble.
The same effect is shown by the portion of the Mohr envelope,
Fig. 7, corresponding to low values of $\sigma_3$, which has a
slope of around $65^\circ$. This confirms the speculations of
Terzaghi (1962) on the $\theta$ value of randomly jointed rock
masses in connection with the stability of steep slopes.
The whole Mohr envelope is shown in Fig. 7 and shows a very
rapid decrease of $\theta$ from the steep initial slope. The
flattening at the higher values of $\sigma$ is associated with
the onset of ductile behaviour.

The types of failure are very similar to those of the
original marble, (Paterson, 1958) being shear fractures on
a single plane for $\sigma_3 < 2000$ psi grading into a pattern of
conjugate shears for greater values of $\sigma_3$. The planes make
angles of $22^\circ$ to $31^\circ$ with the axis.
Fig. 6. Curve I: Maximum differential stress attained, $\Delta \sigma_{\text{max}}$, plotted against confining pressure $\sigma_3$ for "granulated" marble, Curve II: Corresponding average curve for the original marble.

Fig. 7. Mohr envelope for "granulated" marble.
Single shear planes appeared at confining pressures as low as 20 psi although it was not possible to measure their inclination because of disintegration of the samples when handled.

The variation of the tangent modulus with confining pressure is given in Table I and its value at various fractions of $\sigma_{\text{max}}$ is plotted in Fig. 8 as a function of confining pressure $\sigma_3$. It appears that, while the tangent modulus increases substantially with confining pressure, this increase is not so marked as that of strength and it always remains less than one third of that of the original marble which has a value of $10^7$ psi.

This general behaviour is similar in kind, though very different in degree, to that described by Walsh (1965) for solid material containing cracks. For uniaxial compression he derives the formula

$$\frac{E_f^{-1} - E_g^{-1}}{E_0^{-1} - E_g^{-1}} = \frac{1}{5} \left[ \frac{2+3\mu^2+2\mu^4}{(1+\mu^2)^{3/2}} - 2\mu \right] \quad (1)$$

where $E_0$ is the initial tangent modulus, $E_f$ is the tangent modulus when the stress-strain curve has become linear and $E_g$ is the unloading modulus from this region, and $\mu$ is the coefficient of friction across the crack surfaces. His theory is not strictly applicable to the
present case, but it is interesting to see whether there is any serious discrepancy. Taking the initial state as that of hydrostatic pressure of 415 psi, Fig. 4, it follows from this curve that \( \frac{E_g}{E_f} \) and \( \frac{E_o}{E_f} \approx 0.25 \) approximately. It follows from (1) that \( \mu = 0.4 \) which is of the expected order of magnitude.

**OTHER SYSTEMS OF STRESS**

A number of other tests was made to indicate the response of the material to other systems of stress and, in particular, inhomogeneous stresses.

Diametral compression (Brazilian) tests were made on disks 1" thick and 2" in diameter compressed between flat platens. In this case, if \( W \) is the load per unit length on a cylinder of radius \( R \), there is, on the theory of elasticity, a uniform tensile stress \( \frac{W}{\pi R} \) across the loaded diameter and a compressive stress along this diameter increasing from \( \frac{3W}{\pi R} \) at the centre.

The behaviour of the specimens was characteristic and reproducible; at a load \( W \) of around 130 lb a crack could be observed at the centre of the specimen which as \( W \) was increased would gradually extend (following grain boundaries) along the diameter and a maximum load of about 180 lb would be attained when the crack had reached the platens. Clearly, the compressive stresses in the disk
assists it to support greater loads. The "tensile strength" calculated from the usual formula would be 40 psi at first appearance of the crack or about 60 psi at maximum load.

This effect is accentuated in the three-point Brazilian test (Jaeger, 1967) in which load \( W \) per unit length is applied to a disk supported on two planes at an angle of 120° to one another. In this case, the load \( W \) at which cracking appeared was 400 lb. or about three times that in the conventional Brazilian test. This ratio is much greater than that (approximately unity) for the original marble, indicating, again, strengthening of the material by the increased compressive stresses.

**AXIAL LOADING OF HOLLOW CYLINDERS WITH EXTERNAL CONFINING PRESSURE**

This provides a simple method of studying the effect of the intermediate principal stress under inhomogeneous stress conditions. If \( p_a \) is the axial stress, \( p \) is the confining pressure applied at the external surface, and \( \rho \) is the ratio of the internal to external diameter of the cylinder, the principal stresses on the inner surface (where the stress difference is greatest) are, on elastic theory, \( p_a, \frac{2p}{1-\rho^2}, 0 \).

Experiments were made on cylinders 5" long, 2" external diameter and 1" internal diameter with rubber jackets. With
p = 400 psi, the stresses at failure were 5600, 1070, 0, psi, while with p = 1000 psi they were 8500, 2670, 0, respectively. In the first case, failure was by a plane shear fracture, and in the second the fracture was conical. These results show a very considerable increase of strength with increase of the intermediate principal stress \( \sigma_2 \), the minor principal stress remaining zero. They are very similar to those for solid rocks described by Jaeger and Hoskins (1966).

It is also possible to collapse hollow cylinders by sealing off their ends, jacketing them, and exposing them to external hydrostatic pressure as was done by Robertson (1955). Cylinders of "granulated" marble of the dimensions given above collapsed at an average value of the external pressure of 9300 psi, while similar cylinders of solid marble collapsed at an external pressure of 15600 psi. At these stresses the marble becomes ductile so the theory is more complicated but the results again show the great increase of strength caused by confining pressure.

In general, these observations show how strong interlocking granular material may be in a situation of some practical significance, namely the region outside a circular hole subjected to external compressive stress.

**INDENTATION TESTS**

The plate bearing test which is widely used to give a
The modulus of rock in situ can be determined by forcing a circular disk of radius $R$ into the rock by force $F$, measuring the average displacement $u$ over the disk, and calculating Young's modulus $E$ from the formula of the theory of elasticity, Timoshenko and Goodier (1951):

$$E = 0.54(1 - \nu^2)F/\mu,$$  \hspace{1cm} (2)

where $\nu$ is Poisson's ratio. This method has the advantage of being simple to use and always giving reproducible results, even on bad rock, but, since it is based on elastic theory it is not clear what the results mean. Also, it is difficult or impossible to compare the result with laboratory tests.

Indentation of granulated marble provides a reasonable laboratory model of this situation. A plane circular indenter 1" in diameter was forced into a six inch cube of the material which was lightly constrained at its free surfaces by being potted in a steel box with paraffin wax.

A typical load-displacement curve is shown in Fig. 9, Curve I. This shows the effect of cycling the load and bears a considerable resemblance to the corresponding curves for large scale plate-bearing tests. The final failure corresponds to splitting of the block and has no significance. The maximum value of the tangent modulus from Curve I and the formula (2) with $\nu = 0.2$ is $0.38 \times 10^6$ psi which is exactly that obtained for unconfined compression, Table 1.
It appears that the modulus of elasticity deduced from these miniature plate bearing tests agrees well with that determined from uniaxial compression, and this suggests that the same may be true for large scale tests on randomly jointed material.

In an attempt to study the effect of stress on the modulus, a six inch cube, confined as before, was compressed uniaxially with a stress of 1000 psi and a cylinder 1" in diameter was pushed through the compressing platen into the cube. This corresponds to an indentation or plate bearing test with an additional uniaxial stress. The load-displacement curve in this case is shown in Fig. 9, Curve II. The main difference from Curve I is that the large displacements at low loads disappear, but the maximum value of the tangent modulus attained is only very slightly increased. This implies that the principal effect of uniaxial stress is in closing cracks in the perpendicular direction, but that other cracks are involved in the increase of modulus under triaxial conditions shown in Table 1.

To check this point, some indentation tests under hydrostatic pressure were made with a plane indenter 0.5" in diameter on a 2" x 2" cylinder in the apparatus previously used for punching tests under hydrostatic pressure, Jaeger (1962). At confining pressures of 100, 200, 400, 1000 psi
the maximum tangent moduli were, respectively, 0.37, 0.71, 0.95, 1.48 x 10^6 psi. This shows an increase comparable with that of Table 1 and reasonable agreement, having regard to the small size of the present specimens.

**SIZE EFFECTS**

Recently, a great deal of attention has been paid to the variation of specimen strength with size. Evans and Pomeroy (1958) reported a variation of crushing stress $\sigma_a$ of coal cubes with side length $a$ of the form

$$\sigma_a = Ca^\alpha$$

where $C$ and $\alpha$ are constants and $\alpha$, from their experiments, is of the order of 0.3. Other workers have found a similar law with other values of $\alpha$ up to 0.5. These results are in agreement with the theory of Weibull (1939) and are ascribed to the influence of flaws in the material.

An alternative approach due to Protodyakonov (1964) relates the crushing strength $\sigma_a$ to the ratio $k = a/b$ of the side length $a$ to the average spacing $b$ between discontinuities in the material. His formula is

$$\frac{\sigma_a}{\sigma_m} = \frac{m + k}{1 + k}$$

where $\sigma_m$ is the crushing strength of the rock in bulk, $k \to \infty$, and $m$ is a constant. The crushing strength of a single element of the rock, $k = 1$, is $\frac{1}{2}(m+1)\sigma_m$. 
The explanation of these results involves the assumption of flaws or discontinuities in the rock, sometimes identified with Griffith cracks. "Granulated" marble provides an interesting material for studying size effects since in it the nature and spacing of the discontinuities is known.

An adequate study would require very extensive sampling and careful statistical control and is complicated by the considerable variation in grain size of the marble from block to block. The material remaining from the block used in the other experiments was cut into thirty 1" cubes, twelve 2" cubes, two 4" cubes and one 6" cube, and because of this small number the results are regarded as suggestive only. A further difficulty arose from the fact that the 1" cubes showed a tendency on heating to lose an occasional grain at their edges and a number were rejected for this reason.

The mean values $\sigma_a$ for crushing strength of cubes of side $a$ were found to be as follows

<table>
<thead>
<tr>
<th>$a$ (in)</th>
<th>1</th>
<th>2</th>
<th>4</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_a$ (psi)</td>
<td>4400</td>
<td>4500</td>
<td>3560</td>
<td>3100</td>
</tr>
</tbody>
</table>

Values of $\log_{10} \sigma_a$ plotted against $\log_{10} a$ are shown in Fig. 10. These values are all much higher than that (2300 psi) found for 5" x 2" cylinders, giving another
manifestation of the effects of size and shape.

Ignoring the result for \( a = 1\)", the results fit the law (3) reasonably well with \( \alpha = 0.3\). Alternatively, the dotted curve in Fig. 10 is calculated from the formula (4) with \( b = 0.067"\) and the results for \( a = 2"\) and \( a = 4"\). In both cases, the measured value for \( a = 1"\) is much too low. This may be due to the difficulty of working with 1" cubes mentioned above. However, no specimen gave a value of \( \sigma_a \) greater than 5000 psi and it seems likely that this is a genuine effect and that for specimens with a continuous network of discontinuities, the strength decreases for small values of the ratio \( k \) of the side length to the spacing of discontinuities.

**PERMEABILITY**

It is of interest to study the effect of confining pressure on the permeability of this material. Brace, Orange and Madden (1965) found that the electrical conductivity of water-saturated crystalline rocks decreased with confining pressure. This they attributed to the closing of cracks; the permeability in these cases would be too low to measure without great difficulty. Pomeroy and Robinson (1967) have studied the effect of applied stress on coal; in their experiments the coal was oriented so that water was forced in over bedding planes and external stresses were applied perpendicular
to the two sets of primary cleats.

For "granulated" marble specimens in the standard form of cylinders 5" long and 2" in diameter were used. They were placed in a rubber jacket with a porous disk at each end through which water could be introduced. They were initially saturated with water by soaking for several hours in a vacuum.

The permeability without confining pressure was measured using the falling-head permeameter method and was found to be $10^{-2}$ cm/sec. It is possible that water movement between the jacket and the specimen contributed to this relatively high value but this seems unlikely since the jacket fitted very snugly.

To measure the permeability under confining pressure the jacketed specimen was put into the triaxial pot which had provision for supplying water at constant pressure at one end of the specimen and measuring the outflow from the other end. The confining pressure $\sigma_3$ has to be greater than the inlet water pressure $p$ and it should be remarked that the effective pressure varies from $\sigma_3 - p$ at one end of the specimen to $\sigma_3$ at the other end so that conditions for the closing of cracks vary along the specimen.

Curve I of Fig. 11 shows the variation of permeability with confining pressure $\sigma_1 = \sigma_2 = \sigma_3$ for an inlet pressure $p$ of 300 psi. This shows clearly the large decrease of
permeability caused by closing of cracks by hydrostatic pressure. Since the differential stress might be expected preferentially to close cracks normal to it while restricting flow in its own direction much less, a number of runs under triaxial conditions were made. Fig. 11, Curve II, corresponds to $p = 300$ psi, $\sigma_2 = \sigma_3 = 800$ psi with $\sigma_1$ increased steadily, and Curve III to $p = 300$, $\sigma_2 = \sigma_3 = 1100$. It appears that the effect of increasing $\sigma_1$ alone on inhibiting flow is relatively small. It might, indeed, be expected that the permeability would increase as $\sigma_1$ approaches the strength $\sigma_{\text{max}}$, corresponding to the rapid increase in circumferential strain in Fig. 3. This effect has not been observed, probably because the experimental arrangements were not sufficiently sensitive.

REFERENCES


A MEDIUM-SCALE DIRECT FRICTION EXPERIMENT

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(Received 1 August 1967)

Abstract—Studies of the friction between rock surfaces 12 in. square have been made by sliding a block with plane parallel surfaces between two others in a testing machine. The normal stress was applied by flat jacks. The frictional behaviour of the materials was found to be very greatly influenced by the nature of the surfaces. With moderately rough surfaces the tangential force rose slowly with the relative displacement of the surfaces. With finely ground surfaces, regular stick-slip relaxation oscillations occurred whose amplitude was determined by the coefficients of static and dynamic friction and the stiffness of the testing machine. Such oscillations could be produced or inhibited by decreasing or increasing the roughness of the surface.

1. INTRODUCTION

Most laboratory measurements of the coefficient of sliding friction of rock surfaces have been made by slicing cylinders at various angles to their axes and stressing the resulting specimen in a triaxial pot. Jaeger [1] studied artificial joints filled with plaster, experimentally produced shear fractures, and roughly ground saw-cut joints in this way. Lane and Heck [2] made similar results on natural and grouted rock. Brace and Byerlee [3] have given a preliminary report of similar experiments on sawn surfaces and surfaces of shear fracture.

While this method is very convenient and allows high normal stresses to be attained it is unsatisfactory for two reasons. Firstly, it is only applicable to surfaces of small area and its results may not be applicable to larger surfaces in which macro-contact may occur only at widely spaced intervals. Secondly, the geometry and stress-system change as sliding proceeds so that the results are only strictly accurate for the initiation of sliding. The situation may be seen from Fig. 1. Without spherical seats; Fig. 1(a), large and unknown stresses depending on the lateral stiffness of the machine are imposed when sliding begins. With one spherical seat, Fig. 1(b), slip rotates the system into the situation of Fig. 1(c). With two spherical seats, Fig. 1(d), the geometry is preserved but lateral stresses are imposed. It appears that none of these systems is suited to the study of the variation of the frictional properties of the surface with displacement, which may be described as 'running in', and that, while a few measurements at different confining pressures may be made with a single specimen for purposes of establishing the variation of shear stress across the joint with normal stress, this number should be kept as small as possible. Jaeger [1], Lane and Heck [2] and Murrell [4] all use the system of Fig. 1(b), usually increasing the load necessary to produce slip at several confining pressures. Jaeger [1] showed that after extensive slip (perhaps 0.5 cm for a specimen 5 cm in diameter) the stress-distribution between the surfaces had changed greatly and that the specimen would ultimately fail along a number of fracture planes intersecting the original surface of sliding.
Appendix F

DIRECT FRIC TION EXPERIMENTS

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Studies of the friction between rock surfaces have been made by sliding some parallel surfaces between two others in a testing machine. The normal stress on the faces of the joint and the oscillatory displacement of the surfaces were determined by the coefficients of friction and the stiffness of the machine. Such oscillations could be caused by decreasing or increasing the normal stress on the surface.

1. INTRODUCTION.

Most laboratory measurements of the coefficient of sliding friction of rock surfaces have been made by slicing cylinders at various angles to their axes and stressing the resulting specimen between two axial ports. JAEGER [1] studied artificial joints filled with plaster, experimentally obtained fractures, and roughly-ground saw-cut joints in this way. BLAKE and HECK [2] used joints in natural and ground rock. BRACE and BYERS [3] have given a preliminary account of similar experiments on rough surfaces and stressed fractures.

While this method is very convenient and shows the normal stress at the attained it is satisfactory for two reasons. Firstly, it can be applied to surfaces of small area and its succeeding geometry presents an apparatus which can be applied to larger areas in the macro-contact. Secondly, the stresses and the system change as the machine proceeds thence only strictly account of the friction of sliding. The situation may be analogy that without spherical seats, as in Fig. 1(a), large and unknown stresses depending on the geometry of the machine are imposed when sliding begins. With one spherical seat, slip rotates the system into the situation of Fig. 1(c). With two spherical seats, the geometry is preserved but lateral stresses are imposed. It appears that none of these systems is suited to the study of the variation of the frictional properties of the surface with displacement, which may be described as 'running in', and that, while a few measurements of different confining pressures may be made with a single specimen for purposes of obtaining the variation of shear stress across the joint with normal stress, this number must be kept as small as possible. JAEGER [1], BLAKE and HECK [2] and MURRELL [4] all used the system of Fig. 1(b), usually increasing the load to produce slip at several confining pressures. JAEGER [1] showed that after extensive pre-loading (0.5 cm for a specimen 1 cm in diameter) the stress-distribution between the joint faces changed greatly and that the specimen would ultimately fail along a number of fractures planes intersecting the original surface of slides.
To avoid these difficulties and, in particular, to allow considerable displacement under the same conditions, RAE [5] has used a slider on the curved surface of a rotating cylinder and COOK [6] has used sliding of a hollow cylinder on the plane end of a rotating cylinder. The present experiments use the simplest system of all, namely that of forcing a block of rock with plane and parallel surfaces between two other blocks as in Fig. 2.

This experiment can be performed on any scale and the main purpose of the present work was to use fairly large surfaces, of the order of a foot square, which might be more nearly comparable with natural joint surfaces than the smaller ones previously used. Since the cost of preparation of each specimen is fairly high, only a limited number could be used.

A second object of the work was to study the effect of surface roughness on frictional behaviour. JAEGE [1] reported differences in the behaviour of sliding rock surfaces that appeared to be related to the roughness of the surfaces: sawn and ground planar surfaces exhibited stick-slip phenomena, whilst artificially filled joint surfaces and natural fracture surfaces did not. BRACE and BYERLEE [3] have reported stick-slip behaviour on both sawn and naturally fractured surfaces. It seemed desirable to study this behaviour under carefully controlled conditions.

2. APPARATUS AND MATERIALS

The apparatus is shown in Fig. 2. A centre specimen C, 12 in. square is pushed between two outer specimens A, B of the same materials 12 in. x 10.5 in. supported by packers P on the lower platen of a testing machine. This arrangement allows 1.5 in. travel of the centre specimen with unchanged contact area, and this could easily be increased if necessary. Normal load is applied to the specimens by a pair of flat jacks F, F held in position by two box girders E, E connected by heavy bolts. The displacement of the specimen C is measured both by the machine recorder and by a linear variable differential transformer (LVDT). The load
A MEDICAL DIRECT FRICTION EXPERIMENT

L on the specimen is measured both by the machine chart recorder and by an LVDT attached to this recorder. This latter arrangement, though very simple, is not altogether satisfactory since it includes a component associated with the pendulum movement of the testing machine (Avery 500 ton hydraulic) but this is easily recognized. The LVDT outputs are fed to the axes of a Houston Omnigraph X Y recorder which permits almost any desired magnification of the load-displacement curves on the testing machine recorder. The flat jacks were calibrated by loading them in the testing machine under conditions in which the distance between their surfaces was maintained constant. The normal load on the specimens is obtained from this calibration curve and is of the order of 95 per cent of the hydraulic pressure in the jacks.

The materials used were commercial building stones with the following descriptions:

'Red Granite.' A coarse-grained igneous rock consisting of 50 per cent orthoclase, 40 per cent quartz and 10 per cent accessories with grain size ranging from a few tenths of 1 mm to 2-3 cm.

'Gabbro.' A medium-grained igneous rock consisting of 60 per cent plagioclase, 30 per cent pyroxene and 10 per cent accessories with an average grain size of 0.5 to 2 mm.

'Trachyte.' An even-grained isotropic igneous rock consisting of 80 per cent orthoclase with minor quartz and slightly altered ferromagnesian minerals. Grain size averages about 1 mm.

'Sandstone.' Gosford sandstone. A uniform, weakly cemented quartz sandstone. Grain size is approximately 1 mm.

'Carrara marble'. An essentially pure calcite marble with a grain size of approximately 0.2 mm.

'Womheyan marble.' A marble of very irregular grain size varying between 2 and 10 mm. The surface of this material was not flat to the tolerance described below.

Two experiments were made with wet surfaces and one with mixed surfaces, i.e. sandstone on trachyte.

The final macroscopic area of contact between the surfaces, that is, the area of surface damaged by sliding was approximately 95 per cent of the total area except for the two marbles for which it was much less.

It became apparent on commencing the experiments that, for a given lateral load, the frictional force varied considerably with the amount of 'running in' that the surfaces had had. For this reason a standard order of experiment was adopted. All experiments were begun with new surfaces at a normal stress of 500 psi and a considerable displacement was made under these conditions. The tangential load was released before making any change in the normal loads. In some cases it was found that normal stresses greater than 1000 psi caused cracking of the specimens so greater stresses than this were not used. All runs were made at a standard rate of platen advance of approximately $\frac{1}{2}$ in/hr.

3. SURFACE FINISH

The standard machine shop terminology and techniques for the designation of surface quality are used.

The term 'roughness' refers only to relatively closely spaced surface irregularities such as those produced by sawing, grinding and lapping. Roughness may be specified by either direct measurements of the height of these irregularities or by their average or root-mean-square deviation from the 'mean surface'. The mean surface is the surface located so that the volume of peaks above it is equal to the volume of valleys below. The 'arithmetic average
deviation' of the surface is the result of taking a great many uniformly spaced measurements of the distance of the actual surface from the mean surface and averaging them. It is expressed in micro-inches. 'Flatness' refers to the larger scale warping in the mean surface over a relatively much larger area and is a completely separate measurement from roughness.

The blocks as received from the suppliers were specified to have surfaces flat and parallel to $\frac{1}{16}$ in. and were sometimes improved by reworking by ourselves.

The instrument used to measure surface 'roughness' was a Brush Surfindicator (Model 185). This instrument has a probe with a diamond tipped stylus which is traversed over the specimen. Vertical movement of the stylus is measured, averaged and displayed on a meter directly as the 'arithmetic average deviation' of the surface. Standard surfaces are furnished with the instrument and it is calibrated on these standards before each measurement. The lower limit of measurement is about $5 \mu$in. so we are not measuring the very small irregularities of the order of a few hundred Angstroms reported by Bowden and Tabor [8] which were observed by optical and electron microscope techniques. There are many other aspects of surface quality that are also not measured with a machine of this type such as the accurate shape of the irregularities or minor periodicities. It does give a semi-quantitative means of comparing surfaces, however, which was its purpose in these experiments.

The blocks designated 'rough' in Table I were far too rough for the Surfindicator to be used without damage to its stylus. Irregularities of the order of $\frac{1}{32}$ in. were present on the otherwise 'flat' surfaces.

The behaviour of the various rocks divided sharply into two types apparently determined by surface roughness:

(i) A steady and smooth increase of frictional force asymptotically towards a final value determined by the normal stress.

(ii) Regular 'stick-slip' relaxation oscillations.

4. ROUGH SURFACES

The characteristic behaviour for moderately rough surfaces is shown in Fig. 3 which shows the load–displacement curve for trachyte with a surface roughness of $350 \mu$in.

The fresh rock surfaces are brought together under a jack pressure of 500 psi and the system loaded. The load at first increases rapidly with displacement and subsequently at a steadily decreasing rate, Fig. 3, curve (i). Part of this increase, of course, may be due to the fact that fresh surface is continually being moved into the rubbing area. After considerable displacement in curve (i) so that the surfaces may be regarded as well 'run-in' the tangential load is released and the normal load changed so that the same portions of the surface are in contact under a different normal load. The experiment is then repeated, Fig. 3, curves (ii)–(viii). There may be small variations in behaviour between different materials at this stage.

In some cases, there is a very slight peak in the load which can just be seen, e.g. in Fig. 3, curves (vi), (vii), (viii) followed by an approximately constant value: in others, the load rises slightly with displacement.

The suggestion from the curves of Fig. 3 is that the tangential stress between sliding surfaces varies with the amount of 'running in' that these surfaces have had and that ultimately a surface characteristic of the material, and perhaps of the normal stress, is established. It is the tangential stress across this surface, that is, the value to which the curves of Fig. 3 are asymptotic which is given in the Tables below. Some of the surfaces on which sliding has taken place are discussed in Section 9: it appears that they are slickensided and contain detrital material. These considerations make it very difficult to define and measure friction...
between rock surfaces. Measurements during the early stages of sliding correspond to removing some of the major protuberances and so are not representative; in the later stages the surfaces will include fractured crystals, slickensides and detrital material. However, it is probably such surfaces that are in question in rock mechanics.

Plotting the values of the tangential stress $\tau$ taken from Fig. 3, curves (ii)-(viii) against the normal stress $\sigma$ gives a very good straight line of the form

$$\tau = c + \mu \sigma$$

where $\mu$ is the coefficient of friction and $c$ may be called the cohesion.

Plots of $\tau$ against $\sigma$ for various materials are given in Fig. 4 and the values of $\mu$ and $c$ for a number of surfaces behaving in this way are given in Table 1.

---

**Fig. 3.** Testing machine recorder load–displacement curves for rough Trachyte. Flat jack pressures (i) 500 psi; (ii) 250 psi; (iii) 125 psi; (iv) 230 psi; (v) 375 psi; (vi) 500 psi; (vii) 625 psi; (viii) 750 psi.

**Fig. 4.** Typical shear stress–normal stress relations for surfaces which do not show stick–slip. A, Wombeyan marble; B, Trachyte rough; C, Trachyte 350 $\mu$m; D, sandstone.
TABLE I. COEFFICIENT OF FRICTION \( \mu \) AND COHESION \( c \) FOR ROCK SURFACES WHICH GIVE REGULAR SLIDING

<table>
<thead>
<tr>
<th>Rock</th>
<th>Surface roughness (( \mu ))</th>
<th>( \mu ) (psi)</th>
<th>( c ) (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Red Granite</td>
<td>180 ± 20</td>
<td>0.64</td>
<td>45</td>
</tr>
<tr>
<td>Gabbro Rough</td>
<td></td>
<td>0.66</td>
<td>55</td>
</tr>
<tr>
<td>Trachyte</td>
<td>300–350</td>
<td>0.58</td>
<td>65</td>
</tr>
<tr>
<td>Trachyte Rough</td>
<td></td>
<td>0.68</td>
<td>60</td>
</tr>
<tr>
<td>Wet trachyte</td>
<td>300–350</td>
<td>0.56</td>
<td>60</td>
</tr>
<tr>
<td>Sandstone Rough</td>
<td></td>
<td>0.51</td>
<td>40</td>
</tr>
<tr>
<td>Wet sandstone Rough</td>
<td></td>
<td>0.61</td>
<td>45</td>
</tr>
<tr>
<td>Sandstone on trachyte Rough</td>
<td></td>
<td>0.57</td>
<td>40</td>
</tr>
<tr>
<td>Wombeyan marble</td>
<td>200</td>
<td>0.75</td>
<td>160</td>
</tr>
</tbody>
</table>

It appears that in all cases, except the Wombeyan marble, the coefficient of friction lies in the range 0.5–0.7 and that there is a small cohesion of the order of 50 psi.

For Wombeyan marble both \( \mu \) and \( c \) are relatively high. This is probably due in part to the fact that for the block used contact was only over about 0.25 of the area so that the normal stresses would be sufficient to bring the material into the ductile state. There was great surface damage to the material and fracture extended some distance into the material below the area of contact.

5. THE EXPRESSION FOR THE LAW OF FRICTION

The most natural extension of Amonton’s law is the linear law (1) used to interpret the preceding experiments. It has the advantage of leading to simple mathematical formulae and of being in line with the simple Coulomb theory of soil and rock mechanics. This representation was used by JÄGER [1] and LANE and HECK [2] and is seen to represent the present results adequately.

ARCHARD [7] has provided theoretical justification for a power law

\[
\tau = \mu \sigma^n
\]

where \( \mu \) is a constant and \( 2/3 < n < 1 \). MURRELL [4] interprets his results on sandstone in this way obtaining the result

\[
\tau = 2.1 \sigma^{0.89}
\]

but in fact his results for \( \sigma < 20,000 \) psi may be equally well represented by

\[
\tau = 500 + 0.70 \sigma
\]

and his results for \( \sigma > 20,000 \) psi are rather scattered and also in a region in which the material may be ductile. Other writers, e.g. BOWDEN and TABOR [8] RALEIGH and PATerson [9] and MAURER [10] determine \( \mu \) from \( \mu = \tau/\sigma \) in each experiment and plot \( \mu \) against \( \sigma \) obtaining values of \( \mu \) which decrease rapidly with increasing \( \sigma \). This would be the case if \( \tau \) and \( \sigma \) were connected by either of the laws (1) or (2). MAURER [10] has performed a double-shear experiment on rods of cross section 1 \( \times \) 0.5 in. and has continued sliding after shear failure under conditions rather similar to those of the present experiments. His results seem to fit the formula (1) reasonably well.

The present experiments indicate that the linear law (1) is applicable for the relatively low normal stresses used.
6. SMOOTHER SURFACES

The typical behaviour is shown in Fig. 5 for the same blocks of Red Granite as those described in Table 1 but with the surface lapped to a roughness of $35 \pm 5 \mu\text{in}$. After an initial rise in load a sudden slip occurs and stick-slip proceeds regularly with increasing amplitude, as in Fig. 5 (i) taken from the machine recorder which is shown on a larger scale.

![Graph](image1)

---

**Fig. 5.** Testing machine recorder load displacement curves for smooth Red Granite with surface roughness $35 \pm 5 \mu\text{in}$. Flat-jack pressures: (i), (v) 515 psi; (ii) 250 psi; (iii) 750 psi; (iv) 125 psi; (v) 250 psi; (vi) 375 psi; (vii) 500 psi; (viii) 625 psi.

---

In Fig. 6 taken from the X-Y recorder fed by the LVDTs. In Fig. 6 the lower portion of the oscillations (below the dotted curve PQ) is caused by motion of the testing machine pendulum, and the actual amplitude of the oscillation is the distance between PQ and the upper part of the curve.

![Graph](image2)

---

**Fig. 6.** The oscillations of Fig. 5(i) on a magnified scale.

---

As before, the normal stress $\sigma$ on the joint can be varied as in Fig. 5(ii)–(viii) and the tangential stress $\tau$ at slip plotted against $\sigma$ which gives a straight line corresponding to a coefficient of friction $\mu$ and cohesion $c$. Similarly $\tau$ at the lower limit PQ can be plotted giving values $\mu^*$ and $c^*$. Plots of this type are shown in Fig. 7 and numerical values in Table 2. As before, the values of $\tau$ used are the asymptotic values reached after a considerable surface displacement.

Comparison of the results in Tables 1 and 2 indicate surprising variability for the same rock with different surface finishes. For the trachyte much the same values are obtained. On the other hand, the gabbro shows a wide variation in $\mu$ from 0·18 to 0·66. The first experiment on gabbro with a roughness of $35 \mu\text{in}$ gave $\mu = 0·32$. This block was apparently not
sufficiently flat and widespread cracking occurred during the test. The experiment was therefore repeated on another set of surfaces lapped together to a roughness of 50 µin. and this gave the even lower value of $\mu = 0.18$. The blocks did not crack during this test. It is possible that the minerals in contact between the gabbro surfaces have lower coefficients of friction than those on the other rocks tested but we can offer no explanation for the wide variation in friction values.

The fact that stick-slip behaviour can be inhibited by reworking the surface to a different roughness suggests that it might be possible to observe a transition in behaviour. An example

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**Table 2. Coefficient of Friction and Cohesion for Rock Surfaces Exhibiting Stick-Slip Behaviour**

<table>
<thead>
<tr>
<th>Rock</th>
<th>Surface roughness (µin)</th>
<th>$\mu$</th>
<th>$\mu^*$</th>
<th>$\mu'$</th>
<th>$c$</th>
<th>$c^*$</th>
<th>$(F-F^*)/X$</th>
<th>$\text{lb/in}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Red Granite</td>
<td>35 ± 5</td>
<td>0.53</td>
<td>0.31</td>
<td>0.42</td>
<td>40</td>
<td>40</td>
<td>5.1 x 10^4</td>
<td></td>
</tr>
<tr>
<td>Red Granite</td>
<td>80 ± 20</td>
<td>0.53</td>
<td>0.48</td>
<td>0.50</td>
<td>50</td>
<td>50</td>
<td>4.7 x 10^4</td>
<td></td>
</tr>
<tr>
<td>Gabbro</td>
<td>35 ± 5</td>
<td>0.32</td>
<td>0.25</td>
<td>0.28</td>
<td>35</td>
<td>30</td>
<td>3.4 x 10^4</td>
<td></td>
</tr>
<tr>
<td>Gabbro</td>
<td>50</td>
<td>0.18</td>
<td>0.15</td>
<td>0.16</td>
<td>40</td>
<td>30</td>
<td>2.9 x 10^4</td>
<td></td>
</tr>
<tr>
<td>Trachyte</td>
<td>30</td>
<td>0.63</td>
<td>0.54</td>
<td>0.58</td>
<td>60</td>
<td>70</td>
<td>5.0 x 10^4</td>
<td></td>
</tr>
<tr>
<td>Carrara marble</td>
<td>55</td>
<td>0.41</td>
<td>0.39</td>
<td>0.40</td>
<td>120</td>
<td>110</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Fig. 7.** Typical shear stress-normal stress relations for surfaces which show stick-slip. A, Red Granite 35 µin; B, gabbro 35 µin.

**Fig. 8.** X–Y recorder load-displacement curve for Red Granite 180 µin.
of this on a large scale from the X-Y recorder is shown in Fig. 8 for the Red Granite with a surface roughness of 180 ± 20 μm. The small number of discrete slips on the increasing curve do not initiate a relaxation oscillation, presumably because the difference between the static and dynamic coefficients of friction is too small.

7. STICK-SLIP OSCILLATIONS

It is well known from studies of metallic friction, Bowden and Tabor [8] Morgan, Morikat and Reed [11] that regular relaxation oscillations frequently occur in apparatus for measuring friction. The simplest model for these is that of a material whose dynamic coefficient of friction \( \mu' \) is less than the static coefficient \( \mu \). In this case the period is determined by \( \mu - \mu' \) and the elastic properties of the system.

From the present point of view the model is that of mass \( M \) pressed against a surface by force \( W \) and moved across the surface by means of a spring \( \beta M \) of stiffness \( \lambda \) whose end \( 0 \) is moved with constant speed \( V \), Fig. 9(a).

The mass will remain at rest relative to the plane until the force \( F \) exerted by the spring reaches the value

\[
F = \mu W. \quad (5)
\]

At this time it will slip, and its motion will be resisted by the force of dynamic friction \( \mu' W \). During slipping, if \( V \) is small, its displacement \( x \) relative to the plane is given, very nearly, by

\[
x = (\mu - \mu')W [1 - \cos nt] / \lambda \quad (6)
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where \( n = (\lambda/M)\). The mass comes to rest again at time \( t = \pi/n \), approximately, and the spring begins to compress again. The displacement \( X \) at each slip is

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X = 2(\mu - \mu')W / \lambda. \quad (7)
\]

The force \( F^* \) exerted by the spring when motion has just ceased is

\[
F^* = (2\mu' - \mu)W. \quad (8)
\]

It follows from (5), (7), (8) that

\[
x = (F - F^*) / \lambda. \quad (9)
\]
The period of the oscillations is

$$2(\mu - \mu^*) W/\lambda V.$$  

(10)

The force-displacement curve is shown by Fig. 9(b), and the displacement-time curve by Fig. 9(c).

Clearly, the curves of Fig. 6 show a considerable resemblance to the simple theoretical ones of Fig. 9(a). On a still larger scale a single oscillation of Fig. 6 takes the form $OAC$, Fig. 10(a) of which $OA$ represents a small displacement while the load is building up, $AC$ represents the slipping phase, and $BC$ is an effect of the machine pendulum. A small displacement before slip has been observed with metals by Bowden and Tabor [8]. Figure 10(b) shows the displacement-time curve for the same test. A high-speed record of this type suggests that slip takes place in a time less than 2 msec corresponding to $n > 1500$ sec$^{-1}$.

![Diagram](image)

**Fig. 10**

(a) One cycle of the oscillation of Fig. 6 on an enlarged scale.
(b) The same showing displacement against time.

It appears from Figs. 5 and 6 that the displacement $X$ at each slip increases with the amplitude of the oscillation and in Table 2 the mean values of $(F - F^*)/X$, where $F$ is the machine load at slip and $F^*$ that at the end of slip, are shown. By (9) these should equal the machine stiffness and measurements of the stiffness of the testing machine in this range give $\lambda = 3 - 5 \times 10^6$ lb/in so that there is good agreement.

It follows from (5) and (8) that the coefficient of dynamic friction $\mu'$ is the arithmetic mean of $\mu$ and $\mu^*$ as defined in Section 6 and this quantity $\mu'$ is shown in Table 2.

It appears that the simple mathematical model of this section fits the experimental facts very well.

Finally, it has been suggested by Brace and Byerlee [3] that stick-slip oscillations may provide a mechanism for earthquakes. If this is the case a generalization of the present theory must apply: there must be a more or less uniform driving force which will determine
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8. FRICTION BETWEEN WET SURFACES

The present system is not a good one for studying friction in the presence of water pressure. Some experiments on trachyte were made in which surfaces on which sliding had taken place were wetted and the experiment repeated. This did not have a significant effect on their behaviour, e.g. Table 1. In the case of sandstone water was admitted to the sliding surfaces through a hole at the centre of the blocks. Since the distribution of water pressure across the blocks was unknown it was difficult to interpret measurements under water pressure and its use was confined to thoroughly flushing the surfaces before a measurement.

9. SURFACE EFFECTS

With the 'rough' surfaces after 1/16 in. of surface displacement a great deal of surface damage and slickensiding was observed, Fig. 11.

For the smooth surfaces on which stick-slip oscillations had taken place there was very little evidence of surface damage. The measured surface roughness does not increase and the behaviour of the two marbles was quite different. The surfaces of the blocks of Wombeyan marble were not so flat as those of the other specimens and contact only occurred over widely separated regions whose total area was of the order of 1 of the total area of the surface. The stresses over the region of contact must have been quite high. One of these regions is shown in Fig. 12 and is associated with much powdered material and slickensiding.

Carrara marble, again, behaved differently. In this case the surfaces of macro-contact were numerous and smaller: they covered about 25 per cent of the surface area. In this case stick-slip oscillations of small amplitude were observed and the final surfaces were strongly slickensided. There is also extensive cracking of the material below the slickensides. This is shown in Fig. 13 which is a cross section of one of the contact regions made after immersing in penetrant dye.

10. CONCLUSIONS

It seems apparent from the above measurements that the frictional behaviour of surfaces is a variable quantity that depends very much on the nature of the surface. All the surfaces used here have been good ones compared with those likely to occur in rock mechanics and even with these considerable movement on the surface is necessary before approximately constant values of the shear stress necessary to cause sliding are achieved.

So far as can be observed for the low normal stresses used here Amonton's law (with added cohesion) is obeyed and there is no evidence of non-linear behaviour.

Again, from the point of view of rock mechanics, stick-slip oscillations do not seem to be important since they require a high degree of surface finish. It is, of course, possible that slickensided surfaces or some natural damage surfaces do possess this degree of finish. One question which cannot be answered with the present experimental system is the ultimate behaviour of the materials after a great deal of sliding. It might be expected that with smooth surfaces when sufficient surface damage had occurred and detrital material accumulated stick-slip oscillations would disappear: on the other hand, it is possible with some rough surfaces that uniform slickensides might develop on which stick-slip oscillations would be possible.
REFERENCES

Fig. 11. Surface of rough Trachyte after sliding.

Fig. 12. Contact region of Wombean marble showing surface damage and slickensiding.
Fig. 13. Section through connect region in C represents marble showing depth of crack.
To avoid these difficulties and, in particular, to allow considerable displacement under the same conditions, RAE [5] has used a slider on the curved surface of a rotating cylinder and COOK [6] has used sliding of a hollow cylinder on the plane end of a rotating cylinder. The present experiments use the simplest system of all, namely that of forcing a block of rock with plane and parallel surfaces between two other blocks as in Fig. 2.

This experiment can be performed on any scale and the main purpose of the present work was to use fairly large surfaces, of the order of a foot square, which might be more nearly comparable with natural joint surfaces than the smaller ones previously used. Since the cost of preparation of such specimens is fairly high, only a limited number could be used.

A second object of the work was to study the effect of surface roughness on frictional behaviour. JAEGER [1] reported differences in the behaviour of sliding rock surfaces that appeared to be related to the roughness of the surfaces: sawn and ground planar surfaces exhibited stick-slip phenomena, whilst artificially filled joint surfaces and natural fracture surfaces did not. BRACE and BYERLEE [3] have reported stick-slip behaviour on both sawn and naturally fractured surfaces. It seemed desirable to study this behaviour under carefully controlled conditions.

2. APPARATUS AND MATERIALS

The apparatus is shown in Fig. 2. A centre specimen C, 12 in. square is pushed between two outer specimens A, B of the same materials 12 in. × 10.5 in. supported by packers P on the lower platen of a testing machine. This arrangement allows 1.5 in. travel of the centre specimen with unchanged contact area, and this could easily be increased if necessary. Normal load is applied to the specimens by a pair of flat jacks F, F held in position by two box girders E, E connected by heavy bolts. The displacement of the specimen C is measured both by the machine recorder and by a linear variable differential transformer (LVDT). The load
L on the specimen is measured both by the machine chart recorder and by an LVDT attached to this recorder. This latter arrangement, though very simple, is not altogether satisfactory since it includes a component associated with the pendulum movement of the testing machine (Avery 500 ton hydraulic) but this is easily recognized. The LVDT outputs are fed to the axes of a Houston Omnigraph X-Y recorder which permits almost any desired magnification of the load–displacement curves on the testing machine recorder. The flat jacks were calibrated by loading them in the testing machine under conditions in which the distance between their surfaces was maintained constant. The normal load on the specimens is obtained from this calibration curve and is of the order of 95 per cent of the hydraulic pressure in the jacks.

The materials used were commercial building stones with the following descriptions:

‘Red Granite.’ A coarse-grained igneous rock consisting of 50 per cent orthoclase, 40 per cent quartz and 10 per cent accessories with grain size ranging from a few tenths of 1 mm to 2–3 cm.

‘Gabbro.’ A medium-grained igneous rock consisting of 60 per cent plagioclase, 30 per cent pyroxene and 10 per cent accessories with an average grain size of 0.5 to 2 mm.

‘Trachyte.’ An even-grained isotropic igneous rock consisting of 80 per cent orthoclase with minor quartz and slightly altered ferromagnesian minerals. Grain size averages about 1 mm.

‘Sandstone.’ Gosford sandstone. A uniform, weakly cemented quartz sandstone. Grain size is approximately 1 mm.

‘Carrara marble.’ An essentially pure calcite marble with a grain size of approximately 0.2 mm.

‘Wombeyan marble.’ A marble of very irregular grain size varying between 2 and 10 mm. The surface of this material was not flat to the tolerance described below.

Two experiments were made with wet surfaces and one with mixed surfaces, i.e. sandstone on trachyte.

The final macroscopic area of contact between the surfaces, that is, the area of surface damaged by sliding was approximately 95 per cent of the total area except for the two marbles for which it was much less.

It became apparent on commencing the experiments that, for a given lateral load, the frictional force varied considerably with the amount of ‘running in’ that the surfaces had had. For this reason a standard order of experiment was adopted. All experiments were begun with new surfaces at a normal stress of 500 psi and a considerable displacement was made under these conditions. The tangential load was released before making any change in the normal loads. In some cases it was found that normal stresses greater than 1000 psi caused cracking of the specimens so greater stresses than this were not used. All runs were made at a standard rate of platen advance of approximately 1⁄2 in/hr.

3. SURFACE FINISH

The standard machine shop terminology and techniques for the designation of surface quality are used.

The term ‘roughness’ refers only to relatively closely spaced surface irregularities such as those produced by sawing, grinding and lapping. Roughness may be specified by either direct measurements of the height of these irregularities or by their average or root-mean-square deviation from the ‘mean surface’. The mean surface is the surface located so that the volume of peaks above it is equal to the volume of valleys below. The ‘arithmetic average
deviation' of the surface is the result of taking a great many uniformly spaced measurements of the distance of the actual surface from the mean surface and averaging them. It is expressed in micro-inches. 'Flatness' refers to the larger scale warping in the mean surface over a relatively much larger area and is a completely separate measurement from roughness.

The blocks as received from the suppliers were specified to have surfaces flat and parallel to $\frac{1}{16}$ in. and were sometimes improved by reworking by ourselves.

The instrument used to measure surface 'roughness' was a Brush Surfindicometer (Model 185). This instrument has a probe with a diamond-tipped stylus which is traversed over the specimen. Vertical movement of the stylus is measured, averaged and displayed on a meter directly as the 'arithmetic average deviation' of the surface. Standard surfaces are furnished with the instrument and it is calibrated on these standards before each measurement. The lower limit of measurement is about 5 $\mu$in so we are not measuring the very small irregularities of the order of a few hundred Angstroms reported by Bowden and Tabor [8] which were observed by optical and electron microscope techniques. There are many other aspects of surface quality that are also not measured with a machine of this type such as the accurate shape of the irregularities or minor periodicities. It does give a semi-quantitative means of comparing surfaces, however, which was its purpose in these experiments.

The blocks designated 'rough' in Table 1 were far too rough for the Surfindicometer to be used without damage to its stylus. Irregularities of the order of $\frac{1}{16}$ in. were present on the otherwise 'flat' surfaces.

The behaviour of the various rocks divided sharply into two types apparently determined by surface roughness:

(i) A steady and smooth increase of frictional force asymptotically towards a final value determined by the normal stress.

(ii) Regular 'stick-slip' relaxation oscillations.

4. ROUGH SURFACES

The characteristic behaviour for moderately rough surfaces is shown in Fig. 3 which shows the load-displacement curve for trachyte with a surface roughness of 350 $\mu$in.

The fresh rock surfaces are brought together under a jack pressure of 500 psi and the system loaded. The load at first increases rapidly with displacement and subsequently at a steadily decreasing rate, Fig. 3, curve (i). Part of this increase, of course, may be due to the fact that fresh surface is continually being moved into the rubbing area. After considerable displacement in curve (i) so that the surfaces may be regarded as well 'run-in' the tangential load is released and the normal load changed so that the same portions of the surface are in contact under a different normal load. The experiment is then repeated, Fig. 3, curves (ii)–(vii). There may be small variations in behaviour between different materials at this stage. In some cases, there is a very slight peak in the load which can just be seen, e.g. in Fig. 3, curves (vi), (vii), (viii) followed by an approximately constant value: in others, the load rises slightly with displacement.

The suggestion from the curves of Fig. 3 is that the tangential stress between sliding surfaces varies with the amount of 'running in' that these surfaces have had and that ultimately a surface characteristic of the material, and perhaps of the normal stress, is established. It is the tangential stress across this surface, that is, the value to which the curves of Fig. 3 are asymptotic which is given in Tables 1 and 2. Some of the surfaces on which sliding has taken place are discussed in Section 9: it appears that they are slickensided and contain detrital material. These considerations make it very difficult to define and measure friction.
A MEDIUM-SCALE DIRECT FRICTION EXPERIMENT

between rock surfaces. Measurements during the early stages of sliding correspond to removing some of the major protuberances and so are not representative; in the later stages the surfaces will include fractured crystals, slickensides and detrital material. However, it is probably such surfaces that are in question in rock mechanics.

Plotting the values of the tangential stress $\tau$ taken from Fig. 3, curves (ii)–(viii) against the normal stress $\sigma$ gives a very good straight line of the form

$$\tau = c + \mu \sigma$$  \hspace{1cm} (1)

where $\mu$ is the coefficient of friction and $c$ may be called the cohesion.

Plots of $\tau$ against $\sigma$ for various materials are given in Fig. 4 and the values of $\mu$ and $c$ for a number of surfaces behaving in this way are given in Table 1.

![Figure 3. Testing machine recorder load–displacement curves for rough Trachyte. Flat jack pressures (i) 500 psi; (ii) 250 psi; (iii) 125 psi; (iv) 250 psi; (v) 375 psi; (vi) 500 psi; (vii) 625 psi; (viii) 750 psi.](image)

![Figure 4. Typical shear stress–normal stress relations for surfaces which do not show stick–slip. A, Wombeyan marble; B, Trachyte rough; C, Trachyte 350 µin; D, Sandstone.](image)
148

E. R. HOSKINS, J. C. JAEGGER AND K. J. ROSENGREN

Table 1. Coefficient of friction $\mu$ and cohesion $c$ for rock surfaces which give regular sliding

<table>
<thead>
<tr>
<th>Rock</th>
<th>Surface roughness</th>
<th>$\mu$</th>
<th>$c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Red Granite</td>
<td>180 ± 20</td>
<td>0.64</td>
<td>45</td>
</tr>
<tr>
<td>Gabbro</td>
<td>Rough</td>
<td>0.66</td>
<td>55</td>
</tr>
<tr>
<td>Trachyte</td>
<td>300-350</td>
<td>0.58</td>
<td>65</td>
</tr>
<tr>
<td>Trachyte Rough</td>
<td>Rough</td>
<td>0.68</td>
<td>60</td>
</tr>
<tr>
<td>Wet trachyte</td>
<td>300-350</td>
<td>0.56</td>
<td>60</td>
</tr>
<tr>
<td>Sandstone</td>
<td>Rough</td>
<td>0.51</td>
<td>40</td>
</tr>
<tr>
<td>Wet sandstone</td>
<td>Rough</td>
<td>0.61</td>
<td>45</td>
</tr>
<tr>
<td>Sandstone on trachyte</td>
<td>Rough</td>
<td>0.57</td>
<td>40</td>
</tr>
<tr>
<td>Wombeyan marble</td>
<td>200</td>
<td>0.75</td>
<td>160</td>
</tr>
</tbody>
</table>

It appears that in all cases, except the Wombeyan marble, the coefficient of friction lies in the range 0.5-0.7 and that there is a small cohesion of the order of 50 psi.

For Wombeyan marble both $\mu$ and $c$ are relatively high. This is probably due in part to the fact that for the block used contact was only over about 0.25 of the area so that the normal stresses would be sufficient to bring the material into the ductile state. There was great surface damage to the material and fracture extended some distance into the material below the area of contact.

5. THE EXPRESSION FOR THE LAW OF FRICTION

The most natural extension of Amonton's law is the linear law (1) used to interpret the preceding experiments. It has the advantage of leading to simple mathematical formulae and of being in line with the simple Coulomb theory of soil and rock mechanics. This representation was used by JAEGGER [1] and LANE and HECK [2] and is seen to represent the present results adequately.

ARCHARD [7] has provided theoretical justification for a power law

$$\tau = \mu \sigma^n$$

where $\mu$ is a constant and $2/3 < n < 1$. MURRELL [4] interprets his results on sandstone in this way obtaining the result

$$\tau = 2.1 (\sigma)^{0.99}$$

but in fact his results for $\sigma < 20,000$ psi may be equally well represented by

$$\tau = 500 + 0.70 \sigma$$

and his results for $\sigma > 20,000$ psi are rather scattered and also in a region in which the material may be ductile. Other writers, e.g., BOWDEN and TABOR [8] RALEIGH and PATTERSON [9] and MAURER [10] determine $\mu$ from $\mu = \tau/\sigma$ in each experiment and plot $\mu$ against $\sigma$ obtaining values of $\mu$ which decrease rapidly with increasing $\sigma$. This would be the case if $\tau$ and $\sigma$ were connected by either of the laws (1) or (2). MAURER [10] has performed a double-shear experiment on rods of cross section $1 \times 0.5$ in and has continued sliding after shear failure under conditions rather similar to those of the present experiments. His results seem to fit the formula (1) reasonably well.

The present experiments indicate that the linear law (1) is applicable for the relatively low normal stresses used.
6. SMOOTHER SURFACES

The typical behaviour is shown in Fig. 5 for the same blocks of Red Granite as those described in Table 1 but with the surface lapped to a roughness of 35 ± 5 µin. After an initial rise in load a sudden slip occurs and stick-slip proceeds regularly with increasing amplitude, as in Fig. 5 (i) taken from the machine recorder which is shown on a larger scale in Fig. 6 taken from the X–Y recorder fed by the LVDTs. In Fig. 6 the lower portion of the oscillations (below the dotted curve PQ) is caused by motion of the testing machine pendulum, and the actual amplitude of the oscillation is the distance between PQ and the upper part of the curve.

As before, the normal stress σ on the joint can be varied as in Fig. 5(ii)–(viii) and the tangential stress τ at slip plotted against σ which gives a straight line corresponding to a coefficient of friction μ and cohesion c. Similarly τ at the lower limit PQ can be plotted giving values μ* and c*. Plots of this type are shown in Fig. 7 and numerical values in Table 2. As before, the values of τ used are the asymptotic values reached after a considerable surface displacement.

Comparison of the results in Tables 1 and 2 indicate surprising variability for the same rock with different surface finishes. For the trachyte much the same values are obtained. On the other hand, the gabbro shows a wide variation in μ from 0·18 to 0·66. The first experiment on gabbro with a roughness of 35 µin gave μ = 0·32. This block was apparently not...
sufficiently flat and widespread cracking occurred during the test. The experiment was therefore repeated on another set of surfaces lapped together to a roughness of 50 µin and this gave the even lower value of $\mu = 0.18$. The blocks did not crack during this test. It is possible that the minerals in contact between the gabbro surfaces have lower coefficients of friction than those on the other rocks tested but we can offer no explanation for the wide variation in friction values.

The fact that stick–slip behaviour can be inhibited by reworking the surface to a different roughness suggests that it might be possible to observe a transition in behaviour. An example
of this on a large scale from the X-Y recorder is shown in Fig. 8 for the Red Granite with a surface roughness of $180 \pm 20 \mu\text{in.}$ The small number of discrete slips on the increasing curve do not initiate a relaxation oscillation, presumably because the difference between the static and dynamic coefficients of friction is too small.

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![Fig. 9](image_url)

(a) Simple model for stick-slip oscillations.
(b) Force-displacement curve.
(c) Displacement-time curve.

The mass will remain at rest relative to the plane until the force $F$ exerted by the spring reaches the value

$$F = \mu W.$$  (5)

At this time it will slip, and its motion will be resisted by the force of dynamic friction $\mu'W$. During slipping, if $V$ is small, its displacement $x$ relative to the plane is given, very nearly, by

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\[ \text{(a)} \]

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(a) One cycle of the oscillation of Fig. 6 on an enlarged scale.
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It appears from Figs. 5 and 6 that the displacement X at each slip increases with the amplitude of the oscillation and in Table 2 the mean values of \((F - F^*)/X\), where \( F \) is the machine load at slip and \( F^* \) that at the end of slip, are shown. By (9) these should equal the machine stiffness and measurements of the stiffness of the testing machine in this range give \( \lambda = 3-5 \times 10^6 \text{ lb/in} \) so that there is good agreement.

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E. R. HOSKINS, J. C. JAEGGER AND K. J. ROSENGREN

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6. COOK N. G. W. Personal communication (1966).
Fig. 11. Surface of rough Trachyte after sliding.

Fig. 12. Contact region of Wombeyan marble showing surface damage and slickensiding.
Fig. 13. Section through contact region in Carrara marble showing depth of cracking.
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Simple photographic equipment for borehole surveying

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ms received 20 June 1967

Abstract Two easily constructed instruments are described which are capable of measuring the direction of boreholes with an accuracy of one degree for a number of orientations of practical importance.

The problem of surveying boreholes, that is, finding their orientation at any depth, has always been a major one for the mining industry. For many years clockwork operated instruments, such as the Tropari, were the standard borehole surveying instruments. Such instruments suffer from the major disadvantage that they are of the 'single shot' type and must be withdrawn and reinserted in the hole after each measurement. This makes the surveying operation very slow particularly in nearly horizontal holes into which the instrument has to be pushed on rods. Recently, more sophisticated instruments, using compass needles with multishot photography or fluxgate magnetometers with remote recording, have become available but these are usually very costly and are only suitable for the larger size holes.

For these reasons many holes drilled by mining companies and in engineering construction work are not surveyed and there is always a possibility that they have deviated from their original direction. In a programme of geothermal measurements in Australia we have measured many such holes and have felt the need of an instrument to measure inclination: this is all that is needed in these and many other geophysical measurements.

The simple instrument shown in figure 1 is one inch in diameter and so will fit into the very common EX holes, approximately 1·5 inches in diameter. It will work satisfactorily in holes whose inclination to the vertical is as much as 30° and this range could easily be extended. It has photographic recording, current being supplied to a lamp by a cable of the same type as that used with thermistor probes so that the same winch and depth measuring equipment may be used.

The arrangement is shown in figure 1. The cable C actuating the lamp L is held in the measuring head by an araldite seal. Since there is no restriction on length, no elaborate optics are necessary and light from L passes through a pinhole P in thin foil on to the mirror M in the photographic chamber. The mirror M is attached to a hollow conical weight which rests on a needle point so that the mirror is always horizontal. The reflected ray is received on photographic paper: a flat piece F with a central hole is used on the end of the cassette and a cylindrical piece G on the sides. The ring R acts partly as a stop to ensure that the cone does not move too far from the pin and partly to secure the film G. The outer tube T is sealed by O-rings O, and held in position by screws S. The external diameter of the instrument is 1 in., which provides a scale for the drawing. Its total length is 2 ft.

Despite the crude optical and photographic arrangements the instrument is accurate to one degree and quite rapid to operate. A sketch of a typical record obtained from the flat paper F is shown in figure 2. The inclination of the hole at a given depth is measured by the distance of the spot from the central hole surrounding the pinhole. The radius of this paper is equivalent to 10°, the intermediate scale being determined by a calibration run. Different readings are distinguished by the size of the spot which is varied by different exposure times for each reading. Up to six measurements can be made with one loading in this manner. The paper is developed on the site with the aid of a black cloth bag, which is also used when loading and unloading the instrument.

As two examples of the value of the instrument, the following may be given. (i) A vertical hole 1·5 inches in diameter was drilled to a depth of 1700 ft in dolerite in Tasmania. The petrology of the core suggested that the hole might have deflected as could well be expected with so long a hole of this small diameter. The inclination to the vertical was found to vary quite regularly being 2° at 1000 ft and 3° at
Notes on experimental technique and apparatus

Figure 2

1600 ft. (ii) A hole, collared vertically, was drilled to 700 ft in steeply dipping cherts which it was felt might have deflected the drill. In fact, the inclination increased to 8° at 500 ft and had returned to 6° at 700 ft.

Following the success of the simple inclination instrument, the more elaborate one shown in figure 3 was built. This was intended for measurement of both inclination and azimuth in nearly horizontal holes but it may be used for inclined holes. The outside diameter is now 2.5 in which sets the scale of the figure. As shown, the instrument is used for measurement of azimuth. A compass needle N is mounted with its axis normal to a cassette C. This is mounted in a pivoted inner tube T which contains a weight W adjusted so that the axis of rotation of the compass needle is in a vertical plane. Because of the increased diameter, it has been possible to use three lamps L, the light from any one of which passes through the pinhole P and is reflected from the mirror M on to photographic paper located around the circumference of the cassette C. Current is supplied to the lamps through slip-rings S from the 4-core cable A. The tube T is adjusted in its bearings by the nut B, but once this adjustment has been made the system is disassembled by unscrewing the head at thread D, and on reassembly the inner tube will locate itself in position. The instrument can be attached to rods, if necessary, by suitable fittings at the end of the rear tube E.

For the measurement of inclination an alternative cassette is used in which the compass needle N is replaced by a weighted needle which acts as a pendulum. The weight W on the inner tube T is moved to an alternative position for which the axis of this needle is horizontal. In both cassettes the compass needle and the weighted needle, respectively, are attached to the axis by a friction joint so that suitable initial settings can be achieved. Measurements with these two cassettes allow the azimuth and inclination of a hole to be calculated.

Since the compass needle does not remain in the horizontal plane when the instrument is tilted, the azimuth is not read directly in inclined holes. The compass needle aligns itself so that it is at the minimum angle to the earth's field consistent with the geometry of the instrument. Knowing the dip of the hole and the resultant direction of the earth's field the true azimuth can be calculated, most conveniently by means of a stereonet. However, the instrument is not suitable for holes inclined at nearly 90° to the direction of the earth's field since the torque which aligns the compass needle becomes very small. These difficulties do not arise with horizontal holes.

This instrument has been used for precise orientation of a number of horizontal holes 700 ft in length drilled for careful measurements for purposes of structural petrology.

A typical calibration record from this instrument is drawn in figure 4. The spacing between the groups of spots corresponds to 10° rotations of the needle. Different readings are distinguished by coding the combinations of spots as shown and also by varying the size of spot with different exposure times. Because of the small deviations of most boreholes, the practical limit is usually about ten readings with one loading of paper. The small hole H which locates the paper around the pinhole in the cassette serves as a reference mark and the instrument is calibrated on the surface before each set of readings.

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